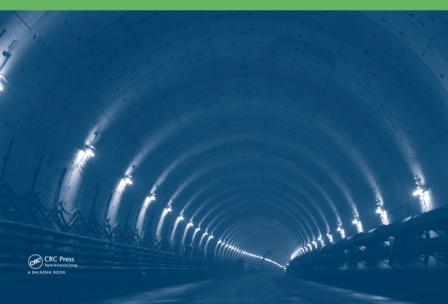
C.W.W. Ng, H.W. Huang & G.B. Liu, editors

# GEOTECHNICAL ASPECTS OF UNDERGROUND CONSTRUCTION IN SOFT GROUND



## GEOTECHNICAL ASPECTS OF UNDERGROUND CONSTRUCTION IN SOFT GROUND

# Geotechnical Aspects of Underground Construction in Soft Ground

**Editors** 

C.W.W. Ng Hong Kong University of Science and Technology, Hong Kong Special Administrative Region

H.W. Huang & G.B. Liu Tongji University, Shanghai, China



CRC Press is an imprint of the Taylor & Francis Group, an **informa** business A BALKEMA BOOK

CRC Press/Balkema is an imprint of the Taylor & Francis Group, an informa business

© 2009 Taylor & Francis Group, London, UK

Typeset by Charon Tec Ltd (A Macmillan Company), Chennai, India Printed and bound in Great Britain by Cromwell Press Ltd, Trowbridge, Wiltshire.

All rights reserved. No part of this publication or the information contained herein may be reproduced, stored in a retrieval system, or transmitted in any form or by any means, electronic, mechanical, by photocopying, recording or otherwise, without written prior permission from the publishers.

Although all care is taken to ensure integrity and the quality of this publication and the information herein, no responsibility is assumed by the publishers nor the author for any damage to the property or persons as a result of operation or use of this publication and/or the information contained herein.

Published by: CRC Press/Balkema P.O. Box 447, 2300 AK Leiden, The Netherlands e-mail: Pub.NL@taylorandfrancis.com www.crcpress.com – www.taylorandfrancis.co.uk – www.balkema.nl

ISBN: 978-0-415-48475-6 (Hardback) ISBN: 978-0-203-87998-6 (eBook)

## Table of Contents

Preface	XIII
Sponsors	XV
Special lectures	
Processes around a TBM A. Bezuijen & A.M. Talmon	3
Supporting excavations in clay – from analysis to decision-making <i>M.D. Bolton, S.Y. Lam &amp; A.S. Osman</i>	15
Overview of Shanghai Yangtze River Tunnel Project R. Huang	29
Underground construction in decomposed residual soils I.M. Lee & G.C. Cho	45
General reports	
Safety issues, risk analysis, hazard management and control C.T. Chin & H.C. Chao	67
Calculation and design methods, and predictive tools <i>F. Emeriault &amp; R. Kastner</i>	77
Analysis and numerical modeling of deep excavations R.J. Finno	87
Construction method, ground treatment, and conditioning for tunneling <i>T. Hashimoto, B. Ye &amp; G.L. Ye</i>	99
Physical and numerical modelling P.L.R. Pang	109
Case histories A. Sfriso	121
Theme 1: Analysis and numerical modeling of deep excavations	
Optimization design of composite soil-nailing in loess excavation <i>G.M. Chang</i>	133
Three-dimensional finite element analysis of diaphragm walls for top-down construction J. Hsi, H. Zhang & T. Kokubun	141
Numerical evaluation of dewatering effect on deep excavation in soft clay L. Li & M. Yang	147
Analysis of the factors influencing foundation pit deformations Y.Q. Li, K.H. Xie, J. Zhou & X.L. Kong	153

Construction monitoring and numerical simulation of an excavation with SMW retaining structure <i>Z.H. Li &amp; H.W. Huang</i>	159
A simplified spatial methodology of earth pressure against retaining piles of pile-row retaining structure <i>Y.L. Lin &amp; X.X. Li</i>	165
Consideration of design method for braced excavation based on monitoring results <i>H. Ota, H. Ito, T. Yanagawa, A. Hashimoto, T. Hashimoto &amp; T. Konda</i>	173
Ground movements in station excavations of Bangkok first MRT N. Phienwej	181
Numerical modelling and experimental measurements for a retaining wall of a deep excavation in Bucharest, Romania <i>H. Popa, A. Marcu &amp; L. Batali</i>	187
3D finite element analysis of a deep excavation and comparison with in situ measurements <i>H.F. Schweiger, F. Scharinger &amp; R. Lüftenegger</i>	193
The effect of deep excavation on surrounding ground and nearby structures A. Siemińska-Lewandowska & M. Mitew-Czajewska	201
Multi-criteria procedure for the back-analysis of multi-supported retaining walls J. Zghondi, F. Emeriault & R. Kastner	207
Monitoring and modelling of riverside large deep excavation-induced ground movements in clays <i>D.M. Zhang, H.W. Huang &amp; W.Y. Bao</i>	215
GPS height application and gross error detection in foundation pit monitoring <i>H. Zhang, S.F. Xu &amp; T.D. Lu</i>	223
Study on deformation laws under the construction of semi-reverse method J. Zhang, G.B. Liu & T. Liu	227
Comparison of theory and test on excavation causing the variation of soilmass strength <i>J. Zhou, J.Q. Wang &amp; L. Cong</i>	235
Theme 2: Construction method, ground treatment, and conditioning for tunnelling	
Ten years of bored tunnels in The Netherlands: Part I, geotechnical issues <i>K.J. Bakker &amp; A. Bezuijen</i>	243
Ten years of bored tunnels in The Netherlands: Part II, structural issues <i>K.J. Bakker &amp; A. Bezuijen</i>	249
The influence of flow around a TBM machine A. Bezuijen & K.J. Bakker	255
Mechanisms that determine between fracture and compaction grouting in sand <i>A. Bezuijen, A.F. van Tol &amp; M.P.M. Sanders</i>	261
Research of non-motor vehicle-rail transit-tube interchanging transport system pattern A.Z.G. Deng & Q.H. Zhang	269
Shotcrete excavations for the Munich subway – Comparison of different methods of face support in settlement sensitive areas <i>J. Fillibeck &amp; N. Vogt</i>	275
Fracturing of sand in compensation grouting K. Gafar, K. Soga, A. Bezuijen, M.P.M. Sanders & A.F. van Tol	281

Historical cases and use of horizontal jet grouting solutions with 360° distribution and frontal septum to consolidate very weak and saturated soils <i>G. Guatteri, A. Koshima, R. Lopes, A. Ravaglia &amp; M.R. Pieroni</i>	287
The effects of sample dimension and gradation on shear strength parameters of conditioned soils in EPBM M. Hajialilue-Bonab, M. Ahmadi-adli, H. Sabetamal & H. Katebi	295
Experimental study on compressibility behavior of foamed sandy soil <i>M. Hajialilue-Bonab, H. Sabetamal, H. Katebi &amp; M. Ahmadi-adli</i>	301
Study on earth pressure acting upon shield tunnel lining in clayey and sandy grounds based on field monitoring <i>T. Hashimoto, G.L. Ye, J. Nagaya, T. Konda &amp; X.F. Ma</i>	307
The double-o-tube shield tunnel in Shanghai soil C. He, L. Teng & J.Y. Yan	313
Frozen soil properties for cross passage construction in Shanghai Yangtze River Tunnel X.D. Hu & A.R. Pi	319
The influence of engineering-geological conditions on construction of the radioactive waste dump <i>J. Kuzma &amp; L. Hrustinec</i>	325
Critical ventilation velocity in large cross-section road tunnel fire Z.X. Li, X. Han & K.S. Wang	331
Metro tunnels in Buenos Aires: Design and construction procedures 1998–2007 A.O. Sfriso	335
Study on the earth pressure distribution of excavation chamber in EPB tunneling <i>T.T. Song &amp; S.H. Zhou</i>	343
Backfill grouting research at Groene Hart Tunnel A.M. Talmon & A. Bezuijen	349
Longitudinal tube bending due to grout pressures A.M. Talmon, A. Bezuijen & F.J.M. Hoefsloot	357
Theme 3: Case histories	
Tunnel face stability and settlement control using earth pressure balance shield in cohesionless soil <i>A. Antiga &amp; M. Chiorboli</i>	365
Displacements and stresses induced by a tunnel excavation: Case of Bois de Peu (France) S. Eclaircy-Caudron, D. Dias & R. Kastner	373
Shield tunneling beneath existing railway line in soft ground Q.M. Gong & S.H. Zhou	381
Case history on a railway tunnel in soft rock (Morocco) A. Guiloux, H. Le Bissonnais, J. Marlinge, H. Thiebault, J. Ryckaert, G. Viel, F. Lanquette, A. Erridaoui & M.Q.S. Hu	385
Observed behaviours of deep excavations in sand B.C.B. Hsiung & H.Y. Chuay	393
Environmental problems of groundwater around the longest expressway tunnel in Korea S.M. Kim, H.Y. Yang & S.G. Yoon	399

Measurements of ground deformations behind braced excavations T. Konda, H. Ota, T. Yanagawa & A. Hashimoto	405
Research on the effect of buried channels to the differential settlement of building <i>D.P. Liu, R. Wang &amp; G.B. Liu</i>	413
Performance of a deep excavation in soft clay G.B. Liu, J. Jiang & C.W.W. Ng	419
Deformation monitoring during construction of subway tunnels in soft ground S.T. Liu & Z.W. Wang	427
The construction and field monitoring of a deep excavation in soft soils <i>T. Liu, G.B. Liu &amp; C.W.W. Ng</i>	433
Excavation entirely on subway tunnels in the central area of the People's Square <i>Y.B. Mei, X.H. Jiang, Y.M. Zhu &amp; H.C. Qiao</i>	441
The benefits of hybrid ground treatment in significantly reducing wall movement: A Singapore case history <i>N.H. Osborne, C.C. Ng &amp; C.K. Cheah</i>	447
3D deformation monitoring of subway tunnel D.W. Qiu, K.Q. Zhou, Y.H. Ding, Q.H. Liang & S.L. Yang	455
Challenging urban tunnelling projects in soft soil conditions H. Quick, J. Michael, S. Meissner & U. Arslan	459
Supervision and protection of Shanghai Mass Rapid Line 4 shield tunneling across the adjacent operating metro line <i>R.L. Wang, Y.M. Cai &amp; J.H. Liu</i>	465
Kowloon Southern Link – TBM crossing over MTR Tsuen Wan Line tunnels in HKSAR K.K.W. Wong, N.W.H. Ng, L.P.P. Leung & Y. Chan	471
Application of pile underpinning technology on shield machine crossing through pile foundations of road bridge <i>Q.W. Xu, X.F. Ma &amp; Z.Z. Ma</i>	477
Characteristics of tunneling-induced ground settlement in groundwater drawdown environment <i>C. Yoo, S.B. Kim &amp; Y.J. Lee</i>	485
Effect of long-term settlement on longitudinal mechanical performance of tunnel in soft soil <i>H.L. Zhao, X. Liu, Y. Yuan &amp; Y. Chi</i>	491
Theme 4: Safety issues, risk analysis harzard management and control	
Research on stochastic seismic analysis of underground pipeline based on physical earthquake model <i>X.Q. Ai &amp; J. Li</i>	499
Risk assessment for the safe grade of deep excavation X.H. Bao & H.W. Huang	507
Multi-factors durability evaluation in subway concrete structure C. Chen, L. Yang & C. Han	513
The use of artificial neural networks to predict ground movements caused by tunneling <i>I. Chissolucombe, A.P. Assis &amp; M.M. Farias</i>	519
Research and application of road tunnel structural optimization <i>W.Q. Ding &amp; Y. Xu</i>	525

Floor heave behavior and control of roadway intersection in deep mine <i>B.H. Guo &amp; T.K. Lu</i>	531
Squeezing potential of tunnels in clays and clayshales from normalized undrained shear strength, unconfined compressive strength and seismic velocity <i>M. Gutierrez &amp; C.C. Xia</i>	537
Framework of performance-based fire protection design method for road tunnel <i>X. Han &amp; G.Y. Ding</i>	545
Prediction of surface settlements induced by shield tunneling: An ANFIS model J. Hou, M.X. Zhang & M. Tu	551
Experimental studies of a geological measuring system for tunnel with ultrasonic transducer <i>D.H. Kim, U.Y. Kim, S.P. Lee, H.Y. Lee &amp; J.S. Lee</i>	555
Performance review of a pipe jacking project in Hong Kong <i>T.S.K. Lam</i>	561
Geotechnical control of a major railway project involving tunnel works in Hong Kong W. Lee, S.S. Chung, K.J. Roberts & P.L.R. Pang	567
Research on structural status of operating tunnel of metro in Shanghai and treatment ideas J.P. Li, R.L. Wang & J.Y. Yan	573
Maximising the potential of strain gauges: A Singapore perspective N.H. Osborne, C.C. Ng, D.C. Chen, G.H. Tan, J. Rudi & K.M. Latt	579
Discussion on design method for retaining structures of metro station deep excavations in Shanghai <i>R. Wang, G.B. Liu, D.P. Liu &amp; Z.Z. Ma</i>	587
Risk analysis for cutterhead failure of composite EPB shield based on fuzzy fault tree Y.R. Yan, H.W. Huang & Q.F. Hu	595
Risk assessment on environmental impact in Xizang Road Tunnel C.P. Yao, H.W. Huang & Q.F. Hu	601
Risk analysis and fuzzy comprehensive assessment on construction of shield tunnel in Shanghai metro Line <i>H.B. Zhou, H. Yao &amp; W.J. Gao</i>	607
Theme 5: Physical and numerical modelling	
Tunnel behaviour under seismic loads: Analysis by means of uncoupled and coupled approaches <i>D. Boldini &amp; A. Amorosi</i>	615
Investigating the influence of tunnel volume loss on piles using photoelastic techniques <i>W. Broere &amp; J. Dijkstra</i>	621
Assessment of tunnel stability in layered ground P. Caporaletti, A. Burghignoli, G. Scarpelli & R.N. Taylor	627
Reinforcing effects of forepoling and facebolts in tunnelling <i>K. Date, R.J. Mair &amp; K. Soga</i>	635
Mechanical behavior of closely spaced tunnels — laboratory model tests and FEM analyses <i>J.H. Du &amp; H.W. Huang</i>	643
Stability analysis of masonry of an old tunnel by numerical modelling and experimental design <i>J. Idris, T. Verdel &amp; M. Alhieb</i>	649

Excavation with stepped-twin retaining wall: Model tests and numerical simulations N. Iwata, H.M. Shahin, F. Zhang, T. Nakai, M. Niinomi & Y.D.S. Geraldni	655
Stability of an underwater trench in marine clay under ocean wave impact <i>T. Kasper &amp; P.G. Jackson</i>	663
A study on behavior of 2-Arch tunnel by a large model experiment S.D. Lee, K.H. Jeong, J.W. Yang & J.H. Choi	669
Behavior of tunnel due to adjacent ground excavation under the influence of pre-loading on braced wall <i>S.D. Lee &amp; I. Kim</i>	677
Two distinctive shear strain modes for pile-soil-tunnelling interaction in a granular mass <i>Y.J. Lee &amp; C.S. Yoo</i>	683
Stability analysis of large slurry shield-driven tunnel in soft clay Y. Li, Z.X. Zhang, F. Emeriault & R. Kastner	689
Effects of soil stratification on the tunneling-induced ground movements F.Y. Liang, G.S. Yao & J.P. Li	697
Centrifuge modelling to investigate soil-structure interaction mechanisms resulting from tunnel construction beneath buried pipelines <i>A.M. Marshall &amp; R.J. Mair</i>	703
Ground movement and earth pressure due to circular tunneling: Model tests and numerical simulations <i>H.M. Shahin, T. Nakai, F. Zhang, M. Kikumoto, Y. Tabata &amp; E. Nakahara</i>	709
Analysis of pre-reinforced zone in tunnel considering the time-dependent performance K.I. Song, J. Kim & G.C. Cho	717
Vault temperature of vehicle fires in large cross-section road tunnel K.S. Wang, X. Han & Z.X. Li	725
Effects of different bench length on the deformation of surrounding rock by FEM X.M. Wang, H.W. Huang & X.Y. Xie	729
The effects of loaded bored piles on existing tunnels J. Yao, R.N. Taylor & A.M. McNamara	735
3D FEM analysis on ground displacement induced by curved pipe-jacking construction <i>G.M. You</i>	743
Theme 6: Calculation and design methods, and predictive tools	
Calculation of the three dimensional seismic stressed state of "Metro Station–Escalator–Open Line Tunnels" system, which is located in inclined stratified soft ground <i>R.B. Baimakhan, N.T. Danaev, A.R. Baimakhan, G.I. Salgaraeva, G.P. Rysbaeva,</i> <i>Zh.K. Kulmaganbetova, S. Avdarsolkyzy, A.A. Makhanova &amp; S. Dashdorj</i>	751
A complex variable solution for tunneling-induced ground movements in clays <i>H.L. Bao, D.M. Zhang &amp; H.W. Huang</i>	757
Simulation of articulated shield behavior at sharp curve by kinematic shield model <i>J. Chen, A. Matsumoto &amp; M. Sugimoto</i>	761
Deformation and pore pressure model of the saturated silty clay around a subway tunnel <i>Z.D. Cui, Y.Q. Tang &amp; X. Zhang</i>	769
Analytical solution of longitudinal behaviour of tunnel lining <i>F.J.M. Hoefsloot</i>	775

Design of tunnel supporting system using geostatistical methods S. Jeon, C. Hong & K. You	781
Comparative study of software tools on the effects of surface loads on tunnels <i>D.K. Koungelis &amp; C.E. Augarde</i>	785
Geologic Model Transforming Method (GMTM) for numerical analysis modeling in geotechnical engineering X.X. Li, H.H. Zhu & Y.L. Lin	791
Review and interpretation of intersection stability in deep underground based on numerical analysis <i>T.K. Lu, B.H. Guo, L.C. Cheng &amp; J. Wang</i>	799
Analysis of surface settlement due to the construction of a shield tunnel in soft clay in Shanghai Z.P. Lu & G.B. Liu	805
Urban tunnels in soil: Review of current design practice in Brazil A. Negro	811
A study on loads from complex support system using simple 2D models Z. Shi, W. Bao, J. Li, W. Guo & J. Zhu	817
Ground reaction due to tunnelling below groundwater table Y.J. Shin, J.H. Shin & I.M. Lee	823
Basal stability of braced excavations in $K_0$ -consolidated soft clay by upper bound method X.Y. Song & M.S. Huang	829
Analytical two and three dimension models to assess stability and deformation magnitude of underground excavations in soil <i>L.E. Sozio</i>	837
Dynamic response of saturated silty clay around a tunnel under subway vibration loading in Shanghai <i>Y.Q. Tang, Z.D. Cui &amp; X. Zhang</i>	843
Lateral responses of piles due to excavation-induced soil movements C.R. Zhang, M.S. Huang & F.Y. Liang	849
Elastic-plastic analysis for surrounding rock of pressure tunnel with lining based on material nonlinear softening <i>L.M. Zhang &amp; Z.Q. Wang</i>	857
Modification of key parameters of longitudinal equivalent model for shield tunnel <i>W. Zhu, X.Q. Kou, X.C. Zhong &amp; Z.G. Huang</i>	863
Author Index	869

## Preface

Under the Chairmanship of Professor K. Fujita, the first symposium purposely addressing geotechnical issues related to underground construction in soft ground was held in 1994, prior to the 13th International Conference on Soil Mechanics and Geotechnical Engineering held in New Delhi. Following the success of the first symposium, Professor R. Mair succeeded the Chairmanship of TC28 and he initiated a series of three-day International Symposia on Geotechnical Aspects of Underground Construction in Soft Ground including technical site visits to underground construction projects. In total, four three-day International Symposia have been held very three years since 1996. These include the ones held in London, UK (1996), in Tokyo, Japan (1999), in Toulouse, France (2002) and in Amsterdam, the Netherlands (2005).

This volume includes a collection of four invited special lectures delivered by Dr A. Bezuijen (The Netherlands), Mr Huang Rong (China), Professor M.D. Bolton (UK) and Professor I.M. Lee (Korea). The titles of their lectures are "Processes around a TBM", "Overview of Shanghai Yangtze river tunnel project", "Supporting excavations in clay – from analysis to decision-making" and "Underground construction in decomposed residual soils", respectively.

In addition, this volume contains 112 papers grouped under six themes including (i) Analysis and numerical modelling of deep excavations; (ii) Construction method, ground treatment, and conditioning for tunnelling; (iii) Case histories; (iv) Safety issues, risk analysis, hazard management and control; (v) Physical and numerical modelling and (vi) Calculation and design methods, and predictive tools. Six general reports discussing and commenting papers grouped under the six themes were contributed orally during the Symposium by Professor Richard Finno, Professor Tadashi Hashimoto, Mr Alejo Sfriso, Dr C.T. Chin, Dr Richard Pang and Professor Richard Kastner, respectively. The written versions of their six general reports are also included in this volume.

Y.S. Li Chairman of the Symposium

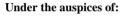
C.W.W. Ng, H.W. Huang and G.B. Liu *Vice-Chairmen of the Symposium and Editors* 

## Sponsors

### Organized by:



Tongji University





Technical Committee 28 of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE)

Supported by



China Civil Engineering Society



Chinese Society for Rock Mechanics and Engineering





香港科技大學 THE HONG KONG UNIVERSITY OF SCIENCE AND TECHNOLOGY Geotechnical Division, the Hong Kong Institution of Engineers

Hong Kong Geotechnical Society

Hong Kong University of Science and Technology







Science and Technology Commission of Shanghai Municipality

Shanghai Changjiang Tunnel & Bridge Development Co., Ltd.

Shanghai Society of Civil Engineering

Special lectures

## Processes around a TBM

A. Bezuijen & A.M. Talmon

Deltares and Delft University of Technology, Delft, The Netherlands

ABSTRACT: Processes that occur around a TBM during tunnelling have been investigated while tunnelling in saturated sand. The pore pressure in front of the TBM increases due to a lack of plastering during drilling. This has consequences for the stability of the tunnel face, or the soil in front of the tunnel. A bentonite flow is likely alongside the TBM from the tunnel face, and/or grout flow from the back. It seems that virtually no investigation has been made of this part of the TBM, but it is important to understand the volume loss that occurs around a tunnel. The lining is constructed behind the TBM and the tail void grout is applied. Pressures measured in the tail void grout will be discussed, as well as the consequences for loading on the soil and the lining. Most of the results described are based on field measurements performed at various tunnels constructed in the Netherlands.

#### 1 INTRODUCTION

Dutch experience of using TBM tunnelling is relatively recent. The first TBM tunnel was constructed in the Netherlands between 1997 and 1999 (the Second Heinenoord Tunnel). In the early 1990s, Dutch engineers were uncertain whether the soft saturated soil in the western parts of their country was suitable for TBM tunnelling. The decision was therefore taken to include a measurement programme in the first tunnelling projects. An overview of this programme and some results are presented by Bakker & Bezuijen (2008). In the programme, results from the measurements were predicted using existing calculation models. The measurement results were analysed at a later date, and discrepancies with the predictions were explained where possible.

An important part of the measurement and analysis programme was dictated by the processes that occur around the TBM. This paper deals with some of these processes. It does not cover all aspects of TBM tunnelling as this would not fit within the limits of this paper (see Bezuijen & van Lottum, 2006, for more information). The paper focuses on certain areas where ideas concerning the mechanisms involved have changed over the last decade, and where a better understanding is now apparent.

In order to structure this paper, we 'walk' along the TBM. We start with a process at the front of the TBM: the creation and stability of the tunnel face under the influence of excess pore pressures. The paper then discusses what happens next to the TBM. The last part of the paper deals with the tail void grout that is injected at the end of the TBM. The paper describes the current state of the art of these processes, and discusses how knowledge gained about these processes may influence the design of a TBM tunnel in soft soil.

#### 2 PORE PRESSURES IN FRONT OF A TBM

#### 2.1 Flow in coarse and fine granular material

During TBM tunnelling, it is essential that the tunnel face is stabilised by pressurised slurry (slurry shield) or muck (EPB shield). The pressure must be adapted to the ground pressure to stabilise the front. If pressure is too low, this will lead to an instable tunnel front resulting in collapse of the tunnel face. If pressure is too high, a blow-out will occur. Various calculation methods have been proposed to calculate the stability of the tunnel face. Most of these methods do not take the influence of pore water flow into account. It is assumed that the bentonite slurry or muck at the tunnel face creates a perfect seal that prevents water flow from the face into the soil. Experience with tunnels built in areas where the subsoil contains gravel has shown that the bentonite slurry can penetrate into the subsoil over more than 7 m (Steiner, 1996). Steiner advises that the sand and fines should be retained in the slurry (instead of removing them in the separation plant), and that sawdust should be used in the bentonite (Steiner, 2007). Anagnostou & Kovari (1994) propose a calculation method for such a situation. However, this method only takes the viscous behaviour of the slurry into account, and not the stiffening that occurs during standstill. The results of this calculation method may therefore lead to the prescription of bentonite

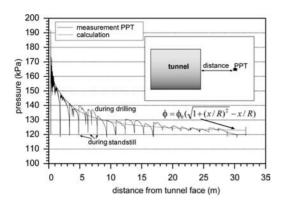


Figure 1. Measured excess pore pressure in front of a slurry shield and approximation.

with viscosity that is too high (Steiner, 2007). The state of the art for such a situation involving coarse granular material is still trial and error, but the trial can be performed in the laboratory to avoid errors in the field.

Usual tunnelling conditions in the Netherlands are a saturated sandy soil in medium-fine sand. In such soil conditions, the groundwater flow influences the plastering. There will be virtually no plastering of the tunnel face by the bentonite or the muck during drilling, because the groundwater in front of the TBM prevents water in the bentonite slurry or muck flowing into the soil. Plastering will only occur during standstill of the TBM process.

Figure 1 shows measured pore pressure in front of a slurry shield as a function of the distance from the TBM front. Plastering occurs during standstill, resulting in a pressure of 120 kPa (the hydrostatic pressure). Higher pore pressures were measured during drilling, because the TBM's cutter head removes a cake before it can form at the tunnel face.

Figure 2 shows the same phenomenon measured in front of an EPB shield. Here, only the pressure during drilling was recorded.

Bezuijen (2002) shows that the amount of excess pore pressure measured in the soil in front of the TBM (apart from pressure at the tunnel face) also depends on soil permeability, the quality of the bentonite or muck, and the drilling speed. Where EPB drilling takes place in sand with a low permeability ( $k = 10^{-5}$  m/s), the pore pressure measured in sand in front of the TBM is virtually equal to pressure in the mixing chamber. The pressure is lower in sand with higher permeability ( $k = 3 \cdot 10^{-4}$  m/s), because some plastering of the face occurs during drilling. Soil permeability also influences the foam properties. Muck in the mixing chamber will be dryer in sand with a higher permeability. Where the permeability of the sand is lower, the water content in the muck is nearly entirely

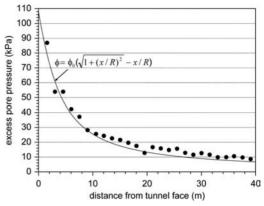


Figure 2. Measured excess pore pressure in front of an EPB shield (•) and approximation (Botlek Rail Tunnel, MQ1 South). Relatively impermeable subsoil.

determined by water in the soil and much less by the foam properties (also see Bezuijen, 2002).

Figure 1 and Figure 2 also show a theoretical curve (Bezuijen, 2002):

$$\phi = \phi_0 \left( \sqrt{1 + (x/R)^2} - x/R \right)$$
(1)

Where  $\phi_0$  is the piezometric head at the tunnel face,  $\phi$  the piezometric head at a distance *x* in front of the tunnel face, and *R* the radius of the tunnel. This relationship is valid for situations where the permeability of soil around the tunnel is constant. In the Netherlands, the sandy layers used for tunnelling are sometimes overlain with soft soil layers of peat and clay with a low permeability. In such a situation, the pressure distribution in the soil can be evaluated as a semi-confined aquifer. This is described by Broere (2001).

#### 2.2 Influence on stability

Bezuijen et al (2001) and Broere (2001) have shown that the groundwater flow in front of the TBM implies that a larger face pressure is necessary to achieve a stable front. According to Bezuijen et al (2001), the difference is approximately 20 kPa for a 10-m-diameter tunnel constructed in sand, where the top is situated 15 m below the ground surface.

Knowledge of this groundwater flow appeared essential during the Groene Hart Tunnel (GHT) project, not to prevent collapse of the tunnel face but to prevent a form of blow-out (Bezuijen et al, 2001). This tunnel enters a deep polder where the piezometric head in the sand layers underneath the soft soil layers is higher than the surface level (see Figure 3). As a result, the effective stresses beneath the soft soil layers are extremely small. The calculated excess pore pressure 23.7 24.0 24.3 km

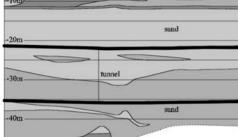


Figure 3. Geotechnical profile GHT tunnel in polder. Tunnel is drilled from right to left in this picture.

in the sand layer induced by the tunnelling process could cause 'floating' of the soft layers. The contractor made detailed numerical calculations (Aime et al, 2004). As a result of these calculations, a temporary sand dam was constructed at the point where the tunnel entered the polder. This dam delivered the necessary weight to prevent lifting of the soft soil layers due to excess pore pressure generated at the tunnel face during drilling.

#### 3 FLOW AROUND THE TBM

#### 3.1 Calculation model

Until recently, only limited attention has been given to pressure distribution and flow around the TBM shield. It was assumed that the soil was in contact with the TBM shield across the shield. During drilling of the Western Scheldt tunnel, however, it appeared that the TBM deformed at large depths and high water pressures (the tunnel is constructed up to 60 m below the water line). This could not be explained by the concept of a TBM shield in contact with the soil. Furthermore, tunnelling technology has advanced to a level where the ground loss due to tunnelling is less than the volume difference caused by tapering of the TBM. TBMs are usually tapered, with a slightly larger diameter at the head compared with the tail. This allows the TBM to manoeuvre and to drill with a certain curvature. Table 1 shows the volume difference due to tapering for different TBMs.

The volume losses measured during these projects varied, but negative volume losses were sometimes measured in all the projects (there was actually heave). It is clear that the measured volume loss can be less than the volume loss due to tapering. This leads to

Table 1. Percentage of tapering of the TBM in 3 tunnel projects in The Netherlands.

Tunnel project	Tapering %
Second Heinenoord	0.95
Botlek	0.77
Sophia	0.79

the idea (Bezuijen, 2007) that the soil is not in contact with the TBM all over the TBM. Overcutting at the tunnel face can lead to bentonite flow over the TBM shield from the face towards the tail. Grout pressure during grout injection is usually higher at the tail than the soil pressure. The soil is therefore pushed away from the TBM, and grout will flow from the tail over the shield. It is possible to describe flow on the shield, if it is assumed that both the bentonite and the grout are Bingham liquids, that the yield stress is dominant in the flow behaviour, and that there is linear elastic soil behaviour. A more or less conceptual model is developed, assuming a cylindrical symmetrical situation around the tunnel axis. Changes in the soil radius for such a situation can be described as (Verruijt, 1993):

$$\Delta \sigma = 2 \frac{\Delta r}{r} G \tag{2}$$

Where  $\Delta \sigma$  is the change in pressure,  $\Delta r$  the change in radius, *r* the radius of the tunnel and the grout, and *G* the shear modulus of the soil around the tunnel.

The flow around the TBM shield can be described as:

$$\Delta P = \alpha \frac{\Delta x}{s} \tau_{\gamma} \tag{3}$$

Where  $\Delta P$  is the change in pressure due to the flow,  $\Delta x$  a length increment along the TBM, *s* the gap width between the tunnel and the soil, and  $\tau_{\gamma}$  the yield stress of the grout around the TBM.  $\alpha$  is a coefficient indicating whether there is friction between the soil or bentonite and the grout ( $\alpha = 1$ ) only, or also between the TBM and the grout or bentonite ( $\alpha = 2$ ). Viscous forces are neglected in this formula. This is permissible due to the low flow velocities that can be expected.

With no grout or bentonite flow around the TBM, tapering will lead to an effective stress reduction proceeding from the TBM's face to the tail according to equation (2). The grout and bentonite flow will change this pressure distribution. In order to calculate the pressure distribution under flow, the flow direction of both the bentonite and the grout must be known. These flow directions can vary during the tunnelling process (Bezuijen, 2007). On average, however, the TBM

Table 2. Input parameters used in calculation with bentonite and overcutting.

Length TBM shield	5	m
Diameter	10	m
Diameter reduction	0.2	%
Overcutting	0.015	m
Asymmetric (1) or symmetric (2)	2	
Grain stress	150	kPa
Grout pressure	400	kPa
Pore pressure	200	kPa
Pressure on tunnel face	250	kPa
Shear modulus (G)	90	MPa
Yield stress grout	1.6	kPa
Yield stress bentonite	0.01	kPa

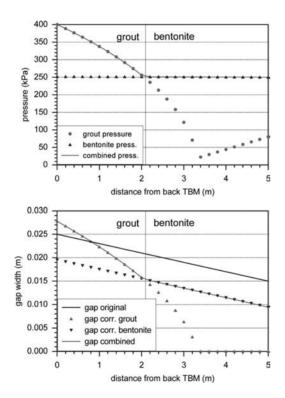


Figure 4. Pressures and gap width along a TBM. Grout pressures and bentonite pressures. Parameters see Table 2. Plots show pressures and gap width for the bentonite and grout pressure separately and the combined result.

advances and therefore the bentonite and grout front must also advance in the same direction to achieve a stable situation. This means that grout and bentonite only move with respect to the soil, and not with respect to the TBM. Therefore  $\alpha = 1$  for both the bentonite and the grout. The result of an example calculation using the parameters given in Table 2 is shown in Figure 4. The Figure shows that the gap width for a completely stiff soil mass would increase from 0.015 m at the front to 0.025 m at the tail of the TBM. If there were only grout pressures, the gap width would be 0.028 m at the tail of the TBM, due to the grout pressure that is larger than the total stress. However, the gap would close at 3.4 m from the tail. If the influence of the bentonite is included, there is still a gap width of 0.01 m at the tunnel face (5 m from the tail). The line through the triangles presents the gap width due to the combined effects of both the bentonite and the grout. The plot above presents the pressures in the same way.

#### 3.2 Consequences and status

The model shows that the volume loss is not determined by tapering of the TBM (as suggested for example by Kasper & Meschke, 2006), but is influenced by the pressure distribution of the bentonite and grout. With sufficient grout pressure, it is possible to have a 'negative' volume loss (the surface level rises after the TBM passes). It also explains that bentonite is sometimes found in the tail void, and grout is found in the pressure chamber. The first situation occurs when bentonite pressure is relatively high and grout pressure is low (we will see that it is quite difficult to control grout pressure, especially during ring building). The second situation occurs when grout pressures in the tail void are relatively high (which may occur during drilling).

Contrary, however, to the model described for the pore pressures in front of the TBM and the grout pressure, to be described in the following sections, the experimental evidence for this model is still limited. To our knowledge, pressure distribution around the TBM shield has never been measured. The shield was perforated during construction of the Western Scheldt tunnel but no grout was found between the shield and the soil (Thewes, 2007). The fact that no grout was found during this investigation may be caused by the fact that, in reality, the TBM will not be placed as symmetrically in the drilled hole as suggested in this simple model. The TBM must be in contact with the soil at some point to maintain mechanical equilibrium. There will be no grout around the shield at that location.

Guglielmetti (2007) rightfully argues that more research is needed in this field, because: 'The topic (flow of bentonite and grout around the TBM) is definitely one of the most important in the field of mechanised tunnelling, being the management of the void around the shield of a TBM as one of the major sources of concern for both designers and contractors involved in urban tunnelling projects'.

There is some evidence from the results of extensometer measurements carried out at the Sophia Rail Tunnel. The results of the extensioneters (shown in Figure 5) are presented in Figure 6 during passage of

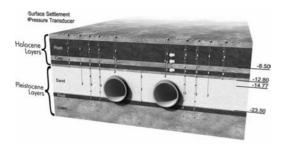


Figure 5. Sophia Rail Tunnel, soil stratification and location of extensometers at the measurement location (picture Arne Bezuijen).

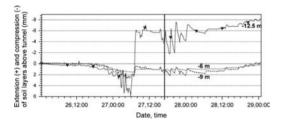


Figure 6. Extensioneter results. The vertical line shows when the tail of the TBM passes. Soil above the TBM is already compressed before the tail passes.

the TBM. The results show that there is initially some extension of the soil in front of the TBM due to the relatively low stresses at the tunnel face. However, the soil above the tunnel (see the extensometer at -12.5 m) is compressed several rings before the tail of the TBM passes (the vertical line) indicating heave, and there is therefore no settlement due to the tapering. When the TBM has passed, the extensometer at -12.5 m follows the course of the grout pressures measured around the lining. This will be discussed in more detail in the next section, and shows that a change in grout pressure indeed leads to a change in soil deformation.

We are currently working on the possibility of measuring pressures around the shield.

#### 4 TAIL VOID GROUTING

#### 4.1 Introduction tail void grouting

Coming at the end of the TBM, the tail void grouting process is important. The process determines the loading on the soil and on the lining.

The pressure distribution caused by tail void grouting has been studied during construction of the Sophia Rail Tunnel (Bezuijen et al, 2004) and the Groene Hart Tunnel. Here, we will describe the fundamental mechanisms using measurements from the Sophia Rail

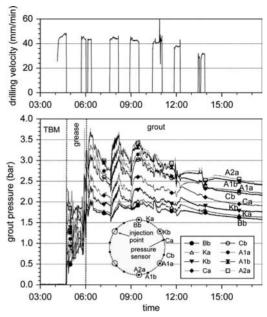


Figure 7. First tube Sophia Rail Tunnel: drilling velocity and measured grout pressures at the right side of the tunnel as a function of time.

Tunnel, as they have provided the most complete data set until now.

The study of grout pressures was initiated by earlier measurements performed at the Second Heinenoord Tunnel and the Botlek Rail Tunnel. These measurements did not match the generally accepted assumption at that time – at least in the Netherlands – that the vertical pressure gradient in liquid grout must be dictated by the density of the grout, and that the pressure distribution after hardening must reflect the  $K_0$  (the ratio between the horizontal and vertical soil pressure). In reality, the vertical pressure gradient was lower and the influence of  $K_0$  could not be detected.

#### 4.2 Measurements

The Sophia Rail Tunnel was constructed in sandy subsoil overlain with soft soil layers (see Figure 5). The water table is close to the surface. During construction of the Sophia Rail tunnel, two rings in the lining were each equipped with 14 pressure sensors. The pressures measured with one of these instrumented rings are shown in Figure 7.

These measurements are discussed in detail in Bezuijen et al (2004): we will only describe the main phenomena here. The upper plot in Figure 7 shows the drilling velocity, when drilling occurs, and when there was a standstill for ring building. It can be seen that an increase in pressure is measured as soon as the pressure gauges (built into the lining elements) moved from the grease into the grout. Pressure increases as long as drilling continues, and decreases when drilling stops during ring building.

#### 4.3 Grout pressures

The mechanism that leads to these pressure variations is explained in Bezuijen & Talmon (2003). Grout bleeding or consolidation of the grout leads to a volume loss of grout. Experiments showed that this volume loss is between 3% and 8%, depending on the type of grout (Bezuijen & Zon, 2007). This consolidation leads to stress reduction in the relatively stiff sand layer. This stress reduction is measured as a reduction of grout pressure. The effective stresses will ultimately be very small: the minimum stress that is necessary to keep the hole in the ground open. Leca & Dormieux (1990) calculate this for a tunnel opening in sand. They calculate that a cylindrical cavity in the ground remains open when effective stresses of only a few kPa are applied.

The consequence is that grout pressures around the lining will decrease to values that are only a few kPa above the pore water pressure. It is therefore clear that the original  $K_0$  can no longer be found in the grout pressures. The pressure decrease due to volume loss in the grout has changed the original stress state, and unloading of the soil leads to much lower stresses. Since the stresses in the sand around the tunnel decrease, the sand reaction will be the reaction of a very stiff material. Only a small volume decrease in the grout will lead to a large decrease in stresses. Calculation methods quite often still use the original in-situ stresses to calculate loading on the lining. For a tunnel in sand, this leads to a calculated loading that is much too high, as shown by Hashimoto et al (2004).

For slow hardening or non-hardening grouts, the strength increase in the grout is caused by grout bleeding or consolidation. It should be realised that this strength increase is only present when the tunnel is drilled through a permeable soil. When drilling takes place through less permeable soils such as clay, this consolidation will be much lower and the grout will be in liquid form over a greater part of the tunnel's length. This has consequences for loading on the lining, as we will discuss later.

#### 4.4 Pressure gradients

The vertical pressure gradient over the tunnel lining is important when calculating the longitudinal loading on the lining. The vertical pressure gradient that was measured during construction of the first tunnel tube of the Sophia Rail Tunnel is shown in Figure 8. The pressure gradient starts at nearly 20 kPa/m and decreases to values under the pore water pressure gradient of

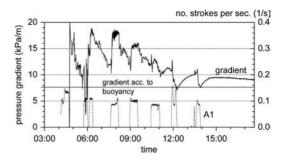


Figure 8. First tube Sophia Rail Tunnel: pressure gradient over the tunnel lining at one location, and pump activity for one of the injection points (A1) as a function of time.

9.81 kPa/m. The tail void grout used for this tunnel had a density of  $2190 \text{ kg/m}^3$ . If the vertical pressure were to increase with depth in accordance with this density, the pressure gradient should be 21.5 kPa/m. Results showed that the measured vertical density is always lower. This is because the grout is a Bingham liquid, with a viscosity and a yield stress. The grout has to flow downwards if more grout is injected in the upper half of the tunnel. This downward flow needs a driving force to overcome the yield stress, and the pressure gradient will therefore be less than the gradient that is calculated from the density. Talmon et al (2001) developed a numerical program to calculate the pressure distribution in the tail void due to injection. We only describe some of the consequences here. If the viscosity is not taken into consideration, the maximum pressure gradient (dP/dz) that can be expected is:

$$\frac{dP}{dz} = \rho_{gr}g - 2\frac{\tau_{\gamma}}{s} \tag{4}$$

Where  $\rho_{gr}$  is the density of the grout, *g* the acceleration of gravity,  $\tau_{\gamma}$  the yield strength of the grout, and *s* the width of the tail void gap between the soil and the lining. If the yield stress in the grout is low, the vertical pressure gradient is determined by the grout density (21.5 kPa/m for the Sophia Rail Tunnel, slightly higher than the maximum value measured in Figure 8). Consolidation or hardening of the grout leads to a higher yield stress, and thus to a lower gradient.

A complicating factor is that the maximum shear stress that can be developed is a vector. If the maximum shear stress is developed in one direction, there will be no shear stress perpendicular to that direction. When drilling starts for a new ring and the grout pumps are activated, the elastic soil reaction will lead to an increase of the tail void and grout will therefore flow backwards from the TBM. Ring shear stresses barely develop in this situation, and the vertical gradients therefore increase during drilling. They decrease again when drilling stops (Figure 8).

Further from the TBM, the vertical gradients decrease and become equal to the gradient according to the buoyancy forces. This has to be the case, because the total force on the lining far away from the TBM must be zero. The vertical pressure gradient therefore compensates for the weight of the lining. As a result, the gradient becomes lower than the gradient in the pore water. This is because the average density of the lining is lower than the density of pore water. One remarkable result is that the vertical pressure gradient at some distance from the TBM (at 12:00 in Figure 8, 5 rings behind the TBM) decreases during drilling. The flow no longer has any influence at this point, but drilling and grout injection lead to higher gradients in the first part of the lining and therefore to higher buoyancy forces. The first rings have the tendency to move upwards, which must be compensated by the TBM and the rings further away. This partly compensates for the weight of the rings further from the TBM, so that the effective weight of these rings and also the vertical gradient is less.

#### 5 INFLUENCE ON PORE WATER PRESSURES

Section 2.1 describes how no plastering occurs at the front when drilling takes place in fine to mediumfine saturated sand, because the bentonite filter cake is destroyed by the cutting wheel before it is able to form. As a result, water flows from the tunnel face into the soil. Section 4.3 describes how consolidation of the grout also leads to a water flow from the tunnel lining into the soil, because water expelled from the grout will flow into the surrounding soil. A grout cake will form however, because the consolidated grout is no longer disturbed. It is therefore reasonable to assume that examination of the variation in pore pressure in soil next to a tunnel under construction will show pore pressures that are dominated by pressures existing at the tunnel face. This theory was tested at the Groene Hart Tunnel. Pore pressure transducers (PTTs) were installed as close as 0.75 m from the tunnel lining. The PTTs were placed in one plane, with the grout pressure gauges on Ring 2117 of the tunnel (see Figure 9).

Figure 10 shows the measurement results. The grout pressure gauges on Ring 2117 give no signal before they are in the grout. The PPTs show a slight increase during drilling due to the excess pore pressure generated at the tunnel face. As drilling stops, the pore pressure reduces to the hydrostatic pressure. The various construction cycles can be seen. There is a sharp increase in grout pressure when Ring 2117 leaves the TBM, followed by a decrease due to consolidation. It is remarkable however that this has virtually no influence on the measured pore pressures at less than a metre from these gauges. This result is confirmed by numerical calculations. The quantity of water expelled

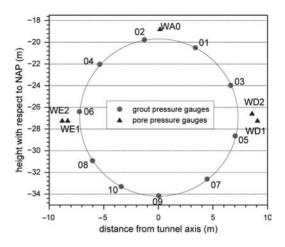


Figure 9. Position of pore pressure gauges and grout pressure gauges at ring 2117 of the GHT.

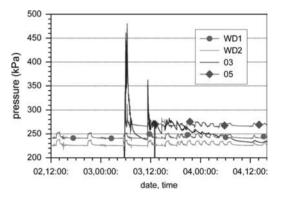


Figure 10. Pore pressures and grout pressures measured at GHT (also see text).

from the grout is far less than the water flow from the tunnel face. The latter dominates the pore pressures.

The measurements show another remarkable feature. Grout pressure gauge 05 follows the water pressure after 3.20:00, but this is not the case for gauge 03. This may indicate that there is no 'sealing' grout layer around gauge 05, so that it is possible to measure the pore water pressure.

#### 6 LOADING ON TUNNEL LINING

We have seen in Section 4.4 that vertical pressure gradients exist in the zone where the grout is not yet consolidated or hardened which are higher than corresponds to the weight of the lining. Measurements at the Sophia Rail Tunnel showed that the gradient decreases more or less linear with the distance (see Figure 11). As a result, that part of the lining is pressed upwards by

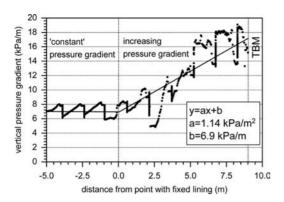


Figure 11. Example of gradient in the grout pressure as a function from the distance (0 on the X-axis represents the point where the lining is more or less fixed. The TBM is at 9 m). Results from Sophia Rail Tunnel (Bezuijen et al, 2004).

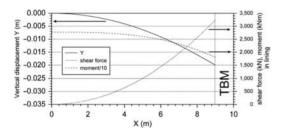


Figure 12. Calculated shear force and moment in the lining, and displacement where the grout has not yet hardened. Calculated moments are divided by 10.

the buoyancy forces. It is necessary to mobilise shear forces from the TBM to achieve a stable tunnel lining. This will lead to moments in the lining.

Bezuijen & Talmon (2005) have shown that the moments in the liquid grout zone increase backwards from the TBM (see Figure 12). A positive moment means here that the force on the lower part of the tube is higher than on the upper part. At the TBM, this moment is created by the TBM itself. This is because face pressure is higher at the bottom due to larger soil stresses.

At the Groene Hart Tunnel the bending moment in the lining was measured for a large distance behind the TBM using strain gauges installed in the lining segments. There is an increase in the moment for a few rings, in accordance with the calculations previously mentioned. There is subsequently a decrease, with the moments becoming negative at a greater distance from the tunnel. Bogaards & Bakker (1999) and Hoefsloot (2008) argue that the remaining bending moment is a result of the staged construction of the tunnel. They developed a calculation model to take into account the different stages in construction.

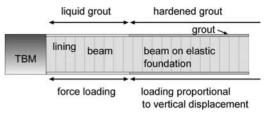


Figure 13. Boundary condition for beam calculation.

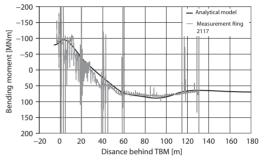


Figure 14. Bending moment ring 2117, measurement and calculation. Groene Hart Tunnel (Hoefsloot, 2008).

However, Talmon (2007) has shown that such a 'staged' calculation is not necessary to find the same results.

According to Talmon, the negative moment appears at some distance from the TBM because the reaction force to compensate the buoyancy in the fluid grout zone is situated further from the TBM than the buoyancy force itself. The tunnel lining is 'pushed' a bit higher in the soil than in the equilibrium situation far behind the TBM.

Hoefsloot and Talmon both model the tunnel lining as a beam on an elastic foundation, except for lining elements inside the TBM and lining elements in the liquid grout zone, see Figure 13. The exact boundary conditions and the transition between liquid and solid grout are still the subject of debate.

Although example calculations have been presented that show good correlation with measurements (see Figure 14), there are still uncertainties with this type of calculation that need further research:

- An important input parameter is the moment and shear force that is transferred from the TBM to the lining. While the moment can be derived from the jack forces, the shear force is not determined.
- With generally-accepted parameters for the lining stiffness and the soil's elastic parameters, the calculated movement of the lining is much smaller than the measured movement.
- The grout pressures are only measured when the grout is more or less in the liquid phase. This results

Table 3. Specification of grout mixtures used in fracture tests (WCR = water-cement ratio). Coclay D90 Ca activated bentonite is used.

Mixture	WCR	Bentonite %	k (m/s)
1	1	7	$5.10^{-8}$ $6.10^{-0}$
2	10	7	$6.10^{-0}$

in loading on the tunnel lining as shown in Figure 11. However, loading on the lining in situations where the grout has hardened is less known. This is because the instruments used were not suitable to measure pressures when grout has hardened.

Conclusions that can be drawn from this type of calculations are:

- The length of the liquid grout zone and the density of the grout are extremely important parameters when calculating bending moments in the lining. If this length is too long, loading will be too high and tunnelling will not be possible (also see Bezuijen & Talmon, 2005).
- The shear force that is exerted on the lining by the TBM is an important parameter. It is therefore worthwhile to measure this shear force.

#### 7 COMPENSATION GROUTING

Grout consolidation also appeared to be important when describing compensation grouting. Experiments (Gafar et al, 2008) showed that the fracturing behaviour in compensation grouting depends on the specification of the grout. If more cement is added, the permeability of the grout is higher and there will be more consolidation and leak-off during grout injection. Gafar et al describe how this influences the fracturing behaviour. Recent tests carried out as part of the research project on compensation grouting present proof of the suggested grout consolidation mechanism. At Delft University, the density of grout bodies made in two compensation grouting experiments was analysed in a CT-scan. Such a CT-scan can be used to determine the density of the material tested. The grout mixtures used in the experiments are shown in Table 3.

The results of the CT-scans are shown in Figure 15 and Figure 16. The results of the first grout mixture clearly show an increase in density at the boundary of the grout body. Grout at the boundary of the sample is consolidated. The grout body made with the second mixture has a more constant density across the fracture (the middle section). In the second experiment, the CT-scan was performed while the grout body was still in the sand. The more homogeneous density of

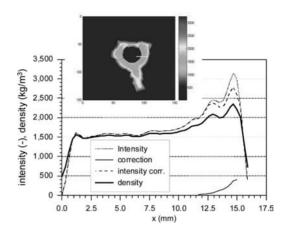


Figure 15. Density measured with a CT-scan. Raw data (inset) and density. Correction for beam hardening effect and calculated value of the density of the grout along the line shown in inset. Mixture 1 in Table 3.

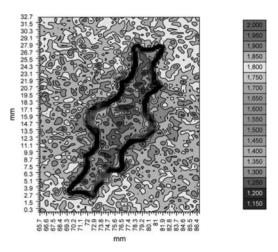


Figure 16. Grout density in a fracture measured with a CT-scan. Mixture 2 in Table 3.

the grout body in the second test is understandable if the permeabilities of the grout are considered. The lower permeability of the second grout sample results in much less grout consolidation within the limited injection time. The grout density in the fracture therefore does not increase at the boundary of the grout as is the case for mixture 1.

The permeabilities were determined using the procedure suggested by McKinley and Bolton (1999), a form of oedometer test with drainage on one side. This procedure can also be used to test the consolidation properties of tail void grout. However, the thickness of the grout layer in the test should be identical to that in the field. This is to avoid scaling effects that occur because hardening of the grout is independent of the sample size (Bezuijen & Zon, 2007).

#### 8 DISCUSSION

The research described above has increased understanding of the processes that occur around a TBM during tunnelling. This has already had consequences for practical aspects of tunnelling. Examples are the excess pore pressures in front of the TBM: extra sand was added locally above the planned tunnel trajectory of the Groene Hart Tunnel to prevent a blow-out. and the grout was changed in a tunnel project in London where it appeared that the liquid zone of traditional grout for a tunnel drilled in clay with no possibility of consolidation was too long to achieve the desired drilling speed. However, the authors believe that the results can make an even greater contribution to improving shield tunnelling. Knowledge about the influence of excess pore pressures on face stability can improve definition of the pressure window at the tunnel face, so preventing a blow-out due to excessively high pressures and instability caused by pressures that are too low. In combination with research on EPB tunnelling in clay (Merrit & Mair, 2006), foam research for EPB tunnelling in sand can lead to better control of the EPB process. It has already been discussed how flow around the TBM is important for TBM design, and that more experimental evidence is needed. Research into grouting can lead to smaller settlement troughs and optimisation of loading on the lining. This last aspect may lead to cheaper lining construction.

The results must be discussed with tunnel builders and contractors if improvements to the shield tunnelling process are to be achieved. Discussion about certain aspects has already started, but we hope that this paper will stimulate the involvement of more parties.

#### 9 CONCLUSIONS

To understand the processes that are important when tunnelling with a TBM, the flow processes around a TBM must be considered: groundwater flow at the tunnel face, bentonite and grout flow around the TBM, and grout flow and grout consolidation around the tunnel lining. The research described in this paper has brought about progress with regard to these flow processes during tunnelling in soft ground:

- The groundwater flow at the tunnel face is described.
- The muck in the mixing chamber is described as a function of drilling speed and permeability.

- A conceptual model for the flow of bentonite and grout has been developed. Although this model must still be verified using the results of measurements, it shows some promising results.
- Considerable information has been obtained about the grouting process and the resultant lining loading.

Although not unusual, it is interesting to see that this research also raises new questions: what is the exact position of the TBM during the tunnelling process, what is the interaction between the TBM and the lining, are the predicted pressures around the TBM correct, and what are the consequences for our design methods? Even in a relatively simple beam calculation for calculating loading on the lining in a longitudinal direction it appears that uncertainties in the boundary conditions determine the outcome of the calculation. As long as these uncertainties remain, more sophisticated numerical calculations will present the same uncertainties.

#### ACKNOWLEDGEMENTS

The research described in this paper was sponsored by COB, the Dutch Centre for Underground Construction, and Delft Cluster. We would like to thank these organisations for giving us the opportunity to perform this research. We also wish to thank the project organisations of the different tunnels for giving permission to use tunnelling data in our research. And last but not least, we would like to thank our fellow members in the COB committees for their stimulating discussions on the various subjects.

#### REFERENCES

- Aime, R., Aristaghes, P., Autuori, P. & Minec, S. 2004. 15 m Diameter Tunneling under Netherlands Polders. Proc. Underground Space for Sustainable Urban Development (ITA Singapore), Elsevier.
- Anagnostou, G. & Kovári, K. 1994. The face stability of Slurry-shield-driven Tunnels. *Tunelling and Underground Space Technology* 9(2): 165–174.
- Bakker, K.J. & Bezuijen, A. 2008. 10 years of bored tunnels in the Netherlands. Proceeding 6th Int. Symposium on Underground Construction in soft Ground, Shanghai.
- Bezuijen, A. 2002. The influence of soil permeability on the properties of a foam mixture in a TBM. 3rd. Int. Symp. on Geotech. Aspects of Underground Construction in Soft Ground, IS-Toulouse.
- Bezuijen, A. 2007. Bentonite and grout flow around a TBM. *Proc. ITA 2007, Prague.*
- Bezuijen, A., Pruiksma, J.P. & Meerten, H.H. van. 2001. Pore pressures in front of tunnel, measurements, calculations and consequences for stability of tunnel face. *Proc. Int. Symp. on Modern Tunneling Science and Techn. Kyoto.*

- Bezuijen, A. & Talmon, A.M. 2003. Grout the foundation of a bored tunnel. Proc ICOF 2003 Dundee.
- Bezuijen, A., Talmon, A.M., Kaalberg, F.J. & Plugge, R. 2004. Field measurements of grout pressures during tunneling of the Sophia Rail tunnel. *Soils and Foundations* 44(1): 41–50.
- Bezuijen, A. & van Lottum, H. (eds). 2006. Tunnelling A Decade of Progress. GeoDelft 1995-2005, Taylor and Francis/Balkema, Leiden, ISBN 0 415 39113 4.
- Bezuijen, A. & Zon, W. van der. 2007. Volume changes in grout used to fill up the tail void. Proc. ITA 2007, Prague.
- Bogaards, P.J. & Bakker, K.J. 1999, Longitudinal bending moments in the tube of a bored tunnel. *Numerical Models* in Geomechanics Proc. NUMOG VII: 317–321.
- Broere, W. 2001. Tunnel Face Stability & New CPT Applications. Ph.D. Thesis, Delft University of Technology, Delft University Press.
- Gafar, K., Soga, K., Bezuijen, A., Sanders, M.P.M. & van Tol, A.F. 2008. Fracturing of sand in compensation grouting. Proceeding 6th Int. Symposium on Underground Construction in soft Ground, Shanghai.
- Guglielmetti, V. 2007. Tunnels and Tunnelling International, October, P32.
- Hashimoto, T., Brinkman, J., Konda, T., Kano, Y. & Feddema, A. 2004. Simultaneous backfill grouting, pressure development in construction phase and in the long term. *Proc. ITA Singapore*.
- Hoefsloot, F.J.M. 2008. Analytical solution longitudinal behaviour Tunnel lining. Proceeding 6th Int. Symposium on Underground Construction in soft Ground, Shanghai.

- Kasper, T. & Meschke, G. 2006. On the influence of face pressure, grouting pressure and TBM design in soft ground tunnelling. *Tunn. and Undergr. Space Techn.* 21: 160–171.
- Leca, E. & Dormieux, L. 1990. Upper and lower bound solutions for the face stability of shallow circular tunnels in frictional material. *Géotechnique* 43: 5–19 (in French).
- Merritt, A.S. & Mair, R.J. 2006. Mechanics of tunnelling machine screw conveyors: model tests. *Geotechnique* 56(9): 605–615.
- McKinley, J.D. & Bolton, M.D. 1999. A geotechnical description of fresh cement grout – Filtration and consolidation behaviour. *Magazine of Concrete Research* 51(5): 295–307.
- Steiner, W. 1996. Slurry penetration into coarse grained soils and settlements from a large slurry shield tunnel. *Proc. Geotech. Aspects of Underground Construction in Soft Ground*, London, Mair and Taylor (eds). Balkema, Rotterdam, ISBN 9054108568: 329–333.
- Steiner, W. 2007. Private communication.
- Talmon, A.M. 2007. Notes on analytical beam model. Delft Hydraulics report Z3934/Z4145.
- Talmon, A.M., Aanen, L. Bezuijen, A & Zon, W.H. van der. 2001. Grout pressures around a tunnel lining *Proc. Int. Symp. on Modern Tunneling Science and Techn. Kyoto.*
- Thewes, M. 2007. Private communication.
- Verruijt, A. 1993. Soil Dynamics. Delft University of Technology, b28.

## Supporting excavations in clay – from analysis to decision-making

M.D. Bolton & S.Y. Lam University of Cambridge, UK

A.S. Osman Durham University, UK

ABSTRACT: Finite Element Analysis (FEA) is used to calibrate a decision-making tool based on an extension of the Mobilized Strength Design (MSD) method which permits the designer an extremely simple method of predicting ground displacements during construction. This newly extended MSD approach accommodates a number of issues which are important in underground construction between in-situ walls, including: alternative base heave mechanisms suitable either for wide excavations in relatively shallow soft clay strata, or narrow excavations in relatively deep soft strata; the influence of support system stiffness in relation to the sequence of propping of the wall; and the capability of dealing with stratified ground. These developments should make it possible for a design engineer to take informed decisions on the relationship between prop spacing and ground movements, or the influence of wall stiffness, or on the need for and influence of a jet-grouted base slab, for example, without having to conduct project-specific FEA.

#### 1 INTRODUCTION

The Mobilizable Strength Design (MSD) method has developed following various advances in the use of plastic deformation mechanisms to predict ground displacements: (Milligan and Bransby, 1975; Bolton and Powrie, 1988; Bolton et al. 1989, 1990a, 1990b). MSD is a general, unified design methodology, which aims to satisfy both safety and serviceability requirements in a single calculation procedure, contrasting with conventional design methodology which treats stability problems and serviceability problems separately. In the MSD method, actual stress-strain data is used to select a design strength that limits ground deformations, and this is used in plastic soil analyses that satisfy equilibrium conditions without the use of empirical safety factors.

Simple plastic mechanisms are used to represent the working state of the geotechnical system. The mechanisms represent both the equilibrium and deformation of the various soil bodies, especially at their junction with the superstructure. Then, raw stress-strain data from soil tests on undisturbed samples, taken from representative locations, are used directly to link stresses and strains under working conditions. Constitutive laws and soil parameters are unnecessary.

The MSD approach has been successfully implemented for shallow foundations (Osman and Bolton, 2005), cantilever retaining walls (Osman and Bolton, 2005), tunneling-induced ground displacements (Osman et al. 2006) and also the sequential construction of braced excavations which induce wall displacements and ground deformations (Osman and Bolton, 2006).

Consider the imposition of certain actions on a soil body, due to construction activities such as stress relief accompanying excavation, or to loads applied in service. The MSD method permits the engineer to use simple hand calculations to estimate the consequential ground displacements accounting for non-linear soil behavior obtained from a single well-chosen test of the undisturbed soil.

The MSD approach firstly requires the engineer to represent the working states of the geotechnical system by a generic mechanism which conveys the kinematics (i.e. the pattern of displacements) of the soil due to the proposed actions. Analysis of the deformation mechanism leads to a compatibility relationship between the average strain mobilized in the soil and the boundary displacements.

The average shear strength mobilized in the soil due to the imposed actions is then calculated, either from an independent equilibrium analysis using a permissible stress field (equivalent to a lower bound plastic analysis), or from an equation balancing work and energy for the chosen mechanism (equivalent to an upper bound plastic analysis).

The location of one or more representative soil elements is then selected, basing this judgment on the soil profile in relation to the location and shape of the selected mechanism. The centroid of the mechanism can serve as a default location if a single location is to be employed. Stress-strain relationships are then obtained from appropriate laboratory tests on undisturbed soil samples taken from the selected locations and carried out with precise strain measurements. Equivalent in-situ tests such as self-boring pressuremeter tests can alternatively be carried out. The mode of deformation in the soil tests should correspond as closely as possible to the mode of shearing in the MSD mechanism. Otherwise, anisotropy should somehow be allowed for.

Finally, the mobilized shear strength required for equilibrium under working loads is set against the representative shear stress-strain curve in order to obtain the mobilized soil strain, and thereby the boundary displacements of the simplified MSD mechanism.

#### 2 MSD FOR DEEP EXCAVATION PROBLEM

Osman and Bolton (2006) showed for an in-situ wall supporting a deep excavation that the total deformation could be approximated as the sum of the cantilever movement prior to propping, and the subsequent bulging movement that accretes incrementally with every sequence of propping and excavation.

A method for estimating the cantilever movement had been suggested earlier in Osman and Bolton (2004). It begins by considering the lateral earth pressure distribution for a smooth, rigid, cantilever wall rotating about a point some way above its toe, in undrained conditions. A simple mobilized strength ratio is introduced to characterize the average degree of mobilization of undrained shear strength throughout the soil. By using horizontal force and moment equilibrium equations, the two unknowns - the position of the pivot point and the mobilized strength ratio - are obtained. Then, a mobilized strain value is read off from the shear stress-strain curve of a soil element appropriate to the representative depth of the mechanism at the mid-depth of the wall. Simple kinematics for a cantilever wall rotating about its base suggests that the shear strain mobilized in the adjacent soil is double the angle of wall rotation. Accordingly, for the initial cantilever phase, the wall rotation is estimated as one half of the shear strain required to induce the degree of mobilization of shear strength necessary to hold the wall in equilibrium. Osman and Bolton (2004) used FEA to show that correction factors up to about 2.0 could be applied to the MSD estimates of the wall crest displacement, depending on a variety of non-dimensional groups of parameters ignored in the simple MSD theory, such as wall flexibility and initial earth pressure coefficient prior to excavation.

A typical increment of bulging, on the other hand, was calculated in Osman and Bolton (2006) by considering an admissible plastic mechanism for base heave. In this case, the mobilized shear strength was deduced from the kinematically admissible mechanism itself, using virtual work principles. The energy dissipated by shearing was said to balance the virtual loss of potential energy due to the simultaneous formation of a subsidence trough on the retained soil surface and a matching volume of heave inside the excavation. The mobilized strength ratio could then be calculated, and the mobilized shear strain read off from the stress-strain curve of a representative element, as before. The deformation is estimated using the relationship between the boundary displacements and the average mobilized shear strain, in accordance with the original mechanism.

The MSD solutions of Osman and Bolton (2006) compared quite well with some numerical simulations using the realistic non-linear MIT-E3 model, and various case studies that provided field data. However, these initial solutions are capable of improvement in three ways that will contribute to their applicability in engineering practice.

- 1 The original mechanism assumed a relatively wide excavation, whereas cut-and-cover tunnel and subway constructions are likely to be much deeper than their width. The MSD mechanism therefore needs to be adapted for the case in which the plastic deformation fields for the side walls interfere with each other beneath the excavation.
- 2 The structural strain energy of the support system can be incorporated. This could be significant when the soil is weak, and when measures are taken to limit base heave in the excavation, such as by base grouting between the supporting walls. In this case, the reduction of lateral earth pressure due to ground deformation may be relatively small, and it is principally the stiffness of the structural system itself that limits external ground displacements.
- 3 Progressively incorporating elastic strain energy requires the calculation procedure to be fully incremental, whereas Osman and Bolton (2006) had been able to use total energy flows to calculate the results of each stage of excavation separately. A fully incremental solution, admitting ground layering, will permit the accumulation of different mobilized shear strengths, and shear strains, at different depths in the ground, thereby improving accuracy.

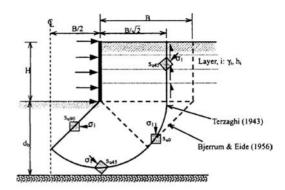
It is the aim of this paper to introduce an enhanced MSD solution that includes these three features. This is then compared with existing FEA of braced excavations which featured a range of geometries and stiffnesses. It will be suggested that MSD provides the ideal means of harvesting FEA simulations for use in design and decision-making.

#### 3 PLASTIC FAILURE MECHANISMS

Limit equilibrium methods are routinely used in stability calculations for soft clay which is idealised, unrealistically, as rigid-plastic. Slip surfaces are selected as the assumed focus of all plastic deformations. Failure mechanisms should be kinematically admissible, meaning that unwanted gaps and overlaps should not be produced. Furthermore, in the case of undrained shearing of clays, a constant-volume condition should be respected at every point. A consequence is that undrained plane-strain failure mechanisms must comprise only slip planes and slip circles. The soil on such failure surfaces is taken to mobilize its undrained shear strength divided by a safety factor, to maintain the mechanism in limiting equilibrium under the action of gravity, and any other applied loads. Calculated in this way, the safety factor literally offers an estimate of the factor by which the strength of the soil would have to drop before the soil construction would collapse. Such estimates might err either on the high side or the low side, depending on the particular assumptions that were made.

In the case of base heave in braced excavations, plastic solutions were derived from slip-line fields based on the method of characteristics. Such solutions comprise both slip surfaces, as before, and plastic fans which distribute plastic strains over a finite zone in the shape of a sector of a circle. Notwithstanding these zones of finite strain, the additional presence of slip surfaces still restricts the application of these solutions to the prediction of failure. Furthermore, no such solution can be regarded automatically as an accurate predictor of failure, notwithstanding their apparent sophistication. All that can be said is that they will lead to an unsafe estimate of stability. Their use in practice can only be justified following backanalysis of actual failures, whether in the field or the laboratory.

Two typical failure mechanisms as suggested by Terzaghi (1943) and Bjerrum and Eide (1956) are shown in Figure 1. They have each been widely used for the design of multi-propped excavations. Terzaghi (1943) suggested a mechanism consisting of a soil column outside the excavation which creates a bearing capacity failure. The failure is resisted by the weight of a corresponding soil column inside the excavation and also by adhesion acting along the vertical edges of the mechanism. Bjerrum and Eide (1956) assumed that the base of the excavation could be treated as a negatively loaded perfectly smooth footing. The bearing capacity factors proposed by Skempton (1951) are used directly in the stability calculations and are taken as stability numbers,  $N = \gamma H/c_u$ . Eide et al. (1972) modified this approach to account for the increase in basal stability owing to mobilized shear strength along the embedded length of the rigid wall.



 a) Without wall embedment (principal stress directions after Clough & Hansen, 1981)

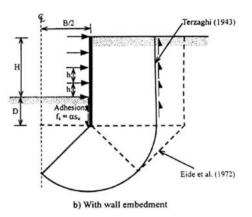


Figure 1. Conventional basal stability mechanism and notation (after Ukritchon et al. 2003).

O'Rourke (1993) further modified the basal stability calculations of Bjerrum and Eide (1956) to include flexure of the wall below the excavation level. It was assumed that the embedded depth of the wall does not change the geometry of the basal failure mechanism. However, an increase in stability was anticipated due to the elastic strain energy stored in flexure. This gave stability numbers that were functions of the yield moment and assumed boundary conditions at the base of the wall.

Ukritchon et al. (2003) used numerical limit analysis to calculate the stability of braced excavations. Upper and lower bound formulations are presented based on Sloan and Kleeman (1995) and Sloan (1988), respectively. The technique calculates upper bound and lower bound estimates of collapse loads numerically, by linear programming, while spatial discretization and interpolation of the field variables are calculated using the finite element method. No failure

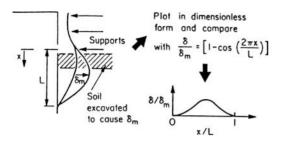


Figure 2. Incremental displacements in braced excavation (after O'Rourke, 1993).

mechanism need be assumed and failure both of the soil and the wall are taken care of. However, both soil and wall are again assumed to be rigid perfectly plastic so the failure mechanism includes a plastic hinge at the lowest level of support.

All these collapse limit analyses provide useful guidance on the possible geometry of plastic deformation mechanisms for service conditions. But the key requirement for MSD mechanisms is that displacement discontinuities (slip surfaces) must be avoided entirely. In that way, small but finite ground displacements are associated at every internal point with small but finite strains.

#### 4 WALL DEFORMATIONS

Consider now the *deformations* of a multi-propped wall supporting a deep excavation in soft, undrained clay. At each stage of excavation the incremental displacement profile (Figure 2) of the ground and the wall below the lowest prop can be assumed to be a cosine function (O'Rourke, 1993) as follows:

$$\delta w = \frac{\delta w_{\text{max}}}{2} \left( 1 - \cos(\frac{2\pi y}{l}) \right) \tag{1}$$

Here  $\delta w$  is the incremental wall displacement at any distance y below the lowest support,  $\delta w_{max}$  is its maximum value, and *l* is the wavelength of the deformation, regarded as proportional to the length *s* of the wall below the lowest level of current support:

$$l = \alpha s$$
 (2)

O'Rourke (1993) defined the wavelength of the deformation as the distance from the lowest support level to the fixed base of the wall. Osman and Bolton (2006) suggested a definition for the wavelength of the deformation based on wall end fixity. For walls embedded into a stiff layer beneath the soft clay, such that the wall tip is fully fixed in position and direction, the wavelength was set equal to the wall length ( $\alpha = 1$ ). For short walls embedded in deep soft clay, the maximum wall displacement occurs at the tip of the wall so the wavelength was taken as twice the projecting wall length ( $\alpha = 2$ ). Intermediate cases might be described as restrained-end walls ( $1 < \alpha < 2$ ).

However, these definitions applied only to very wide excavations. When a narrow excavation is considered, the wavelength will be limited by the width of the excavation. In addition, in the case of the partially restrained wall, the depth of a relatively stiff soil stratum may also limit the depth of the deformation pattern.

#### 5 GEO-STRUCTURAL MECHANISMS

An incremental plastic deformation mechanism conforming to Equation 1 was proposed by Osman and Bolton (2006) for an infinitely wide multi-propped excavation in clay. In this mechanism, the wall is assumed to be fixed incrementally in position and direction at the lowest prop, implying that the wall has sufficient strength to avoid the formation of a plastic hinge. The wall and soil are deforming compatibly and the soil deformation also follows the cosine function of Equation 1. The dimensions of this mechanism depend on the wavelength *l*.

Figure 3(a) shows the complete displacement field for the mechanism proposed by Osman and Bolton (2006). The solution includes four zones of distributed shear which consist of a column of soil adjoining the excavation above the level of the lowest prop, a circular fan zone centred at the lowest prop, another circular fan zone with its apex at the junction of the wall and the excavation surface and a 45 degree isosceles wedge below the excavation surface. It is required that the soil shears compatibly and continuously with no relative sliding at the boundaries of each zone. The dotted lines with arrows show the direction of the flow. Along each of these lines the displacement is constant and is given by the cosine function of Equation 1. It is assumed that the zone outside the deformation zones is rigid. This mechanism is simple and neat, but it only applies to very wide excavations. In the case of a narrow excavation, the width of the triangular wedge could be bigger than the actual width of the excavation. In view of this, a new mechanism for narrow excavations is proposed in Figure 3(b). The mechanism in the passive zone (zone EFHI) is replaced. The new mechanism meets the condition for undrained shearing, which means that the volumetric strain remains zero throughout the zone.

The following solution approach is an extension of Osman and Bolton (2006). In their original solution, soils are assumed to be homogenous. The average shear strain increment in each zone is calculated by taking the derivative of the prescribed displacement equation. Then, the undrained shear strength ( $c_{u,mob}$ ) mobilized at any location for any excavation heightwas expressed using a single mobilization ratio  $\beta$ ( $\beta = c_{u,mob}/c_u$ ) to factor the strength profile. With the use of the virtual work principle, the plastic work done by shearing of the soil was equated to the virtual change of gravitational potential energy of the soil. A  $\beta$  factor can then be found so that a corresponding mobilized shear strain can be read off from the chosen stress-strain curve. The incremental displacement can then be calculated by the correlation between the average shear strain increment and the incremental wall displacement.

This approach offered a straightforward way to estimate the bulging displacement of the retaining wall. However, the approach requires refinement in order to include some additional features that may be significant in deep excavations. Firstly, the approach did not consider the elastic strain energy stored in the support system. Secondly, it is common to find a non-uniform soil stratum with undrained shear strength varying irregularly with depth. Furthermore, the geometry of the deformation mechanism changes as the construction proceeds, so the representation of mobilization of shear strength through the whole depth, using a single mobilization ratio, is only a rough approximation. In reality there will be differences in mobilization of shear strength at different depths for calculating incremental soil displacement. Lastly, the original mechanism of Osman and Bolton (2006) shown in Figure 3(a) only applied to wide excavations; narrow excavations called for the development of the alternative mechanism of Figure 3(b).

In view of these issues, a new fully incremental calculation method has been introduced, allowing for the storage of elastic strain energy in the wall and the support system, and respecting the possible constriction of the plastic deformations due to the narrowness of an excavation.

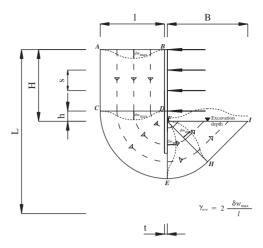
#### 5.1 Deformation pattern in different zones

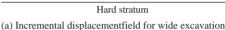
From Figure 3, the soil is assumed to flow parallel to the wall at the retained side above the level of the lowest support (zone ABDC) and the incremental displacement at any distance x from the wall is given by the cosine function of Equation 1, replacing y by x.

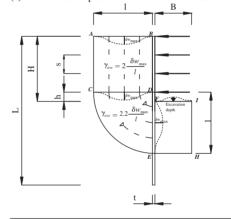
By taking the origin as the top of the wall, the deformation pattern of retained soil ABDC is given in rectangular coordinates as follows:

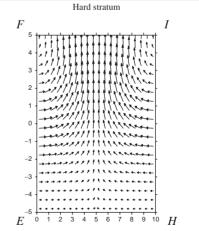
$$dw_{y} = -\frac{dw_{max}}{2}(1 - \cos(\frac{2\pi x}{l}))$$
(3)

 $dw_x = 0$  (4)









(b) Incremental displacement field for narrow excavation

Figure 3. Incremental displacement fields.

In fan zone, CDE, by taking the apex of the fan zone as the origin

$$dw_{y} = \frac{dw_{\max}}{2} \left( 1 - \cos\left(\frac{2\pi\sqrt{x^{2} + y^{2}}}{l}\right) \right) \left(\frac{-x}{\sqrt{x^{2} + y^{2}}}\right)$$
(5)

$$dw_x = \frac{dw_{\max}}{2} \left( 1 - \cos\left(\frac{2\pi\sqrt{x^2 + y^2}}{l}\right) \right) \left(\frac{y}{\sqrt{x^2 + y^2}}\right)$$
(6)

For fan zone EFH in very wide excavations as indicated in Figure 3(a), by taking the junction of the wall and the current excavation level as the origin:

$$dw_{y} = \frac{dw_{max}}{2} \left( 1 - \cos\left(\frac{2\pi \left[h + \sqrt{x^{2} + y^{2}}\right]}{l}\right) \right) \left(\frac{x}{\sqrt{x^{2} + y^{2}}}\right)$$
(7)

$$dw_{z} = \frac{dw_{\max}}{2} \left( 1 - \cos\left(\frac{2\pi \left[h + \sqrt{x^{2} + y^{2}}\right]}{l}\right) \right) \left(\frac{y}{\sqrt{x^{2} + y^{2}}}\right)$$
(8)

For the triangular zone FHI in very wide excavations, again taking the junction of the excavation and the wall as the origin:

$$dw_{y} = dw_{x} = \frac{\sqrt{2}dw_{max}}{4} \left( 1 - \cos\left(\frac{2\pi \left[h + \frac{\sqrt{2}}{2}(x - y)\right]}{l}\right) \right)$$
(9)

For narrow excavations as shown in Figure 3(b), a rectangular zone EFHI of 2D shearing is now proposed. The origin is taken as the mid-point of FE, mid-wavelength in the excavation, at the wall.

$$dw_{y} = \frac{l \times dw_{\max}}{4B} \left( \pi + \frac{2\pi y}{l} + \sin\left(\frac{2\pi y}{l}\right) \right) \left( \sin\left(\frac{\pi y}{B}\right) \right)$$
(10)

$$dw_x = \frac{dw_{max}}{2} \left( 1 + \cos\left(\frac{2\pi y}{B}\right) \right) \left( \cos\left(\frac{\pi x}{l}\right) \right)$$
(11)

In order to get more accurate solutions, it is supposed that the soil stratum is divided into *n* layers of uniform thickness  $\overline{t}$  (Figure 4). The average shear strain  $d\gamma(m,n)$  is calculated for *n* layers in *m* excavation stages. The incremental engineering shear strain in each layer is calculated as follows:

$$d\gamma(m,n) = \frac{\int \left(\sqrt{\left(\frac{\partial w_x}{\partial y} + \frac{\partial w_y}{\partial x}\right)^2 - 4\frac{\partial w_x}{\partial x}\frac{\partial w_y}{\partial y}}\right) dVol}{\int dVol}$$
(12)

In order to get a better idea of the deformation mechanism, the relationship between the maximum incremental wall displacement and the average shear strain mobilized in each zone of deformation should

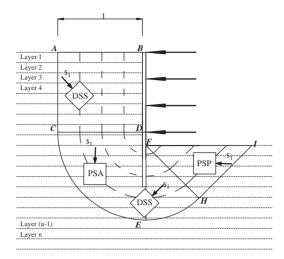


Figure 4. Mobilizable shear strength profile of an excavation stage in an layered soil.

be obtained. On the active side of the excavation, the spatial scale is fixed by the wavelength of deformation l, and all strain components are proportional to  $dw_{max}/l$ . The average engineering shear strain increment  $\gamma_{mob}$  mobilized in the deformed soil can be calculated from the spatial average of the shear strain increments in the whole volume of the deformation zone. For a wide excavation i.e. Figure 3(a), the average shear strain is equal to  $2dw_{max}/l$ . For a narrow excavation, the average shear strain ( $\gamma_{ave}$ ) of active zone ABCD and fan zone CDE is  $2dw_{max}/l$  and  $2.23dw_{max}/l$ , respectively, while  $\gamma_{ave}$  in the passive zone EFHI depends both on the wavelength l of the deformation and the width B of the excavation. The relationship between the normalized average shear strain in EFHI and the excavation geometry is shown in Figure 5. The correlations are as follows:

$$\frac{\gamma_{ave}}{2w_{\max}/B} = 0.38 \left(\frac{2l}{B}\right) + 0.61 \quad \text{for} \quad \frac{l}{B} \ge 1$$
(13)

$$\frac{\gamma_{ave}}{2w_{max}/B} = 0.98 \left(\frac{2l}{B}\right)^2 - 3.16 \left(\frac{2l}{B}\right) + 3.83 \quad \text{for } \frac{l}{B} \le 1$$
(14)

Apart from the first excavation stage, all subsequent deformation mechanisms must partially overlay the previous ones (Figure 6). Due to the non-linearity of soil it is important to calculate the accumulated mobilized shear strain in each particular layer of soil in order to correctly deduce the mobilized shear strength of that layer. This is done by an area average method described as follows:

$$\gamma(m,n) = \frac{d\gamma(m,n) \times A(m,n) + \gamma(m-1,n) \times A(m-1,n)}{A(m,n)}$$
(15)

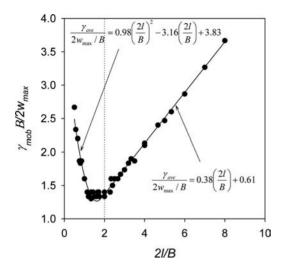


Figure 5. Correlation between normalized average shear strain and excavation geometry for a narrow excavation.

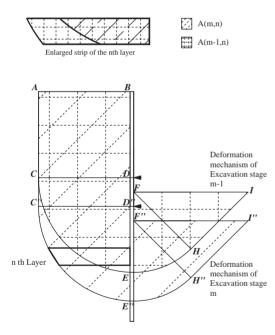


Figure 6. Overlapping of deformation field.

where  $\gamma(m, n)$  is the total shear strain of the nth layer in the mth excavation stage, and A(m, n) is the area of deformation in the nth layer in the mth excavation stage.

With the help of some suitable stress-strain relation for the soil (discussed in the following section), the mobilized strength ratio  $\beta(m,n)$  at excavation stage m for soil layer n can be found (Figure 7).

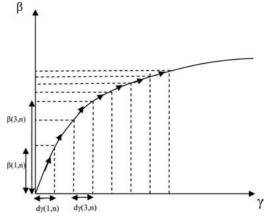


Figure 7. Typical stress strain relationship of soft clay.

## 5.2 Shear strength mobilized in mechanism

In soft clay, the undrained shear strength generally varies with depth, and with orientation of shear direction. The strength matrix  $c_{mob}(m,n)$  mobilized for excavation stage m for layer n can be expressed using a matrix  $\beta(m,n)$  on the appropriate undrained shear strength profile. Regarding orientation, anisotropy of soft soil can be a significant factor for excavation stability. For example, Clough & Hansen (1981) show an empirical factor based on the observation that triaxial extension tests give roughly one half the undrained shear strength of triaxial compression, with simple shear roughly half way between. Figure 4 shows the orientation of the major principal stress direction within the various zones of shearing in the assumed plastic mechanism for wide excavations, and indicates with a code the soil test configuration that would correctly represent the undrained shear strength of at the specific orientation. For locations marked DSS the assume directions of shearing are either vertical or horizontal, so the ideal test on a vertical core is a direct simple shear test. In areas marked PSA and PSP, shearing takes place at 45 degrees to the horizontal and these zones are best represented by plain strain active and passive tests, respectively. Since the undrained shear strength of the direct simple shear test is roughly the average of that of PSA and PSP, the relative influence of the PSA and PSP zones is roughly neutral with respect to direct simple shear. As a result, the design method for braced excavation can best be based on the undrained shear strength of a direct simple shear test. A similar decision was made by O'Rourke (1993).

The equilibrium of the unbalanced weight of soil inside the mechanism is achieved by mobilization of shear strength. For each excavation stage, mobilization of shear strength of each layer is considered by the following:

$$c_{u,mob}(m,n) = \beta(m,n) c_u(n) \tag{16}$$

where  $c_{u,mob}(m,n)$  is the mobilized undrained shear strength for layer *n* in excavation stage *m*;  $c_u(n)$  is the undrained shear strength for layer n; and  $\beta(m,n)$  is the mobilized strength ratio for excavation stage *m* and soil layer *n*.

### 5.3 Incremental energy balance

By conservation of energy, the total loss of potential energy of the soil ( $\Delta P$ ) must balance the total dissipated energy due to plastic shearing of the soil ( $\Delta D$ ) and the total stored elastic strain energy in bending the wall ( $\Delta U$ ).

$$\Delta P = \Delta D + \Delta U \tag{17}$$

The potential energy loss on the active side of the wall and the potential energy gain of soil on the passive side can be estimated easily. The net change of gravitational potential energy  $(\Delta P)$  is given by the sum of the potential energy changes in each layer:

$$\Delta P = \sum_{i=1}^{n} \left[ \int_{\text{volume}} \gamma_{sul}(m, i) dw_{y}(m, i) dVol \right]$$
(18)

where  $dw_y(m, i)$  is the vertical component of displacement of soil in the ith layer for the mth construction;  $\gamma_{sat}(m, i)$  is the saturated unit weight of soil in the ith layer for the mth construction.

Since there are no displacement discontinuities, the total plastic work done by shearing of soil is given by summing the internal dissipation in each layer:

$$\Delta D = \sum_{i=1}^{n} \left[ \int_{Volume} \beta(m,i) c_u(m,i) | d\gamma(m,i) | dVol \right]$$
(19)

where  $c_u(m,i)$  is the undrained shear strength of soil in the ith layer for the mth construction;  $d\gamma(m,i)$  is the shear strain increment of soil in the ith layer for the mth construction; and the corresponding mobilized strength ratio is given by:

$$\beta(m,i) = \frac{c_{u,mob}(m,i)}{c_u(m,i)}$$

The total elastic strain energy stored in the wall,  $\Delta U$ , can be evaluated by repeatedly updating the deflected shape of the wall. It is necessary to do this since U is a quadratic function of displacement:

$$\Delta U = \frac{EI}{2} \int_{0}^{s} \left[ \frac{d^2 w_{\chi}}{dy^2} \right]^2 dx$$
(20)

where E and I are the elastic modulus and the second moment of area per unit length of wall, and s is the length of the wall in bending. L can be the length of wall s below the lowest prop.

By assuming the cosine waveform equation (Equation 1), the strain energy term can be shown to be as follows:

$$\Delta U = \frac{\pi^3 E I d w_{\max}^2}{l^3} \left( \frac{\pi s}{l} + \frac{\sin(\frac{4\pi s}{l})}{4} \right)$$
(21)

where l is the wavelength of deformation,  $dw_{max}$  is the maximum deflection of the wall in each excavation increment.

### 5.4 Calculation procedure

The following calculation procedure is programmed in Matlab 2006b.

- 1 At each stage of excavation, a maximum deformation  $w_{max}$ , which is bounded by an upper and a lower bound, is assumed. The soil stratum is divided into *n* layers. The areas on both the active side and the passive side in each layer are calculated.
- 2 For each layer, with the help of the numerical integration procedure in Matlab, the mobilized shear strain and the change in PE on both active and passive sides in different zones is calculated. (Equation 18) The total mobilized shear strain is updated according Equation 15.
- 3 With the use of a suitable stress-strain curve (Figure 7), the mobilizable strength ratio  $\beta$  can be found.
- 4 Total change in PE and total energy dissipation and elastic bending energy in the wall can be calculated by Equations 18, 19 & 21, respectively.
- 5 By considering the conservation of energy of a structure in statical equilibrium, the sum of total energy dissipation and elastic strain energy in the wall balances the total change in PE. To facilitate solving the solution, an error term is introduced as follows:

$$\text{Error} = \Delta D + \Delta U - \Delta P \tag{22}$$

- 6 When the error is smaller than a specified convergence limit, the assumed deformation is accepted as the solution; otherwise, the method of bisection is employed to assume another maximum displacement and the error term is calculated again using steps 1 to 5.
- 7 Then, the incremental wall movement profile is plotted using the cosine function of equation
- 8 The cumulative displacement profile is obtained by accumulating the incremental movement profiles.

## 6 VALIDATION BY NUMERICAL ANALYSIS

The finite element method can provide a framework for performing numerical simulations to validate the extended MSD method in evaluating the performance of braced excavations. However, finite element analysis of retaining walls is potentially problematic. One the most difficult problems is the constitutive model used for the soil. The stress-strain relationship can be very complicated when considering stress history and anisotropy of soil (Whittle, 1993).

The validation of the extended MSD method is examined by comparing its predictions with results of comprehensive FE analyses of a plane strain braced excavation in Boston Blue Clay carried out by Jen (1998). In these analyses, the MIT-E3 constitutive model is used (Whittle, 1987). The model is based on Modified Cam clay (Roscoe and Burland 1968). However, several modifications had been made to improve the basic critical state framework. The model can simulate small strain non-linearity, soil anisotropy and the hysteretic behaviour associated with reversal of load paths. Whittle (1993) also demonstrated the ability of the model to accurately represent the behaviour of different clays when subjected to a variety of loading paths.

Jen (1998) extended the use of the MIT-E3 model for analyzing cases of deep excavation in a great variety of situations. She considered the effect of excavation geometry such as wall length, excavation width and depth of bed rock, the effect of soil profile such as  $c_u/OCR$  ratio and layered soil, and the effect of structural stiffness such as wall stiffness and strut stiffness. This provides a valuable database for validation of the extended MSD method.

#### 6.1 An example of MSD calculation

The following example shows the extended MSD calculation of wall deflections for a 40 m wall retaining 17.5 m deep and 40 m wide excavation (Figure 8). The construction sequence comprises the following steps:

- 1 The soil is excavated initially to an unsupported depth (h) of 2.5 m.
- 2 The first support is installed at the ground surface.
- 3 The second level of props is installed at a vertical spacing of 2.5 m, and 2.5 m of soil is excavated.

The undrained shear strength of the soil is expressed by the relationship suggested by Hashash and Whittle (1996) for Boston Blue Clay (BBC) as follows:

$$c_{y} = 0.21[8.19z + 24.5]kPa$$
 (23)

The cantilever mode of deformation and the bulging movements are calculated separately using the mechanism of Osman & Bolton (2006) and the extended

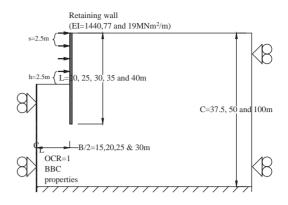


Figure 8. Scope of parametric study to examine excavation width effect.

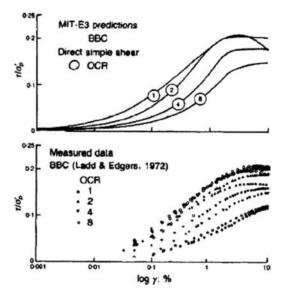


Figure 9. Stress-strain response for Ko consolidated undrained DSS tests on Boston blue clay (After Whittle, 1993).

MSD method as described above. The total wall movements are then obtained by adding the bulging movements to the cantilever movements to the cantilever movement according to Clough et al. (1989).

## 6.1.1 Cantilever movement

By solving for horizontal force equilibrium and moment equilibrium about the top of the wall, the mobilized shear stress ( $c_{mob}$ ) is found to be 11.43 kPa. The mobilized strength ratio  $\beta$  is 0.2886. With the help of direct simple shear stress-strain data for Boston blue clay by Whittle (1993) (Figure 9), the mobilized strain is read off for an appropriate preconsolidation pressure  $\sigma'_p$  and an appropriate OCR. The mobilized shear

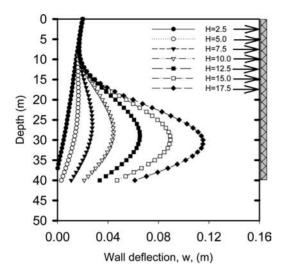


Figure 10. Wall deflections from MSD with different excavation depths.

strain ( $\gamma_{mob}$ ) is found to be 0.2%. By considering the geometrical relationship, the wall rotation is found to be 0.1%. The displacement at the top of the wall is found to be 39 mm.

#### 6.1.2 Bulging movement

The first support is installed at the top of the wall. The length of the wall below the support is 40 m. By adopting the iterative calculation procedure, and using the deformation mechanism for a narrow excavation, the bulging movement at each stage of excavation can be obtained. Then, the incremental bulging movement profile in each stage is plotted using the cosine function, using the maximum incremental displacement in each stage together with the corresponding wavelengths. The total wall movement is obtained by accumulating cantilever movement and the total bulging movement. Figure 10 shows the final deformation profile of the accumulated wall movement of an excavation with a width of 40 m. The maximum wall deflection at an excavation depth of 17.5 m is 115 mm. The position of the maximum wall displacement is located at 0.75 L, where L is the length of the wall.

#### 6.2 Effect of excavation width

The effect of excavation width on predicted ground movements is the focus of this section. Underground transportation systems may have excavation widths ranging from 25 m (a subway station) to 60 m (an underground highway). The most widely used design charts generally incorporate the effect of excavation width in estimation of factor of safety against base heave (Bjerrum and Eide, 1956) or as a multiplication factor in estimating the maximum settlement (Mana and Clough, 1981).

The scope of the excavation analyses are shown in Figure 8. In the analyses, the excavation was carried out in undrained conditions in a deposit of normally consolidated Boston Blue Clay with depth C taken to be 100 m. A concrete diaphragm wall of depth L = 40 m, and thickness 0.9 m, supported by rigid props spaced at h = 2.5 m, was used for supporting the simulated excavation. The excavation width varies from 20 m to 60 m. The wavelength of deformation is chosen according to the  $l = \alpha s$  rule, where  $\alpha$  was taken to be 1.5 and s is the length of wall below the lowest prop. Computed results by Jen (1998) are used for comparison. Full details of the analysis procedures, assumptions and parameters are given in Jen (1998). In the following section, only results of wall deformation will be taken for comparison.

Figure 11(a) and (b) show the wall deflection profile with different excavation widths at an excavation depth of 17.5 m, as calculated by the extended MSD method and the MIT-E3 model. Figure 11(a) shows that the excavation width does not have any effect on the deflected shape of the wall as calculated by the extended MSD method. Figure 11(b), simularly, shows a limited effect on the deflected shape of the wall by the MIT-E3 model. While the MSD-predicted maximum wall deflection increases by a factor of 1.5 as the width is increased from 30 m to 60 m, the MIT-E3 computed maximum wall deformation increased by a factor of 1.6 with the same increase in excavation width.

## 6.3 Effect of bending stiffness of the wall

In general, structural support to excavations is provided by a wall and bracing system. Soldier piles and lagging, sheet piles, soil mix and soldier piles, drilled piers (secant piles), and reinforced concrete diaphragm walls are examples of wall types that have been used to support excavations. The various types of wall exhibit a significant range of bending stiffness and allowable moment. Support walls composed of soldier piles and sheet piles are generally more flexible and capable of sustaining smaller loads than the more rigid drilled piers and reinforced diaphragm walls.

The preceding sections have all assumed a 0.9 m thick concrete diaphragm wall with elastic bending stiffness  $EI = 1440 \text{ MNm}^2/\text{m}$ . Although it is possible to increase this bending stiffness by increasing the wall thickness and reinforcement, or by using T-panels (barettes), most of the walls used in practice have lower bending stiffnesses. For example, the typical bending stiffness of sheet pile walls is in the range of 50 to  $80 \text{ MNm}^2/\text{m}$ . This section assesses the effect of wall bending stiffness on the excavation-induced displacements.

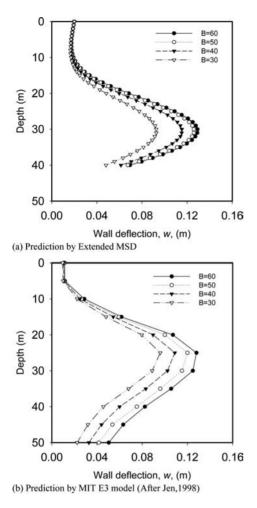
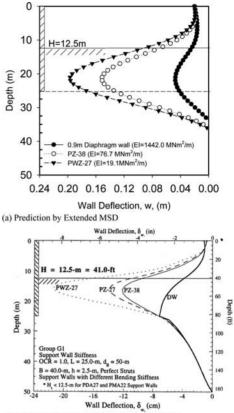


Figure 11. Wall deflection profile of different excavation widths at H = 17.5 m.

Excavation in soft clay with a width of 40 m supported by a wall of length 25 m and of various bending stiffness (EI = 1440, 70 and 20 MNm<sup>2</sup>/m) are studied. Results generated by the MSD method and FEA are compared. Figure 12 (a) and (b) presents the deflection profiles of the excavations predicted by extended MSD and the MIT-E3 model, respectively. As the bending stiffness of the wall decreases, there is no pronounced change in the overall shape of the wall; the maximum wall deflection increases and its location migrates towards the excavated grade. At H = 12.5 m, the maximum wall displacement is 47 mm for the concrete diaphragm wall with the maximum deflection located at 7.5 m below the excavation level, while the result for the most flexible sheet pile wall shows 197 mm of maximum wall deflection occurring at 5.5 m below the excavation level. In additional to this, changes in



(b) Prediction by MIT E3 model (After Jen, 1998)

Figure 12. Deflection profiles of walls with various bending stiffnesses.

wall stiffness also affect the transition from a subgrade bending mode to a toe kicking-out mode. As the wall stiffness decreases, the influence of embedment depth reduces, and hence the tendency for toe kickout to occur is less. Again, a fairly good agreement can be seen when comparing extended MSD results and numerical results by the MIT-E3 model, though kinks are usually found at the wall toe in the numerical predictions, which implies localization of large shear strains developed beneath the wall toe.

#### 6.4 Effect of wall length

Wall length is one of the geometrical factors affecting the behaviour of a supported excavation. Previous analyses were done by Osman and Bolton (2006). The studies showed that the wall end condition should be assumed to be free for short walls (L = 12.5 m) since the clay is very soft at the base and the embedded length is not long enough to restrain the movement at the tip of the wall (kick-out mode of deformation). For long walls (L = 40 m), the embedded depth was assumed

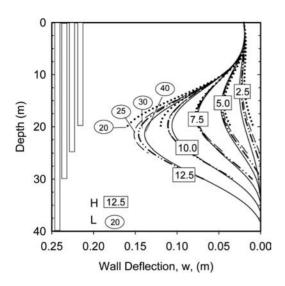


Figure 13. Wall deflection profile of excavation with different support wall lengths, by Extended MSD method.

to be sufficient to restrain the movement at the wall base (bulging model of deformation). However, the effect of structural stiffness was not considered in the old MSD method, though similar observations were made by Hashash and Whittle (1996) in their numerical analyses.

In this section, the effect of wall length will be considered. Excavations with widths of 40 m supported by a 0.9 m thick concrete diaphragm wall with varying length (L = 20, 25, 30 and 40 m) are studied. Figure 13 shows the wall displacement profiles against depth with different lengths of wall. For H < 7.5 m, the deflected wall shapes are virtually identical for all four wall cases of wall length. This agrees with the conclusion made by Hashash (1992) that wall length had a minimal effect on pre-failure deformations. As H increases to 10 m, the toe of the 20 m long wall begins to kick out with maximum incremental deformations occurring at the toe of the wall. The movements of the 25, 30 and 40 m long walls are quite similar. At H = 12.5 m, the toe of the 20 m and 25 m long wall kick out, while the two longer walls (L = 30 and 40 m)continue to deform in a bulging mode. The difference in deformation mode shape demonstrates that the wall length has a significant influence on the failure mechanism for a braced excavation.

Figure 15 shows a similar set of analyses by using the MIT-E3 model. Similar observations about the wall shape can be made.

Figure 14 summarizes the variation of the normalized excavation-induced deflection ( $w_{max}/H$ ) with the width to length ratio (B/L) for different bending stiffnesses of the support wall, for H = 17.5 m.

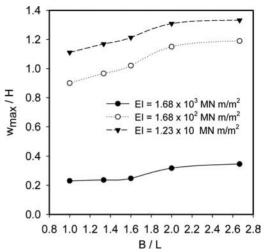


Figure 14. Variation of maximum wall deflection with width to length ratio of wall.

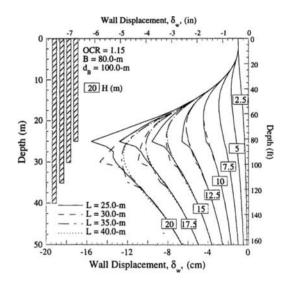


Figure 15. Wall deflection profile of excavation with different support wall lengths, by MIT-E3 method (After Jen (1998)).

For a flexible wall (EI = 12.3 MN m<sup>2</sup>/m), the normalized maximum wall deflection increases linearly as the B/L ratio increases from 1 to 2. The gradient changes and w<sub>max</sub>/H increases in a gentle fashion as the B/L ratio increases from 2 to 2.8. For a rigidly supported wall, the increase in w<sub>max</sub>/H ratio is less significant as the B/L ratio increases. In other words, the maximum wall deflection is less sensitive to a change of B/L ratio for a rigid wall.

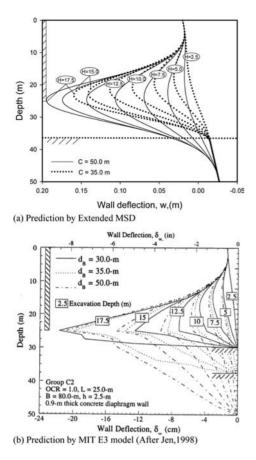


Figure 16. Wall deflection profiles of excavation with different depths to the firm stratum.

## 6.5 Effect of the depth of bearing stratum

The depth to bedrock, C, is an important component of the excavation geometry. The preceding analyses have assumed a deep clay layer with bedrock located at C = 100 m which represents a practical upper limit on C. In practice, however, the clay layer is usually less than 100 m deep. The following results focus on the discussion of the geometrical parameter C. The analysis involves plane strain excavation in normally consolidated Boston blue clay supported by a 0.9 m thick concrete diaphragm wall with rigid strut supports spaced at 2.5 m.

The wall deflection profiles for excavations predicted by both MSD and MIT-E3 with two depths of the clay stratum (C = 35 m and 50 m) are compared in Figure 16(a) and (b).

In general, the depth of the firm stratum would only affect wall deformations below the excavated grade, hence the largest effects can be seen at the toe of the

wall. For situations where the wavelength of deformation is restricted less by excavation width than by the depth of the firm stratum (B > C), the magnitude of maximum wall deflection increases with the depth of the firm stratum (C). The MSD method predicts that the 'kick-out' displacement of the wall toe is limited by the restriction of developing a large deformation mechanism. As a result, the maximum wall deflection is also limited. The increase in incremental wall deflection decreases in later stages of excavation when H increases from 12.5 m to 17.5 m due to the reduction of wavelength of deformation. On the other hand, when the depth of the firm stratum is much larger than the width of the excavation (B < C), the depth of the bed rock has a minimal effect on the magnitude of wall deflection. Results by FEA by Jen (1998) (Figure 16(b)) also showed the same observation. Despite the shortcoming of MSD not being able to model the correct shape of wall, the maximum wall deflection is predicted reasonably well. The net difference in maximum wall displacement between MSD and the full FEA is generally less than 20%.

## 7 CONCLUDING REMARKS

An extended MSD method is introduced to calculate the maximum wall displacement profile of a multipropped wall retaining an excavation in soft clay. As with the earlier MSD approach, each increment of wall bulging generated by excavation of soil beneath the current lowest level of support is approximated by a cosine function. The soil is divided into layers in each of which the average shear strain increments are compounded so that the mobilized strength ratio in each layer can be tracked separately as excavation proceeds, using stress-strain data from a representative element test matched to the soil properties at mid-depth of the wall. The incremental loss in potential energy associated with the formation of a settlement trough, due to wall deformation and base heave, can be expressed as a function of those ground movements at any stage. By conservation of energy, this must always balance the incremental dissipation of energy through shearing and the incremental storage of elastic energy in bending the wall. By an iterative procedure, the developing profile of wall displacements can be found.

A reasonable agreement is found between predictions made using this extended MSD method and the FEA results of Jen (1998) who created full numerical solutions using the MIT-E3 soil model. In particular, the effects of excavation width, wall bending stiffness, wall length, and the depth of the clay stratum, were all quite closely reproduced.

It is important to draw the right lessons from this. The excellent work at MIT over many years, on soil element testing, soil constitutive models, and Finite Element Analysis, have provided us with the means to calibrate a very simple MSD prediction method. This was based on an undrained strength profile, a single stress-strain test, and a plastic deformation mechanism. Were it not for the multiple level of props the calculation of ground displacements could be carried out in hardly more time than is currently required to calculate a stability number or factor of safety. Allowing for the need to represent various levels of props, the calculations then call for a Matlab script or a spreadsheet, and the whole process might take half a day to complete.

An engineer can therefore anticipate that important questions will be capable of approximate but reasonably robust answers in a sensible industrial timescale. For example:

- Will a prop spacing of 3m be sufficient for a wall of limited stiffness and strength?
- Should the base of the wall be fixed by jet-grouting prior to excavation?
- Will a particular construction sequence cause the soil to strain so much that it indulges in post-peak softening?
- Is it feasible to prop the wall at sufficiently close spacings to restrict strains in the retained ground to values that will prevent damage to buried services?

This may lead engineers to take soil stiffness more seriously, and to request accurate stress-strain data. If so, in a decade perhaps, our Codes of Practice might be updated to note that MSD for deep excavations provides a practical way of checking for the avoidance of serviceability limit states.

## ACKNOWLEDGEMENT

The authors would like to acknowledge the earmarked research grant # 618006 provided by the Research Grants Council of the HKSAR Government, and also the Platform Grant (GR/T18660/01) awarded by the UK Engineering and Physical Sciences Research Council.

## REFERENCES

- Bjerrum, L. & Eide, O. 1956. Stability of strutted excavations in clay. *Geotechnique* 6: 115–128.
- Bolton, M.D. & Powrie, W., 1988. Behaviour of diaphragm walls retaining walls prior to collapse. *Geotechnique* 37(3): 335–353.
- Bolton, M.D., Powrie W. & Symons, I.F. 1989. The design of stiff in-situ walls retaining overconsolidated clay part. *Ground engineering* 22(8): 44–48.
- Bolton, M.D., Powrie, W. & Symons, I.F. 1990a. The design of stiff in-situ walls retaining over-consolidated clay part 1, short term behaviour. *Ground Engineering* 23(1): 34–39.

- Bolton, M.D., Powrie, W. & Symons, I.F. 1990b. The design of stiff in-situ walls retaining over-consolidated clay, part II, long term behaviour. *Ground engineering* 23(2): 22–28.
- Clough, G.W. & Hansen, L.A. 1981. Clay anisotropy and braced wall behavior. ASCE Journal of Geotechnical Engineering 107(7): 893–913.
- Clough, G.W., Smith, E.W. & Sweeney, B.P. 1989. Movement control of excavation support system by iterative design. *Foundation Engineering Current Principles and Practices* Vol. 2 ASCE, New York, NY: 869–882.
- Eide, O., Aas, G. & Josang, T. 1972. Special application of cast-inplace walls for tunnels in soft clay. *Proceedings* of the 5th European Conference on Soil Mechanics and Foundation Engineering, Madrid, Spain, 1: 485–498.
- Hashash, Y.M.A. & Whittle, A.J. 1996. Ground movement prediction for deep excavations in soft clay. ASCE Journal of Geotechnical Engineering 122(6): 474–486.
- Jen, L.C. 1998. The design and performance of deep excavations in clay. PhD thesis, Dept. of Civil and Environmental Engineering, MIT, Cambridge, Mass.
- Mana, A.I. & Clough, G.W. 1981. Prediction of movements for braced cut in clay. J. Geo. Engrg. ASCE 107(GT8): 759–777.
- Milligan, G.W.E. & Bransby, P.L. 1976. Combined active and passive rotational failure of a retaining wall in sand. *Geotechnique* 26(3): 473–494.
- O'Rourke, T.D. 1993. Base stability and ground movement prediction for excavations in soft clay. *Retaining Structures*, Thomas Telford, London: 131–139.
- Osman, A.S. & Bolton, M.D. 2004 A new design method for retaining walls in clay. *Canadian Geotechnical Journal* 41(3): 451–466.
- Osman, A.S. & Bolton, M.D. 2005 Simple plasticity-based prediction of the undrained settlement of shallow circular foundations on clay. *Geotechnique* 55(6): 435–447.
- Osman, A.S. & Bolton, M.D. 2006 Ground movement predictions for braced excavations in undrained clay. ASCE Journal of Geotechnical and Geo-environmental Engineering 132(4): 465–477.
- Osman, A.S., Bolton, M.D. & Mair, R.J. 2006 Predicting 2D ground movements around tunnels in undrained clay. *Geotechnique* 56(9): 597–604.
- Skempton, A.W. 1951. The bearing capacity of clays. Proc., Building Research Congress, London: 180–189.
- Sloan, S.W. 1988. Lower bound limit analysis using finite elements and linear programming. *International Journal* for Numerical and Analytical Methods in Geomechanics 12(1): 61–77.
- Sloan, S.W. & Kleeman, P.W. 1995. Upper bound limit analysis using discontinuous velocity fields. *Computer Methods in Applied Mechanics and Engineering* 127: 293–314.
- Terzaghi, K.1943. Theoretical soil mechanics, John Wiley & Sons, Inc., New York, N.Y.
- Ukritchon, B., Whittle, A.J. & Sloan, S.W. 2003. Undrained stability of braced excavations in clay. ASCE Journal of Geotechnical and Geoenvironmental Engineering 129(8): 738–755.
- Whittle, A.J. 1987. A constitutive model for overconsolidated clays with application to the cyclic loading of friction piles. PhD thesis, Dept. of Civil and Environmental Engineering, MIT, Cambridge, Mass.
- Whittle, A.J. 1993. Evaluation of a constitutive model for overconsolidated clays. *Geotechnique* 43(2): 289–313.

## Overview of Shanghai Yangtze River Tunnel Project

## R. Huang

Commanding Post of Shanghai Tunnel & Bridge Construction, Shanghai, P.R. China

ABSTRACT: In the paper, an introduction of the construction background and scale of Shanghai Yangtze River Tunnel and Bridge Project and natural conditions of Shanghai Yangtze River Tunnel construction are given. The overall design concept and some critical technical solutions such as segment structure of large diameter bored tunnel, water proofing of segment under high depth and water pressure, long tunnel ventilation system and fire fighting system are described. Characteristics of two mixed TBM with a diameter of 15,430 mm are described. The overall construction methods of tunnel, and critical technical solutions and risk provision measures for large and long river-crossing tunnel such as the front surface stability for bored tunnel construction, floating resistance of large diameter tunnel, long distance construction survey, synchronous construction of internal structure, and cross passage construction of fresh/salty alternating geological/environmental condition are discussed.

## 1 INTRODUCTION

Shanghai Yangtze River Tunnel and Bridge project is located at the South Channel waterway and North Channel waterway of Yangtze River mouth in the northeast of Shanghai, which is a significant part of national expressway, as shown in Figure 1. It is an extremely major transport infrastructure project at seashore area in China at Yangtze River mouth and also the largest tunnel and bridge combination project worldwide. The completion of the project will further promote the development space for Shanghai, improve the structure and layout of Shanghai traffic system, develop resources on Chongming Island, accelerate economic development in the north of Jiangsu Province, increase the economy capacity of Pudong, accelerate the economy integrity of Yangtze River Delta, boom the economic development of Yangtze River area and even the whole country and upgrade the comprehensive competence of Shanghai in China and even in the global economy.

Shanghai Yangtze River Tunnel and Bridge (Chongming Crossing) alignment solution is the planned western solution which is implemented firstly based on the Shanghai overall urban planning, and comparison between east and west alignment and in combination of various aspects. The western alignment starts from Wuhaogou in Pudong, crossing Yangtze River South Channel waterway to Changxing Island and spanning Yangtze River North Channel waterway to east of Chongming Island.

Yangtze River begins to be divided into 3 levels of branches and have 4 mouths flowing into the sea: The



Figure 1. Site location of Chongming Crossing.

South Channel waterway is mixed river trench. The intermediate slow flow area forms Ruifeng shoal which is relatively stable for a long time. The natural water depth makes it as the main navigation channel. However, the North Channel waterway is located in the middle part of river, which is influenced by the south part and branch transition into North Channel waterway. So the trench varies alternatively and the river map is not as stable as South Channel waterway. Therefore, after iterative discussion by several parties, finally the solution of 'Southern Tunnel & Northern Bridge' is selected. The total project is 25.5 km long, among which 8.95 km is tunnel with a design speed of 80 km/h and 9.97 km is bridge and 6.58 km is land connection with a design speed of 100 km/h, as shown

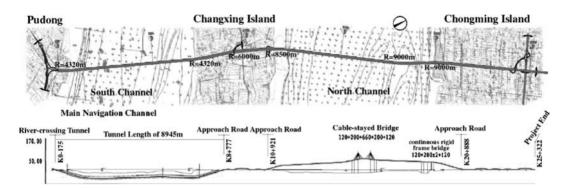


Figure 2. Diagram of Shanghai Yangtze River Tunnel and Bridge.

in Figure 2. The total roadway is planned as dual 6 lanes.

## 2 CONSTRUCTION BACKGROUND AND PLANNING

The planning study of Shanghai Yangtze River Tunnel and Bridge Project (Chongming Crossing) was incepted from 90s of last century. The preliminary preparatory work has lasted 11 years. In May 1993, the National Scientific Committee held the 'Yangtze River mouth crossing significant technical-economical challenges - early stage work meeting'. After one year special investigation, the 'Preliminary study report of significant technical challenges of 'Yangtze River Crossing' was prepared. The pre-feasibility study report was prepared in March 1999. In August 2001, the international concept competition was developed and the 'Southern Tunnel & Northern Bridge' solution was defined. The National Planning Committee approved the project proposal in December 2002. The feasibility study report was approved by the National Development and Reform Committee in November 2004. The preliminary design was approved by the Ministry of Communication in July 2005 and total investment of 12.616 billion RMB was approved for the project.

For the project construction investment, 5 billion was funded by Shanghai Chengtou Corporation (60%) and Shanghai Road Construction Cooperation (40%), and 7.6 billion was financed from Bank Consortium.

Based on the characteristics of the national major project, Commanding Post of Shanghai Tunnel & Bridge Construction was established with approval of Shanghai Municipal Committee. The post is directed by the vice major and composed of staff from Pudong New Area, Chongming County and other committees and bureaus. The main responsibility is to make decision on significant problems and coordinate important items. In order to improve the depth of daily management, office was set up under the commanding post, working together with established 'Shanghai Yangtze River Tunnel and Bridge Construction Development Co., Ltd.' which is mainly in charge of the implementation of the project and daily work of commanding post and performs the investment management on behalf of the client. The specific work is responsible for the financing, investment, construction, operation and transfer of the project. To detail the technical assurance measures, the clients sets up the technical consultant team which provides theoretical support, technical assistance and consultancy service for significant technical challenges during the implementation. Meanwhile, the team is involved in the investigation of significant technical solutions, review of construction method statement and treatment of technical problems to ensure the high quality and safety. International well-known consultancy companies are entrusted for the purpose of application of state-of-art philosophy, most successful experience, optimal concept and most mature management to make the Yangtze River Tunnel and Bridge Project as Century Elite Project.

The project finally initiated on 28th, December 2004 and planned to be open to traffic in July 2010. The main civil structure of the bridge is planned to be closed in June 2008, and tunnel in April 2009.

## 3 NATURAL CONDITIONS OF TUNNEL PROJECT

### 3.1 Environmental conditions

Shanghai Yangtze Tunnel Project starts from Wuhaogou of Waigaoqiao in Pudong New Area, connected with Shanghai main fast roads such as Middle Ring, Outer Ring and Suburb Ring through Wuzhou Aveneu, crossing southern water area and lands on Changxing Island 400 m west of Xinkaihe Harbour, connected with Changxing Island road net through Panyuan Interchange. The main building on land is the flood prevention wall on Pudong side and Changxing Island. Others are farm fields. The river-crossing section is mainly the

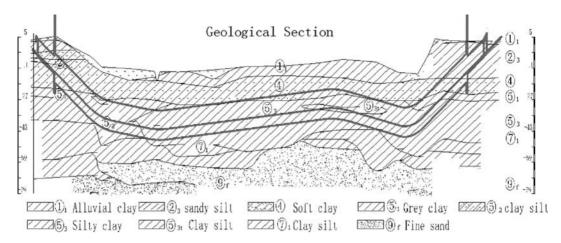


Figure 3. Longitudinal profile of tunnel.

southern water way for navigation which is an important passage for connecting Yangtze Waters with other seashore area in China and oceans worldwide.

There are two sea cables arranged along the bored tunnel axis with a depth of 3 m below natural river bed. One cable is basically located at the west side of the tunnel and goes into the river near Wuhaogou on Pudong side, which is about 1,500 m away from the tunnel. It becomes closer to the tunnel gradually to the north and crosses the tunnel to its east at 240 m from Changxing Island and lands on Changxing Island at 350 m west of Xinkaihe Harbour. The other cable goes into the river near Wuhaogou, 1,300 away from the tunnel. Then it turns to NE first and N at 2,600 m way from Pudong Land Connections, almost identical with the tunnel alignment. And it changes from the west of tunnel to east of tunnel gradually and lands on Changxing Island about 300 m west of Xinkaihe Harbor.

Furthermore, two sunken boats close to Chainage XK2+350 and XK1+500 have been salved before bored tunnel construction. Earth was also filled back at corresponding locations; however, there may be still some remains.

## 3.2 River regime and hydrological conditions

At the mouth of Yangtze River it is tide area with intermediate level. Outside of mouth is regular half day tide and inside is irregular half day shallow tide due to the change of tide wave. Average flood tide time is 5 h and average ebb tide time is 7 h, so total time for ebb and flux is 12 h. The average currency flow is 1.05 m/s for flood tide during flood season and 1.12 m/s for ebb tide. The maximal flow for flood tide is 1.98 m/s and 2.35 m/s for ebb tide.

The underground water type in the shallow stratum at tunnel site is potential water, which has close hydraulic relation with river water. The potential water level is mainly influenced by the Yangtze River flux and ebb. The average water level for Waigaoqiao and Changxing Island is 2.8 m and 2.4 m, respectively.

In the stratum  $\overline{\mathbb{O}}$  and B at site area, the confined water is rich. At most area, the confined water is directly continuous. The confined water level is between -4.15 m and -6.76 m. Furthermore, slight confined water distributes in  $\textcircled{B}_2$ , which has certain hydraulic relations with confined water in  $\overline{\mathbb{O}}$ .

## 3.3 Geological conditions

The relief of onshore area of the project is 'river mouth, sand mouth, sand island' which is within the major four relief units in Shanghai. The ground surface is even with a normal elevation of 3.5 m (Wusong Elevation). The water area is classified as river bed relief.

The project site has a seismic fortification intensity of 7, classified as IV site. The stratum  $@_3$  and  $@_2$ sandy silt distributing on Pudong land area is slightly liquefied.

Main geological layers (refers to Figure 3) TBM crosses are:  $\textcircled{1}_1$  grey muddy clay,  $\textcircled{5}_1$  grey muddy clay,  $\textcircled{5}_2$  grey clayey silt with thin silty clay,  $\textcircled{5}_3$  silty clay,  $\textcircled{5}_3$  tlens,  $\textcircled{7}_{1-1}$  grey clay silt,  $\textcircled{7}_{1-2}$  grey sandy silt, etc. Unfavorable geological conditions are experienced along the axis of the tunnel, such as liquefied soil, quick sand, piping, shallow gas (methane), lens and confined water, etc.

## 4 TUNNEL DESIGN SOLUTION

### 4.1 Scale

Shanghai Yangtze River Tunnel is designed as dual 6 lanes expressway, and rail traffic provision is made below the road deck. Seismic fortification level is 7.

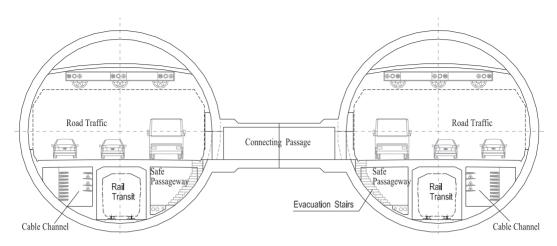


Figure 4. Cross section of bored tunnel.

Design service life is 100 years. The project consists of land connections of Pudong side (657.73 m), river-crossing tunnel (east tube 7,471.654 m and west tube 7,469.363 m) and land connections on Changxing Island (826.93 m). Total length is 8,955.26 m and investment is 6.3 billion RMB. The river-crossing part is twin-tube bored tunnel.

## 4.2 Tunnel alignment

The longitudinal profile of bored tunnel is in a shape of 'W' with a longitudinal slope of 0.3% and 0.87%. The land connections have a longitudinal profile of 2.9%. The minimal curvature radius of horizontal plane is 4,000 m and vertical profile 12,000 m.

## 4.3 Building design

### 4.3.1 Cross section of bored tunnel

Based on structural limit of traffic passage and equipment layout requirement, the internal diameter of lining for bored tunnel is determinated as 13.7 m considering the fitted tolerance of lining at curved section, construction tolerance, differential settlement, and combining the design and construction experience. On the top of tunnel, smoke discharge ducts are arranged for fire accident with an area of 12.4 m<sup>2</sup>. Each tunnel has three lanes with a structural clear width of 12.75 m and road lane clear height of 5.2 m. The central part below road deck is for rail traffic provision in future. On the left side, beside the buried transformer arrangement, it also serves as main evacuation stairs. The right side is cable channel, including provision space for 220 kV power cable, as shown in Figure 4.

## 4.3.2 Cross-section of land connections

Working shaft is underground four-floor building: -1 is for ventilation pipe and pump plant for fire fighting; -2 is for road lane with cross over; -3 is

for rail traffic provision and power cable gallery and -4 is for waste water pump plant.

The cut-and-cover is designed with a rectangular shape consisting of two tubes and one cable channel. 3 lanes are arranged in each tube. The structural limit is 13.25 m in width and 5.5 m in height, as shown in Figure 5. Upper area with a height of 0.6 m is for equipment provision. The upper part of central gallery is for cable channel, middle part for evacuation and lower part for pipe ditch. Ventilation shaft and building for equipments are arranged above the cut-and-cover tunnel close to the working shaft.

The approach consists of light transition zone and open ramp. The structural limit of cross section is identical with that of cut-and-cover tunnel. Both sides have a slope section with a slope of 1:3 with green planting for protection. The light transition zone is designed as steel arch structure.

## 4.4 Structural design

## 4.4.1 Structural design of bored tunnel

The external diameter of bored tunnel lining is 15,000 mm and internal diameter 13,700 mm, as shown in Figure 6. The ring width is 2,000 mm and thickness is 650 mm. Precast reinforced concrete common tapered segments are assembled with staggered joint. Concrete strength class is C60 and seepage resistance class is S12. The lining ring consists of 10 segments, i.e. 7 standard segments (B), 2 adjacent segments (L), and 1 key segment (F). According to the different depth, segments are classified as shallow segments, middle-deep segments, deep segments and extremely deep segments. Skew bolts are used to connect segments in longitudinal and circumferential direction. 38 × M30 longitudinal bolts are used to connect the rings.  $2 \times M39$  circumferential bolts are used to connect the segments. Shear pins are added between

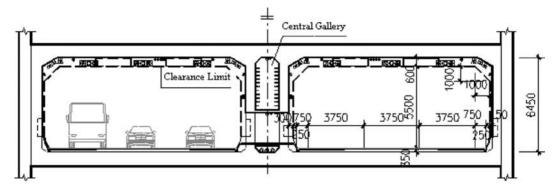


Figure 5. Cross-section of cut-and-cover.

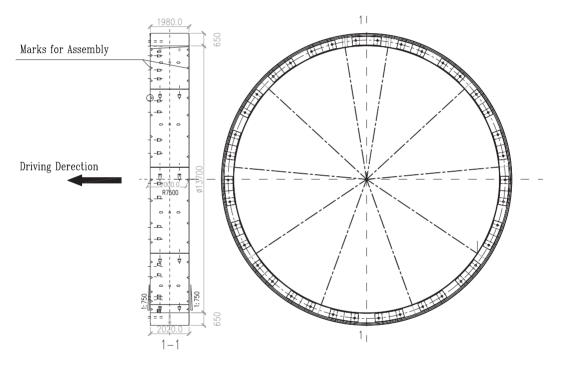


Figure 6. Lining structure.

lining rings at shallow cover area, geological condition variation area and cross passage to increase the shear strength between rings at special location and reduce the step between rings.

#### 4.4.2 Structural design of land connections

The working shaft and cut & cover tunnel share the same wall. The thickness of diaphragm of working shaft is 1,000 mm, and the inner wall is 500 mm, 1,200 mm, respectively. For the cut-and-cover tunnel, the thickness of diaphragm is 1,000 mm, 800 mm, and 600 mm respectively depending on the excavation depth. The inner structure thickness is 600 mm.

For the open cut ramp, the bottom plate structure thickness is around 500–1,100 mm. Under the bottom plate, bored piles are arranged as up-lifting resistance pile to fulfil the structural floating resistance requirement. The slope uses in-situ cast reinforced concrete grid and fill earth and green planting in the grid for protection.

## 4.5 Structural water-proof and durability design

## 4.5.1 Requirement and standard

For the bored tunnel and working shaft, the water proof standard of slightly higher than level II is required. For

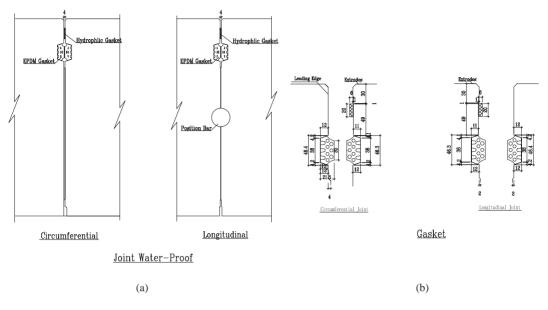


Figure 7. Segment joint water proofing sketch.

the entire tunnel, the average leakage should be less than  $0.05 \text{ L/m}^2$ ·d. For each random  $100 \text{ m}^2$ , the leakage should be less than  $0.1 \text{ L/m}^2$ ·d. The inner surface wet spots should not be more than 4‰ of total inner specific surface area. In each random  $100 \text{ m}^2$ , the wet spots should not be more than 4 locations. The maximal area of individual wet spot should not be large than  $0.15 \text{ m}^2$ .

The chloride diffusion coefficient of concrete lining structure of bored tunnel is not more than  $12 \times 10^{-13}$  m<sup>2</sup>/s. Concrete seepage resistance class is not less than S12. Furthermore, it is required that under 1 MPa water pressure which is equivalent to 2 times of water pressure for the tunnel with the largest depth, no leakage is occurred when the lining joint opens 7 mm and staggers 10 mm. The safety service life of water proof material is 100 years.

The seepage resistance class of onshore tunnel structure is not less than S10.

## 4.5.2 Water proofing design

The segment joint water proof arrangement consists of EPDM rubber strip with small compressive permanent deformation, small stress relaxation and good aging resistance performance and hydrophilic rubber strip, as shown in Figure 7.

The deformation joint at cut-and-cover tunnel uses embedded water stop gasket, outer paste gasket and inserted sealing glue forming enclosed system. The top plate uses water proof paint as outer water proof layer.

#### 4.6 Tunnel operation system

#### 4.6.1 Ventilation system

The road tunnel uses jet fan induced longitudinal ventilation combined with smoke ventilation.

The longitudinal ventilation area in tunnel is  $82 \text{ m}^2$ . Jet fans are suspended above the deck lane and below the smoke discharge duct, supporting induced ventilation in normal operation and congested condition. 78 jet fans with a diameter of 1,000 mm are arranged in each tube from Pudong access to Changxing Island access, every 3 as a group.

Ventilation shafts are arranged on Pudong side and Changxing Island, respectively, housing large ventilation machine and special smoke discharge axial fan. The fans are connected with main tunnel through air inlet and ventilation duct. During normal operation and congested condition, the ventilation machine is turned on to discharge the polluted air in the tunnel. 6 large axial fans with a capacity of  $75 \text{ m}^3/\text{s} - 150 \text{ m}^3/\text{s}$  are housed in the working shaft on Changxing Insland and Pudong, respectively.

For normal operation of lower rail traffic, piston ventilation mode is used.

## 4.6.2 Water supply and drainage system

The fire water, washing waste water, and structural leakage are collected by the waste water sump at the lowest point of river. Sump is arranged at upper and lower level, respectively. The lower waste water is drained by the relay of upper sump. The upper sump is arranged on two sides of rail traffic area, housing four pumps which are used alternatively under normal operation and turned on entirely during fire fighting. For lower level, 4 sumps with a dimension of  $1,000 \times 1,000 \times 550$  mm are arranged at the lowest point of tunnel where SGI segment is used and above the sump water collection trench with a length of 7 m and a width of 1 m is arranged. One waste water pump is placed in each pit which are used alternatively at normal condition and three are used, one spare during fire fighting.

At each access of tunnel, one rain water sump is arranged to stop water and drain it out of the tunnel. The rain amount is designed based on a return period of 30 years for rainstorm.

## 4.6.3 Power supply system

The electricity load in tunnel is classified as three levels: level I is for ventilation fan, valve, water pump, lighting and monitoring & control system and direct current screen, etc; level II is for tunnel inspection and repair, and ventilation fan in transformer plant; level III is for air conditioning cold water machines.

On Pudong side and Changxing Island, two transformers are arranged. Two independent 35 kV power circuits are introduced respectively and can be used as spare power for the other through two connection cables. Each route ensures the electricity load of level I and II in the tunnel. For the dynamical and lighting load far away from transformers, the power is supplied through 10 kV power supply network in the tunnel and embedded transformers underneath the tunnel to ensure the long distance power supply quality and reduce energy losses. 6 kV power is supplied for the concentrated ventilation fan. Lighting electricity is supplied by independent circuit in power supply system.

### 4.6.4 *Lighting system*

Light belt is used for lighting in the tunnel. At portal area, natural light transition and artificial light combination is used for lighting. Fluorescence lamp is the main light source in the tunnel. Strengthening lighting uses the high pressure sodium lamp. Taking account of the energy consumption, the application research of LED with high power is being developed. The shift time for emergency lighting in the tunnel should not be larger than 0.1 s and the emergency time is 90 min.

### 4.6.5 Monitoring and control system

The comprehensive monitoring system consists of traffic monitoring system, equipment monitoring system, CCTV monitoring system, communication system, fire automatic alarming system, central computer management system, monitoring and control center. Equipment monitoring system is classified as ventilation subsystem, water supply and drainage subsystem, lighting subsystem, and electrical monitoring subsystem. Monitoring system has access provision for health monitoring system, and expressway net traffic monitoring emergency center, rail traffic monitoring and 220 kV, etc.

The information collected by the tunnel monitoring system, bridge monitoring system, and toll station system is transferred to the monitoring and control center in the tunnel and bridge administration center on Changxing Island. Furthermore, one administration center is arranged at Wuhaogou on Pudong side assisting the daily management and emergency treatment, establishing the three level frame of 'monitoring and control center – administration center – outfield equipment'.

## 4.7 Fire-fighting system

The fire fighting sytem design cosists of balanced and redundant design of safety and function for the entire tunnel structure, building, water supply and drainage and fire fighting, emergency ventilation and smoke discharge, lighting, power supply and other subsystems. The details are as follows:

- Cross passage is arranged every 830 m connecting the upchainage and downchainage tunnel for passenger evacuation with a height of 2.1 m and width of 1.8 m. Three evacuation ladders are arranged between two cross passages connecting the upper and lower level.
- The passive fire proof design uses the German RABT fire accident temperature rising curve. The fire accident temperature is 1,200°C. Fire proof inner lining which ensures the surface temperature of protected concrete segment is not more than 250°C within 120 minutes is selected to protect the arch above smoke duct, smoke duct and crown above the finishing plate. For rectangular tunnel, fire proof material which ensures the structure top plate safety within 90 minutes is selected to protect the top plate and 1.0 m below the top plate. To ensure the passenger evacuation, fire proof bursting resistance fibre is mixed in the concrete bulkhead to achieve no damage of structure when structure is exposed to fire for 30 minutes.
- The ventilation system is designed based on only one fire accident in road tunnel and rail traffic area. The marginal arch area of bored tunnel is used for smoke duct. Special smoke ventilation valve is arranged every 60 m for the smoke ventilation in case of fire accidents on road level. When fire accident occurs in lower level, ventilation fan in the working shaft is turned on to ventilate the smoke to the side of fire source while passengers evacuate towards the fresh air.
- The emergency lighting is arranged on two sides with the same type. As the basic lighting, inserted into the basic lighting uniformly. Meanwhile, normal lighting and emergency lighting are installed in

the cable passage. Evacuation guidance signs are arranged on the two sides of road, cross passage and safety passage. Emergency telephone guidance signs are arranged above the telephones in tunnel.

- Fire water supply at both ends of tunnel is from the DN250 water supply pipe introduced from two different municipal water pipes without fire water pond. One fire fighting pump plant is arranged in working shaft on Pudong side and Changxing Island, respectively. The fire hydrant system is continuous in the longitudinal evacuation passage. Fire hydrant group is arranged every 50 m at one lane side in each tunnel and fire extinguisher group every 25 m. Foam-water spraying system is used in the tunnel which can provided foam liquid continuously for 20 min and arranged every 25 m.
- The communication and linkage of each subsystem of comprehensive monitoring and control system can realize the monitoring, control and test of the whole tunnel such as fan, water pump, electrical and lighting equipment. Fire automatic alarming system can detect the possible hazards such as fire fast, real-time identify and alarm and has the function of passage alarming and tunnel closed. Furthermore, corresponding equipments can be automatically activated to extinguish the fire at early time and organize the hazard prevention to reduce the loss to the minimum extent.

### 5 Φ15, 430 MM SLURRY MIXED TBM

Two large slurry pressurized mixed shield machines with a diameter of 15.43 m are used for the construction of 7.5 m long bored tunnels.

### 5.1 TBM performance and characteristics

The TBM consists of shield machine and backup system with a total length of 13.4 m and weight of 3,250 t, including cutter head system, shield body, tailskin, main drive, erector, synchronized grouting system, transport system, guidance system and data acquisition system and slurry system.

The TBM has excavation chamber and working chamber. During advancing, the air bubble in the working chamber is adjusted through the control unit to stabilize the slurry level thus balance the water/soil pressure in excavation chamber, as shown in Figure 8.

The thrust system consists of 19 groups thrust cylinders with a total thrust force of 203,066 kN. Cutter head is drived by 15 motors with 250 kW power, so the total power is 3,750 kW. Installation position for two spare motors is also provided. Tailskin seal structure is composed of three rows steel wire brushes and one steel plate brush, forming 3 grease chambers. The erector system is centrally supported with 6 freedom



Figure 8. Bulkhead of Mixshield TBM.

degrees. Vacuum suction plated is used to grasp the segment. 6-point grouting is used for simultaneous grouting.

Backup system consists of 3 gantries: gantry 1 housing the power equipment and control system, gantry 2 housing 3 cranes and bridge section for segment, road element, and other construction material transport, gantry 3 is pipe laying gantry for carrying the extension of the different services such as cable hose, slurry, air and industrial water pipes.

Excavated soil is transported from excavation chamber to the slurry treatment plant (STP) through the slurry pipe in the slurry circulation system. After separation by the treatment equipment, excavated soil with large size is separated and then the recycled slurry is pumped back into excavation chamber and working chamber.

# 5.2 Adaptability to the 'large, long and deep' characteristics

For the TBM construction, firstly the project and crew safety should be ensured. The key for safety of TBM is to protect the cutter head and tailskin, mainly including cutter head design, main bearing seal and tailskin seal assurance. Furthermore, the maintenance and repair of these parts are risk and difficult to access, so the inspection and possibility for maintenance in case of failure must be considered.

### 5.2.1 Cutter head and cutting tools

Cutter head is for soft ground and can be rotated in two directions. The cutter head is pressure resistant steel structure and specific wear protection is arranged for the periphery area. Special wear protection is also designed for cutting tools.

The closed type cutter head is designed with 6 main arms and 6 auxiliary arms, 12 large material opening and 12 small material opening. The opening ratio is around 29%. 209 cutting tools are arranged on

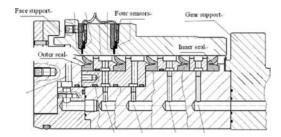


Figure 9. Main bearing seal arrangement.

the cutter head, among which 124 fixed scrapers, 12 buckets, 2 copy cutters, 7 replaceable center tools and 64 replaceable tools.

The scrapers are custommade large tools with features of 250 mm width, wear-resistance body and high quality carbide alloy cutting edges whose angle matches the parameter of excavated ground. The scrapers at the edge are used to remove the excavated soil at edge and protect the cutter head edge from direct wear. Copy cutter can automatically extend and retract. The multiple over-cut areas can be setup in the control cabin and corresponding cutting tools position are displayed. The replaceable cutting tools have special seal to prevent the slurry at the front surface enter into the cutter head chamber. During operation, the workers can enter the cutter head chamber to replace the cutting tools under atmospheric condition with high safety, good operation possibility and low risk.

In order to avoid clogging at cutter head center, the opening at center is designed as chute to ease the material flowing. Meanwhile, one bentonite hole is arranged at each opening to ease flushing in case of clogging.

#### 5.2.2 Main bearing seal

Two sets seal system are arranged for the main bearing seal design. The outer seal is for the excavation chamber side and inner seal for the shield body with normal pressure. The special seal combination can bear a pressure of 8.5 bar.

Outer seal is to separate the main bearing and excavation chamber. Seal type is axial seal with large diameter, totally 4 lip seals and one pilot labyrinth, thus forming 4 separate areas, as shown in Figure 9.

The inner seal one the gear box side is special axial seal which can carry the pressure of gear chamber.

The seal system has grease lubrication and leakage monitoring system which can monitor the grease amount by pressure and flow monitoring. The seal system has been proved successfully in many projects for several years and become a standard configuration.

#### 5.2.3 Tailskin

The tailskin is sealed off by 3 rows steel wire brush and 1 steel plate brush, as shown in Figure 10.

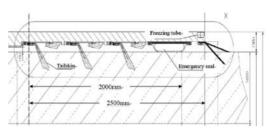


Figure 10. Tailskin structure.

Furthermore, 1 emergency seal is arranged between the 3rd row steel wire brush and the steel plate brush. The emergency seal has the function to protect the ring building area from water ingress while changing the first three steel brush seals. Due to no practical application references of this technology, modeling test has been carried out for the emergency seal installation to confirm the reliability of the emergency seal when the inflatable seal is pressurized to 1 MPa.

Simultaneous grouting lines are arranged at the tail skin, including one standard grout pipeline and one spare pipeline for filling the annulus gap outside the segment after excavation. Furthermore, 19 chemical grout pipes are added for special hardening grout (simultaneous slurry penetrating into cement) or polyurethane for leakage block in emergency condition.  $19 \times 3$  grease pipes have the function of steel wire brush lubrication and tail skin sealing. The seal system is controlled from the cabinet in automatic and manual modes through time and pressure control.

Furthermore, freezing pipelines are arranged at the tailskin to ease the ground treatment by means of freezing measures in case of leakage and ensure the seal treatment and repair safety.

### 5.2.4 Man lock and submerged wall

During long distance advancing, there is a possibility of operation failure of mixing machine due to large obstacles blocking such as stones, main bearing seal replacement due to wear, submerged wall closed or leakage examination in the air bubble chamber. These maintenance and repair work need workers access the air bubble chamber with a pressure up to 5.5 bar. Therefore, two man locks are arranged to ensure the maintenance and repair workers can access.

The main chamber of manlock can house one 1.8 m stretcher. Under pressure-reducing condition, the medical staff can access the main chamber and organize rescue in case of emergency. Meanwhile, the other man lock can transport the tools, material and equipment from TBM to the air bubble chamber.

The man lock is equipped with poisonous gas detection system which can take the sample of enclosed gas in the man lock. The system information will be displayed at the working position where outside staff is. The man lock also provides the flange connection. Once the rescue and injuries enters into temporary rescue chamber, the temporary chamber can be disassembled fast and transported out of the tunnel, connected with large medical chamber for the convenience of medical work to rescue.

The submerged wall uses hydraulic drive and is equipped with air pressure seal strip. When normal operation in the working chamber is needed, the submerged wall can be closed thus the excavation chamber and working chamber can be separated, and then the valve can be opened for reducing the pressure. At this time, pipe for supplementing slurry which penetrates working chamber can maintain the slurry pressure in the excavation chamber.

## 6 TUNNEL CONSTRUCTION METHOD

## 6.1 Overall arrangement and time schedule

Based on the overall programming, the construction of working shafts, bored tunnel, synchronous construction of road structure, operation equipment installation and commissioning are the main works and secondary works such as receiving shaft and crosspassage in parallel.

In May 2006, the launching shaft and onshore structures on Pudong side were completed and site assembly of two TBMs started. The east tunnel starts advancing in September 2006, while west tunnel in January 2007. During construction of these two tunnels, the prefabricated road element erection and TBM advancing are synchronous, which on one hand resist the tunnel floating during construction stage and on the other hand provide special truck passage for segments, prefabricated road elements and materials to realize the fast bored tunnel construction. In parallel with bored tunnel construction, the road deck structure construction is also carried out 200-250 m back from segment erection and top smoke duct will start construction in January, 2008, forming gradually working flow in tunnel. After west tube TBM advancing 3 km, the first crosspassage started construction in October, 2007. After the tunnel is through, final connection work of working shaft and road structure is carried out and operation equipment and finishing and pavement work will start.

## 6.2 Main critical technical issues during bored tunnel construction

## 6.2.1 TBM launching and arriving technology

## 6.2.1.1 TBM launching

## (1) Tunnel eye stabilization

3-axial mixing pile and RJP injection procedure is used surrounding the working shaft to stabilize the ground forming a stabilized area of 15 m in length. 6 dewatering wells for bearing water are supplemented

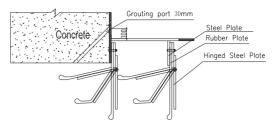


Figure 11. Water stop tank sketch.

beyond the treated ground area and holes are bored for grouting the annulus to ensure the safety during tunnel gate removal. These three measures application has achieved good performance. During TBM launching, the treated soil is stable.

## (2) Tunnel annulus seal

The diameter of tunnel eye is up to 15,800 mm. To prevent the slurry enters into the working shaft from the circular build gap between tunnel eye and shield or segment during launching thus affect the establishment of front face soil and water pressure, good performance seal water stopping facility is arranged. The facility is a box structure with 2 layers water stop rubber strip and chain plate installed, as shown in Figure 11. The outside chain plate is adjustable with 50 mm adjustment allowance. Furthermore, 12 grout holes are arranged uniformly along the outside between two layer water stop on the box for the purpose of sealing in case of leakage at the tunnel eye. The outer end surface of water stop facility shall be vertical to the tunnel axis.

## (3) Back support for TBM

The back up shield support includes 7 rings, among which -6 is steel ring composed of 4 large steel segments with high fabrication quality to ensure the circularity and stiffness of the reference ring, as shown in Figure 12. After precise positioning of steel ring, it is supported on the concrete structure of cut and cover tunnel by 19 steel struts with a length of 1.2 m. Other 6 minus closed rings segments are assembled with staggered joint. Inserts are embedded on the inside and outside surface. After each ring building, the circumferential ring and longitudinal ring are connected with steel plate to improve the integrate stiffness and ensure the circumferential plane of each minus ring shall be vertical to the design axis.

## 6.2.1.2 TBM receiving

## (1) Arrangement in receiving shaft

Before TBM receiving, the diaphragm between receiving shaft and cut & cover tunnel and the diaphragm in the receiving shaft between upchainage and downchainage tunnel shall be completed to make the receiving shaft as an enclosed shaft structure. Then MU5 cement mortar is cast in the working shaft with a height

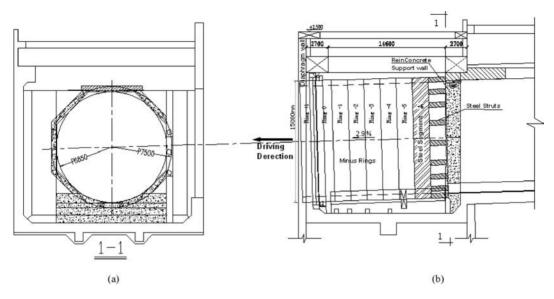


Figure 12. Back supports for TBM.

of 3 m higher than the TBM bottom. Steel circumferential plate is arranged along the steel tunnel annulus. The inner diameter of steel plate is 5 cm larger then TBM. 18 grout holes are arranged surrounding the tunnel annulus and inflatable bag is installed in the tunnel eye.

#### (2) TBM arriving

When the cutting surface of TBM is close to the concrete wall of tunnel eye, advancing is stopped. Then pump water in the receiving shaft to the underground water level. Meanwhile, inject double grout into the annulus 30 m back from tailskin through the preset grout hole on the segment to stabilize the asbuilt tunnel and block the water/soil seepage passage between untreated ground and TBM.

After above work, the TBM starts excavation of C30 glass fibre reinforced concrete and accesses the working shaft. The cutting surface accesses into the working shaft and the cutter head will cut the MU5 cement mortar directly and sit on the mortar layer. During accessing into the working shaft, polyurethane is injected through the chemical grouting holes.

## (3) Tunnel eye sealing and water pumping

When 2/3 of TBM accesses the receiving shaft, water pumping is started. After pumping the water in the working shaft, continue the TBM advancing and inject the grout timely. When the TBM is in the working shaft, fill air in the inflatable bag in time to make the inflated bag seal the circumferential gap. Meanwhile, grouting is performed through the 18 holes on the tunnel annulus. Grout material is polyurethane. After the gap is fully filled with the grout, the air in inflated bag could be released slowly under close observation. If any water leakage is observed, the polyurethane shall be injected again for sealing.

When the tunnel gate ring is out of the tailskin, the welding work between ring steel plate, seal steel plate and embedded steel plates shall be done immediately to fill the gap between tunnel gate ring and tunnel.

## 6.2.2 TBM advancing management

## 6.2.2.1 Main construction parameters

During TBM construction, the construction parameters shall be defined and adjusted based on theoretical calculation and actual construction conditions and monitored data to realize dynamical parameter control management.

The advancing speed at beginning and before stop shall not be too fast. The advancing speed shall be increased gradually to prevent too large starting speed. During each ring advancing, the advancing speed shall be as stable as possible to ensure the stability of cutting surface water pressure and smoothness of slurry supply and discharge pipe. The advancing speed must be dynamically matching with the annulus grout to fill the build gap timely. Under normal boring condition, the advancing speed is set as 2–4 cm/min. If obstacles varying geological conditions are experienced at the front face, the advancing speed shall be reduced approximately according to actual conditions.

Based on the theoretical excavation amount calculated from formula and compared to actual excavated amount which is calculated according to the soil density, slurry discharge flow, slurry discharge density, slurry supply density and flow, and excavation time, if the excavation amount is observed too large, the slurry density, viscosity and cutting face water pressure shall be checked to ensure the front surface stability.

In order to control the excavated soil amount, the flow meter and density meter on the slurry circuit shall be checked periodically. The slurry control parameters are: density  $\rho = 1.15 - 1.2 \text{ g/cm}^3$ , viscosity = 18–25 s, bleeding ratio <5%.

Single type grout is used to inject at 6 locations, which is controlled by both pressure and grout amount. The grout pressure is defined as 0.45–0.6 Mpa. Actual grout amount is around 110% of theoretical build gap. 20 h-shear strength of grout shall not be less than 800 Pa and 28 day strength shall be above the original soil strength.

## 6.2.2.2 Shallow cover construction

At the launching section, the minimum cover depth is 6.898 m, i.e. 0.447 D, which is extremely shallow. To ensure the smooth advancing, 1-2 m soil is placed above the top. Meanwhile, in order to prevent slurry blow-out, leakage-blocking agent is mixed in the slurry and surface condition is closely monitored.

## 6.2.2.3 Crossing the bank of Yangtze River

Before the TBM crossing, the terrain and land feature in the construction surrounding area are collected, measured and photographed for filing. 155 monitoring points are arranged along the bank in 7 monitoring sections. During TBM crossing, the pressure is set according to the water pressure at excavation surface calculated for each ring. The slurry parameter is also adjusted timely based on the surface monitoring information. Grease injection at tail skin is performed well to avoid leakage and synchronous grout amount and quality are strictly controlled.

## 6.2.2.4 Adverse geological condition

### (1) Shallow gas

When the TBM is crossing the deposit on Pudong side, methane gas may be experienced in the shallow area. At this time, the ventilation in the tunnel shall be increased to ensure good ventilation conditions of TBM. The concentration test of methane and combustible gas are carried out.

### (2) Lens

Prior to the construction, geological investigation is carried out to learn the general location of prism. During construction, the TBM is set with suitable speed to cross the stratum as fast as safely possible.

## (3) Bored hole

Due to the tunnel alignment adjustment, 9 geological bored holes will be experienced along the TBM advancing. During crossing, slurry with large density is used and polyurethane is injected surrounding the tunnel after crossing.

## 6.2.3 Quality assurance technical measures for large tunnel

## 6.2.3.1 Segment prefabrication

Nine sets steel formwork with high preciseness are used for segment prefabrication to fulfill the technical requirement to segment such as allowable width tolerance  $\pm 0.40$  mm, thickness tolerance +3/-1 mm, arc length  $\pm 1.0$  mm, circular surface and end surface plainness  $\pm 0.5$  mm. In order to control prefabrication preciseness strictly and ensure the production quality, special laser survey system is introduced to conduct accurate measurement of segment profile dimension beside traditional survey measurement tools and segment trial assembly.

Fly ash and slag are mixed in the concrete for segment prefabrication. Strictly concrete casting, vibrating and curing procedures are used to control cracks and achieve the water proofing and durability requirement.

## 6.2.3.2 Segment assembly

The segment assembly shall satisfy the fitted tunnel design axis requirement by segment selection (rotation angle) and meanwhile make the longitudinal joint not on the same line. During the whole assembly process, for straight line, the principle is to erect on left and right at intervals. For curved section, the suitable segment rotation angle shall be selected based on TBM attitude, and segment lipping data.

Secondly, the relative dimension between segment and shield shall be checked to correct the positioning of each ring segment.

Then, each segment building shall be closely contacted. The ring plane and 'T' joint shall be even.

Finally, strictly control the lipping of ring. When the segment lipping exceeds the control value, the rotational angle of segment shall be adjusted timely to ensure the verticality between segment and tunnel axis.

## 6.2.3.3 Floating-resistance of tunnel

Due to the tunnel diameter up to 15 m, the floating resistance and deformation control during construction for large diameter tunnel are very challenging. The technical measure is mainly to improve the synchronous grouting management. Mortar type grouting material with cementation property is injected at multi-points. Furthermore, grout package with certain strength shall be formed surrounding the tunnel timely to resist the tunnel upfloating. Meanwhile, the tunnel axis shall be strictly controlled during construction and the tight connect between segments shall be improved to achieve the tunnel-floating resistance.

## 6.2.3.4 Ground deformation control

The ground settlement during TBM construction is mainly contributed by the front surface slurry pressure setup, annulus grouting and shield body tamper.

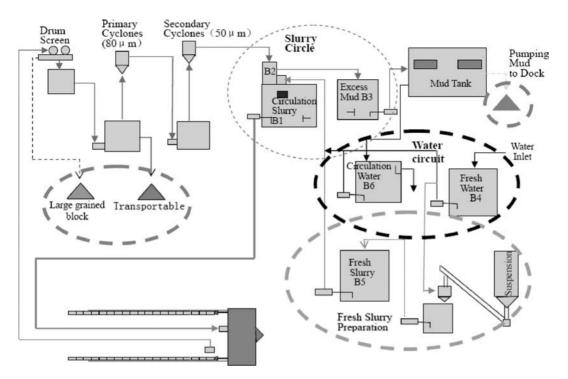


Figure 13. STP system flow chart.

Therefore, the ground settlement variation can directly reflect the TBM construction parameters setup. The crew can correct the construction parameter based on settlement monitoring to increase the deformation.

## 6.2.4 Back-up technology for long distance TBM construction

## 6.2.4.1 Slurry treatment and transport

The slurry separation system consists of subsystems of treatment, conditioning, new slurry generation, slurry discharge and water supply; with a capacity of  $3.000 \text{ m}^3/\text{h}$  to fulfill the advancing requirement of 45 mm/min, as shown in Figure 13. Based on the geological conditions along the tunnel alignment, the treatment system selects 2 level treatment methods. The initial treatment uses two rolling shieve to separate soil with a size of larger than 7 mm. For secondary treatment, firstly grain with a size of large than 75 is separated by  $4 \times \phi$ 750 mm cyclones and then grain with a size of large than  $40 \,\mu m$  is separated by  $12 \times \phi 300$  mm cyclones. The slurry spilled at the top of cyclone is transported to conditioning tank for reuse. After adjustment, the density of supplied slurry is 1.05-1.35 g/cm<sup>3</sup>. The maintained optimal value is between 1.20 and 1.30 and d50 is between 40 and 50 µm. The STP system circulation efficiency is up to 70%. Discharged slurry and waste is transported to the bardge at riverside by pipes and trucks. The slurry

supply pipe has a diameter of 600 mm and discharge pipe 500 mm. To ensure the long distance slurry supply velocity of 2.5 m/s and discharge velocity of 4.2 m/s to avoid slurry settlement in pipe and maintain not too high pressure in the pipe, one relay pump is arranged every 1 km. The pressure at pump outlet is controlled within 10 bar.

## 6.2.4.2 Axis control and construction survey guidance

Static measurement with GPS control net is used for surface control survey. For elevation control, GPS elevation fit method is used for elevation transfer. Part of basic traverse mark every 500 m is selected as main traverse. In the tunnel, level II subtraverse is used for the plane control, i.e. construction traverse and control parallel traverse. The control mark has a spacing of 600–900 m. The elevation control survey in tunnel uses level II. The fixed level mark is arranged with a spacing of 80 m.

6.2.4.3 Construction ventilation and fire protection Due to the large diameter, long distance and 'W' longitudinal slope, especially when the TBM is advancing with a upgrading slope, the heat and humidity generated at the working face can not be discharged naturally thus concentrate at the working face in a shape of fog. Meanwhile, heavy trucks for construction material transport also cause a large amount of waste air in the tunnel. Bad environment will have unfavorable influence on TBM equipment and crew, and also affect the smooth progressing of survey activity.

During construction stage, 2 special axial fans (SDF-No18) are arranged on the surface to provide fresh air to the space below road deck in the tunnel, then the relay fan and ventilation system equipped on the gantry will transport the fresh air to working surface. Meanwhile, other ventilation equipment on the gantry provides fresh air to main secondary equipments of TBM such as transformer, hydraulic equipment and electrical installations.

Adequate fire extinguishers are arranged in the shield and gantry and also oxygen, poisonous gas protection mask are equipped. Fire extinguisher is equipped on each transport truck. Safety staff is equipped with portable gas analysis device for check the air quality in tunnel every day.

## 6.2.4.4 Material transport

Segment, grout and prefabricated elements, etc are transported to the working area by special trucks from ramp area, through cut & cover tunnel and road deck which is constructed synchronously. Truck transport can avoid the derailing problems during traditional electrical truck transport. Furthermore, the truck has two locos, so the transport efficiency is high.

Prefabricated road element is transported to the gantry 2 by trucks and then lifted and erected by the crane on the bridge beam. Segments are transported to gantry 2 and then transferred to the segment feeder by the crane on the bridge beam and then transported to erection area.

# 6.2.5 Critical equipment examination and replacement technology

## 6.2.5.1 Main bearing sealing

Four supersonic sensors are installed in the seal arrangement for monitoring the main seal wear condition. Once the abrasion reaches certain value or grease leakage is monitored in the tank, it indicates the main seal needs to be rotated to another oritentation.

Once the seal wear is observed beyond preset value, the surface could be moved to ensure the replacement of main bearing seal. During replacing, the slurry in the chamber must be drained and provide effective support to excavation face. The operation staff shall go to the slurry camber to replace the seal under certain pressure.

## 6.2.5.2 Abrasion measurement and replacement of cutting tools

The system will be installed on 8 selected scraper positions as well as on two bucket positions. It will be connected to a plug at the rear of the cutting wheel to allow for simple condition diagnosis from a readout device. Conductor loop is embedded in the device. The wear condition of cutting tools can be indicated by checking the closed/open status of loops.

The worker accesses the cutting wheel arms from the center of the main drive. The worker installs the lowering/ lifting frame (with bolts) and screws it to the fixing plate of the tool. The fixing plate is then unscrewed. The worker will then lower the tool using the frame (with bolts). The pressure-tight gate will be closed down. The worn out tool shall be then exchanged with a new one. The tool will be lifted to position behind the gate. The gate will be opened. Then the tool will be put in its final position. The fixing plate is then screwed to the tool support. The frame is transferred to the next tool.

6.2.5.3 Tail seal and steel wire brush replacement When the leakage is experienced at tail skin, and steel brush is defined to be replaced necessarily, open the emergency sealing and erect special segments. Strengthen the surrounding soil at tail skin with freezing method and then replace first 2 or 3 rows steel brush.

## 6.3 Synchronous construction of road deck

The synchronous construction of road structure includes erection of road element, segment roughening and drilling for inserting rebar, prefabrication of two side ballast, insitu cast corbel and road deck on two sides. According the variation and trend of asbuilt ring deformation and settlement, and the construction progress of 12 m (6 rings) per day and based on the requirement of deformation joint arrangement every 30 m, the construction is organized and arranged as flowing operation every 15 m. As shown in Figure 14, the basic construction procedure is as follows:

- Road element installation, 25 rings later than segment erection.
- Segment roughening includes the junction surface between ballast and segment and segment inner surface at corbel. The insert bar placing includes the  $\Phi 16$  bar at ballast and  $\Phi 20$  bar at corbel. The roughening works at ballast position is carried out at gantry 2, and the roughening operation platform at corbel is fixed to gantry 2. Insert bar placing is following gantry 3. The roughing machine is equipped with dust suction facility which can eliminate the dust to maximum extent.
- Reinforcement placing, formwork erection and concrete casting for ballast is carried out at 15 m behind the gantry 3 and 15 m more behind for corbel, and then another 15 m for road deck. Road deck concrete casting works are located at 250 m–300 m from the segment erection area. After casting, the curing with frame lasts 3 days and formwork is removed on the 4th day. After 28 days curing, the road deck can be open to traffic. During curing, the

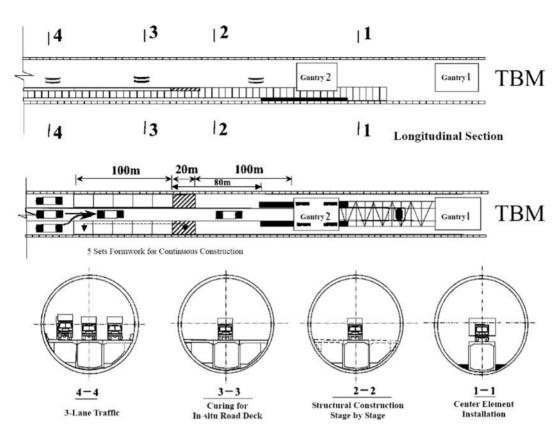


Figure 14. Synchronous construction flow chart.

road deck area is separated. Concrete mixing truck is used for concrete casting.

## 6.4 Cross passage construction

The cross passage which connects the two main tunnels has a length of around 15 m and diameter of 5 m. The construction will be by freezing method for soil strengthening and mining method for excavation.

The freezing holes are arranged as inside and outside rows which are drilled from two sides. The freezing is done from one side or both sides. Inside row holes are drilled from upchainage tunnel, 22 in total and outside row holes are drilled from downchainge tunnel, 18 in total.

Mining method will be used for excavation by area division. Firstly, pilot with a horn opening is excavated, and then the cross passage is excavated to design dimension. The fullsection excavation is done with a step of 0.6 m or 0.8 m.

When the main structure concrete strength reaches 75%, enforced thawing will be carried out. The hot brine for thawing circulates in the freezing pipe and

the frozen soil is thawed by section. Based on the informational monitoring system, the soil temperature and settlement variation is monitored. Grouting pipe is arranged at shallow and deeper area for dense grouting. The overall principle for thawing is to thaw the bottom part, then middle part, and lastly the top part, as shown in Figure 15. When thawing by section is done in sequence, one section is being thawed and subsequent sections maintain the freezing for the purpose of maintaining the cross passage structure and main tunnel as an integrated part thus settlement avoidance before the section grouted.

## 6.5 Land connections construction

The profile dimension of working shaft is  $22.4 \times 49$  m, with a depth of 25 m. 1.0 m thick diaphragm with a depth of 45 m is used for retaining structure. Open cut is used for excavation. The support system consists of 5 layers reinforced concrete and 1 layer steel support. Inside the pit, 3 m below the bottom, injection is done interval to make the strength not lower than 1.2 MPa. 13.5–16.0 m outside the working shaft is treated. For Thawing Area

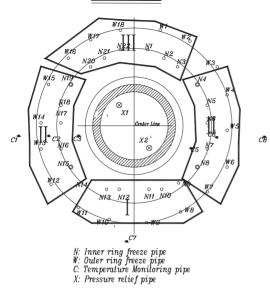


Figure 15. Divided thawing area of cross passage.

diaphragm at the TBM accessing into the receiving shaft, GFPR is used instead of normal reinforcement so that the TBM can cut the retaining wall directly and thus avoid the reinforcement cutting and tunnel eye concrete removal, which simplifies the construction procedure, accelerates construction progress and reduces the construction risk.

The excavation depth of pit for Pudong cut-andcover is 23.1-9.9 m, and Changxing cut-and-cover 17.2 m-8.4 m. According to the excavation depth, diaphragm with thickness of 1.0 m, 0.8 m and 0.6 m is selected respectively. The support system is composed of reinforced concrete support and steel support. 3 m underneath the pit bottom is strengthened by rotating injection and also the junction between working shaft and cut-and-cut outside the pit to ensure the pit excavation stability.

The ramp is open cut with a slope of 1:3. The slope is protected through green planting in the reinforced concrete grid which is anchored in soil by anchors to prevent from sliding. In order to avoid slope sliding, the slope is strengthened by cement mixed piles with a diameter of 700 mm.

### 7 CONCLUSION

During the process from planning to implementation. Shanghai Yangtze River Tunnel has experienced various challenges. Technical support of tunnel construction from China and abroad is provided. With independently developed and owned IPR and featured TBM tunnel construction theory and core technology is established, forming the core technology of large and long river-crossing TBM tunnel in China. Special technical issues such as lining structure design of extremely large tunnel, long distance TBM construction and hazard prevention system for long and large tunnel achieve to be internally state-of-art. Relevant standards, codes, guidance, specification and patent technology are developed to improve the technical system of tunnel construction in China and upgrade the internal competence of tunnel engineering.

## REFERENCES

- Cao, W.X. et al. 2006. Shanghai Yangtze River Tunnel Project design. Shanghai Construction Science and Technology 5: 2–6.
- Chen, X.K. & Huang, Z.H. 2007. Shanghai Yangtze River Tunnel TBM cutting tools wear detection and replacement technology. *The 3rd Shanghai International Tunneling Symposium Proceedings: Underground project construction and risk provision technology*: 152–157. Tongji University Publication Company.
- He, R. & Wang, J.Y. Shanghai Yangtze River Tunnel synchronous construction method statement. *The 3rd Shanghai International Tunneling Symposium Proceedings: Underground project construction and risk provision technology*: 168–177. Tongji University Publication Company.
- Sun, J. & Chen, X.K. 2007. Discussion of TBM selection for Shanghai Yangtze River Tunnel. *The 3rd Shanghai International Tunneling Symposium Proceedings: Underground project construction and risk provision technology*: 91–98. Tongji University Publication Company.
- Yu, Y.M. & Tang, Z.H. 2007. Shanghai Yangtze River Tunnel construction survey technology. *The 3rd Shanghai International Tunneling Symposium Proceedings: Underground project construction and risk provision technology*: 158–167. Tongji University Publication Company.
- Zhang, J.J. et al. 2007. Shanghai Yangtze River Tunnel TBM launching construction technology. The 3rd Shanghai International Tunneling Symposium Proceedings: Underground project construction and risk provision technology: 144–151. Tongji University Publication Company.

## Underground construction in decomposed residual soils

## I.M. Lee

Department of Civil Engineering, Korea University, Seoul, Korea

## G.C. Cho

Department of Civil and Environmental Engineering, KAIST, Daejeon, Korea

ABSTRACT: Large scale underground construction projects, including subway construction projects in six major cities, have been ongoing in Korea, where residual and granite soils are the most common soil type. Characteristics of decomposed granite soils are different from those of pure sand and/or clay. This paper presents an overview of geotechnical aspects of underground construction in urban areas where mostly decomposed residual soils are present, focusing on mechanical properties, apparent earth pressure, effect of groundwater, and effect of spatial variability in geotechnical properties. Although several important aspects are theoretically, numerically, and experimentally discussed herein, it remains a challenge to fully understand residual soils, particularly in relation to the practice of underground construction, because of their complexity and richness.

## 1 INTRODUCTION

Large scale subway construction projects have been ongoing in six major cities in Korean peninsular. In particular, underground construction work of subway line No.9 is being carried out under the Seoul Metropolitan Government along with extension of subway lines No.7 and No.3. Construction of a new subway line (Line No.2) will be launched in Incheon this year and will be finished before the Asian Games are held in 2014.

Residual and granite soils are the most common soil type in Korea. Characteristics of decomposed granite soils are different from those of sand and/or clay. Their mechanical properties and behaviors vary depending on the parent rock types and weathering processes. Moreover, the profile of the ground in Korea is generally not uniform, isotropic, or homogeneous; multilayered conditions are common, with ground being composed of successive layers of fill and/or sedimentary layers, weathered residual soils, and soft to hard rock. Therefore, conventional/classic soil mechanics cannot be directly applied to these ground conditions. In order to provide data and methodologies to enhance underground construction in areas characterized by the predominant presence of decomposed residual soils, this paper presents an overview of geotechnical aspects of underground construction in urban areas where decomposed residual soils are the main ground component, focusing on mechanical properties, apparent earth pressure, effect of ground water, and effect of spatial variability in geotechnical properties.

# 2 MECHANICAL PROPERTIES OF RESIDUAL SOILS

## 2.1 General

We have intensively studied the characteristics of the following two residual soils: Shinnae-dong and Poidong soils. The characteristics of the two soils are documented in Table 1 (i.e., the type of mineral, compaction properties, plasticity, and soil classification). The parent rock of the Shinnae-dong soil is a granite while that of the Poi-dong soil is a banded biotite gneiss. The Shinae-dong soil is closer to a cohesionless soil, predominantly consisting of primary minerals, with only about 10% of fine particles. On the other hand, the Poi-dong soil shows clay-like characteristics due to a large percentage of fine particles and secondary minerals. The particle size distributions of the two soils are shown in Figure 1.

Lee (1991) studied the behavior of a Bulam soil in his Ph.D. dissertation. The characteristics of the Bulam soil are similar to those of the Shinna-dong soil, since two areas are very close to each other and share the same rock origin. Kim (1993) studied the mechanical behavior of Andong and Kimchun soils. These soils are classified as SM in the Unified Soil

Table 1. Characteristics of two residual soils.

Characteristics	Poi-dong	Shinnae-dong
Primary minerals		
Quartz (%)	17.7	33.3
Feldspar (%)	15.0	50.0
Mica (%)	9.8	9.0
Secondary minerals		
Kaolinite (%)	23.5	6.0
Illite (%)	20.7	
Vermiculite (%)	8.4	2.0
Chlorite (%)	4.5	
Montmorilonite (%)		
Porosity	0.409	0.358
Maximum dry density (kN/m <sup>3</sup> )	16.68	18.64
Percent passing #200 sieve (%)	47.36	10.05
Plasticity		Nonplastic
LI	34.0	Ĩ
PL	19.84	
PI	14.16	
Specific gravity	2.74	2.65
USCS	SC	SW-SM

Note: LI = liquidity index; PL = plastic limit; PI = plasticity index; USCS = Unified Soil Classification System.

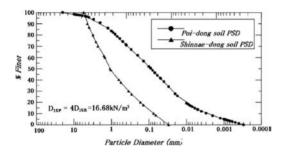


Figure 1. Particle size distribution of two residual soils.

Classification System with the percent passing a #200 sieve being  $14 \sim 17\%$ , and are also included in the following discussion on mechanical characteristics of residual soils.

## 2.2 Strength characteristics

Figure 2 presents a summary of the peak internal friction angle of each residual soil. The internal friction angle decreases with an increase of fine contents.

Because of capillarity, partial saturation affects the strength of residual soils. Lee et al. (2005) performed triaxial tests to obtain the strength properties of unsaturated residual soils. Figure 3 presents typical results of failure envelopes at different matric suctions. The internal friction angle as well as the apparent cohesion increase with an increase in the matric suction (i.e., when the soil is unsaturated).

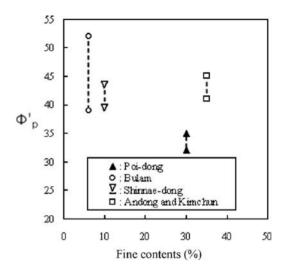


Figure 2. Peak internal friction angle versus fine contents passing the #200 sieve.

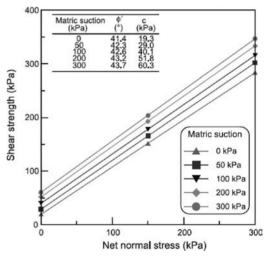


Figure 3. Failure envelopes at different matric suctions.

Lee et al. (2002) performed shear tests on unsaturated residual soils and found that the apparent cohesion can increase from 20 kPa at a saturated condition (i.e., matric suction = 0 kPa) up to 200 kPa at an unsaturated condition (i.e., matric suction = 400 kPa).

As an unsaturated soil is re-saturated, its apparent cohesion can be eliminated. Thus, during tunnel construction, cohesion loss can be induced by resaturation (e.g., seepage hindrance, drainage clogging and groundwater change), and may result in tunnel face instability. Utilizing the limit equilibrium analysis proposed by Leca & Dormieux (1990), as shown in Figure 4, the required support pressure to stabilize the

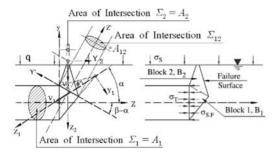


Figure 4. Collapse mechanism of a tunnel face with two conical blocks.

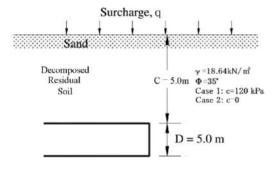


Figure 5. Ground condition for limit equilibrium analysis.

tunnel face can be calculated by replacing the specific apparent cohesion with a value of zero. As an example, the ground condition of a site is shown in Figure 5. The required support pressure  $\sigma_{\tau}$  increases from zero up to  $\sigma_{\tau} = 9.0$  kPa as the cohesion decreases from 120 kPa to 0 kPa. The results show that the apparent cohesion is a key factor in tunnel face stability.

#### 2.3 Deformation characteristics

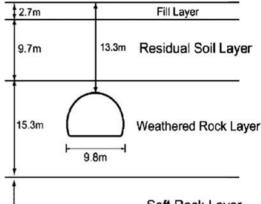
One of the most difficult tasks in geotechnical engineering is estimating the deformation-related soil properties properly. Typical material properties commonly used at the design stage in Korea are summarized in Table 2.

Cho et al. (2006) proposed an analytical method to estimate soil parameters from relative convergence measurements. As an example, the ground conditions of the Busan subway site are shown in Figure 6. Initial estimates are taken from Table 2. The crown and sidewall convergence data measured by a tape extensometer, presented in Figure 7, are used as observed values. Results obtained from back-analyses are summarized in Table 3 along with initial estimates. There is a large discrepancy between the initial input properties and the properties obtained from back-analyses. In particular, the initial inputs of Young's modulus and earth pressure coefficient at rest of the residual

Table 2. Typical material properties.

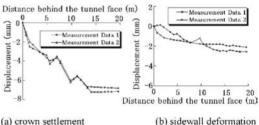
Layer	E (MPa)	μ	γ (kN/m <sup>3</sup> )	K <sub>0</sub>	c (kPa)	φ (°)
Filling	19.6	0.35	18.6	0.5	0	35
Residual soil	29.4	0.33	18.6	0.5	49.1	35
Weather rock	196	0.23	21.6	0.5	98.1	35
Soft rock	981	0.2	23.5	0.7	196	40

Notation: E = Young's modulus,  $\mu$  = Poisson's ratio,  $\gamma$  = unit weight of soil,  $K_0 =$  lateral earth pressure coefficient, c = apparent cohesion, and  $\phi =$  internal friction angle.



Soft Rock Laver

Figure 6. Ground conditions of the Busan subway site.



(b) sidewall deformation

Figure 7. Relative convergence behind tunnel face.

Table 3. Results of parameter estimation.

Properties	E <sub>r</sub> (MPa)	E <sub>w</sub> (MPa)	K <sub>ow</sub>
Initial input	29.4	196	0.5
Back-analysis	83.3	210	0.76

Notation:  $E_r$  and  $E_w$  = Young's moduli of the residual soil and weathered rock, respectively; and Kow = the earth pressure coefficient of the weathered rock.

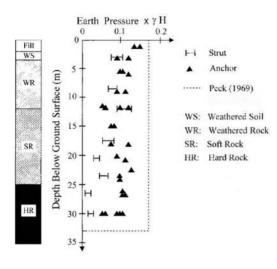


Figure 8. Apparent earth pressure distribution for braced and anchored walls.

soil are too small. This example clearly demonstrates the importance of the observational method in tunnel engineering.

### **3 APPARENT EARTH PRESSURE**

## 3.1 Apparent earth pressure in braced and anchored walls

Design of an in situ wall system requires a lateral earth pressure distribution behind the wall to estimate support loads and wall bending moments. One of the most well-known apparent earth pressures is that proposed by Peck (1969); however, his suggestion is only applicable to either ground that is purely sand and/or clay, and cannot be directly applied to cohesive soils that have cohesion as well as an internal friction angle, or to multilayered ground conditions.

Many Korean researchers have attempted to collect field data to propose the earth pressure in multilayered ground (for example, Lee & Jeon 1993, Yoo 2001). Figure 8 presents a typical set of results, showing the apparent earth pressure distribution of a 33m deep excavation site along with the distribution proposed by Peck (1969) (Yoo 2001). The actual (measured) earth pressure is about 68 to 83% of Peak's. Figure 9 shows the maximum earth pressures obtained from 62 excavation sites. In this data, the weighted average values of the internal friction angle and the unit weight of soil are used for the multilayer ground. The average value of the measured apparent pressures is approximately 75% of Peck's suggestion (i.e., earth pressure =  $0.65 \text{ K}_a \gamma \text{H}$ ).

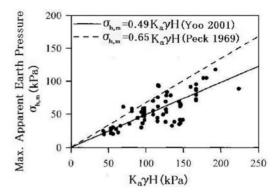


Figure 9. Maximum apparent earth pressure versus  $K_a \gamma H$ .

#### 3.2 Apparent earth pressure in a vertical shaft

It is well known that the earth pressure acting on a vertical shaft is less than that on a retaining wall, because of the three dimensional arching effect. The existing equations of earth pressures acting on vertical shafts consider only either purely cohesionless or cohesive soils. These solutions are not directly applicable to estimation of earth pressures for multi-layered ground.

Lee et al. (2007) proposed an equation to estimate earth pressures in multi-layered ground, assuming that the failure shape is conical, as shown in Figure 10(a). For equilibrium of horizontal forces and vertical forces, as shown in Figure 10(b), the earth pressure ( $P_i$ ) can be expressed as follows

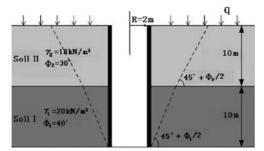
$$P_i = K_w \quad \sigma_v \tag{1}$$

$$\sigma_{v} = (q - \frac{\gamma}{T})e^{-Tz} + \frac{\gamma}{T}$$
<sup>(2)</sup>

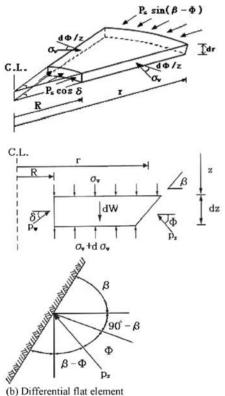
and

$$T = \frac{2\pi}{A} \left[ R \tan \delta \quad K_w + \frac{1}{\tan(\beta - \phi)} \left\{ K_w R + \lambda (r - R) \right\} \right]$$
(3)

where  $K_w =$  the coefficient of radial earth pressure,  $\lambda =$  the coefficient of tangential earth pressure, and  $\delta =$  the wall friction angle. Figure 11 presents schematic drawings of a construction site in multilayered ground and three vertical shafts along with the locations of measuring instruments. Earth pressures are measured at different shafts. The earth pressures calculated from Eq. (1) are compared with the measured values, as shown in Figure 12. The measured earth pressures are even smaller than those obtained from the theoretical equation.



(a) Failure plane in multi layered ground



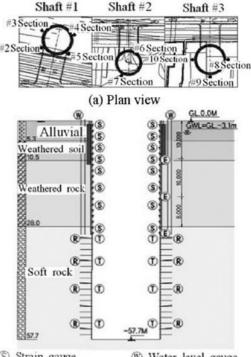
(b) Differential flat element

Figure 10. Derivation of earth pressure in vertical shaft.

## 4 EFFECT OF GROUNDWATER

## 4.1 Effect of seepage pressure

Unexpected groundwater inflow and seepage forces often cause tunnel failures during construction. Shin et al. (2006) presented and reviewed five cave-in collapses that occurred while constructing Line No.5 of the Seoul Metropolitan subway. Figure 13 shows the general features of the collapses and failure details are summarized in Table 4. A comprehensive review



- Strain gauge Water level gauge
- ① Shotcrete stress cell
- R Transducer for measuring axial force of rockbolt
- (E) Earth pressure cell and piezometer

## (b) Vertical section

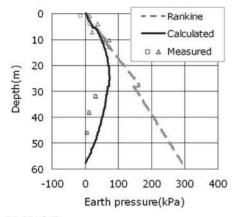
Figure 11. Schematic drawings of a construction site in multi layered ground – Three shafts with locations of measuring instruments.

on such collapse mechanisms reveals the following common features:

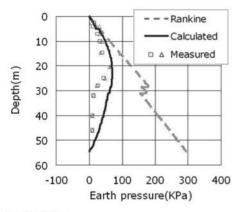
- Thin soil/rock cover and/or mixed faced ground conditions including decomposed granite soils;
- Collapse initiated at the tunnel shoulder during excavation of the upper half of the tunnel section; and
- 3. A considerable amount of groundwater inflow with soil.

In particular, it is observed in these sites that tunnel face collapses always occurred along with seepage ahead of the tunnel face.

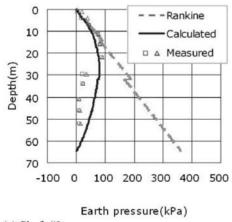
Lee et al. (2003) modified the upper bound solution originally proposed by Leca & Dormieux (1990), taking into account seepage forces in a stability assessment of a tunnel face (refer to Figure 4). The horizontal components of seepage pressures acting on the tunnel face,  $\sigma_{S,F}$ , can be simply considered as an external load



(a) Shaft #1

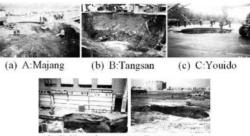


(b) Shaft #2



(c) Shaft #3

Figure 12. Earth pressures measured at vertical shafts.



(d) D:Yongdungpo (e) E:Anyangcheon

Figure 13. Cave-in collapses in the Seoul Subway Line No.5.

in the opposite direction of the support pressure  $\sigma_{T}$ . A modified upper bound solution with consideration of seepage forces becomes

$$N_{s}\left[(K_{p}-1)\frac{P}{\sigma_{c}}+1\right]+N_{\gamma}(K_{p}-1)\frac{\gamma D}{\sigma_{c}}\leq(K_{p}-1)$$

$$\times\frac{\sigma_{\tau}-\sigma_{S,F}}{\sigma_{c}}+1,$$
(4)

where P is the surcharge,  $\sigma_c$  is the unconfined compressive strength of the soil,  $\sigma_T$  is the required support pressure applied to the tunnel face,  $\sigma_{S,F}$  is the seepage pressure acting on the tunnel face,  $K_p$  is Rankine's earth pressure coefficient for passive failure,  $\gamma$  is the unit weight of soil, D is the tunnel diameter, and  $N_s$ and  $N_{\gamma}$  are the weighting coefficients, respectively.

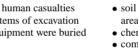
When tunnel excavation is performed below the groundwater level, the stress condition in front of the tunnel face becomes the summation of the effective stress and the seepage pressure. The effective stress can be calculated by the upper bound solution while the seepage pressure can be obtained from numerical analyses. The effective support pressure at the tunnel face can be obtained by Eq. (4) with use of the submerged unit weight  $\gamma_{sub}$  instead of  $\gamma$ .

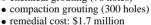
As an example analysis, consider a virtual tunnel with a diameter D driven horizontally under a depth C, as shown in Figure 14. Ground material properties used for the analysis are c = 0,  $\phi' = 35^{\circ}$ , and  $\gamma_{sub} = 5.4 \text{ KN/m}^3$ . Figure 15 shows the total head distribution around the tunnel face, determined by seepage analyses, and the failure zone, estimated from a limit equilibrium analysis. The seepage pressure is calculated by using  $J = i\gamma_w A$ , where *i* is the hydraulic gradient and A is the area. The total support pressure is then obtained by summing up the effective support pressure and the seepage pressure. Figure 16 shows the support pressure change with variation of the H/D ratio (For the case of a dry condition, the dry unit weight  $\gamma_{\rm d} = 15.2 \,\rm KN/m^3$  is used for the analysis). The results suggest that the total support pressure is little affected by the tunnel depth and increases significantly with

Table 4. Collapse mechanisms, damage, and remedial work.

Case	Failure mechanism	Damage	Remedial work
A	<ul> <li>17 Nov 1991, 18:50 : blasting <ul> <li>21:05: total collapse (1,000 m<sup>3</sup>)</li> </ul> </li> <li>thin weathered rock cover <ul> <li>inflow of soil and groundwater</li> </ul> </li> <li>weathered granite (WG) at the face</li> <li>very close to an existing stream</li> </ul>	<ul> <li>no human casualties</li> <li>2-1ane road collapsed</li> <li>stop of gas supply (5,000 households)</li> <li>damage to lighting poles and traffic light poles</li> </ul>	<ul> <li>soil dumping immediately after collapse</li> <li>face shotcrete and invert Close (t = 1.5 ~ 2.0 m)</li> <li>cement milk grouting and curtain wall grouting</li> </ul>
В	• 27 Nov 1991, 10:40: blasting - 16:00 :rock falls at the face - 22:00: soil and water inflow (D = 25 m) • 28 Nov 1991, - 03:20 :additional collapse (D = 20 m) • WG at the face • permeability: $(1.0 \times 10^{-4} \sim 2.0 \times 10^{-5} \text{ cm/sec})$	<ul> <li>no human casualties</li> <li>80 households evacuated</li> <li>electricity and water main collapsed</li> <li>(3-story bldg) slipped into crater</li> </ul>	<ul> <li>grouting: cement mortal → cement milk → chemical grout</li> <li>lowering of groundwater level(3 m/day)</li> <li>pumping/fore poling for re-excavation</li> <li>remedial cost: \$4.5million</li> </ul>
С	<ul> <li>11 Feb 1992, road header excavation - 04:30: raveling</li> <li>significant inflow of ground water (100-130 ℓ/min</li> <li>about 4.5 ton of soil flew into face: D = 10 m</li> <li>heavily WG at the face</li> </ul>	<ul> <li>no human casualties</li> <li>4-lane road collapsed</li> <li>service culvert (6 m × 3 m) exposed (including 154 kv electricity cable)</li> <li>passengers delay</li> </ul>	<ul> <li>dumping soils into collapsed area (240 tons)</li> <li>dumping ready-mixed- concrete (105 tons)</li> <li>grouting</li> </ul>
D	<ul> <li>7 Jan 1993, 03:30: blasting <ul> <li>collapse after removing materials collapse size: 0.7 m × 1.2 m</li> <li>started at the left side of crown</li> <li>soil inflow: 900 m<sup>3</sup>, groundwater: 300 ℓ/min</li> </ul> </li> <li>WG &amp; DGS at the face</li> </ul>	<ul> <li>no human casualties</li> <li>2-lane road collapsed</li> <li>supply stop of water main(Φ = 200 mm)</li> <li>sewer culverts were broken</li> <li>40 households evacuated</li> </ul>	<ul> <li>grouting: cement mortal, cement milk and LW(160 holes)</li> <li>mortal injection beneath sewer culve</li> <li>reduce inflow of groundwater using chemical grouting</li> </ul>
Е	<ul> <li>1 Feb 1993, ring cut</li> <li>-08:30 : rock fall and collapse (oval shape D = 10–30 m)</li> </ul>	<ul> <li>no human casualties</li> <li>6 items of excavation equipment were buried</li> </ul>	<ul> <li>soil dumping into collapsed area (5,500 m<sup>3</sup>)</li> <li>chemical grouting (76 holes)</li> </ul>

- inflow of soil with groundwater
- alluvium & DGS at the face
- · beneath an existing stream





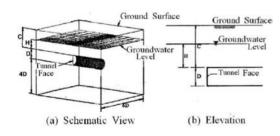


Figure 14. Dimensional condition for seepage analysis.

an increase in the groundwater level ratio. As the total support pressure is related to the tunnel face stability, the seepage force seriously affects the tunnel face stability. While the effective overburden pressure is reduced slightly by the arching effect during tunnel excavation, the seepage pressure remains at the same level during tunnel excavation. This explains why the

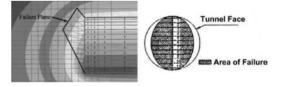


Figure 15. Hydraulic head distribution and failure zone.

effect of seepage plays an important role in tunnel face stability problems.

#### 4.2 Particle transport characteristics of granite residual soils

A soil is said to be internally stable if it is self-filtering and if its fine particles do not move/migrate through

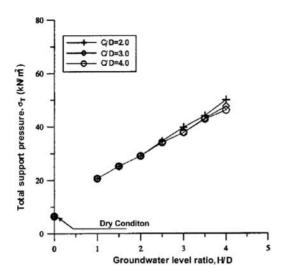


Figure 16. Change of support pressure with variation of the H/D Ratio.

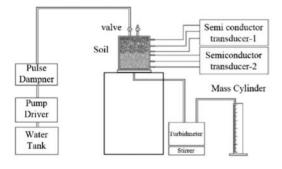


Figure 17. Schematic drawings of experimental set-up.

the pores of its own coarser fraction. Previous investigations into the internal stability of cohesionless soils suggest that soils with a uniformity coefficient ( $C_u$ ) > 20 and with concave upward grain size distributions tend to be internally unstable (Lee et al. 2002). Most residual soils in Korea, including those listed in Table 1, have uniformity coefficients much greater than 20, suggesting that they are internally unstable.

Lee et al. (2002) studied the nature of particle transport and erosion in residual soils. Two types of residual soils introduced in Section 2 are used: Shinnae-dong soil and Poi-dong soil. The experimental setup is shown in Figure 17. In selected experiments, a cylindrical hole 7 mm in diameter is drilled into the compacted specimens to induce erosion only in the hole and simulate surface erosion of the soils. An electronic pump is used to achieve a constant flow rate of the influent from the water tank. The effluent from

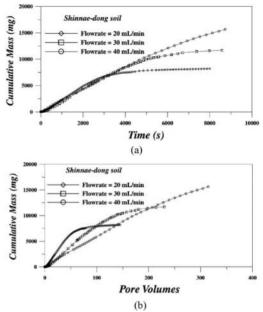


Figure 18. Cumulative mass versus (a) time and (b) pore volume for Shinnae-dong soil.

the cell is characterized with respect to its turbidity (in terms of nephelometric turbidity units, NTUs) and particle size distribution.

The cumulative mass of particles in the effluent eroded from the base soils is plotted with respect to time in Figures 18(a) & 19(a) for the two residual soil types. Some important differences can be observed in the internal erosion behavior of the two soil types. The Shinnae-dong soil exhibits almost the same rate of erosion during the initial stages of the experiment for the three different flow rates used (Figure 18a). Particle redeposition in the soil sample appears to compensate for the increased erodibility at higher flow rates. This is even more apparent when the cumulative mass is plotted in terms of pore volume (Figure 18b). It is seen that at low flow rates the erosion rates are higher, because of the reduced particle deposition. There appears to be a maximum limit for the cumulative mass of internal erosion for each flow rate, beyond which the soil "protected" itself from further erosion, perhaps through the formation of a filter bridge. For the relatively cohesive Poi-dong soil (Figure 19a & 19b), "self-protection" due to particle redeposition is not apparent. There is no cap on the maximum eroded quantities during the period of testing.

In contrast to the internal erosion behavior, the surface erosion from the two samples (as observed in experiments where erosion is induced in a cylindrical hole) follows almost linear trends, with the rates

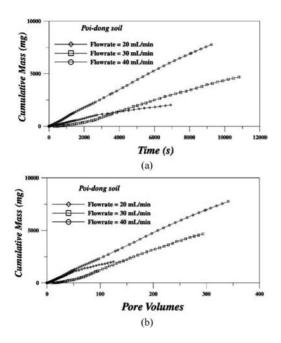


Figure 19. Cumulative mass versus (a) time, and (b) pore volumes, for Poi-dong soils.

of erosion increasing as the flow rate increases. The rates of erosion are also considerably higher than those obtained in the internal erosion experiments discussed above.

Particle transport characteristics of residual soils might be among the factors that result in the instability of underground structures.

## 4.3 Difficulties in penetration grouting

Tunnelling works in soft ground frequently require grouting technology, either to prevent groundwater or to improve mechanical properties of the ground. However, grouting is not available in many cases in decomposed residual soils due to low groutability. Burwell defines the groutability (N) of suspension grouts by the following simple equation (Kim et al. 2007):

$$N = \frac{D_{15}(soil)}{d_{85}(grout)}$$
(5)

where  $D_{15}$  is the particle size of base soils corresponding to 15% finer and  $d_{85}$  is the particle size of grouts corresponding to 85% finer. If *N* is larger than 25, grout can be successfully injected into the soil formation. However, Burwell notes that even in

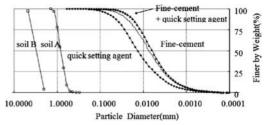


Figure 20. Grain-size distribution of soils and grouts.

Table 5. Soil and grout properties used in chamber injection tests.

Material	D <sub>10</sub> (mm)	D <sub>15</sub> (mm)	d <sub>85</sub> (μm)	d <sub>95</sub> (µm)	N
Soil A	0.60	0.64	_	_	32
Soil B	2.10	2.22	_	_	111
Fine cement	_	_	16	27	_
Quick setting agent	_	_	37	70	_
Fine cement + Quick setting agent	-	-	20	39	-

case of N > 25, the following requirement should be additionally satisfied for the soil to be groutable:

$$\frac{D_{10}(soil)}{d_{95}(grout)} > 11$$
(6)

where  $D_{10}$  is the particle size of base soils corresponding to 10% finer and  $d_{95}$  is the particle size of grouts corresponding to 95% finer. Kim et al. (2007) performed pilot-scale chamber injection tests to investigate the groutability of two granular soils that satisfy the groutability criteria proposed by Burwell.

The grain-size distributions of soils and grouts are shown in Figure 20 and their properties are summarized in Table 5. The experimental set-up for pilot-scale chamber injection tests is shown in Figure 21. Typical results of the experiments are shown in Figure 22. Although the N value of the soil A (N = 32) is greater than 25, the grout could not be sufficiently injected into soil A. Meanwhile, groutability is fairly good for soil B (N = 111). These results suggest that the consideration of filtration phenomena is indispensable to reasonably evaluating the potential of grout penetration. The N value of the Shinnae-dong soil shown in Figure 1 is 2.6 and that of the Poi-dong soil is 0.2. These values mean that penetration grouting in these soils is almost impossible. Therefore, finding an appropriate grouting method has presented a considerable challenge in granite residual soils.

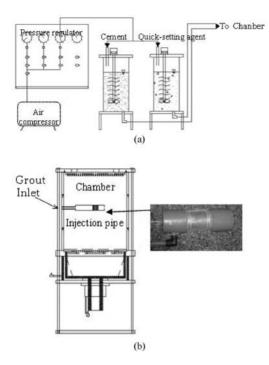


Figure 21. Experimental set-up for pilot-scale chamber injection test.

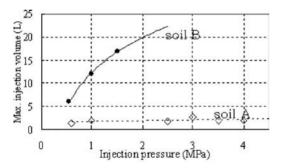


Figure 22. Maximum injection volume with injection pressure.

## 4.4 Ground reaction curve with consideration of seepage forces and grouting

Theoretical analyses of seepage around tunnels suggest that a loss in hydraulic heads occurs at the shotcrete lining and concentration of seepage force at the shotcrete lining in the radial direction induces unfavorable ground reaction (Shin, 2007). Thus, when seepage problems are anticipated during tunnel construction, proper grouting around tunnels can provide effective reduction of seepage force acting on the shotcrete lining and also increases the stiffness and strength of the surrounding ground. When grouting

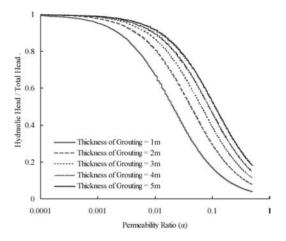


Figure 23. Hydraulic head at soil-grouting interface depending on the permeability ratio.

is applied around the tunnel, a loss of hydraulic heads occurs in the grouting zone around the tunnel; this reduces the seepage force acting on the shotcrete lining, and results in a favorable ground reaction.

Following Darcy's continuity equation, the hydraulic head acting on the soil-grouting interface can be written as

$$H_{I} = H_{T} \cdot L_{g} \frac{1}{L_{g} + \alpha \cdot L_{s}}$$
(7)

where  $H_I$  is the total head at the soil-grouting interface,  $H_T$  is the total head of a site,  $L_g$  is the thickness of the grouting,  $L_s$  is the length across which water travels through the soil media, and  $\alpha$  is the permeability ratio between the soil and grouting area (i.e.,  $\alpha = K_g/K_s$ ). Figure 23 shows the variation of the hydraulic head at the soil-grouting interface with different permeability ratios. As the permeability ratio decreases and the grouting thickness increases, the hydraulic head acting on the interface increases.

Finite element analyses were performed in order to explore the effect of grouting on the ground reaction with consideration of seepage. Seepage force acting on the grouting-soil interface can be modeled by fully coupled mechanical-hydraulic analyses, as shown in Figure 24. Material properties used in this analysis are summarized in Table 6. It was assumed that shotcrete is not applied, the groundwater flow is in a steady-state condition, the grouting thickness is 1 m, and the permeability ratio is  $\alpha = 0.1$ . Four cases were simulated numerically: 1) Grouting with seepage; 2) Grouting without seepage; 3) No grouting with seepage; and 4) No grouting without seepage. Figure 25 presents the effect of grouting and seepage force on the ground reaction curve as given by the numerical analysis results. The case of seepage force without grouting

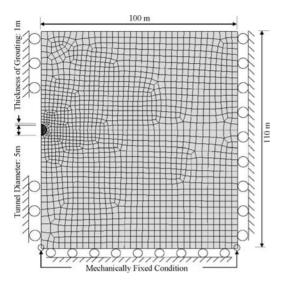


Figure 24. Finite element model for seepage force analysis with consideration of grouting.

Table 6. Material properties used in numerical simulation.

	E (MPa)	μ	γ (kN/m <sup>3</sup> )	K <sub>0</sub>	c (kPa)	φ (°)
Weathered soil	50		18.64	0.5	10	35
Grouted zone	500		18.64	0.5	100	35

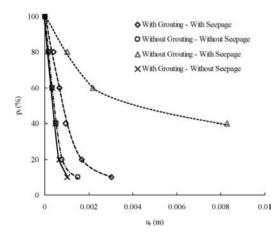


Figure 25. Effect of grouting and seepage force on ground reaction curve (i.e.,  $\alpha = 0.1$ ).

yields a very unfavorable ground reaction curve, which induces a large deformation and requires substantial internal support. However, if the seepage force is not considered, the ground reacts almost elastically even though grouting is not applied. This means that the seepage force significantly affects the ground reaction behavior. In the case where grouting is applied, unfavorable ground reactions induced by the seepage force could be considerably reduced.

## 5 CHARACTERIZATION AND MODELING OF GROUTED RESIDUAL SOIL

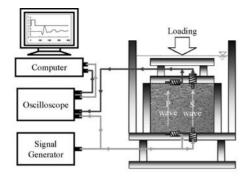
## 5.1 Experimental study on time-dependent characteristics of grouted residual soil

In Korea, in conventional tunnelling in residual soils, pre-reinforcement (grout injection) is typically applied ahead of the tunnel face to enhance the construction safety. In addition, a 1 to 2 day time interval is given between one face and the next face. During this time interval, it is known that changes in the material properties occur due to effects of the curing of the grouting material. However, the stiffness and strength at 28 curing days after the grout injection are generally applied as the material properties for pre-reinforced zones in the design stage without considering the effect of the time-dependent behavior of the injected grout material. Thus, this paper present a new method to characterize the timedependent behavior of pre-reinforced zones around a large-section tunnel in residual soil using elastic waves and to consider time-dependent characteristics in numerical modeling for tunnel design (Song, 2007).

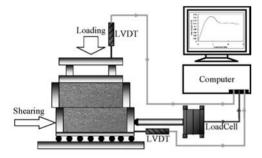
Figure 26 presents schematic drawings of the experimental setup for investigation of the time-dependent characteristics of grouted residual soils: (a) Setup for elastic wave measurements; (b) Setup for shear strength parameter measurements. Bimorph bender elements were installed in the testing device and used to send and receive P- and S-waves (Figure 26a). The specimens were prepared by mixing a residual soil with 5% cement (by weight; the cement-water ratio is the same as that used in the field).

Figure 27 shows typical results for the elastic wave velocity according to the curing time when the normal stress is  $\sigma_n = 160$  kPa. The results show that the wave velocity increases drastically according to the curing time and is almost constant after 7 days. P-wave velocity is faster than S-wave velocity and Poisson's ratio can be readily determined from the two wave velocities.

Figure 28 shows the time-dependent characteristics of shear strength parameters obtained from the direct shear test. As shown in Figure 28(a), the friction angle does not change in accordance with the curing time. On the other hand, it is apparent that the cohesion increases with the curing time; after a certain amount of curing time the cohesion converges, as shown in Figure 28(b). It is deduced that the bonding of cement increases the cohesion and, after with



(a) Measurement setup for elastic wave



(b) Schematic diagram of simple shear test

Figure 26. Experimental setup for investigation of time-dependent characteristics of grouted residual soils.

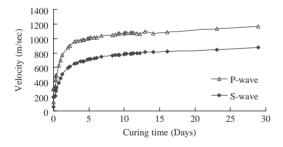
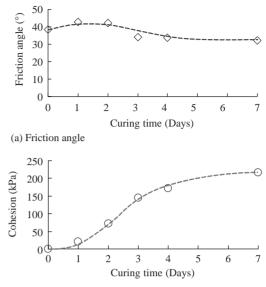


Figure 27. Elastic wave velocity according to curing time ( $\sigma_n = 160 \text{ kPa}$ ).

the elapse of time, the cohesion maintains a uniform value with the end of cementation. The early stage of this phenomenon is controlled by the normal stress, but as curing time increases the cementation controls the friction angle and cohesion.

The wave velocity and cohesion of grouted residual soils can be respectively correlated with the curing time as follows:

$$V_{s}, V_{p} = \alpha (1 - e^{-\beta t}) + V_{0}$$
(8)



(b) Cohesion

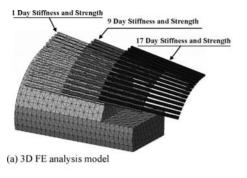
Figure 28. Time-dependent characteristics of shear strength parameters.

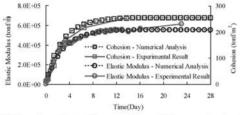
$$e = A(1 - e^{-B \cdot t}) \tag{9}$$

where  $\alpha$ ,  $\beta$ , A, and B are the fitting parameters and t is the curing time. These fitting parameters can be determined by best-fitting the experimental data with Eq. (8) and Eq. (9). Also, the shear strength and strength parameters (i.e., the cohesion and friction angle) can be uniquely correlated to the elastic wave velocities.

# 5.2 Numerical simulation of time-dependent characteristics of grouted residual soil

The construction of underground space in residual soil entails many risk factors such as difficulties in predicting arching effects and determination of various uncertain underground properties. Researchers have suggested various techniques for auxiliary support systems such as the reinforced protective umbrella method (RPUM), which has the advantage of combining a modern forepoling system with a grouting injection method (Barisone, 1982). This method is used for pre-reinforcement design before the underground excavation: not only for small section tunneling within weathered and crashed zones, but also for large underground spaces. In addition, to decrease the risk of a collapse or failure in large excavation caverns, researchers have developed various techniques and construction methods. Some examples include: a tunneling method using an advanced reinforcing system where a double steel pipe is used for water-proofing





(b) Time-dependent stiffness and strength for numerical analysis and obtained experimental results

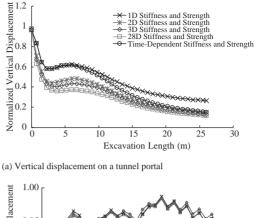
Figure 29. 3D tunnel model and time-dependent material properties of the pre-reinforced zone after 12m excavation.

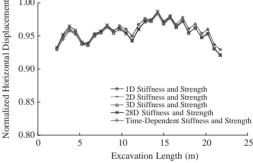
and a urethane injection is used for reinforcement; the Trevi jet method, which involves constructing an archshell structure around a tunnel crown with cement grout; and steel pipe reinforced multi-step grouting, where a beam arch is constructed around the tunnel crown with large diameter steel pipes, and multilayer cement grouting injection is employed.

Three dimensional FE analyses were performed to examine the time-dependent behavior of the grouted zone. The results obtained from laboratory tests were applied to a numerical simulation of a tunnel, taking into account its construction sequence. Figure 29(a) shows a simulated 3D four-lane tunnel model, where the same stress state and stress level as used in the experiment were assumed. Figure 29(b) shows the time-dependent elastic modulus and cohesion values obtained from the experimental study as well as those used in the numerical analysis.

The time-dependent behavior of a pre-reinforced zone can be modeled using the following procedure. The material properties (i.e., stiffness and strength) of the pre-reinforced zone are considered as the boundary conditions from Day 1 to Day 28. The registered initial boundary conditions are applied to a pre-assigned mesh in pre-reinforcement construction. The boundary conditions are then updated according to the field construction sequence.

For a quantitative analysis, the displacements of each case are normalized with the results of a pipeonly case. Figure 30(a) shows the normalized vertical





(b) Horizontal displacement on a tunnel face

Figure 30. Variation of normalized displacement.

displacement at the portal. The trend of the normalized vertical displacement curve for the time-dependent condition is similar to that of the one day curing case within the initial excavation section (<8 m). As the excavation continues, the results of the time-dependent condition become similar to those of the  $2 \sim 3$  days curing case, until vertical displacement eventually converges. The stiffness and strength of the pre-reinforced zone for the  $1 \sim 2$  days curing case are roughly  $30 \sim 50\%$  of those of the 28 days curing case. In other words, a reduction of the material properties of the pre-reinforced zone makes it possible to model the time-dependent effect of the pre-reinforced zone on the global tunnel behavior upon initial tunnel excavation.

Figure 30(b) shows the normalized horizontal displacement at the tunnel face. It is found that the normalized horizontal displacement for the time-dependent condition varies within a range of  $0.94 \sim 0.98$ , which is very similar to that of other cases during excavation. Therefore, pre-reinforcement can be considered for prevention of collapse rather than as a means of displacement reduction control at the tunnel face. Thus, it can be concluded that grouting reduces the horizontal displacement by approximately  $2 \sim 6\%$  at the tunnel face with the pre-reinforcement method.

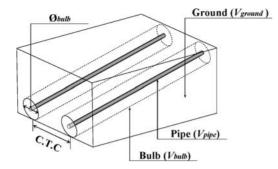


Figure 31. Diagram of simplified pre-reinforced zone.

An analysis method combining experimental and numerical procedures that consider the timedependent effect on the pre-reinforced zone on tunnel behavior will provide a reliable and practical design basis and means of analysis for tunnels in soft ground.

#### 5.3 Determination of equivalent design parameters for the pre-reinforced zone in residual soil

The design and analysis of pre-reinforcement techniques require design assumptions that are problematic at best, resulting in increased uncertainty in tunnel design. The pre-reinforcement effect is typically modeled by simulating the construction sequence of setting the reinforced zone and then increasing the stiffness, thereby obviating the need for complex modeling of each bulb and steel pipe. However, this approach has been found to be unsuitable when the center-to-center distance between pipes is larger than the expansion of the grout bulb. This approach assumes that the stiffness of the pre-reinforced zone is the same as the stiffness of the grout bulb, which may be either completely hardened or arbitrary, and that the pre-reinforced zone becomes two to four times stronger than before reinforcement. Therefore, this study presents a new technique for determining a reasonable equivalent parameter of the pre-reinforced zone.

A pre-reinforced zone consists of ground, grout bulbs, and steel pipes. It may be simplified, as shown in Figure 31, to a condition where the space between the pipes is wider than the grout expansion range. In sandy soil and weathered soil, the bulb may be cylindrical. We assessed five cases comprising various conditions to model the pre-reinforced zone, as summarized in Table 7.

For the strengthening of the grout bulb reinforcement, we followed the method of Kikuchi et al. (1995). The grout injection consequently produced a tenfold increase in the stiffness of the weathered soil. Table 8 shows the equivalent design parameters for the various compositions of the ground, the bulb, and the steel pipes. A precisely-simulated model (Figure 32a)

Table 7. Summary of equivalent design methods.

	<b>5</b> 1	e
Case	Equivalent section	Detail
0	Eground C Esteel	Precisely-simulated model
1	E ground E m	Bulb+Steel pipe SSS [series stiffness system]
2	E <sub>et</sub>	Ground+Bulb+Steel pipe SPSS [series-parallel stiffness system]
3	E <sub>see</sub> E <sub>et</sub>	Ground+Bulb SSS [series stiffness system]
4	E <sub>eq</sub>	Ground+Bulb+Steel pipe SSS [series stiffness system]
5	$E_{see} \rightarrow \bigcirc E_{et}$	Supposition of a pre-reinforced zone with a high level of stiff- ness

Table 8. Equivalent design properties used for numerical modeling.

Equivalent design properties	E <sub>eq</sub> (MPa)	C <sub>eq</sub> (kPa)
Case 1	1748.21	1065.04
Case 2 Case 3	107.35 117.81	91.21 141.22
Case 4	315.22	216.73
Case 5	490.35	588.42

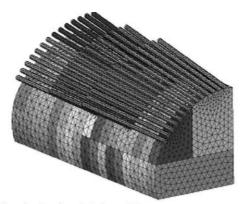
is compared with various equivalent stiffness models (Figure 32b).

Figure 33 shows the vertical displacement at the tunnel crown, the horizontal displacement at the springline, and ground surface settlement. To obtain these values, we averaged the five nodes located at the center and left areas of the tunnel crown at a depth of 4.5 m from the portal. The DRM/DEM parameter is herein defined as the fraction of the displacement of the referential model (DRM) to the displacement of the equivalent model (DEM):

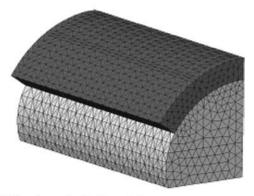
$$DRM / DEM = \left(\frac{Disp_{case}}{Disp_{real}} - 1\right) \times 100(\%)$$
(10)

The precisely-simulated model is represented by a value of 0%.

From the vertical displacement results, case 1 and case 2 give the closest result to the precisely-simulated model in weathered soil. However, general methods tend to overestimate the effect of pre-reinforcement. Although there is only a slight difference between Case 2 and Case 3, Case 2 predicts a similar horizontal displacement at the springline relative to the



(a) Precisely-simulated model



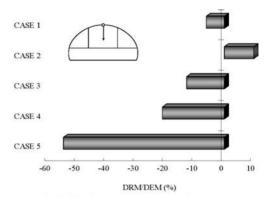
(b) Simple equivalent model

Figure 32. 3D FE analysis model for comparison of equivalent model and precisely modeled.

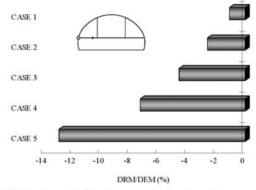
precisely-simulated model and the design is rather safe. The equivalent model offers a satisfactory prediction of the ground surface settlement, as the DRM/DEM value ranges from 0.55% to 0.7% in Case 1 and Case 2.

Case 1 exemplifies a proper equivalent modeling technique for simulating the pre-reinforcing mechanism in residual soils. Case 2, a SPSS in which the stiffness of the bulb and steel pipes are coupled in parallel and then connected to the stiffness of the ground in series, exemplifies a simple equivalent modeling technique that predicts the vertical displacement at the tunnel crown, the horizontal displacement at the springline, and the ground surface settlement.

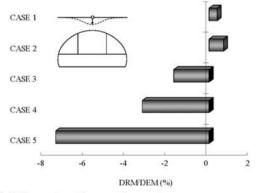
When the ground, grout bulbs, and steel pipes are regarded as an individual support system, the prereinforced zone is not a series or parallel stiffness system but a series-parallel compound stiffness system. Thus, a small degree of stiffness support receives the largest stress; moreover, a linear combination of large stiffness supports resists ground displacement. The SPSS explains the failure mechanism of the



(a) Vertical displacement at tunnel crown



(b) Horizontal displacement at springline



(c) Ground surface settlement

Figure 33. Comparison between equivalent cases.

pre-reinforced zone: namely, the rupture of the steel pipes and the grout bulb follows the yielding of the ground. The SPSS should be used in various studies related to the analysis of pre-reinforced tunnels.

# 6 EFFECT OF SPATIAL VARIABILITY

The mean value of the measurements is often used for design parameters even if there is a noticeable variation. The effect of the variation of geotechnical parameters on tunnel safety or deformation is rarely studied or considered using a statistical concept. There are two kinds of sources for variation in the design parameters: spatial distribution and uncertainty.

Uncertainty means that the material property of the soil/rock has a characteristic unreliability. In reliability-based designs, the uncertainty of the geomaterial is significant for a specific site characterization. There are three major geotechnical uncertainties governing the variability of geoproperties: inherent soil characteristics, measurement errors, and transformation fallacies (Phoon & Kulhawy, 1999).

The spatial distribution of soil has been considered for ecological and environmental modeling (Jury, 1985) and various characterization methods have been suggested by Cho et al. (2004). Likewise, the spatial distribution of the geoproperties is important for the mechanical behavior of underground structures surrounded by variable soils. The soil itself is not an isotropic material, but an anisotropic and non-homogenous material. These characteristics are induced by the formation process and ground stress. Spatial distribution takes the macro scale into the range of interest. Thus, it is expected that, among other soils, weathered residual soils have high spatial variability of their geoproperties due to their origin and weathering process.

The effect of the spatial distribution on the geotechnical parameters of tunnel deformation is studied through numerical analyses based on statistical concepts as shown in Figure 34. The geotechnical parameters that cause the largest deformation of tunnels when the ground material follows the Mohr-Coulomb model and the expected displacement variation characteristics for each geotechnical design parameter are presented in this study.

The coefficient of variation (COV) has long been commonly used to quantify the variability of soil and rock properties (Harr, 1987). The COV is defined as the standard deviation ( $\sigma$ ) divided by the mean ( $\mu$ ) of the parameter:

$$COV (\%) = \left(\frac{\sigma}{\mu}\right) \times 100 \tag{11}$$

Each material property of the residual soil can be regarded as a normal random variable that has a certain probabilistic error. In particular, it has been shown that the spatial distribution of the friction angle and unit weight follows a normal distribution parameterized with the mean and COV (Lumb, 1966; Hoeg & Murarka, 1974; Lacasse & Nadim, 1996; Low &

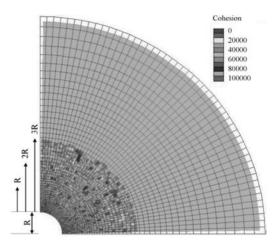


Figure 34. Numerical model of spatial distribution of geoproperty (cohesion, Range = 3R, COV = 40%).

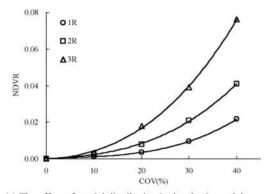
Table 9. Representative coefficient of variation (COV) of the geotechnical parameters.

Properties	COV(%)	Reference
Elastic Modulus	$15\!\sim\!45$	Harr, 1987; Phoon and Kul-hawy,1999
Friction Angle	$24 \sim 32$	Schultze, 1972
Cohesion	$40 \sim 68$	Schultze, 1972; Tan et al. 2000

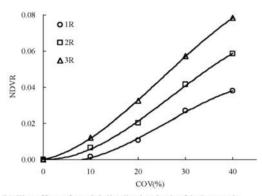
Tang, 1997; Phoon & Kulhawy, 1999; Tanit et al., 2004). Thus, it is assumed herein that all geotechnical parameters, i.e., constitutive components of the Mohr-Coulomb model, have normal distribution characteristics. The representative COV of the geotechnical parameters is summarized in Table 9.

The bulk modulus (B) and shear modulus (G) are important material parameters and can be alternatively changed by the elastic modulus (E) and Poisson's ratio, respectively. The normally distributed elastic modulus generated with a specific COV is converted to a bulk modulus and shear modulus to simulate the spatial distribution effect of the elastic modulus on the tunnel deformation.

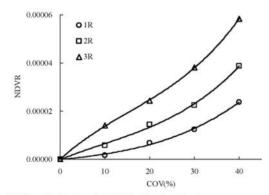
The numerical analysis results are shown in Figure 35(a). A COV of 20% should be considered a critical variation of the elastic modulus effect on the tunnel deformation, as a COV of more than 30% causes a relatively significant deformation variation. The elastic modulus is an influential geoproperty in tunnel deformation. The largest NDVR is more than 0.075 when COV = 40% and range = 3R. The displacement variation induced by the elastic modulus variation can be predicted from Figure 35(a), depending on the range from the tunnel center.



(a) The effect of spatial distribution in the elastic modulus



(b) The effect of spatial distribution in the friction angle



(c) The effect of spatial distribution in cohesion

Figure 35. Effect of spatially distributed geoproperties on tunnel deformation.

The effect of spatial distribution in the friction angle on tunnel deformation is shown in Figure 35(b). The NDVR at the tunnel crown and springline reaches  $0.077 \sim 0.078$  when the variation range is 3R and the COV of the friction angle is 40%. Therefore, the friction angle variability is the most high-ranking factor for the calculation of deformation in the tunnel compared with other geotechnical parameters when the Mohr-Coulomb material model is used. The general COV of the friction angle is small at 12% (Schultze, 1972). Therefore, a range of  $10\% \sim 20\%$  variation of the friction angle is critical for tunnel deformation characteristics. It can be seen that as the variation range increases, the slope of the curvature increases.

The COV of cohesion is near 40% (Fredlund & Dahlman, 1972). As shown in Figure 35(c), the NDVR enlarges in all cases as the COV increases. However, it can be observed that the variation of cohesion has a minor effect on deformation in the tunnel. The maximum NDVR induced by the cohesion variation is lower than 0.00006 when the variation range is 3R and the COV is 40%. From the analysis results, the spatial distribution effect of cohesion on tunnel deformation is smaller than that of the elastic modulus and friction angle. Therefore, the spatial distribution of cohesion is not a critical parameter for the characterization of tunnel behavior.

### 7 CONCLUSIONS

We presented an overview of geotechnical aspects of underground construction in urban areas where mostly decomposed residual soils are present, focusing on mechanical properties, apparent earth pressure, effect of ground water, and effect of spatial variability in geotechnical properties.

The strength and deformation characteristics of residual soils are affected by particle/pore size, fines content, mineralogy, and unsaturation, among others: The internal friction angle decreases with an increase of fine content and partial saturation increases the strength of unsaturated residual soils due to capillarity. Results of back analyses from actual measurements show that geotechnical properties of residual soils, which are commonly used for design and modeling in Korea, are inappropriate in this regard, thus demonstrating the importance of the observational method in tunnel engineering.

As an unsaturated soil is re-saturated, its apparent cohesion can be eliminated. Thus, during tunnel construction, cohesion loss can be induced by resaturation (e.g., seepage hindrance, drainage clogging and groundwater change) and may result in tunnel face instability.

The lateral earth pressure coefficient of residual soils is smaller than the coefficient commonly suggested in the literature (e.g.,  $70 \sim 80\%$  of Peck's suggestion).

Unexpected groundwater inflow and seepage forces often cause tunnel failures during construction. Several case histories suggest that tunnel face collapses always occur along with seepage ahead of the tunnel face. Analytical results suggest that the total support pressure is little affected by the tunnel depth and increases significantly with an increase in the groundwater level ratio. As the total support pressure is related to the tunnel face stability, the seepage force seriously affects the tunnel face stability.

Particle transport characteristics of residual soils might be among the factors that result in the instability of underground structures. Their phenomena are complicated and are involved in erosion versus self-healing (redeposition) processes.

While groutability is affected by the pore size of the residual soils, proper selection of a grouting method has proved a difficult task in granite residual soils. When seepage problems are anticipated during tunnel construction, proper grouting around tunnels can achieve effective reduction of seepage force acting on the shotcrete lining and can increase the stiffness and strength of the surrounding ground.

An analysis method combining experimental and numerical procedures that consider the timedependent effect on the pre-reinforced zone on tunnel behavior will provide a reliable and practical de-sign basis and means of analysis for tunnels in soft ground.

It is expected that, among other soils, weathered residual soils have high spatial variability of geoproperties, because of their origin and weathering process. The numerical results show that tunnel deformation increases with an increase in the spatial variability of geotechnical design parameters and is accelerated with an increase in tunnel size.

Research on decomposed residual soils has been conducted throughout the last few decades. Nonetheless, the current increase of underground construction projects in urban areas requires better understanding of residual soils for safer underground use.

# ACKNOWLEDGEMENTS

This study was funded by the Korea Institute of Construction and Transportation Technology Evaluation and Planning under the Ministry of Construction and Transportation in Korea (Grant No. 04-C01).

#### REFERENCES

- Barisone, G., Pigorini, B. & Pelizza, S. 1982. Umbrella arch method for tunnelling in difficult conditions – Analysis of Italian cases. *Proceedings of the 4th Congress International Association of Engineering Geology*. New Delhi, 4: 15–27
- Cho, G.C., Lee, J.S. & Santamarina, J.C. 2004. Spatial variability in soils: High resolution assessment with electrical needle probe. *Journal of Geotechnical and Geoenvironmental Engineering*. ASCE 130 (8): 843–849
- Cho, K.H., Choi, M.K., Nam, S.W. & Lee, I.M. 2006. Geotechnical parameter estimation in tunneling using relative convergence measurement. *International Journal for*

Numerical and Analytical Method in Geomechanics 30: 137–155

- Fredlund, D.G. & Dahlman, A.E. 1972. Statistical geotechnical properties of glacial lake edmonton sediments, in Statistics and Probability in Civil Engineering. Hong Kong University Press, distributed by Oxford University Press. London.
- Harr, M.E. 1987. *Reliability Based Design in Civil Engineering*, Dover Publications, INC.
- Hoeg, H. & Murarka, R.P. 1974. Probabilistic analysis and design of a retaining wall. *Journal of Geotechnical Engineering Division*. ASCE 100 (3): 349–366
- Jury, W.A. 1985. Spatial variability of soil properties. In: Hern, S.C., Melancon, S.M. (Eds.), Vadose Zone Modeling of Organic Pollutants, Lewis, Chelsea, MI, 245–269
- Kikuchi, K., Mito, Y. & Adachi, T. 1995. Case study on the mechanical improvement of rock masses by grouting. *Rock Foundation*. Balkema 393–398
- Kim, J.S., Choi, Y.K., Park, J.H., Woo, S.B. & Lee, I.M. 2007. Effect of viscosity and clogging on grout penetration characteristics. *Journal of Korean Geotechnical Society* 23 (4): 5–13
- Kim, Y.J. 1993. Mechanical characteristics of decomposed Korean granites. *Ph.D. Thesis*, Seoul: Korea University
- Lacasse, S. & Nadim, F. 1996. Uncertainties in characterizing soil properties. Uncertainty' 96, Geotechnical Special Publication, ASCE 58 (1): 49–75
- Leca, E. & Dormieux, L. 1990. Upper and lower bound solutions for the face stability of shallow circular tunnels in frictional material. *Geotechnique* 40 (4): 581–606
- Lee, C.K. & Jeon, S.K. 1993. earth pressure distribution on retaining walls in the excavation of multi-layered ground, *Journal of Korean Geotechnical Society* 9 (1): 59–
- Lee, I.K. 1991. Mechanical behavior of compacted decomposed granite soil. *Ph.D. Thesis*, London: City University.
- Lee, I.M., Moon, H.P., Lee, D.S., Kim, K.R. & Cho, M.S. 2007. Earth pressure of vertical shaft considering arching effect in layered soils. *Journal of Korean Tunnelling Association* 9 (1): 49–62
- Lee, I.M., Nam, S.W. & Ahn, J.H. 2003. Effect of seepage forces on tunnel face stability, *Canadian Geotechnical Journal* 40: 342–350
- Lee, I.M., Park, Y.J. & Reddi, L.N. 2002. Particle transport characteristics and filtration of granitic residual soils from the Korean peninsula. *Canadian Geotechnical Journal* 39: 472–482
- Lee, I.M., Sung, S.G. & Cho, G.C. 2005. Effect of stress state on the unsaturated shear strength of a weathered granite., *Canadian Geotechnical Journal* 42: 624–631
- Lee, S.J., Lee, S.R. & Jang, B.S. 2002. Unsaturated shear strength characteristics of weathered granite soils. *Journal* of Korean Geotechnical Society 22 (1): 81–88
- Low, B.K. & Tang, W.H. 1997. Reliability analysis of reinforced embankments on soft ground. *Canadian Geotechnical Journal* 34 (5): 672–685
- Lumb, P. 1966. The variability of natural soils. Canadian Geotechnical Journal 3 (2): 74–97
- Peck, R.B. 1969, Deep excavations and tunneling in soft ground, Proc., 7th Int. Conf. on Soil Mech. And Found. Engrg., State-of-the-Art Rep., State-of-the-Art Vol: 225–290
- Phoon, K.K. & Kulhawy, F.H. 1999. Characterization of geotechnical variability. *Canadian Geotechnical Journal* 36 (4): 612–624

- Schultze, E. 1972. Frequency distributions and correlations of soil properties. In Statistics and Probability in Civil Engineering, Hong Kong University Press, distributed by Oxford University Press, London.
- Shin, J.H., Lee, I.K., Lee, Y.H. & Shin, H.S. 2006. Lessons from serial tunnel collapses during construction of the Seoul Subway Line 5. *Tunnelling and Underground Space Technology* 21: 296–297
- Shin, Y.J. 2007. Elasto-plastic ground response of underwater tunnels considering seepage forces, *Ph.D. Thesis*, Seoul: Korea University
- Song, K.I., Kim, J. & Cho, G.C. 2007. Numerical analysis of pre-reinforced zones in tunnel considering the time-dependent grouting performance. *Journal of Korean Tunnelling Association* 9 (2): 109–120
- Tanit, C. 2004. Reliability-based design for Internal stability of mechanically stabilized earth walls. *Journal of Geotechnical and Geoenvironmental Engineering*. ASCE 130 (2): 163–173
- Yoo, C. 2001. Behavior of braced and anchored walls in soils overlying rock. *Journal of Geotechnical and Geoenvironmental Engineering*. ASCE 127 (3): 225–233

General reports

# Safety issues, risk analysis, hazard management and control

C.T. Chin & H.C. Chao

Moh and Associates, Inc., Taipei, Taiwan

ABSTRACT: The occurrence of hazard events in geotechnical practice is often associated with geotechnical uncertainties. To prevent hazard events from happening or reduce the impacts of their consequence, sources of geotechnical uncertainties need to be identified and treated, and appropriate control measure has to be implemented throughout a project. The probabilistic methods are the devices dealing with uncertainties and the risk management is the technique to facilitate achieving the goal of project safety. The papers presented in this theme are summarized and briefly discussed in three categories – risk and decision analysis, geotechnical control, and analysis and control of ground response. In risk analysis, geotechnical uncertainties need to be treated explicitly. Results of risk analysis, qualitatively or quantitatively, not only provide a baseline for decision making but also insights to the problem of concern. In order to make best use of available analysis tools, more case studies are needed.

#### 1 INTRODUCTION

#### 1.1 Geotechnical uncertainty

Uncertainty is the lack of certainty, a state of having limited knowledge where it is impossible to exactly describe existing state or future outcome (Hubbard, 2007). If uncertainty requires to be treated, probability is the device. There are two kinds of probability: relative frequency and subjective, degree-of-belief probability. The probability of an uncertain event is the relative frequency of occurrence when it is obtained through repeated trials or experimental sampling. The subjective, degree-of-belief probability comes from judgment where the probability of an uncertain event is the quantified measure of one's belief or confidence, according to the information available and one's state of knowledge at the time it is assessed (Vick, 2002).

In engineering practice, although uncertainty is surely a certainty and may create some impact, it seems the existence of which is not a bother to many. As a matter of fact, most of the engineers get along quite well without explicitly using the necessary device, probability, to manage it. This probably attributes to the customary practice engineers have. By using the established standards, codes, factors of safety, design criteria, or procedures in which the uncertainties somehow have been accounted for or somewhere hidden behind, deterministic method by which answers are either correct or wrong can comfortably be applied in solving problems.

However, the material in geotechnical engineering is widely known for its significantly abundant

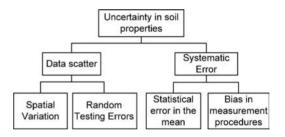


Figure 1. Categories of uncertainty in soil property data (Christian et al. 1994).

uncertainties. Sources of uncertainties can be unknown presence of geologic defects, uncertain value of soil properties, limited knowledge to the mechanisms and processes, and much more. One of the examples as presented by Christian el al. (1994) is demonstrated in Figure 1, in which the uncertainties associated with the characterization of soil properties are attribute to systematic error and data scatter. Systematic error results from statistical error in mean value arising from limited numbers of measurements or bias in measurement procedures like those associated with field permeability test or field van test. Data scatter, on the other hand, is a result of random testing errors or actual spatial variation in the soil profile.

Another source of geotechnical uncertainty is modeling. A model is an appropriate simplification of reality. Good modeling skill is reflected in the ability to identify the appropriate level of simplification – to recognize what features are important and what are not. Very often engineers are unaware of the simplifications that they have made and problems may arise precisely because the assumptions that have been made are inappropriate in a particular application (Wood, 2004).

The types of geomechanical model can be divided into the following categories: empirical model, theoretical model, analytical model, numerical model and constitutive model. Model uncertainty is the extent to which a model incarnates a uniquely correct representation of the physical process it seeks to emulate. Model uncertainties arise from its representative degree to the real field processes, and for a physical process different models and operation can always be found.

In conventional approach, effects of uncertainty are generally accounted for, consciously or not, by standards, codes, design criteria, or factors of safety. The underlying uncertainties may have already been considered or evaluated somehow in the process developing them. This simple strategy seems work quite well for works having been encountered before and lot of experience accumulated. However, when circumstances are unique and uncertainties are not routine, which are often the cases in geotechnical engineering practice, aforementioned procedures or tools may not be fully applicable any more.

#### 1.2 Geotechnical safety

The occurrence of hazard event in major geotechnical engineering projects such as tunneling or deep excavation often draws a lot of attention from the general public. Potential consequences include significant financial loss to the client and contractor, schedule delay and loss of confidence to the general public, and casualty. Thus, it is obvious that safety is an important issue that requires special cares to ensure the objectives of a project can be achieved.

Safety itself is in essence not a measurable quantity. In practice, it is evaluated through the safety indicators. Some of the physical characteristics such as the size of cracks, deformations and differential settlements, are selected to serve as safety indicators for evaluating the safety status. This process requires analysis and interpretation, and judgment plays an important role in it. If the uncertainty is to be accounted for and the effects are accommodated, sources of uncertainty must be understood first. This is where probability comes in. One of the most important tools in dealing with probability is judgment. Judgment provides an interpretative framework that helps guide how uncertainties are comprehended and subsequently managed. When a structure is said to be safe during adjacent excavation, it means the assessor hold some sufficient degree of belief.

In past decades, there has been a greater awareness on need to treat and manage the geotechnical

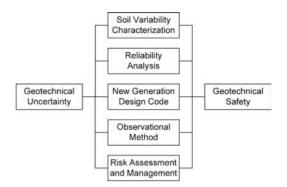


Figure 2. Geotechnical safety.

uncertainty in a rational and explicit way. In response to this demand, active discussion and development have been seen on topics that may be grouped into five categories: soil variability characterization, reliability analysis, new generation design code, observational method, and risk assessment and management. Advancements in these disciplines, on the other hand, are intended to address better issues on geotechnical safety. These concepts are conveniently illustrated by Figure 2.

# 1.3 Risk assessment

Traditionally, risks were managed indirectly through the engineering decisions taken during the project development. More often than not, the information and knowledge used behind the decisions are inexplicit and not easy to trace. To improve this, it is necessary to introduce formal risk management technique. In this practice, risk is defined as the product of failure probability and consequences:

$$Risk = P_f \times C_f$$

Where,  $P_{\rm f}$  is probability of failure and  $C_{\rm f}$  is consequences of failure.

By definition, risk assessment must account for both the probability and extent of adverse consequences of hazard events arising from a given activity. Thus, risk assessment involves identification of hazard events and qualitative or quantitative descriptions of risks. The scope of risk analysis contains the entire process of the causes and effect of adverse events. Such a process is composed of three sequential parts, an initiator that starts it, response to the initiator and the consequences (Vick, 2002) as illustrated in Figure 3.

Initiator is the cause that sets a potential failure process in motion. Response is the event directly resulted from the initiator, and also called the hazard event in some occasions. Response leads to failure if the

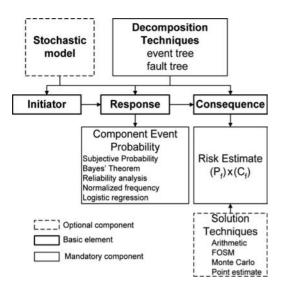


Figure 3. Components of risk analysis (Vick, 2002).

resistance or capacity of a structure is unable to sustain the effects. In risk analysis, failure is treated as consequence and needs to be a physically observable condition like retaining wall collapse, utility pipeline broken, roadway damage et al. Generally, failure cannot be evaluated meaningfully if it is not converted to something measurable such as lives lost or dollar cost. It is also important to realize there are uncertainties associated with the response. As shown in Figure 3, probabilistic methods such as Bayes' approach, reliability analysis, subjective judgment and others are introduced as the tools for evaluating these uncertainties.

As shown also in Figure 3, initiator, response and consequence are linked together by decomposition techniques such as event tree and fault tree. Event tree and fault tree facilitate envision how failure process occurs. As such, they are basic tool of risk identification and analysis. Figure 4(a) shows a two-level of event tree and fault tress. The event starts with initiator event I. If it occurs, the next event to happen is  $R_1$ , followed by  $R_2$  or  $R_3$  or both. This event tree contains two failure modes  $IR_1R_2$  and  $IR_1R_3$ . Figure 4(b) presented the same failure progress by Fault tree. Event uses a "bottom up" or "forward" approach beginning with the initiator and taking it to consequences. This captures failure process by expressing its logical order. Fault tree, on the other hand, uses a "top down" or "backward" approach starting from the top event and identifying the possible events in the 2nd level through a search process. The "and" and "or" logic gates instrumented in Fault tree allow computations using Boolean algebra in coping with complex problem.

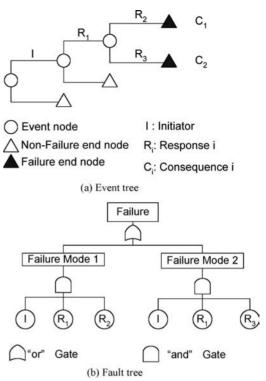


Figure 4. Event tree and Fault tree format.

After risk is identified, the rest of work in risk analysis is to evaluate the probability and consequences such that the corresponding rank-order can be determined based on predefined criteria. Subjective, degree-of-belief probability often are assessed using expert investigation method incorporated with other techniques such as Dephi method, Analytic Hierarchy Process (AHP), Fuzzy set theory in risk analysis. One of the controversial issues arises here in the process quantification of risks. According to Vick (2002), qualitative approaches allows us to discern that one thing is more likely or less likely than some other, but an important property - by how much more or less - in the qualification process is missing. Other than it, the depth of the insight to the problem can be reached is considered in proportion to the efforts made in the process of risk analysis where numerical quantification usually requires more.

Risk response measure and reduction strategy can be developed based on risk acceptable criteria defined in risk policy (ITA, 2004). For risks with high rankorder, it may be necessary to develop design or alternatives in determining risk reduction measure. Decision analysis is essentially a comparative approach based on decision rules and results of risk analysis on various alternatives. Similar techniques in risk analysis

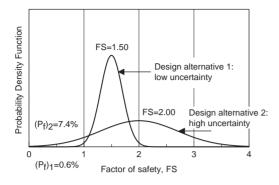


Figure 5. Effect of uncertainty.

can be applied to evaluate the risk associated with each of the alternative. By comparing the results of risk analysis, the alternative having the greatest risk effectiveness can be identified. Results of decision analysis provide a solid baseline for decision maker to make decision when dealing with risk.

# 1.4 Deterministic thinking versus probabilistic thinking

In engineering, a deterministic system is a system that given a particular input, it will always produce the same output, and the underlying machine will always pass through the same sequence of states. In this system, results of thinking are either right or wrong, mathematic algorithms used are either correct or not, and there is no need to be bothered by uncertainty.

However, uncertainty in fact exists and the effects of which may be significant in many occasions and cannot just be neglected. As has been mentioned earlier in this report, uncertainties in geotechnical engineering practice are managed through standards, codes, design criteria, factors of safety or established procedures. With such strategies, the deterministic thinking works and functions well for situations where experience and routine works dominate. For unique situations or non-routine conditions or in risk analysis, uncertainty becomes an unavoidable issue that requires using of the technique of probabilistic methods in obtaining appropriate solution.

Figure 5 demonstrates the effect of uncertainty in design results for two alternatives based on different design models. The computed factors of safety for alternative 1 and alternative 2 are 1.50 and 2.00, respective. In a deterministic system, the factor of safety for alternative 2 is apparently higher, and this alternative is almost certain to be selected as the final scheme. However, if the uncertainties associated with both models are considered, the failure probability of alternative 2 is apparently higher even the computed factor of safety seems better.

Table 1. Risk and decision analysis.

ID	Торіс
IS-022	Risk analysis and fuzzy comprehensive assessment on construction of shield
	tunnel in Shanghai metro line
IS-294	Risk assessment on environmental impact in
	Xizang Road Tunnel
IS-293	Risk analysis for cutterhead failure of composite
	EPB shield based on fuzzy fault tree
IS-070	Risk assessment for the safe grade of deep excavation
IS-055	Multi-factors durability evaluation in subway concrete structure
IS-383	Research on structural status of operating tunnel of metro in Shanghai and treatment ideas

### 2 REVIEW OF PAPERS

#### 2.1 Risk and decision analysis

As listed in Table 1, the papers related to risk and decision analysis are grouped into this section. Among these papers, IS-022, IS-294 and IS-293 discuss tunneling risk analysis, and IS-070 introduces risk analysis for deep excavation, and IS-055 is about decision analysis. While the others apply formal risk or decision analysis techniques in the evaluation approach, the approach presented in paper IS-383 can be taken as an informal risk management approach. As have been used widely in geotechnical practice, risks associated with the identified hazard events were not presented. Nevertheless, the causes of hazard events were identified and the hazard prevention measures were developed.

Paper IS-022 presented a risk analysis approach in which risks were identified thorough the work breakdown and fault tree method. The associated rank-order of the identified risks were analyzed by fuzzy comprehensive evaluation method. The expert investigation method and AHP were used to determine the risk indicators' weight, evaluate the probabilities and consequences of the identified risks. The membership function was applied to determine the membership degree value of each of the risk events. The case investigated in this paper was Shanghai metro, which is a 32.2 km long shield tunnel. For the convenience of risk analysis, the metro line was divided into 12 sections. Results of the evaluation showed the identified risk events include obstruction, tunnel collapse, quicksand, groundwater ingress, ground settlement, et al. The risk levels with respect to each of the sections were from medium to significant. The possible financial loss is between 100,000 RMB to 10,000,000 RMB. Based on the risk classification criteria of this case, the consequences are great but compensable.

Paper IS-294 presented the risk analysis for the construction of Shanghai Xizang Road Tunnel, a rivercrossing tunnel built for the 2010 Shanghai Exposition. Other special features included up-cross and down-cross existing metro lines. Risk analysis was based on expert investigation and confidence index methods. The computer software "TRM 1.0" developed by the Tongji University was used as the tool to analyze the risk data and thereafter determine the corresponding risk level. In the case investigated, the work of risk analysis was focused on the environmental impacts as a result of tunnel construction to the surrounding buildings, roads, and utility pipelines. Based on the analysis results, risk mitigation measures focused on ground settlement control were proposed. The authors also pointed out that the risk is dynamic in nature, which requires constant and cycled assessment and treatment.

Paper IS-293 presented for tunneling in adverse ground condition, the cutterhead of shield machine is exposed to high risk of failure as a result of heavy demand in machine operation. This paper presented a risk analysis approach for cutterhead failure using the fault tree analysis method and fuzzy set theory. In this study, risk identification was conducted by the fault tree analysis method. Results of the identification indicated three major hazard events - cuter disk failure, cutter failure and other system components failure - will lead to cutterhead failure. Expert investigation method and fuzzy set theory were introduced to evaluate the probabilities and consequences of the occurrences for these hazard events. Based on the results of risk analysis, remediation measures for the hazard events with higher risk level were developed and presented.

Historic data in paper IS-070 showed the proportion of accidents in deep excavation projects resulted from design and construction related problems is 87% based on 344 cases investigated. Because effective tools in dealing with geotechnical uncertainty and applicability of current analytical theories are limited, engineers tend to be conservative consciously or not in dealing with geotechnical problems. This is usually reflected in high construction costs. To improve this, a Fuzzy synthetic evaluation process was used in this study for risk analysis. This method follows the procedures in which the risk factors were identified first, and expert investigation method, Analytic Hierarchy Process (AHP) method and Delphi method were adopted to determine the weight for each of the factors and the safety index, and the safety ranking order of an excavation project can thus be decided. The case visited was the Shanghai international passenger transport center project. The size of the excavation area, hydrogeology condition, design feature, construction aspect and surrounding environment are selected as the primary factors affecting the safety of deep excavation. Results of the evaluation indicated that the level of safety of the project as a whole was acceptable. However, size of excavation area and hydrogeology condition were identified as the factors having more significant impact on the safety of the excavation activities. Measures to prevent hazard events from happening were thus developed based on the results of risk assessment.

Paper IS-055 presented a decision analysis approach for determining the optimal design alternative for the concrete mixture of a subway tunnel. Factors having significant influence on the durability of subway concrete structure were identified to be stray current corrosion, chloride ions ingression, sulfate attack and carbonation. While the durability attribution from each of the factors was determined individually, the combined influence of these factors was vague. To obtain the optimal design scheme, five design alternatives were developed. Expert investigation method, AHP and Multiple Attribute Decision Model incorporated with fuzzy set theory were applied in assessing the joint influence of these factors. Results of the evaluation provided a baseline for decision maker in selecting the adequate design scheme.

Paper IS-383 presented the health diagnostic results and hazard prevention or reduction measures for an operating tunnel of Shanghai metro. Results of the diagnosis showed the tunnel structure is suffered from detrimental events such as ingress of groundwater, crack and convergence of tunnel lining, and settlement along the tunnel alignment. The causes of these detrimental events were identified to be adverse geological condition, sequela of the accidents during construction, local ground subsidence, construction activities in the proximity, etc. To prevent these detrimental events from getting worse and reduce the associated risks to the tunnel and surrounding environment, metro passengers and third parties, control measures in terms of regulation, monitoring schemes were proposed.

# 2.2 Geotechnical control

The geotechnical control is a process exercised throughout the planning, design and construction phases of a project to facilitate achieving project objectives. The control measures applied include regular audit inspection, site supervision and risk management to ensure adequate design standards and effective safety protection be performed such that the project can be completed in a manner of optimization. As listed in Table 2, two of the papers were grouped into this section. The paper IS-372 introduced a metro railway under construction and the paper IS-380 presented a retrospect study of a cable duct crossing project. Both of the cases reported were construction projects in Hong Kong.

Paper IS-372 presented a geotechnical control process exercised for a HK\$8.3 billion subway

Table 2.	Geotechnical control.
ID	Торіс
IS-372	Geotechnical control of a major railway project involving tunnel works in Hong Kong
IS-380	Performance Review of a Pipe Jacking Project in Hong Kong

construction project in Hong Kong. This project is packed into three design-built contracts. The content of the project includes the construction of shield tunnels, cut-and-cover tunnels and underground station. The Geotechnical Engineering Office (GEO), Civil Engineering and Development Department of the Hong Kong Special Administration Region (HKSAR) is in charge of controlling the building ordinance and regulations and issuing technical standards. Under of auditing of GEO, the private owner of this project, the Kowloon Canton Railway corporation (KCRC) committed to follow the Joint Code of Practice for Risk Management of Tunneling Works for implementation of risk management process. The geotechnical control process started from the planning stage and will be kept in effect throughout the construction stage. Because the project is considered a private project, the tunnel works may be exempted form the administrative procedures. Thus, a review panel was formed within GEO in the planning stage. The KCRC demonstrated to GEO they had met the requirements for instrument of exemption (IoE) in aspects including risk management, design, and construction. The IoE was issued after the document met with the specified requirements. Under the IoE, the KCRC is required to appoint authorized personnel, employ assurance system and control scheme, prepare site supervision plan, and keep appropriate records and reports for regular GEO inspection during the construction stage. Monthly meeting are scheduled between relevant parties. When significant changes occur in design or working methods, the KCRC needs to report to GEO. The geotechnical control process is currently in progress.

Paper IS-380 review the performance of a cable duct crossing construction project in Hong Kong. With the project, the geotechnical control process was exercised by the GEO, HKSAR. This project involved the construction of a 222 m long and 1.95 m diameter tunnel to serve as cable duct crossing a highway, two MRT tunnels, and two Airport express link tunnels. This cable duct was constructed with pipe jacking method. Prior to the commencement of works, a condition survey was conducted for existing utility pipeline. During construction, an automatic monitoring system was used to take readings for monitoring railway track settlement. The work of monitoring was controlled by a system based on alert level, action level and alarm level management. Specified responses corresponding to each level of the system were also defined before the construction started. During the construction, a close to action level reading on the track settlement was recorded. Under the framework of geotechnical control process, an urgent meeting was held among the client, design and contractor. After the meeting, the type of lubricant used for filling the gap between the tunnel lining and surrounding soils was replaced. With the control process, the maximum settlement of railway tracks was controlled within the maximum allowable range throughout the whole construction phase.

# 2.3 Ground response, analysis and design

Most of the engineering designs are composed of two major components - determining the imposed load and computing the resistance or capacity. Factor of safety in a design problem is defined as the ratio between the resistance and load. Because most of the geotechnical engineering projects are performed underground, the construction activities involving add or remove loads to the ground will inevitably result in stress and strain redistribution. When the resulted stresses exceed the resistance or capacity of the ground, failure could occur. However, ground response before reaching failure state may draw more concern for underground construction project. In urban area, underground construction induced ground deformation make the construction itself and the buildings, utility pipelines and infrastructures adjacent to the construction site exposed to the risk of being damaged. Results of ground response prediction and monitoring can be used as a baseline for developing control and protection schemes.

The papers in this group are further divided into two sub-groups – ground response, analysis and design – for discussion purposes.

# 2.3.1 Ground response

Table 3 lists the papers related to the issue of ground response. Paper IS-048 introduced a prototype and laboratory scale non-destructive scanning technique designed for detecting crack or cavity ahead of the front end of a shield machine. Paper IS-369 presented an analytical approach for evaluating the squeezing potential of soft rock. Paper IS-247 presents a numerical investigation for the floor heave behavior at the T-section of a deep mining tunnel using threedimensional finite element method. Paper IS-376 discusses the application of strain gauge in measuring strut load in deep excavation project in Singapore. Suggestions for maximizing its effectiveness are also proposed. Paper IS-014 and IS-339 present the application of fuzzy set theory and neural network in predicting ground settlement induced by tunneling.

Table 3. Ground response.

ID	Торіс
IS-048	Experimental studies of a geological measuring system for tunnel with ultrasonic transducer
IS-369	Squeezing potential of tunnels in clays and clayshales from normalized undrained shear strength, unconfined compressive strength and seismic velocity
IS-247	Floor heave behavior and control of roadway intersection in deep mine
IS-376	Maximising the Potential of Strain Gauges: A Singapore Perspective
IS-014	Prediction of surface settlement induced by shield tunneling: an ANFIS model
IS-339	The use of artificial neural networks to predict ground movements caused by tunneling

Paper IS-048 presented an experimental model developed for detecting multiple reflection sources based on rotational scanning technique and ultrasonic wave reflection method. In the proposed test configuration, the ground was simulated by plaster blocks, among which the maximum dimension is of 1000 mm in height, 1200 mm in width and 150 mm in thickness. Horizontal and inclined cracks are simulated by drilled holes. Various signal process techniques such as stacking, signal compensation, and demodulation were applied to obtain ultrasonic image of the space ahead of the scanning transducer. Results of the experimental evaluation showed that the proposed method is able to identify the location of multiple reflection sources.

The tunnel squeezing phenomenon was first described by Terzaghi (1946) who associated squeezing mainly with clay-rich rocks. One of the first stability criteria to predict squeezing was developed by Peck (1969) for tunnels in clays based on Broms and Bennermark's (1967) stability criteria for open excavation. For tunnel in rocks, most of the squeezing criteria proposed are empirical or semi-empirical such as Singh et al. (1992), Goel et al. (1995, 2000), Jethwa el al. (2000) and more. The main challenge in use of these semi-empirical approaches is in the determination of the rock mass strength. In addition, most of the proposed methodologies developed thus far are mainly for clays or hard rocks. Few studies have been proposed for intermediate material such as hard soils or soft rocks.

In paper IS-369, simple methods to evaluate the squeezing potential of intermediate soil-rock material based on undrain shear strength, unconfined compressive strength and P-wave velocity was proposed. Evaluation of field measurement and other empirical tunnel squeezing criteria was also performed for comparative purpose.

For large cross section tunnel, the ground deformation resulted from excavation work is usually much more significant at the tunnel intersection than at the regular part. Thus, special measures controlling possible ground movement that might be detrimental to the safety is necessary. Paper IS-247 presented an investigation of the floor heave at a T-section of a ventilation tunnel at the depth of GL. -990 m of Tongkou colliery using three-dimensional finite element modeling. Based on the evaluation results, suggestions for controlling floor heave were given.

Instrumentation plays an important role in ensuring that construction control is maintained during excavation. Comparison between the monitored data and design predictions provides the opportunities for verifying design results and refining design methods. Experiences indicate in most of the excavation project, some of the instrumentation readings are genuine load conditions while some are not. It is important to gain clear understanding of the impact of construction activities and data interpretation and make effort to maximize the quality of the data.

According to paper IS-376, strain gauge is widely used in measuring the change of strut load in deep excavation in Singapore because of the acceptable reliability and low costs compared to other instruments. However, the performance of strain gauge can easily be affected by the location where it is installed, electromagnetic interference, temperature, preloading, welding and other construction activities. To minimize the influence of these factors, measures such as use of load cell as a cross reference, protection against construction induced disturbance, and use of real time system were proposed. For data interpretation, skilled personnel fully aware of the design predictions for the excavation, the excavation process and the potential impact of the excavation on the readings should be assigned.

Due to the complexity and variation of the ground composition, accurate prediction of tunneling induced ground settlement based on conventional geotechnical approaches such as empirical methods, analytical methods or numerical methods is considered by many as a challenge. Empirical approach is easy to use but often precludes the consideration of ground resistance and deformability parameters. Analytical and numerical approach are generally using simplified ground parameters for practical purposes rather than incorporating the complete ground conditions along the whole range of a tunnel alignment into the modeling approach. The applicability of available constitutive models and simulation of three dimensional ground response within 2 dimensional space for ground settlement prediction is controversial to many. An alternative in solving the problems mentioned above is the artificial intelligence technique. Artificial neural network and fuzzy logic were introduced into this field for prediction in recent years.

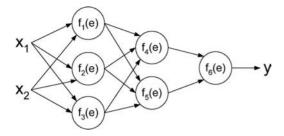


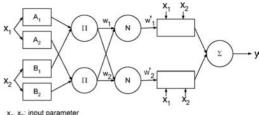
Figure 6. Conceptual model of a neural network with two inputs and one outputs.

An artificial neural network (ANN), often just called a "neural network" (NN), is a mathematical model based on biological neural networks. A conceptual model is shown in Figure 6. It consists of an interconnected group of artificial neurons and processes information using a connectionist approach to computation. In most cases an ANN is an adaptive system that changes its structure based on external or internal information that flows through the network during the learning phase. In more practical terms neural networks are non-linear statistical data modeling tools. They can be used to model complex relationship between inputs and outputs or to find patterns in data. One of the most important properties of ANNs is their ability to learn from environment and to improve their performance with such learning. The learning occurs when the NN reaches a generalized solution for a particular case of problems.

Fuzzy logic is a form of multi-valued logic derived from fuzzy set theory to deal with reasoning that is approximate rather than precise. Just as in fuzzy set theory the set membership values can range between 0 and 1, in fuzzy logic the degree of truth of a statement can range between 0 and 1 and is not constrained to the two truth values, true and false, as in classic predicate logic. When linguistic variables are used, these degrees may be managed by specific functions.

The Adaptive Neural-Fuzzy Inference System (ANFIS) is a hybrid intelligent system combining the ability of a neural network to fuzzy logic. An illustrative architect of ANFIS is given in Figure 7. It uses a given set of input/output data to construct a fuzzy inference system within which the membership function parameters are tuned or adjusted using either a backpropagation algorithm along or in combination with a least square type of method. This adjustment allows the fuzzy system to learn from data and thus it also has the potential in prediction. This paper presented an application of an ANFIS based model in the prediction of shield tunnel induced settlement.

Paper IS-339 presented an application of ANN in predicting ground settlement induced by tunnel excavation. The case visited is the Metro-DF in the city of Brasilia and the tunnel was constructed using the New



v cutput parameter

A, B : linguistic label associated with the node function

Figure 7. Architecture of ANFIS.

Table 4. Input and output parameters.

Input parameters, x <sub>i</sub>	Output parameter, y
Ground settlements along the centerline of tunnel alignment 0 m, -5 m, -10 m, -20 m and -30 m relative to front end of shield machine tunnel lining installed per day in terms of the number of rings	Ground settlement along the centerline of tunnel alignment +5 m relative to front end of shield machine

Austrian Tunneling Method (NATM). Two data sets were used for training the ANN. One of the data set was obtained from the monitored data during the tunnel construction and the other was established based on results of finite element modeling. Validating or testing of the ANN was performed after the completion of training phase. Results of the evaluation with monitored data show the average correlation coefficients between the predicted and measured value were about 0.99 and 0.95 at the completion of training phase and at the validating phase, respectively. Although the prediction with finite element method exhibited better correlation between the results obtained from the training and validating phases, precision was not always achieved. The authors attributed the cause of lacking precision to inadequate constitutive model, difficulty in simulating the real tunnel geometry, and simulate the three dimensional physical reality in two dimensional space. Based on the evaluation results, the authors concluded that the ANN is an effective computational tool in predicting settlement induced by tunneling when good set of training data is available.

Paper IS-014 presented the application of an ANFIS model in the prediction of ground settlement induced by shield tunneling. The case visited is the Shanghai No. 2 subway project. Data set obtained from field measurements was used in model training and validating.

Results of the evaluation showed that the ANFIS predictions are in good agreement with the measured data with relative error within the range from 2% to 7%. For this particular case visited, the author

#### Table 5. Analysis and design.

ID	Торіс
IS-083	Research and application of road tunnel structure optimization
IS-374	Framework of performance-based fire protection design method for road tunnel
IS-125	Discussion on design method for retaining structures of metro station deep excavation in Shanghai
IS-234	Research on stochastic seismic analysis of underground pipeline based on physical earthquake model

also compared the ANFIS predicted results with those derived from Peck approach (1969), Pi-sigma approach (Gupta and Rao, 1994) and Back Propagation based neural network method. According to the authors, the ANFIS prediction exhibited relatively better accuracy and stable in terms of the computed results.

#### 2.3.2 Analysis and design

The papers relating to analysis and design are grouped in this section as listed in Table 5.

According to paper IS-083, conventional road tunnels are designed based on passive analysis approach. This method follows the procedure (1) alternatives development (2) computation and analysis (3) selecting the best design scheme among alternatives. The advantage of this method is conceptually easy. However it is time consuming and may not be able to identify the best alternative if it is not included in the scheme developed. Thus, it is difficult to evaluate the performance of the structure. To improve this, an optimization method is proposed in this paper. In this method, the dimensions of the tunneling are treated as input variables, and the stress of the spring is the objective function. Algorithm of complex analysis programmed by C++ language was used in the optimization process. Results of comparison showed both the stress distribution along the tunnel and the relating structural cost were reduced significantly based on the results evaluation for this method.

Currently, performance based design code has been used in fire protection design for buildings. However, the design code for road tunnels is still prescriptive based. In view of this, Paper IS-374 proposed a framework of performance based design code for road tunnels with large cross sections.

Although there is no significant variation in the construction methods and geological condition for Shanghai metro, results of diaphragm wall and strut design in terms of diaphragm depth, thickness, rebar content, and strut load in deep excavation are apparently varied even for projects with similar excavation depth. The paper IS-125 presents the results of investigation for projects with excavation depth from 14.92 m to 17.28 m. The comparison of the strut loads between the design value and field measurement in Shanghai was also presented. Causes of the deviation were identified to be results of different computation tools, analysis parameters, and lacking communication between designer and contractor. Over-design and unnecessary cost were often the consequences. Suggestions on the issues relating to active earth pressure coefficients, vertical spring coefficient,  $k_v$ , under the toe of the diaphragm wall, equivalent subgrade coefficient,  $k_h$ , strut load, diaphragm thickness, and monitoring scheme were proposed.

#### 3 CONCLUSION AND REMARK

#### 3.1 Treatment for geotechnical uncertainty

In conventional approach, geotechnical uncertainty is managed through codes, standards, design criteria, established procedures and other devices. Uncertainty is recognized somewhat indirectly and inexplicitly. Engineering judgment plays an important role and has been used commonly in defining and setting the aforementioned management devices. For risk management and other unique or non-routine geotechnical problems, judgment still plays an important role but geotechnical uncertainty needs to be treated explicitly for requirements such as safety evaluation. Probabilistic methods including Baye's theorem, reliability method, subjective probability and more provide the necessary tools in coping with it.

#### 3.2 Qualification and quantification for risks

Expert investigation method was used extensively for risk analysis in determining the probability and consequences of risk events. This method requires experts to express their judgment in qualitative terms based on predefined criteria, all information available, and the uncertainties of the risk they perceive. The application of elicitation techniques based on cognitive process is necessary in finding the true belief of the experts. It is also important to reduce the extent of what considered unavoidable bias when using elicitation techniques in expert investigation.

Uncertainty is usually expressed qualitatively and the use of verbal terms like "possible", "probable", "likely" or "unlikely" seems to work well. However, numerical quantification provides the missing link for qualitative approach. The extent to which the uncertainty of an event is greater than the other can be determined through quantitative approach. It is important to note that probability, qualitatively or not, is only a verbal description or the output of numerical quantification process. The insight obtained through the process is what really matters. The insight helps us understand the true meaning of the probability associated with the problem and what behind it.

#### 3.3 Decision analysis

Decision analysis is a technique developed on the framework of risk analysis and comparative skills. Through this technique, a relatively better design alternative can be determined. Nevertheless, it should be noted that decision analysis only provides a baseline for decision maker. A well-informed decision comes about by considering risk magnitudes, risk reduction measures, and feasibility in performing these measures other than the results of decision analysis.

#### 3.4 Need for more case studies

No new theory in soil mechanics can be accepted for practical use without ample demonstration by field observations that it is reasonable accurate under a variety of conditions (Terzaghi and Peck, 1948). To interpret the results of observation, it is necessary to incorporate judgment. Judgment, or subjective probability plays an important role for risk management and hazard control. The acquisition of judgment relies on experience and knowledge that can be derived from field observation or diligent study of published case histories. Because the opportunity of performing personal observation is limited for most in contemporary geotechnical practice, study of case histories becomes the primary source for accessing and developing one's own experience.

For risk management and hazard control, more case studies are needed. Geotechnical engineers should make best use of available tools and present their cases for further studies.

# PAPER IN THEME 4

- Ai, X.Q. & Li, J. 2008. Research on stochastic seismic analysis of underground pipeline based on physical earthquake model.
- Bao, X.H. & Huang, H.W. 2008. Risk assessment for the safe grade of deep excavation.
- Chissolucombe, I., Assis, A.P. & Farisa, M.M. 2008. The use of artificial neural networks to predict ground movements caused by tunneling.
- Cong, C. & Linde, Y. 2008. Multi-factor durability evaluation in subway concrete structure.
- Ding, W.Q. & Xu, Y. 2008. Research and application of road tunnel structural optimization.
- Guo, B.H. & Lu, T.K. 2008. Floor heave behavior and control of roadway intersection in deep mine.
- Gutierrez, M. & Xia, C. 2008. Squeezing potential of tunnels in clays and clayshales from normalized undrained shear strength, unconfined compressive strength and seismic velocity.

- Han, X. & Ding, G.Y. 2008. Framework of performancebased fire protection design method for road tunnel.
- Hou, J., Zhang, M.X. & Tu, M. 2008. Prediction of surface settlement induced by shield tunneling: an ANFIS model.
- Kim, D.H., Kim, U.Y., Lee, S.P., Lee, H.Y. & Lee, J.S. 2008. Experimental studies of a geological measuring system for tunnel with ultrasonic transducer.
- Lam, T.S.K. 2008. Performance review of a pipe jacking project in Hong Kong.
- Lee, W., Chung, S.S., Roberts, K.J. & Pang, P.L.R. 2008. Geotechnical control of a major railway project involving tunnel works in Hong Kong.
- Li, J.P., Wang, R.L. & Yan, J.Y. 2008. Research on structural status of operating tunnel of metro in Shanghai and treatment ideas.
- Osborne, N.H., Ng, C.C., Chen, D.C., Tan, G.H., Rudi, J. & Latt, K.M. 2008. Maximising the potential of strain gauges: A Singapore perspective.
- Wang, R., Liu, G.B. & Liu, D.P. 2008. Discussion on design method for retaining structures of metro station deep excavation in Shanghai.
- Yan, Y.R., Huang, H.W. & Hu, Q.F. 2008. Risk analysis for cutterhead failure of composite EPB shield based on fuzzy fault tree.
- Yao, C.P., Huang, H.W. & Hu, Q.F. 2008. Risk assessment on environmental impact in Xizang Road Tunnel.
- Zhou, H.B., Yao, H. & Gao, W.J. 2008. Risk analysis and Fuzzy comprehensive assessment on construction of shield tunnel in Shanghai Metro line.

# REFERENCES

- ABI/BTS. 2004. A Joint Code of Practice for the Procurement, Design and Construction of Tunnels and Associated Underground Structures, London: The Association of British Insurer, The British Tunneling society.
- Douglas Hubbard "How to Measure Anything: Finding the Value of Intangibles in Business", John Wiley & Sons, 2007.
- David Muir Wood. 2004. Geotechnical Modeling, Spon Press, New York.
- Gupta, M.M. & Rao, D.H. 1994. On the principles of fuzzy neural network. Fuzzy Sets and Systems, 61(1): 1–8.
- Huang, H.W. et al. 2006. Guidelines of risk Management for Metro Tunnelling and Underground Engineering Works, Tongji University.
- Internatinal tunnel Association. 2002. Working Group No. 2, Guidelines for tunneling risk management, Balkema.
- Peck, R.B. 1969. Deep excavation and tunneling in soft ground. Proceedings of the 7th International Conference on soil Mechanics and Foundation Engineering, Mexico.
- Steven G. Vick. 2002. Degrees of Belief-Subjective Probability and Engineering Judgment, ASCE Press, American Society of Civil Engineers, Reston, Virginia, USA.
- Terzaghi, K. & Peck, R. 1948. Soil mechanics in engineering practice, Wiley, New York.
- Whitman, R.V. 2000. Organizing and Evaluating Unceratinty in Geotechnical Engineering, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No. 7, July, 2000.

# Calculation and design methods, and predictive tools

F. Emeriault & R. Kastner

LGCIE, INSA-Lyon, F-69621, France

ABSTRACT: This general report covers 19 papers that are included in session 6 of the symposium, related to the design or calculation methods and predictive tools for tunneling and deep excavations. For this report, the papers have been classified in 3 main subjects: i) excavations, ii) tunneling, iii) general papers on design methods and tools. There are a greater number of papers concerning tunnelling, covering a large numbers of subjects, subdivided in the following topics: T.B.M. simulation, Ground reaction curve, Longitudinal behaviour of segmented lining, Settlement troughs, Effect of vibrations.

#### 1 INTRODUCTION

This session with a very broad theme, contains 19 papers. Tables 1 and 2 present a classification of these papers, by countries and by themes. There are papers from 8 countries, but more than half of the papers (11/19) are from China.

Table 1.	Classification	by	countries.
----------	----------------	----	------------

Countries	Number of papers		
Brazil	2		
China	11		
Japan	1		
Kazakstan	1		
Korea	1		
Netherlands	1		
UK	1		
USA	1		

Table 2. Classification	ı by	subjects.
-------------------------	------	-----------

Topics	Sub-topic	Number of papers
Excavations		3
Tunnels	TBM simulation	1
	Ground reaction curve	2 + 2 (rock)
	Settlement trough	2
	Longitudinal behaviour	2
	Effect of vibrations	3
General		4

The content of the papers can be broadly divided into calculation and design of tunnelling works, excavations, and more general papers. 3 papers deal with excavation, considering 3 different aspects: basal stability, strut loads, and effect on nearby piles. There are a greater number of papers concerning tunnelling, covering a large number of subjects. Among these 12 papers, 1 reports on TBM numerical simulation, 2 present analytical methods for the ground reaction curve, 2 deal with the assessment of the settlement trough, 2 consider the problem of the longitudinal behaviour of the segmented tunnel lining, 3 report on the effect of vibrations or on the seismic response, the 2 remaining concerning more problems related to rock tunnels. Finally there are 4 general papers concerning the simulation tools or presenting a national report.

Considering this large number of subjects, it is not really possible to highlight a main emphasis, some of the papers covering very narrow subjects and others concerning very general topics.

In the following sections of this report, the major findings and key features of each paper are presented and briefly discussed.

# 2 EXCAVATIONS

*Song and Huang* studied the basal stability of an excavation in soft clay by an upper bound approach. The failure mechanism considered (Figure 1) is based on the classical Prandtl failure mechanism. Their original contribution to this problem is to consider the dependence of the short term shear resistance of soft clays on the local orientation of the failure surface. They propose an analytical upper bound solution based on this

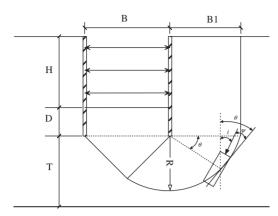


Figure 1. Definition of geometric parameters.

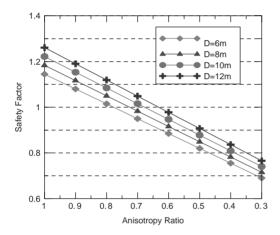


Figure 2. Influence of D/H on the factor of safety  $(\gamma = 18 \text{ kN/m}^3, \text{ undrained shear strength } S_{uv}(z) = 0.33\sigma'_v,$  width of the excavation B = 15 m, depth of the excavation H = 12 m).

kinematical mechanism and on the equation proposed by Casagrande and Carillo (1944) for describing the anisotropy of shear strength:

$$S_{ui} = S_{uh} + (S_{uv} - S_{uh}) \cdot \cos^2 i$$
 (1)

where  $S_{uh}$  and  $S_{uv}$  are obtained by undrained triaxial compression and extension tests.

Figure 2 presents the results of a parametric study on the evolution of the safety factor with the anisotropy ratio, for different values of the embedment depth. It appears clearly that the anisotropy ratio has more influence than the embedment depth which increases only slightly the safety factor. The authors studied also the influence of the depth of the bedrock, when the bedrock limits the extension of the failure mechanism.

This approach compared well with a 2D FE analysis of adeep excavation in Boston Blue Clay a presented by

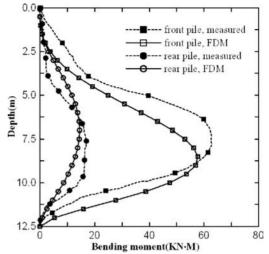


Figure 3. Comparison of free head pile group.

Hashash and Whittle (1996) using an advanced effective stress soil model, MIT-E3. The authors analysed also a case of failure in Shanghai where the standard codes led to safety factors of more than 1.4 and where this approach leads to a safety factor of 0.97, explaining the basal instability.

Zhang and co-authors present a method for estimating the response of piles to lateral soil movements induced by a nearby excavation.

For a single pile, the method is based on the classical two-stage approach (Poulos & Chen 1997):

- in a first step, the free-field soil movement must be determined either by measurement or by calculation;
- in a second step, these soil movements are imposed to the piles through a Winkler subgrade reaction model: the pile is represented by an elastic beam, the pile-soil interaction is modeled using linear elastic soil springs, the effect of axial load on the pile is ignored. The Winkler subgrade reaction equation is solved by a FD approach, permitting to take into account heterogeneous soils.

This classical method has been extended by the authors to pile groups. In the case of pile groups, the shielding effect of piles is modelled by superposing to the free field soil movement the reduction of the displacement due to neighbouring piles. This shielding effect is calculated using an attenuation function based on simplified Mindlin's equation.

The authors present a comparison of their approach with centrifuge model tests and finite element simulation published by Leung et al (2000).

A comparison of calculated and measured bending moments is given on Figures 3 and 4 in the case of

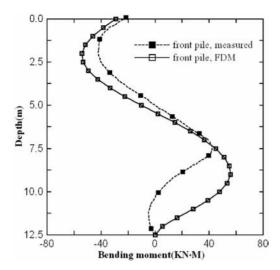


Figure 4. Comparison of front pile in capped head pile group.

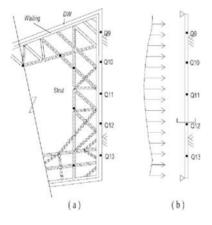


Figure 5. Example of strut system.

pile groups: this simplified model fits quite well with the experimental results for free headed piles but there are some differences in the case of capped piles. The authors explain that this difference could be reduced by using a non-linear elastic spring hypothesis, but perhaps is it due to the Winkler's hypothesis itself.

Finally, it must be noted that this method is not specific to excavations, and could be used for other works inducing lateral soil movements, such as tunnelling or embankments on soft clays.

*Shi and co authors* present a method for estimating by back-analysis the strut loads in a complex concrete strut system, such as the system presented on Figure 5.

The proposed method is based on measured displacements of the wall in the horizontal plane of the

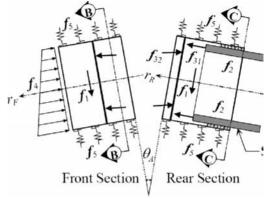


Figure 6. Forces acting on the TBM.

strut system, the load distribution between the different struts at the same level being calculated by back-analysis.

As there are no direct load measurements, the magnitude of the loads obtained by this approach depends strongly on the a priori hypothesis concerning the distribution of soil pressure acting on the wall. Therefore this method can only give an indication on the relative distribution of the loads.

#### 3 TUNNELING

#### 3.1 T.B.M. simulation

*Chen and co-authors* present an interesting paper on the behaviour of aTBM when following a curved alignment. They propose a comprehensive numerical model of an articulated shield which is an extension of a kinematic shield model proposed for a single circular shield (Sugimoto & Sramoon 2002). Their model is focused on the tunnel boring machine, considering all the forces acting on the shield, such as for example (Figure 6)

- the different jack thrust forces,
- the pressure acting on the face,
- the forces acting on the shield periphery.

These latter forces represent the interaction between the shield and the surrounding soil. They are simulated by a spring model. Therefore, in this model, the tunnelling operation is seen mainly from the point of view of the TBM.

The simulation of the TBM behaviour is obtained by imposing to the model the main operation parameters of the shield.

A comparison between an observed and simulated behaviour is presented in Figure 7. The actual shield trajectory, in the vertical and horizontal plane, and in

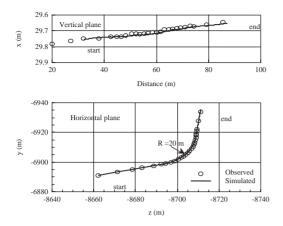


Figure 7. Simulated and observed behaviour.

extreme conditions of a sharp curve, is well simulated by this very comprehensive model.

This model gives also the field of soil pressure acting on the shield, derived from the spring model, as shown on this figure. In a further step, it could be interesting to use these calculated contact stresses between the shield and the soil in a continuum model of the soil mass for modelling the soil deformation and to check if these calculated contact stresses lead to a realistic simulation of observed displacements and settlement troughs.

#### 3.2 Ground reaction curve

*Shin and co-authors* propose an analytical model of the ground reaction curve taking into account the seepage forces. The following classical hypotheses are adopted:

- The tunnel is bored in an infinite soil mass subjected to a hydrostatic in situ stress,
- The soil is linear elastic perfectly plastic with the Mohr-Coulomb yield criteria,
- Radial seepage forces are taken into account, as indicated in the equilibrium equation:

$$\frac{d\sigma'_r}{dr} + \frac{\sigma'_r - \sigma'_{\theta}}{r} + i_r \cdot \gamma_w = 0$$
<sup>(2)</sup>

The hydraulic gradients are calculated separately, considering a steady state of seepage.

Based on these hypotheses, the authors propose an analytical solution of the elasto-plastic state of stress, expressed in terms of stress state and displacement. Due to these assumptions, this model is more adapted to deep tunnels.

The authors present an application of their model for a 50 m deep tunnel, with a diameter of 5 meters. Different cases are examined: fully drained or with

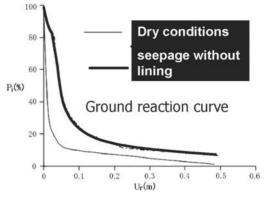


Figure 8. Effect of radial seepage on the ground reaction curve.

a semi impervious lining, having the same mechanical properties than the surrounding soil. A comparison between the dry soil case and the fully drained case is presented in Figure 8:

- in dry conditions, the pressure decreases rapidly with the convergence, as the soil resistance is mobilised,
- when fully drained seepage is considered, there is a marked increase of the convergence for a given internal pressure, as the seepage forces do not depend on the soil convergence and remain constant.

Despite the diverse simplifications of such an analytical model, this paper gives interesting indications on the influence of seepage forces and shows clearly that they should be taken into account for modelling the ground reaction curves and for assessing the stability of the excavation.

In a second paper on ground reaction curves, *Sozio* presents a 2D or 3D analytical model representing the tunnel and the soil cover by a thick sphere or a thick cylinder (Figure 9). The soil model is the classical linear elastic-perfectly plastic Mohr-Coulomb model.

The originality of this model is that the gravity forces are emulated by radial body forces. This enables to take into account a limited cover depth, with a surface load and an internal pressure. The author proposes to use this 3D model for a preliminary assessment of the stability of the unlined length of a tunnel, the problem being to estimate the radius of the sphere equivalent to the tunnel unsupported heading.

Such analytical models are generally based on restrictive hypotheses. It is the case for this model, but it has to be highlighted that in his paper, the author indicates very clearly the limitations of the proposed models. It should be interesting to compare this model to the classical approach based on the assumption of a tunnel bored in an infinite soil mass.

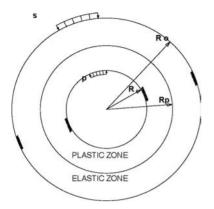


Figure 9. Representation of the soil tunnel interaction.

The papers presented by *Zhang & Wang* and *Lu et al* concern more specifically deep rock tunnels.

Zhang & Wang study the ground reaction in the case of a pressure tunnel, the rock mass being not unloaded but loaded by the internal pressure. In this specific situation, quite far from urban tunnels in soft ground, the softening of the rock considered by the authors can lead to a broken zone around the tunnel.

Lu et al studied by 3D numerical simulations the stability of different types of intersections between deep mine tunnels and the influence of the construction sequences. The method of simulation is not precisely described. If this study is not directly applicable to shallow tunnels in soft ground, some of their results can be considered from a qualitative point of view such as the fact that excavating towards the intersection appears more dangerous than excavating from the excavation.

#### 3.3 Longitudinal behaviour of segmented lining

The paper proposed by *Hoefsloot* is based on the observation that the staged construction of segmented tunnel linings induces a permanent longitudinal bending moment in the lining. Based on solutions proposed by Bogaards & Bakker (1999), and Bakker (2000), the author proposes an analytical solution by considering the segmented tunnel lining as a beam on an elastic foundation.

The longitudinal loading scheme (Bending moment and shear force from jack forces, shear force from steel brushes, weight of lining segments, uniformly distributed load of limited length: back up train) advances with the progress of the TBM (Figure 10). This analytical model has been built in a spreadsheet, and verified using PLAXIS 2D.

The result of this model is compared with strain measurements made in the lining of the GROENE HART tunnel in the Netherlands. As illustrated on Figure 11, a bending moment is induced in the lining by

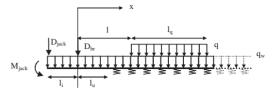


Figure 10. Model used by Hoefsloot to represent the stage construction of the tunnel lining.

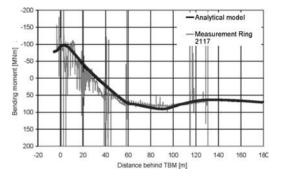


Figure 11. Groene hardt tunnel – evolution of the bending moment with the advance of the TBM.

the advancement of the TBM and becomes permanent after about 60 meters. Despite the simple hypothesis of a spring model, the evolution of the bending moment is quite well modelled. Nevertheless this result has been obtained by adjusting some parameters which are difficult to assess, such as the lining bending stiffness and the effect of grouting (Talmon et al. 2008).

Finally, this analytical model, validated on field measurements, shows that the staged construction of the segmental lining in a straight alignment results in a permanent longitudinal bending moment, that should be considered in the design of the lining and for the installation of the segments.

A second paper on the longitudinal behaviour of segmented lining is presented by *Zhu et al.* The authors examine the problem of the actual longitudinal stiffness of segmented linings which is one of the parameters which was necessary to adjust in the model proposed by *Hoefsloot*.

The assessment of the lining stiffness is based on a 3D numerical model, composed of shell elements (Figure 12) and joints with shear and normal stiffness at all the interfaces between the individual elements. The complete numerical model, loaded as a cantilever beam, is compared to a simplified equivalent continuous model, which is not precisely described in the paper.

The stiffness deduced from the numerical model appears on their example to depend on the segment length and to be lower than the stiffness obtained

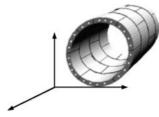


Figure 12. Lining model.

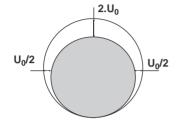


Figure 13. Convergence scheme proposed by Heli Bao et al.

by the equivalent continuous model. But both models are based on hypothesises concerning the joint behaviour, which need to be measured or assessed based on the observation of the actual behaviour of full scale segmented linings.

#### 3.4 Settlement troughs

There is generally a large consensus about the use of the Gaussian type curve for describing the settlement trough.

The direct estimation through elastic calculations or by numerical simulations often leads to larger settlement troughs than observed. The result depends in fact on different assumptions, one being the convergence profile of the ground around the tunnel.

*Heli Bao and his co-authors* present an analytical solution, using conformal mapping of an elastic half space. In order to fit with the observed settlement troughs, they propose an elliptical convergence shape, based on the solution proposed by Park (1974), as indicated on Figure 13.

This approach is compared with the observed settlements during the construction of a 6.2 m diameter tunnel in Shanghai. The calculated settlement trough fits quite well with the observed one. But no indication is given concerning the assessment of the magnitude of the convergence, the authors indicating simply the gap between the TBM and the lining, gap which is in fact certainly partially filled by grouting.

In their paper, Zu and Liu compared different methods for settlement trough assessment: Peck's empirical approach (Peck 1969), stochastic medium theory, and the solution proposed by Verruijt and Booker (1996).

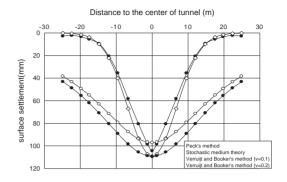


Figure 14. Settlement trough calculated using 3 different methods (Zu and Liu).

The example presented on Figure 14 exhibits very large differences, but it is in fact an extreme case, with a cover depth of less than one meter for a tunnel 6 meters in diameter.

They present also some observed settlement troughs from Shanghai metro line 7 construction. It would have been interesting to have more details on the tunnel works leading to these settlement troughs and to compare the observed settlement troughs to the calculation methods presented in the first part of the paper.

#### 3.5 Effect of vibrations

In their 2 complementary papers, *Cui and co-authors* investigate experimentally the dynamic loading and the development of pore pressure of saturated silty clays near Shanghai subway Line No.2, during the passage of metro trains. On the observed site, settlements exceeding 20 cm where observed, but no details on the evolution of these settlements is given.

The experimental study is based on field observations and dynamic triaxial tests.

Based on in situ measurements, the authors propose an experimental law of attenuation of the dynamic loading with the distance to the tunnel. This law, due to the polynomial approximation adopted, has certainly a limited domain of validity, and cannot be extrapolated outside the range of distance corresponding to the measurement points.

They studied also the development of pore pressure with the dynamic loading, both in the field and by triaxial tests; but the relation of field measurements results with dynamic triaxial tests is not clearly described.

It would have been interesting to have more details concerning the long term evolution of pore pressure in situ, combined also with the evolution of soil deformations during cycling loading tests, in order to explain the observed large settlements.

Baimakhan and co-authors propose a coupled approach (analytical and numerical) to determine the stresses induced by earthquakes on the lining of tunnels of subway lines.

Using a concept of homogeneous anisotropic elastic medium they consider the effect of the succession of different soil or rock layers.

They analyse in a more specific way the case of tunnel or galleries with a longitudinal axis making an angle with the major direction of anisotropy.

# 4 GENERAL PAPERS ON DESIGN METHODS AND TOOLS

*Koungelis & Augarde* compare, on an academic example, the results given by 2 different FE codes, Strand 7 and Plaxis. They investigate the effect of surface loading on wished in place tunnels in soft ground assuming plane strain conditions.

The initial conditions and soil characteristics are the same in all their simulations, except for dilatancy. The two meshes used in their comparative study are quite different: in Plaxis the mesh is more refined and consists of fifteen – noded triangular elements, and in Strand 7 the mesh is coarser, and moreover consists of simple 6 noded triangular elements.

The authors compare the changes in horizontal and vertical diameters for different position of the surface load. There are actually small differences although there is a great difference between the refinement of the meshes and the type of triangular elements.

Although no indication is given in the paper, one can suppose that in this example, plastic zones are certainly very limited or absent around the tunnel. Therefore, this paper compares essentially the influence of the mesh refinement and of the type of element in a linear elastic case, which explains the fact that only minor difference is noted. Such comparisons should be extended to less academic situations, where the tunnel construction is simulated and where small strain behaviour is modelled or where large plastic zones are mobilised.

Jeon and coauthors present an interesting communication on the use of geostatistical methods for assessing the spatial distribution of the rock mass characteristic named RMR. They compare a method named SIS (Juang et al., 2003, Feng et al., 2006) to the more classical kriging (Marinoni, 2003; Pardo-Igúzquiza and Dowd, 2005).

The problem considered here, is how to assess, on the base of limited bore holes, the ground characteristics along the tunnel alignment. The application presented concerns deep rock tunnels, where geostatistical estimations of RMR around a tunnel are compared.

Compared to kriging which gives a deterministic value at each point considered, SIS gives a statistical distribution of the unknown value with different

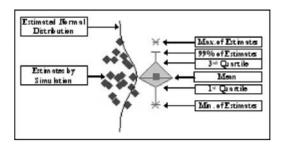


Figure 15. Distribution of estimated value.

characteristics of this statistical distribution as illustrated on Figure 15.

Such a distribution is important information for tunnelling projects, based on a limited number of investigation points, in order to evaluate the limits of the design and for risk assessment.

One of the difficulties for multiplying 2D or 3D numerical simulations on tunnelling projects is that the pre-processing tasks are much time consuming.

Li and co-authors propose in their paper a methodology to derive a FEM model from numerical geological models, which are more and more used in the frame of large geotechnical projects.

The figures presented Figure 16 show some of the stages, beginning from the geological model and ending to a 3D finite element mesh and where the soil characteristics are imported from the geological model.

In such an approach, coupling the geological model with the geostatistical method presented in the previous paper could be certainly a very powerful tool for helping the designer to test different hypothesis of the soil parameters.

*Negro* presents the results of a comprehensive survey of current design practice in Brazil. The analysis of this survey is based on the answers of 20 experts. The topics of the survey concern the main aspects of tunnel design:

- Tunnel heading stability,
- Settlement,
- Damage to existing structures,
- Lining design,
- Account of ground water loading,
- 2D/3D FEM or FDM models,
- Soil models in FEM/FDM models,
- Soil investigations,
- Monitoring.

The results of this survey are clearly summarized and analysed in the paper by Negro, therefore, in this report are given only some typical examples.

From this survey, it results that the typical scenario for tunnel projects in Brazil is the following: tunnels with equivalent diameter larger than 6 m, driven under

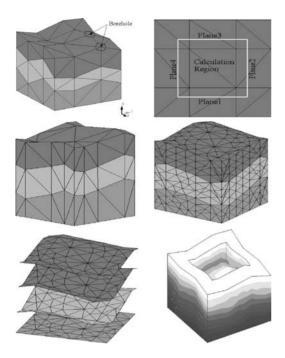
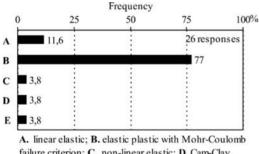


Figure 16. Some stages from the geological model to the 3D FE model.



failure criterion; C. non-linear elastic; D. Cam-Clay family; E others.

Figure 17. Soil models used in numerical analyses.

mixed face condition, in cohesive soils, below water table, and using sprayed concrete as lining (NATM).

Concerning the assessment of tunnel face stability, the survey shows that there is a large range of methods currently used, and it is noticed that some of these methods such as limit equilibrium, upper bound solutions or empirical methods can be unsafe.

It is highlighted also that Practitioners are in fact unhappy with the available methods for stability analysis, which certainly explains this broad range of methods used in practice.

Another interesting result concerns the constitutive models used in numerical analysis (Figure 17).

A large majority still use the linear elastic/plastic Mohr Coulomb model which is well known, but certainly often not adapted to shallow urban tunnels where the limitation of soil movements lead to small strains. Very few or no plasticity will be mobilised, and the model will be equivalent to a simple linear elastic model which very poorly describes the small strain behaviour of the soil. It can be added that this simple model can be unsafe in coupled analysis, as the volume changes in the soil are not correctly described.

This example shows that, with the large availability of 2D or 3D Finite element or finite difference powerful codes, there is certainly a need for clarifications concerning the types of soil models to be used in different situations and also a need for dissemination of this knowledge.

The rich conclusions of such a survey could be certainly a good base for developing and enhancing good practices in the field of tunnel design. For this reason TC28 has proposed to launch national surveys based on the example proposed by Negro, to be collected and analysed for the next TC28 symposium.

# 5 CONCLUSION

After excluding the 4 general papers, the 15 remaining papers allocated to this session, were related to 9 different specific topics.

This highlights that tunnels and deep excavations are complex works, with strong interaction with their environment, and that there is obviously a demand for simple calculation tools addressing specific problems, easy to use, especially at the preliminary design phases.

Concerning the calculation methods, it can be noticed that 9 papers concern analytical or mixed approaches as only 5 concern numerical methods. Therefore, considering the limits of the analytical approaches, due generally to the restrictive hypotheses necessary to obtain a closed form solution, it should be certainly useful now to develop simple numerical tools, easy to use and time saving, dedicated to limited specific problems. These tools could be based on existing codes and therefore able to take into account advanced soil models and realistic geometries. Such tools, after a comprehensive evaluation of their limits, could be certainly useful for practitioners.

Another way to be considered is the development of easy to use pre-processing tools, such as the example presented in this session, to facilitate the use of complex 3D models.

And finally, it should be stressed that all these calculation methods have to be validated carefully and in a scientific way against comprehensive measurements, and that the limitations of these models should be clearly indicated.

### LIST OF PAPERS WITHIN SESSION

- Baimakhan, R.B., Danaev, N.T., Baimakhan, A.R., Salgaraeva, G.I., Rysbaeva, G.P., Kulmaganbetova, Zh.K., Avdarsolkyzy, S., Makhanova, A.A. & Dashdorj, S. Calculation of the three dimensional seismic stressed state of "Metro Station–Escalator–Open Line Tunnels" system, which is located in inclined stratified soft ground.
- Bao, H., Zhang, D. & Huang, H. A Complex Variable Solution for Tunneling-Induced Ground Movements in Clays.
- Chen, J., Matsumoto, A. & Sugimoto, M. Simulation of articulated shield behavior at sharp curve by kinematic shield model.
- Cui, Z.D., Tang, Y.Q. & Zhang, X. Deformation and pore pressure model of the saturated silty clay around a subway tunnel.
- Hoefsloot, F.J.M. Analytical solution of longitudinal behaviour of tunnel lining.
- Jeon, S., Hong, C. & You, K. Design of tunnel supporting system using geostatistical methods.
- Koungelis, D.K. & Augarde, C.E. Comparative study of software tools on the effects of surface loads on tunnels.
- Li, X.X., Zhu, H.H. & Lin, Y.L. Geologic model transforming method (GMTM) for numerical analysis modeling in geotechnical engineering.
- Lu, T.K., Guo, B.H., Cheng, L.C. & Wang, J. Review and interpretation of intersection stability in deep underground based on numerical analysis.
- Lu, Z.P. & Liu, G.B. Analysis of surface settlement due to the construction of a shield tunnel in soft clay in Shanghai.
- Negro, A. Urban Tunnels in Soil: Review of Current Design Practice in Brazil.
- Shi, Z., Bao, W., Li, J., Guo, W. & Zhu, J. A study on loads from complex support system using simple 2D models.
- Shin, Y.J., Shin, J.H. & Lee, I.M. Ground Reaction due to Tunnelling below Groundwater Table.
- Song, X.Y. & Huang, M.S. Basal Stability of Braced Excavations in K0-consolidated Soft Clay by Upper Bound Method.
- Sozio, L.E. Analytical Two and Three Dimension Models to Assess Stability and Deformation Magnitude of Underground Excavations in Soil.
- Tang, Y.Q., Cui, Z.D. & Zhang, X. Dynamic Response of Saturated Silty Clay around a Tunnel under Subway Vibration Loading in Shanghai.
- Zhang, C.R., Huang, M.S. & Liang, F.Y. Lateral Responses of Piles due to Excavation-Induced Soil Movements.
- Zhang, L.M. & Wang, Z.Q. Elastic-plastic analysis for surrounding rock of pressure tunnel with lining based on material nonlinear softening.
- Zhu, W., Kou, X., Zhong, X. & Huang, Z. Modification of Key Parameters of Longitudinal Equivalent Model for Shield Tunnel.

#### REFERENCES

- Bakker, K.J. 2000. Soil Retaining Structures. Rotterdam: Balkema.
- Bogaards, P.J. & Bakker, K.J. 1999. Longitudinal bending moments in the tube of a bored tunnel. *Numerical Models* in Geomechanics Proc. NUMOG VII: p. 317–321.
- Casarande, A. & Carillo, N. 1944. Shear failure of anisotropic soil. J. of the Boston Society of Civil Engineers, 31(4).
- Feng, Y., Tang, S. & Li, Z. 2006. Application of improved sequential indicator simulation to spatial distribution of forest type. *Forest Ecology and Management* 222: 391–398.
- Hashash, Y.M.A. & Whittle, A.J. 1996. Ground movement prediction for deep excavations in soft clay. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 122(6): 474–486.
- Juang, K., Chen, Y. & Lee, D. 2003. Using sequential indicator simulation to assess the uncertainty of delineating heavy-metal contaminated soils. *Environmental Pollution* 127: 229–238.
- Leung, C.F., Chow, Y.K. & Shen, R.F. 2000. Behavior of pile subjected to excavation-induced soil movement. *Journal of Geotechnical and Geoenvironmental Engineering*, 126(11): 947–954.
- Marinoni, O. 2003. Improving geological models using a combined ordinary–indicator kriging approach. *Engineer*ing Geology 69(1–2): 37–45.
- Pardo-Igúzquiza, E. & Dowd, P.A. 2005. Multiple indicator cokriging with application to optimal sampling for environmental monitoring. *Computers & Geosciences* 31(1): 1–13.
- Park, K.H. 2004. Elastic solution for tunneling-induced ground movements in clays. *International Journal of Geomechanics* 4(4): 310–318.
- Peck, R.B. 1969. Deep excavations and tunneling in soft ground. Proceeding of 7th international conference on soil mechanics and foundation engineering. Mexico City: State of the Art Report.
- Poulos, H.G. & Chen, L.T. 1997. Pile response due to excavation-induced lateral soil movement. *Journal of Geotechnical and geoenvironmental engineering*, ASCE, 123(2): 94–99.
- Sugimoto, M. & Sramoon, A. 2002. Theoretical model of shield behavior during excavation I: Theory. *Journal of Geotechnical and Geoenvironmental Engineering* 128(2): 138–155.
- Talmon, A.M., Bezuijen, A. & Hoefsloot, F.J.M. 2008. Longitudinal tube bending due to grout pressures. *Shanghai: TC28*.
- Verruijt, A. & Booker, J.R. 1996. Surface settlements due to deformation of a tunnel in an elastic half plane. *Geotechnique* 46(4): 753–756.

# Analysis and numerical modeling of deep excavations

# R.J. Finno

Department of Civil and Environmental Engineering, Northwestern University, Evanston, IL, USA

ABSTRACT: The sixteen papers comprising the general theme "Analysis and Numerical Modeling of Deep Excavations" of the 6th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground are summarized herein. General characteristics of all papers are presented as are brief summaries of each paper. Most papers included presentation of results of finite element simulations and attendant comparisons with various aspects of observed field performance. Some of the pitfalls for making comparisons between numerical results and field observations of deep excavation performance are discussed briefly. In particular, the effects of modeling construction details and selection of appropriate constitutive models are presented. Recommendations are tendered regarding the essential information that should be conveyed in papers that present results of numerical calculations.

# 1 INTRODUCTION

Sixteen papers concerning analysis and numerical modeling of deep excavations were published in the Proceedings of the 6th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground. Of these papers, ten were presented orally at the symposium. An overview of the papers is made to provide a snapshot of the state of the practice as regards to this topic. All papers are summarized and trends in the contents are discussed. Because most papers included presentation of results of finite element simulations and attendant comparisons with various aspects of observed field performance of deep excavations. general comments regarding factors that can be important in order to make an accurate prediction. Finally, recommendations are tendered regarding the essential information that should be conveyed in papers that present results of numerical calculations.

#### 2 OVERVIEW

The sixteen papers covered the broad topics summarized in Table 1. The classification is somewhat arbitrary and several papers could have fit into more than one topic. It is clear, however, that the majority of papers explicitly included comparisons of computed results and some type of performance data.

Of these 16 papers, results of finite element analyses were reported in ten of them. Table 2 summarizes the FE codes that were used. These results agree with the author's experience that the commercial codes

Т	abl	le l	1. 1	l'opics	and	num	ber	of	papers.
---	-----	------	------	---------	-----	-----	-----	----	---------

Торіс	No. of papers
Numerical analysis and measurements	7
Numerical analysis	3
Back-analysis	2
Measurements	1
Design	1
Stress path	1
Earth pressure	1

Table 2. Summary of finite element codes used in papers.

Analyses	Number and code used			
Three-dimensional simulations	2 -Plaxis 3D Foundations 2- FLAC3 2-Research code (?)			
Plane strain simulations	6-Plaxis 1-GeoTunnel 1-Research code (?)			

FLAC and PLAXIS are most commonly used in both geotechnical practice and research. The ease of use of these codes has progressed to the point where threedimensional analyses have become more common, as suggested by the number of such analyses presented in the papers in this session.

While the use of finite element analyses has become more common in practice, likely as a result of the

Table 3. Constitutive model summary.

Model	No. of applications			
Mohr-Coulomb	6			
Hardening Soil	3			
Duncan-Chang	1			

improved i/o of commercial codes, the accuracy of the results depends the faithful representation of activities that induce stress changes in the ground during excavation and on the constitutive responses assumed in the analyses. Simple models are easy to use, but are limited in the types of computed responses that will agree with observations.

A key aspect to applying finite element analysis to practical problems in geotechnics remains the selection of the soil model and its individual parameters. Of the ten papers that presented results of FE analyses, six assumed soil responded as a Mohr-Coulomb material, as shown in Table 3. This assumption limits the predictive capabilities of a FE simulation of deep excavations or tunneling in that elastic response is assumed until the soil fails. Presumably, this choice was made because of lack of detailed laboratory or field characterizations of the soils considered by the authors in their papers. At the cost of simplicity, the Hardening-Soil and Duncan-Chang models are non-linear and can account for different responses in loading and unloading. But these models also are limited in their predictive capabilities in that they do not account for the incremental non-linearity and small strain stiffness responses that all soils exhibit. In any case, the factors leading to the selection of a soil model should be discussed in a paper to put the results in context. In the same spirit, the parameters and a rationale for their selection also should be included in a paper.

# **3 SUMMARY OF PAPERS**

#### 3.1 Numerical analysis and measurements

Li and Huang presented "Construction monitoring and numerical simulation of an excavation with SMW retaining structure." A SMW retaining structure was used to support two long excavations in Shanghai. Bearing and deformation mechanisms of the SMW were analyzed briefly and the structural analysis of SMW was discussed. Based on the in-situ excavation procedures, the authors simulated construction of the wall numerically using the FE code FLAC3D. They represented soil behavior with a Mohr-Coulomb model and considered two cases. Case 1 was the typical construction situation at the site wherein the supports were installed in a timely fashion. Case 2 considered the situation wherein the supports were not installed in a timely fashion in the lateral direction, thereby leaving excessively large amounts of wall without lateral support. They compared the computed deformations of the retaining structure, the horizontal displacement at the top of SMW and the axial forces of steel pipe supports with the field observation data for both instances. The authors concluded good agreement was shown between the computed and observed results. From a practical point of view, they showed that the normal construction sequencing in case 1 resulted in a stable and safe excavation; the axial forces were lower than the alarm values and the displacement due to excavation were within the permissible range. In case 2, however, the computed results showed the excavation was close to becoming unstable, and measures had to be taken to protect the retaining structure from failure.

Popa et al. presented "Numerical modeling and experimental measurements for a retaining wall of a deep excavation in Bucharest, Romania." They summarized a case history of a diaphragm wall for a deep basement of a new building in the center of Bucharest. The excavation impacted a number of historic structures, leading to the use of "top-down" techniques to support the excavation. The numerical results obtained by plane strain FE simulation were compared with measurements recorded during construction. Soil behavior was assumed to be that of a Mohr-Coulomb material. The computed lateral displacements were 15% and 75% of the observed values, depending on if the comparisons were made in an area with or without a grouted wall - not explicitly modeled in the FE simulations - adjacent to the diaphragm wall.

Schweiger et al. presented "3D finite element analysis of a deep excavation and comparison with in situ measurements." The paper describes the results of FE analyses using Plaxis of a deep excavation project in clayey silt in Salzburg. The excavation was supported by a diaphragm wall, a jet grout panel and three levels of struts. The soil responses were represented by the Hardening-Soil model. Because of insufficient information available at the time of design on the material properties of the jet grout panel, the authors varied its stiffness in a parametric study. The effect of taking into account the stiffness of a cracked diaphragm wall on the deformations also was investigated. In some of the 3D calculations, the authors simulated nonperfect contact between the diaphragm wall and a strut by means of a non-linear behaviour of the strut. The evaluation of the results and comparison with in situ measurements showed that analyses which took into account the reduced stiffness of the diaphragm wall due to cracking achieved the best agreement with the measurements. Furthermore settlements of buildings could be best reproduced by the three-dimensional model, although the predicted settlements were not in good agreement with the observations.

Zhang and Huang presented "Monitoring and modeling of riverside large deep excavation-induced ground movements in clays." They discussed a deep excavation located at the Shanghai international passenger center that was 800 m long and 100-150 m wide with the depth of 13 m. The south side of the deep excavation was within 4.6 m of a parallel flood wall of the Huangpu River. The north side of the excavation was 5 m from a historic building. Because of the differences in the conditions on the two sides of the excavations, Plaxis FE analyses were conducted which explicitly included both sides of the excavation, rather than a centerline symmetric condition. Soil responses were assumed to correspond to that of a Mohr-coulomb material. Computed differences of lateral wall movements on each side differed by as much as 50%, as was verified by field observations made during construction.

Hsi et al. presented "Three-dimensional finite element analysis of diaphragm walls for top-down construction." They discussed the Tugun Bypass Tunnel in Gold Coast, Australia. The tunnel was constructed using diaphragm walls with the top-down cut-andcover method to allow simultaneous construction of an airport runway extension above the tunnel, and excavation of the tunnel beneath. The tunnel was built in deep deposits of saturated, alluvial and estuarine soils with the toes of the walls founded in soil deposits. There was a potential risk for differential settlements between the diaphragm wall panels, caused by the runway fill placed over the tunnel roof during excavation. Three-dimensional numerical modeling was undertaken with Plaxis 3D Foundation to predict the differential settlements of the tunnel arising from the variable subsurface profile, staged excavation and dewatering, non-uniform loading and soil-structure interaction. Soil was assumed to behave as a Hardening-Soil material. Settlements measured after construction were within the range of those computed with the finite element simulations.

Phienwej presented "Ground movements in station excavations of Bangkok first MRT." The characteristics of the lateral movements of the diaphragm walls at excavations for 18 stations of the first Bangkok underground MRT line were evaluated. Three modes of deflected shapes of the walls were observed at different excavation depths, namely a cantilever mode and braced modes with a bulge in soft clay and a bulge in stiff clay. The ratio of maximum lateral wall deflection as a function of excavation depth and the ratio of ground surface settlement to excavation depth and the normalized ground surface settlement varied with the mode of wall deflection. Undrained undrained Young's moduli for a Mohr-Coulomb constitutive response for different soil layers were back-calculated from wall movement data of three selected stations using the 2-D Plaxis FE code. The modulus values, which were

higher than those commonly obtained from conventional triaxial tests, can be used as guideline for future excavations in Bangkok.

Ota et al. presented "Consideration of design method for braced excavation based on monitoring results." They compared observed and design values of wall deflections at several cut-and-cover excavations through soft and sensitive clay ground at the Osaka Subway Line No.8. A beam-spring model was employed in the braced design method which accounted for the characteristics of the Osaka soft ground. While there was good agreement between the observed data and design values in past results, the observed wall deflections in this study were larger than that expected for construction sites wherein the excavations encountered 10 to 20 m thick, soft and sensitive clay layer. The authors discuss how they evaluated the horizontal coefficient of subgrade reaction  $k_h$  on the excavation side of soft clay layer. The authors make new recommendation regarding selection of  $k_h$ , and show that the calculated wall movements with the revised values agree with the observations. These recommendations are applicable to the soft and sensitive Osaka clays.

# 3.2 Numerical analysis

Li and Yang presented the paper "Numerical evaluation of dewatering effect on deep excavation in soft clay." They described a FLAC3D analysis that modeled top-down construction of a 33.7 m deep underground transformer substation in the downtown area of Shanghai. There are both unconfined and confined aquifers on the site of this project and drainage by desiccation in the foundation pit was adopted. Assuming a Mohr-Coulomb soil response, the effective stress methods of analysis incorporated excavation and dewatering of the foundation pit as part of the simulation of construction activities. The computed wall deflections, basal heaves and surface settlements based on analyses that did not consider dewatering were compared to those that did. Results of analyses that considered leakage through the wall and leakage between the aquifers are presented as well. The analysis shows that although the computed differences in lateral wall movement and basal heave were small, due to the low permeability of the soil, dewatering increased the amount of computed surface settlements as a result of drawdown of the water outside the walls of the excavation.

Li et al. contributed the paper "Analysis of the factors influencing foundation pit deformations. They presented results of FE computations based upon 3-D Biot's consolidation theory, assuming the soil responded as a nonlinear Duncan-Chang's material. The finite element equations explicitly considered the coupling of groundwater seepage and soil skeleton deformation during excavation. They presented results that showed the individual effects of the influence of soil permeability, rigidity and levels of lateral supports, rigidity of retaining wall and excavation duration on ground surface settlement, wall horizontal displacement and basal heave of an excavation.

Siemiñska-Lewandowsk and Mitew-Czajewska presented the paper "The effect of deep excavation on surrounding ground and nearby structures." They described problems related with the construction of a 29 m deep excavation of Nowy Swiat Station (S11) of 2nd metro line in Warsaw. A critical section of the project consisted of 7 stations and 6 running tunnels – 6 km length in total. Running tunnels will be constructed using TBM while the stations are to be constructed using cut and cover techniques. Deep excavation will be made with diaphragm walls supported by several levels of slabs and struts. They presented results of 2-D Plaxis FE analyses in terms of ground surface settlements, displacements of surrounding foundations and lateral wall movements, assuming the soil behaves as a Mohr-Coulomb material. Additionally, settlements of the surface were calculated above the TBM (running tunnels). Resulting values of settlements in both cases were discussed, and formed the basis of design predictions that will be verified during construction.

# 3.3 Back analysis

Zghondi et al. presented the paper, "Multi-criteria procedure for the back-analysis of multi-supported retaining walls." They described a numerical backanalysis procedure for multi-supported deep excavations based on the optimization of several indicators, taking in account the forces in the struts and the differential pressures derived from the wall displacement. The evaluation of the procedure is based on results of 1 g small scale laboratory experiments on semi-flexible retaining walls embedded in a Schneebelli material. The proposed numerical procedure was applied to an excavation with 2 levels of struts with low stiffness. The optimized Hardening Soil Model parameters form the basis of calculations of response of 14 different tested configurations. The results are compared with the classical methods, SubGrade Reaction Method, Finite Element analysis with Mohr Coulomb model and with the back-analysis using Hardening Soil Model parameters based on biaxial tests results.

Zhang et al. contributed the paper "Study on deformation laws under the construction of semi-reverse method." Taking a 24.1-m-deep foundation pit of Shanghai Metro Line 1 which uses the semi-reverse construction process of "three open excavating-one tunneling" as an example, they determined deformation laws of a foundation pit under the construction of a semi-reverse method based on analysis of field monitoring data and forward and back analyses methods. They employed Plaxis v8 and assumed the soil acted as a Hardening-Soil material in their computations. The authors stated that results of this approach indicated that the semi-reverse method is an effective way to improve rigidity of the exterior support, control the deformation of excavation, and ensure safety of the surrounding buildings and pipelines.

# 3.4 Measurements

Zhang et al. contributed the paper "GPS height application and gross error detection in foundation pit monitoring." The authors introduced a deformation monitoring model that combined traditional survey technology and GPS measurements. They illustrated foundation pit deformation monitoring based on their experience of deep foundation pit construction of an underground tunnel in Lishui Road, Hangzhou city. When analyzing GPS height conversion to improve the reliability of the GPS datum, they employed Dixon's test in the GPS datum mark to determine potential height anomalies. The authors concluded that this approach is a convenient way to search and delete raw data that includes gross errors.

# 3.5 Design

Chang contributed the paper "Optimization design of composite soil-nailing in loess excavation." Excavations through loess have unique characteristics compared with the others due to its structural properties and collapsibility. To evaluate the mechanisms of support and to develop reasonable methods to design composite soil-nailing in loess excavation, the authors used results of finite element analysis to design a soil nail support system. Their optimization design methods are based on the results of finite element analysis apparently assuming Mohr-Coulomb soil responses. They conducted the simulations to determine the regularity of deformation and the safety factor, as functions of selected design variables. The authors justified their methods by reporting that the lateral deformations of the example excavation were limited to 16 mm.

#### 3.6 Stress path

Zhou et al. contributed the paper "Comparison of theory and test on excavation causing the variation of soil mass strength." In view of the characteristic unloading caused by excavations, they deduced the strength ratio of the unloaded soil to soil subjected to compression loadings. Laboratory tests simulating excavation were carried out based on Hvorslev's strength theory. By comparing theoretical results with the laboratory data, they concluded that the soil mass is overconsolidated. As a consequence, the authors stated that the soil microstructure is damaged, and the soil mass strength is reduced in the unloading process. The authors concluded that analyses of the results are helpful to the understanding of the effect of excavation unloading on the variation of the soil mass strength.

#### 3.7 Earth pressure

Lin and Lee contributed the paper "A simplified spatial methodology of earth pressure against retaining piles of pile-row retaining structure." When using a pile-row retaining structure to support excavation, the authors stressed the importance of obtaining the magnitude and distribution of the earth pressure against the retaining piles. Based on the mode of failure, a new methodology is proposed to evaluate the earth pressure against the retaining piles of such a structure. In the proposed method, both the spatial effect and intermediate principal stress effect are considered. The authors provide an example of the methodology applied to engineering practice. They demonstrated that the strength theory has more influence on earth pressure.

### 4 COMMENTS REGARDING COMPUTED AND OBSERVED RESULTS

Many factors affect ground movements caused by excavations, including stratigraphy, soil properties, support system details, construction activities, contractual arrangements and workmanship. In this theme, most papers described numerical simulations that analyzed ground response arising from excavation. Finite element predictions always contain uncertainties related to soil properties, support system details and construction procedures. Furthermore, while supported excavations commonly are simulated numerically by modeling cycles of excavation and support installation, it generally is necessary to simulate all aspects of the construction process that affect the stress conditions around the cut to obtain an accurate prediction of behavior. This may involve simulating previous construction activities at the site, installation of the supporting wall and any deep foundation elements, as well as the removal of cross-lot supports or detensioning of tiedback ground anchors. Issues of time effects caused by hydrodynamic effects or material responses may be important. The following sections summarize some of the factors that may impact computed responses of ground movements associated with excavations. Proper consideration must be given to such factors when making such analyses, as well as when critically evaluating published results of the same.

#### 4.1 Drainage conditions

An important preliminary decision in any analysis is to match the expected field drainage conditions, which impacts the formulation required. Clough and Mana (1976) and field data have shown that for excavations through saturated clays with typical excavation periods of several months, the clays remain essentially undrained with little dissipation of excess pore pressures.

For undrained conditions, one can employ either a coupled finite element formulation where both displacements and pore water pressures are solved for explicitly (e.g. Carter et al. 1979) or a penalty formulation (e.g. Hughes 1980) wherein the bulk modulus of water - or a sufficiently large number that depends on the precision of the machine making the computation is added to the diagonal terms in the element stiffness matrix during global matrix assembly. This additional term constrains the volumetric strain to nearly zero, i.e., undrained. In both these approaches, the constitutive response of the soil is defined in terms of effective stress parameters. A simpler, alternate approach is to define undrained constitutive response in terms of total stress parameters, with care being taken to make the diagonal terms of the element stiffness matrix large, typically by using a Poisson's ratio close to 0.5. In this case, a Young's modulus corresponds to an undrained value and failure is expressed in terms of an undrained shear strength,  $S_u$  (e.g.,  $\varphi = 0$  and  $c = S_u$ ).

However, there may be cases (e.g., O'Rourke and O'Donnell 1997) where substantial delays during construction occur and excess pore pressures partially dissipate, and in these cases one must use a mixed formulation to account for the pore water effects. When using top-down techniques to excavate, it can take up to several years to reach final grade for large excavations, and hence partially drained conditions would apply therein, requiring a coupled finite element simulation.

# 4.2 Initial conditions

A reasonable prediction of the ground response to construction of a deep excavation starts with a good estimate of the initial stress conditions, in terms of both effective stresses and pore water pressures. The effective stress conditions for excavations in welldeveloped urban areas rarely correspond to at-rest conditions because of the myriad past uses of the land. Existence of deep foundations and/or basements from abandoned buildings and nearby tunnels changes the effective stresses from at-rest conditions prior to the start of excavation. For example, Calvello and Finno (2003) showed that an accurate computation of movements associated with an excavation could only be achieved when all the pre-excavation activities affecting the site were modeled explicitly. They used the case of the excavation for the Chicago-State subway renovation project (Finno et al. 2002), wherein construction of both a tunnel and a school impacted the ground stresses prior to the subway renovation project. Ignoring these effects made a difference of a factor of 3 in the computed lateral movements.

One also must take care when defining the initial ground water conditions. Even in cases where the ground water level is not affected by near surface construction activities, non-hydrostatic conditions can exist for a variety of reasons. For example, Finno et al. (1989) presented pneumatic piezometer data that indicated the presence of a downward gradient within a 20 m thick sequence of saturated clays. This downward flow arose from a gradual lowering since the 1950s of the water level in the upper rock aquifer in the Chicago area. A non-hydrostatic water condition affects the magnitude of the effective stresses at the start of an excavation project.

An engineer has two choices to define such conditions – to measure the *in situ* conditions directly or to simulate all the past construction activities at a site starting from appropriate at-rest conditions. Because both approaches present challenges, it is advantageous to do both to provide some redundancy in the input. In any case, careful evaluation of the initial conditions is required when numerically simulating supported excavation projects, especially in urban areas.

## 4.3 Wall installation

Many times the effects of installing a wall are ignored in a finite element simulation and the wall is "wishedinto-place" with no change in the stress conditions in the ground or any attendant ground movements. However, there are abundant data that show ground movements may develop as a wall is installed.

O'Rourke and Clough (1990) presented data that summarized observed settlements that arose during installation of five diaphragm walls. They noted settlements as large as 0.12% of the depth of the trench. These effects can be evaluated by 3-dimensional modeling of the construction process (e.g., Gourvenec and Powrie 1999), but not without several caveats. The specific gravity of the supporting fluid usually varies during excavation of a panel as a result of excavated solids becoming suspended - increasing the specific gravity above the value of the water and bentonite mixture and subsequently decreasing when the slurry is cleaned prior to the concrete being tremied into place. Consequently, it is difficult to select one value that represents an average condition. Furthermore the effects of the fluid concrete on the stresses in the surrounding soil depend upon how quickly the concrete hardens relative to its placement rate. Some guidance in selecting the fresh concrete pressure is provided by Lings et al. (1994).

It is less straightforward when modeling diaphragm wall installation effects in a plane strain analysis because the arching caused by the excavation of individual panels cannot be taken directly into account. To approximate the effects of this arching when making such an analysis, an equivalent fluid pressure, generally higher than the level of the fluid during construction, can be applied to the walls of the trench to maintain stability. Thus, some degree of empiricism is required to consider these installation effects in a plane strain analysis. One can back-calculate an equivalent fluid pressure corresponding to the observed ground response if good records of lateral movements close to the wall are recorded during construction. More data of this type are needed before any recommendations can be made regarding magnitudes of appropriate equivalent pressures.

The effects of installing a sheet pile wall are different than those of a diaphragm wall, yet the effects on observed responses also can be significant. In this case, ground movements may arise from transient vibrations developed as the sheeting is driven or vibrated into place and from the physical displacement of the ground by the sheeting. The former mechanism is of practical importance when installing the sheeting through loose to medium dense sands, and can be estimated by procedures proposed by Clough et al. (1989). However, these effects are not included in finite element simulations. The latter mechanism in clays was illustrated by Finno et al. (1988). In this case, the soil was displaced away from the sheeting as it was installed. This movement was accompanied by an increase in pore water pressure and a ground surface heave. As the excess pore water pressures dissipated, the ground settled. The maximum lateral movement and surface heave was equal to one-half the equivalent width of the sheet pile wall, defined as the crosssectional area of the sheet pile section per unit length of wall. Sheet-pile installation can be simulated in plane strain by using procedures summarized in Finno and Tu (2006).

In addition to the movements that occur as a wall is installed, installing the walls can have a large influence on subsequent movements, especially if the walls are installed relatively close to each other, as may be the case in a cut-and-cover excavation for a tunnel. Sabatini (1991) conducted a parametric study as a function of the depth, H, to width, B, of an excavation, wherein the effects of sheet-pile wall installation in clays were compared with simulations where the walls were wished into place. The results of the study are shown in Figure 1 where the computed normalized maximum lateral movements,  $\delta H_{(max)}/H$ , are plotted versus H/B.

It is apparent for wide excavations (H/B  $\leq$  0.25) that the decision to include installation effects in a simulation is not critical. However, these effects become pronounced for narrow excavations (H/B  $\gg$  1) and should be explicitly considered. The results also show that for the "wished-in-place" case when the sheet-pile installation effects are ignored, the lateral

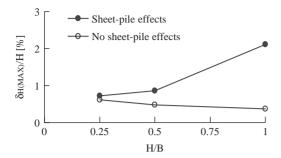


Figure 1. Effects of sheet-pile installation on computed lateral movements.

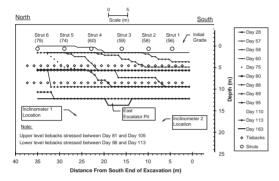


Figure 2. Construction progress at excavation in Chicago (Finno et al. 2002).

movements are larger for wider excavations, a similar trend reported by Mana and Clough (1981).

Sheet-pile installation has two main effects: the soil adjacent to the excavation is preloaded and the shear strength on the passive side is (partially) mobilized prior to the beginning of the cycles of excavation. Wall installation tends to preload the soil on the active side of the excavation as a result of the reduction in shear stress at approximately constant mean normal effective stress. This mechanism provides the soil outside the walls with more available shearing resistance when the cycles of excavation start. However, the soil between the walls has less available passive resistance as a result of the preloading and this promotes the larger movements during excavation as compared to the case of ignoring the sheet-pile effects (Finno and Nerby 1989).

### 4.4 Plane strain versus 3-dimensional analyses

Figure 2 illustrates some of the challenges of using field observations to calibrate numerical models of any kind, even when detailed records exist. This figure summarizes the construction progress at the Chicago-State excavation in terms of excavation surface and support installation on one of the walls of the excavation for selected days after construction started. Also shown are the locations of two inclinometers placed several meters behind the wall. If one is making a computation assuming plane strain conditions, then it is clear that one must judiciously choose data sets so that planar conditions would be applicable to those selected.

Even when a sufficiently extensive horizontal excavated surface is identified, 3-dimensional effects may still arise from the higher stiffness at the corners of an excavation. These boundary conditions lead to smaller ground movements near the corners and larger ground movements towards the middle of the excavation wall. Another, and less recognized, consequence of the corner stiffening effects is the maximum movement near the center of an excavation wall may not correspond to that found from a conventional plane strain simulation of the excavation, i.e., 3-D and plane strain simulations of the excavation do not yield the same movement at the center portion of the excavation, even if the movements in the center are perpendicular to the wall (Ou et al. 1996). This affect can be quantified by the plane strain ratio, PSR, defined herein as the maximum movement in the center of an excavation wall computed by 3-D analyses divided by that computed by a plane strain simulation. Finno et al. (2007) developed the following expression for PSR from the results of a finite element parametric study of excavations through clay:

$$PSR = \left(1 - e^{-k C(L/H_c)}\right) + 0.05(L/B - 1)$$
(1)

where *L* is the excavation length along the side where the movement occurs, *B* is the other areal dimension, and  $H_e$  is the excavation depth. The value of *C* depends on the factor of safety against basal heave,  $FS_{BH}$ , and is taken as:

$$C = 1 - \{0.5 (1.8 - FS_{BH})\}$$
(2)

The value of *k* depends on the support system stiffness and is taken as:

$$k = 1 - 0.0001(\frac{EI}{\gamma h^4})$$
(3)

where *EI* is the bending stiffness of the wall,  $\gamma$  is the total unit weight of the soil and *h* is the average vertical spacing between supports. When  $L/H_e$  is greater than 6, the PSR is equal to 1 and results of plane strain simulations yield the same displacements in the center of an excavation as those computed by a 3-D simulation. When  $L/H_e$  is less than 6, the displacement computed from the results of a plane strain analysis will be larger than that from a 3-D analysis. When conducting an inverse analysis of an excavation with a plane strain

simulation when  $L/H_e$  is relatively small, the effects of this corner stiffening is that an optimized stiffness parameter will be larger than it really is because of the lack of the corner stiffening in the plane strain analysis. This effect becomes greater as an excavation is deepened because the  $L/H_e$  value increases as the excavated grade is lowered. This trend was observed in the optimized parameters for the deeper strata at the Chicago-State subway renovation excavation (Finno and Calvello 2005).

## 5 CONSTITUTIVE MODEL CONSIDERATIONS

When one undertakes a numerical simulation of a deep supported excavation, one of the key decisions made early in the process is the selection of the constitutive model. In general, this selection should be compatible with the objectives of the analysis. If the results form the basis of a prediction that will be updated based on field performance data, then the types of field data that form the basis of the comparison will impact the applicability of a particular model. Possibilities include lateral movements based on inclinometers, vertical movements at various depths and distances from an excavation wall, forces in structural support elements, pore water pressures or any combinations of these data. When used for a case where control of ground movements is a key design consideration, the constitutive model must be able to reproduce the soil response at appropriate strain levels to the imposed loadings.

#### 5.1 Incremental non-linearity

It is useful to recognize that soil is an incrementally nonlinear material, i.e., its stiffness depends on loading direction and strain level. Real soils are neither linear elastic nor elastic-plastic, but exhibit complex behavior characterized by zones of high stiffness at very small strains, followed by decreasing stiffness with increasing strain. This behavior under static loading was identified through back-analysis of foundation and excavation movements in the United Kingdom (Burland, 1989). The recognition of zones of high initial stiffness under typical field conditions was followed by efforts to measure this ubiquitous behavior in the laboratory for various types of soil (e.g., Jardine et al. 1984; Clayton and Heymann 2001; Santagata et al. 2005; Calisto and Calebresi 1998, Cho 2007).

To illustrate this behavior, Figure 3 shows the results of drained, triaxial stress probes conducted on specimens cut from block samples of lightly overconsolidated glacial clays obtained at an excavation in Evanston, IL. Each specimen was reconsolidated under K<sub>0</sub> conditions to the in-situ vertical effective stress  $\sigma'_{v0}$ , subjected to a 36 hour K<sub>0</sub> creep cycle, followed by directional stress probing under drained

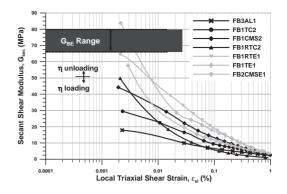


Figure 3. Secant shear modulus as a function of direction of loading.

axisymmetric conditions. Bender element (BE) tests were conducted during all phases of the tests. The secant shear moduli are plotted versus triaxial shear strain in Figure 3 for natural specimens whose stress probes involved changes in the shear stress q. The overconsolidation ratio of these specimens was 1.7, so if one assumes the response is isotropic and elastoplastic, then G should be the same for at least the initial portion of all curves. The stress probes wherein q and the stress ratio,  $\eta = q/p'$ , is increased (" $\eta$  loading") are clearly softer than those where q and  $\eta$  initially decrease (" $\eta$  unloading"). There are no obvious zones of constant  $G_{sec}$  at shear strains greater than 0.002%, and thus no elastic zone is observed in these data for strain levels. Complete details and results of the testing program are presented by Cho (2007).

Burland (1989) suggested that working strain levels in soil around well-designed tunnels and foundations are on the order of 0.1 %. If one uses data collected with conventional triaxial equipments to discern the soil responses, one can reliably measure strains 0.1% or higher. Thus in many practical situations, it is not possible to accurately incorporate site-specific small strain non-linearity into a constitutive model based on conventionally-derived laboratory data.

#### 5.2 Model selection

There are a number of models reported in literature wherein the variation of small strain nonlinearity can be represented, e.g., a three-surface kinematic model develop for stiff London clay (Stallebrass and Taylor 1997), MIT-E3 (Whittle and Kavvadas 1994), hypoplasticity models (e.g. Viggiani and Tamagnini 1999), and a directional stiffness model (Tu 2007). To derive the necessary parameters, these models require either detailed experimental results or experience with the model in a given geology. While the models can be implemented in material libraries in some commercial finite element codes, these routines are not readily

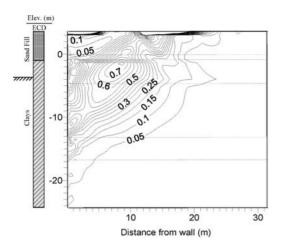


Figure 4. Shear strains behind excavation: 57 mm maximum lateral movement (contours in %).

available to most practitioners. Thus for most current practical applications, one uses simpler, elasto-plastic models contained in material libraries in commercial codes.

For these models, a key decision is to select the "elastic" parameters that are representative of the secant values that correspond to the predominant strain levels in the soil mass. Examples of the strain levels behind a wall for an excavation with a maximum lateral wall movement of 57 mm are shown in Figure 5. These strain levels were computed based on the results of displacement-controlled simulations where the lateral wall movements and surface settlements were incrementally applied to the boundaries of a finite element mesh. The patterns of movements were typical of excavations through clays, and were based on those observed at an excavation made through Chicago clays (Finno and Blackburn 2005). Because the simulations were displacement-controlled, the computed strains do not depend on the assumed constitutive behavior.

As can be seen in Figure 4, maximum shear strains as high as 0.7% occur when 57 mm of maximum wall movement develop. The maximum strains are proportional maximum lateral wall movement; for example, when 26 mm maximum lateral wall movement develops, the maximum shear strain is about 0.35%. These latter strain levels can be accurately measured in conventional triaxial testing, and thus if one can obtain specimens of sufficiently high quality, then secant moduli corresponding to these strain levels can be determined via conventional laboratory testing. Because the maximum horizontal wall displacement can be thought of as a summation of the horizontal strains behind a wall, the maximum wall movements can be accurately calculated with a selection of elastic parameters that corresponds to these expected

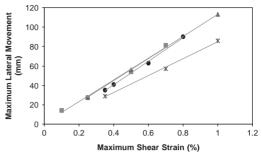


Figure 5. Relation between maximum wall movement and shear strain.

strain levels. In this case, the fact that small strain non-linearity is not explicitly considered will not have a large impact on the computed horizontal wall displacements because the maximum horizontal movement at the wall is dominated by the larger strains in the soil mass. These computed movements would be compatible with those measured by an inclinometer located close to the wall.

However, if one needs to obtain an accurate representation of the distribution of ground movements with distance from the wall, then this approach of selecting strain-level appropriate elastic parameters is not applicable. Small strain non-linearity of soil must be explicitly considered to find the extent of the settlement because the strains in the area of interest vary from the maximum value to zero. As a consequence, many cases reported in literature indicate computed wall movements agree reasonably well with observed values, but the results from the same computations do not accurately reflect the distribution of settlements. Indeed, this was the case in several papers presented as part of this theme. Good agreement at distances away from a wall can be obtained only if the small stain non-linearity of the soil is adequately represented in the constitutive model.

The relation between lateral wall displacements and shear strain levels in the soil behind the wall can be evaluated from results of displacement-controlled finite element simulations. Similar to the results shown in Figure 4, different displacement profiles were studied by imposing lateral wall displacements and settlement profiles, representing conditions with maximum lateral movements at the excavated surface, cantilever movements, deep-seated movements and combination of the latter two (Andrianis 2006). The stratigraphies used in the models were based on typical Chicago soils. The results in Figure 5 show that the relationship between maximum shear strain behind the wall and maximum displacement of the wall is almost linear for lateral wall displacements between 10 and 110 mm. Figure 5 also shows that the results form a narrow band, suggesting that the relation between strain and wall displacement is not greatly affected by the type of movement.

Figure 5 can be used to estimate shear strains for a specified maximum wall movement. With this value of shear strain, the secant shear moduli for use in conventional elasto-plastic models can be estimated based on strain-stress data from high quality laboratory experiments. The values of maximum shear strains, even in the cases with the relatively low values of displacements, are about 0.2% and increase as the specified displacement becomes larger. This is important when one determines soil stiffness in the laboratory. Conventional soil testing without internal instrumentation allows one to accurately measure strains as low as 0.1%. Thus for many cases, the secant shear moduli can be determined from conventional laboratory tests on high quality samples. However, if strain levels are 0.1% or less, then one must select these moduli from test results based on internally-measured strains in equipment not normally available in commercial laboratories.

In summary, using a simulation based on conventional elasto-plastic models limits the type and location of the data that can be used as observations in an inverse analysis. Both vertical and lateral movements measured at some distance from a wall cannot be calculated accurately in this case because the variation of stiffness with strain levels must be adequately represented in the soil model. Only the lateral movements close to a support wall can be reasonably computed with conventional models since that result is dominated by the zones of high strains behind the wall.

## 6 CONCLUDING REMARKS

The papers presented at the symposium included widely variable levels of information regarding the details of the finite element analyses. As such, the author tentatively proposes that the following information be included in any paper describing the results of any finite element simulation of geotechnicallyrelated construction.

- 1 The finite element code used.
- 2 The assumed drainage conditions, e.g., drained, undrained or partially drained.
- 3 The dimensionality of the problem, e.g., plain strain, axisymmetric or three-dimensional.
- 4 The constitutive model(s) employed for both soils and structural elements.
- 5 The parameters for each material and a discussion of the basis of their selection.
- 6 A description of the mesh, including boundary conditions and type of elements used for soil, structural components and interfaces.

7 Construction records, simulation steps and details of how each construction activity was idealized in the finite element simulation.

Finally, comparisons between computed and observed results, as well as a discussion of the comparisons, should be included.

## REFERENCES

- Andrianis, G.E. 2006. Excavation-induced Strains and Cantilever Deflections in Compressible Clays. MS thesis, Northwestern University, Evanston, IL.
- Burland, J.B. 1989. "Small is beautiful" the stiffness of soils at small strains: Ninth Laurits Bjerrum Memorial Lecture. *Canadian Geotechnical Journal* 26: 499–516.
- Calisto, L. & Calebresi, G. 1998. Mechanical behavior of a natural soft clay. *Geotechnique* 48 (4): 495–513.
- Calvello, M. & Finno, R.J. 2003. Modeling excavations in urban areas: effects of past activities. *Italian Geotechnical Journal* 37(4): 9–23.
- Carter, J.P., Booker, J.R. & Small, J.C. 1979. The analysis of finite elasto-plastic consolidation. *International Journal for Numerical and Analytical Methods in Geomechanics* 3: 107–129.
- Cho, W.J. 2007. Recent stress history effects on compressible Chicago glacial clay. PhD thesis, Northwestern University, Evanston, IL.
- Clayton, C.R.I. & Heymann, G. 2001. Stiffness of geomaterials at very small strains. *Geotechnique* 51(3): 245–255.
- Clough, G.W. & Mana, A.I. 1976. Lessons learned in finite element analysis of temporary excavations. *Proceedings*, 2nd International Conference on Numerical Methods in Geomechanics, ASCE, Vol. I: 496–510.
- Clough, G.W., Smith, E.M. & Sweeney, B.P. 1989. Movement control of excavation support systems by iterative design. *Current Principles and Practices, Foundation Engineering Congress*, Vol. 2, ASCE: 869–884.
- Finno, R.J., Atmatzidis, D.K. & Nerby, S.M. 1988. Ground response to sheet-pile installation in clay. *Proceedings, Second International Conference on Case Histories in Geotechnical Engineering*, St. Louis, MO.
- Finno, R.J., Atmatzidis, D.K. & Perkins, S.B. 1989. Observed Performance of a Deep Excavation in Clay. *Journal of Geotechnical Engineering*, ASCE, 115 (8): 1045–1064.
- Finno, R.J. & Blackburn, J.T. 2005. Automated monitoring of supported excavations. Proceedings, 13th Great Lakes Geotechnical and Geoenvironmental Conference, Geotechnical Applications for Transportation Infrastructure, GPP 3, ASCE, Milwaukee, WI.: 1–12.
- Finno, R.J., Blackburn, J.T. & Roboski, J.F. 2007. Threedimensional Effects for Supported Excavations in Clay. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 133(1): 30–36.
- Finno, R.J., Bryson, L.S. & Calvello, M. 2002. Performance of a stiff support system in soft clay. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 128(8): 660–671.
- Finno, R.J. & Calvello, M. 2005. Supported excavations: the observational method and inverse modeling. *Journal of Geotechnical and Geoenvironmental Engineering*. ASCE, 131 (7).

- Finno, R.J. & Nerby, S.M. 1989. Saturated Clay Response During Braced Cut Construction. *Journal of Geotechnical Engineering*, ASCE, 115(8): 1065–1085.
- Finno, R.J. & Tu, X. 2006. Selected Topics in Numerical Simulation of Supported Excavations. Numerical Modeling of Construction Processes in Geotechnical Engineering for Urban Environment, International Conference of Construction Processes in Geotechnical Engineering for Urban Environment, Th. Triantafyllidis, ed., Bochum, Germany, Taylor & Francis, London: 3–20.
- Gourvenec, S.M. & Powrie, W. 1999. Three-dimensional finite element analysis of diaphragm wall installation. *Geotechnique*, 49(6): 801–823.
- Hughes, T.J.R. 1980. Generalization of selective integration procedures to anisotropic and nonlinear media. *International Journal of Numerical Methods in Engineering* 15 (9): 1413–1418.
- Jardine, R.J., Symes, M.J. & Burland, J.B. 1984. The measurement of small strain stiffness in the triaxial apparatus. *Geotechnique* 34 (3): 323–340.
- Lings, M.L., Ng, C.W.W. & Nash, D.F.T. 1994. The lateral pressure of wet concrete in diaphragm wall panels cast under bentonite. *Geotechnical Engineering*, Proceedings of the Institution of Civil Engineers, 107: 163–172.
- Mana, A.I. & Cough, G.W. 1981. Prediction of movements for braced cut in clay. *Journal of Geotechnical Engineering*, ASCE, New York, 107(8): 759–777.
- O'Rourke, T.D. & Clough, G.W. 1990. Construction induced movements of insitu walls. *Proceedings, Design and Performance of Earth Retaining Structures*, Lambe, P.C. and Hansen L.A. (eds). ASCE: 439–470.

- O'Rourke, T.D. & O'Donnell, C.J. 1997. Deep rotational stability of tiedback excavations in clay. *Journal of Geotechnical Engineering*, ASCE, 123(6): 506–515.
- Ou, C.Y., Chiou, D.C. & Wu, T.S. 1996. Three-dimensional finite element analysis of deep excavations. *Journal of Geotechnical Engineering*, ASCE, 122(5): 473–483.
- Sabatini, P.J. 1991. Sheet-pile installation effects on computed ground response for braced excavations in soft to medium clays. MS thesis, Northwestern University, Evanston, IL.
- Santagata, M., Germaine, J.T. & Ladd, C.C. 2005. Factors Affecting the Initial Stiffness of Cohesive Soils. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 131(4): 430–441.
- Stallebrass, S.E. & Taylor, R.N. 1997. The development and evaluation of a constitutive model for the prediction of ground movements in overconsolidated clay. *Geotechnique* 47(2): 235–253.
- Tu, X. 2007. *Tangent stiffness model for clays including small strain non-linearity*. PhD thesis, Northwestern University, Evanston, IL.
- Viggiani, G. & Tamagnini, C. 1999. Hypoplasticity for modeling soil non-linearity in excavation problems. *Prefailure Deformation Characteristics of Geomaterials*, M. Jamiolkowski, M, Lancellotta, R. and Lo Presti, D. (eds.), Balkema, Rotterdam: 581–588.
- Whittle, A.J. & Kavvadas, M.J. 1994. Formulation of MIT-E3 constitutive model for overconsolidated clays. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 120(1): 173–198.

## Construction method, ground treatment, and conditioning for tunneling

T. Hashimoto & B. Ye

Geo-Research Institute, Osaka, Japan

G.L. Ye

Department of Civil Engineering, Shanghai Jiaotong University, Shanghai, P.R. China

ABSTRACT: This general report reviews a selected group of papers of Session 2 which is related to "Construction Method, Ground Treatment, and Conditioning for Tunneling". The papers are divided into 5 groups based on their topics: (1) construction methods with case studies, (2) ground treatment, (3) load and pressure, (4) conditioning additives for EPB, (5) others. Before reviewing, the geotechnical aspects in these fields are firstly summarized, and then the essences of these papers are presented. The deficiencies and future developments are also discussed.

## 1 INTRODUCTION

Session 2 includes 19 papers from China, Japan, UK, the Netherlands, Germany, Iran, Slovakia, Argentina and Brazil. Especially, Tongji University, Shanghai and GeoDelft, the Netherlands contribute to present some papers respectively. It is because that Shanghai and the Netherlands have been performed many tunnels during the past decade in the soft ground. These papers are divided into 5 groups and 8 subgroups based on their topics, which are shown in Table 1. Although all these paper has contributed to some specified aspect of construction method, ground treatment, and conditioning for tunneling, some papers with significant importance are selected to be reviewed in this General Report. The review will be carried out according to the grouping of the paper. Before the review, the geotechnical aspects in these fields are firstly summarized.

## 2 CONSTRUCTION METHOD WITH CASE STUDIES

## 2.1 Bored tunnel by TBM

More and more practices of bored tunnels by TBM bring forward more and more requirements for shield tunnel. Table 2 displays the current trend of development of shield tunnel based on the requirements from the world market of tunneling.

To meet these requirements, technologies of TBM are also developed at the same time. The recent

Table 1.	Grouping	of the	papers in	Session 2.
----------	----------	--------	-----------	------------

Topics	Num. of papers	Authors
1. Construction methods with case studies	6 papers	
1.1 Bored tunnel by TBM (shield tunneling)	(3)	Bakker & Bezuijen (A, B) He et al.
1.2 Shotcrete method (mountain tunneling method, NATM)	(3)	Sfriso Guatteri et al. Fillibeck & Vogt
2. Ground Treatment	5 papers	
2.1 Ground freezing	(2)	Hu & Pi Fillibeck & Vogt
2.2 Grouting	(4)	Guatteri et al. Bezuijen & vanTol Gafar et al Fillibeck & Vogt
3. Load and pressure	7 papers	
3.1 Lining pressure	(5)	Hashimoto et al. Talmon & Bezuijen Talmon et al. Bakker & Bezuijen (A, B)
3.2 Pressure on TBM	(4)	Bezuijen & Bakker Song & Zhou Bakker & Bezuijen (A, B)
4. Conditioning additives for EPB	2 papers	Hajialilue-Bonab et al. (A, B)
5. Ohters	3 papers	Deng & Zhang Kuzme & Hrustinec Li et al.

Table 2. Current trend in shield tunneling.

Long distance	$3 \mathrm{km} \sim 10 \mathrm{km}$
High speed excavation	300 m~1000 m/month
Deep excavation	$40 \mathrm{m} \sim 100 \mathrm{m}$
Large cross section	10 m~15 m of diameter
Deformed cross section	2 faces~4 faces, non-circular
High durability of tunnel	100 years
Cost performance	Not cheap but high quality
	with reasonable cost

Table 3. Geotechnical aspects for bored tunneling (shield tunneling).

TBM type	Both of slurry and EPB type in the soft soil with ground water
Applicable ground	Soft to stiff clay, loose to dense sand, gravel
Ground loss	Possible to be controlled less than $0.1 \sim 1\%$ in normal condition
Face stability	Need some controlling technologies for each slurry type or EPB type
Filling tail void	Simultaneous grouting can reduce ground loss and give an uniform distribution of lining pressure
Segmental lining	Many types of segmental lining have been developed

development of TBM and its technologies are shown as following:

- Durability of TBM
- Durability of cutter bits
- Exchangeable cutter bits
- Installation of linings, new segmental linings
- Driving control system
- Docking method
- Backfill grouting

In the practice of bored tunneling by TBM, the geotechnical aspects shown in Table 3 are of the most importance and should be well considered.

Bakker & Bezuijen (A, B) shared their invaluable experiences and findings on shield tunneling in soft ground obtained in last ten years. During the construction of the 2nd Heinenoord Tunnel that is approximately in the middle underneath the river Oude Maas in the Netherlands. They found out that because "blowout" occurred during TBM driving under the river, face support pressure dropped within 15 seconds after the cutter face working, shown as Figure 1. According to their investigation, they pointed out that face support pressure should be controlled between lower and upper limits for situations with little overburden or the soil cover itself is relatively light. We also are interested in the "15 seconds", which indicted that the front instability occured without any omen, a careful control of front pressure is necessary. Some analysis results of

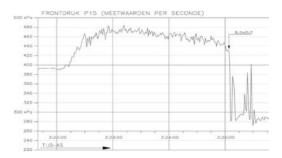


Figure 1. Support pressures before, during and after the "Blow out" at the 2nd Heinenoord tunnel (by *Bakker & Bezuijen* (A)).

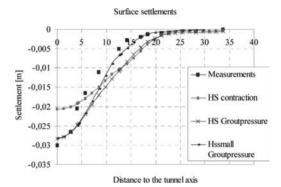


Figure 2. Surface settlements; measured and back-calculated with different material models (by *Bakker & Bezuijen* (A)).

surface settlement were also displayed, shown in Figure 2. It was concluded that for an adequate prediction of deformations it is important to model the grouting pressure as a boundary condition, in combination with the use of small strain material model.

As to the structure issues of the 2nd Heinenoord Tunnel, Bakker & Bezuijen (B) investigated the cracking and palling that occurred due to construction load, see Figure 3. Then a large scale tunnel ring tests was carried out, shown as Figure 4. By combining the model tests as well as numerical tests, it was found that the usage of kaubit in the ring joint was the main reason. The compression of the flexible kaubit strips by jacking force resulted in a slipping of different segment piece, leading to local stress concentration and irregular deformation. By replacing it with stiffer plywood plates, the damage was prevented. The influence of the duration of plywood to the long-term behavior of tunnel, however, is still questionable. During construction of the first tube for the Westernscheldt Tunnel, they found out that high grout pressures and in absence of bedding may cause the buckling of the TBM. Certainly,



Figure 3. Damage to the dowel and notch sockets during the first 150 m of construction of the 2nd Heinenoord tunnel (by *Bakker & Bezuijen (B)*).

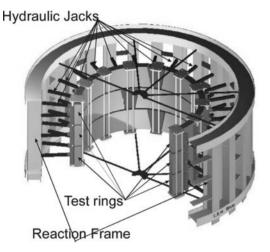


Figure 4. Large-scale tunnel ring testing (by *Bakker & Bezuijen*).

some other factors that were not discussed in the paper may also cause TBM deformation.

*He et al.* studied the first application of DOT tunneling in Shanghai. They conducted an in-situ test to investigate the distribution of stress and displacement around the tunnel. Figure 5 shows the vertical soil stress increment ahead of cutter face. Beautiful distribution of vertical earth pressure increment and settlement troughs were observed. It is expected that more detailed information about the measuring methods can be given out. They also reported a DOT shield passed under a five-floor building with a distance of 1 m successfully by careful operation, shown as Figure 6. The main countermeasures were relative low advancing speed and extra backfill grouting.

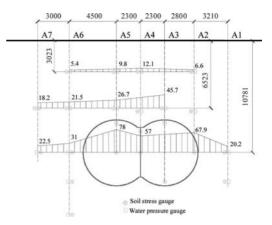


Figure 5. The vertical stress increment in 1.5 m ahead of the opening face (by *He et al.*).

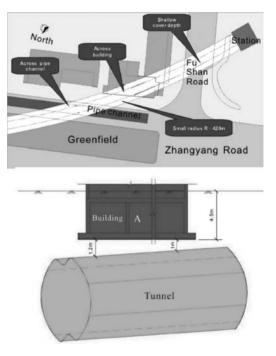


Figure 6. Dot shield tunnel run across the buildings (by *He et al.*).

## 2.2 Shotcrete method (Mountain tunnel method, NATM)

The geotechnical aspects of shotcrete method (Mountain tunnel method, NATM) are summarized in Table 4.

The design and construction procedures of Metro tunnels in Buenos Aires from 1998–2007 were reported by *Sfriso*. The characterization of Buenos Aires soils for tunneling is overconsolidated cemented

Table 4.Geotechnical aspects of shotcrete method (mountain tunnel method, NATM).

Face stability	Prelining, face bolt, horizontal grouting, pipe roof, shotcrete, et al.
Settlement mitigation	Face stabilization, shotcrete, foot pile, rockbolt, minibench, ground improvement (jet grouting, chemical grouting, compensation grouting, et al.)
Geological survey	Geophysical survey (elastic wave, sonic wave, electric resistivity, et al.), pilot boring
Monitoring technology	Extensometer, 3D laser scanning, optical fiber sensing, digital photogrammetric system, et al.
Prediction of ground water inflow and preservation of ground water	Lowering of ground water table, subsidence, drying well

soil with  $N_{spt} > 20$ , which is very favorable for excavation. As shown in Figure 7, shotcrete tunneling methods evolved from German method to Belgian method, and reached an optimal full face excavation. Cut & cover method and underground excavated method were used for stations. According to the filed measurement, the surface settlement is in the range 2–8 mm in general, 4–15 mm at stations.

*Guatteri et al.* described the state-of-the-art of application of ground improvement with all round  $(360^\circ)$  horizontal jet grouting in Sao Paulo and Barcelona, shown as Figure 8. Horizontal jet grouting columns were executed around the excavated section, including the invert, and at the far end of the conical treatment, to create a watertight chamber. This ground improvement achieves good results of pre-consolidation, settlement mitigation, reduction of water flow, and keep of face stability. According to field measurement, ground movements were controlled within 20–30 mm.

Shotcrete excavations with ground freezing, jet grouting, pipe screen and compressed air supporting methods were applied in the construction of Munich Subway. *Fillibech & Vogt* made a comparison of different methods of face support in settlement sensitive urban areas based on the ground deformations. In the case of heading with ground freezing under important structure, measurements for reducing frost heaves were taken, namely reducing operation time and careful temperature control. The recorded vertical displacements in Figure 9 show that a maximum heave of 3–5 mm was achieved. The report of jet grouting displayed a large heave due to installation of jet grouting cover, as shown in Figure 10. Although the face stability increases, the settlement is not reduced so

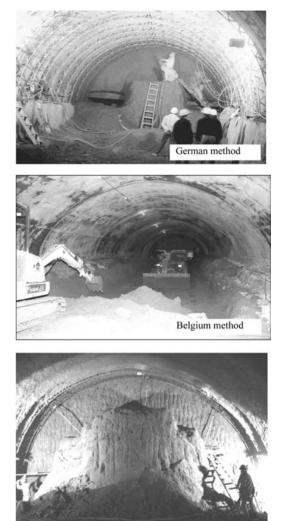


Figure 7. Various shotcrete tunneling methods in the construction of Metro tunnels in Buenos Aires (by *Sfriso*).

Full face excavation

much as expected. In the paper, it is pointed out that the installation of crown supporting measures must lead to higher safety potential, but it is difficult to judge whether these special measures are necessary or not.

## 3 GROUND TREATMENT

The geotechnical aspects of the two sub-subjects of ground treatment, ground freezing and grouting, are summarized in Table 5 and Table 6.

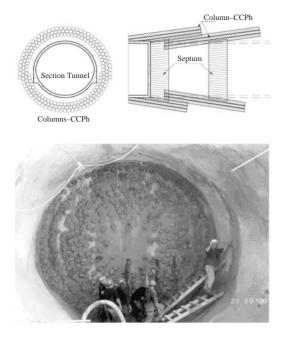


Figure 8. Ground improvement with all round horizontal jet grouting applied in Sao Paulo and Barcelona (by *Guatteri et al.*).

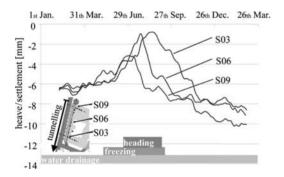


Figure 9. Vertical ground surface displacement along a tunnel protected by ground freezing (by *Fillibech & Vogt*).

The work by *Bezuijen & vanTol* aimed to make clear against the question why fractures can occur more easily in the field than in model tests with the same W/C ratio. Starting from a conceptual model that shown in Figure 11, they demonstrated in an analytical way that heterogeneity of soil in the field and the stress reduction by the installation of pipes (so-called TAM) and other causes before injection are main reasons.

*Garfa et al.* performed a sereies of laboratory scale grout injection tests in which various factors affecting fracturing of sand were studied. Figure 12 shows the schematic diagram of the experimental setup. The experimental results confirmed that fracture initiation

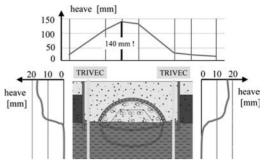


Figure 10. Large heave due to horizontal jet grouting (by *Fillibech & Vogt*).

Table 5.	Geotechnical	aspects on	ground	freezing.

Mechanism of freezing process and evaluation of laboratory freezing test	Segregation potential, ice lens, structure of soil, et al.
Property of frozen soils	Strength, stiffness, freezing point, temperature, thermal conductivity, salinity consistency, et al.
Frost heave and thaw settlement	Laboratory testing, prediction, countermeasure
Application of freezing method on underground construction	Cross passage, docking of TBMs, launch and arrival of TBM

Table 6. Geotechnical aspects on grouting.

Jet grouting	Uniformity of improved soil, ground deformation during jet grouting, applicable ground condition (boulder, obstacle, et al.)
Ground injection	
• Material	Chemical grout, micro-cement, CB, LW, polyurethane, et al.
• Grouting method	Penetration grouting, compaction grouting, double packer, et al.
• Evaluation of improvement	Fracturing, compaction effects, uniformity, strengthening, reduction of permeability
• Settlement control	Compensation grouting

in sand requires some local inhomogeneity around the injection point, rapid development of a filter cake with a limited thickness and a grout with low viscosity and a limited yield stress. Grouts with high w/c (watercement) ratio will exhibit fractures with the formation of filter cake. If the w/c is low, no fractures will be formed.

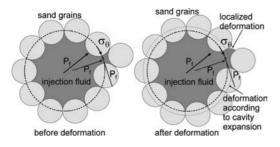


Figure 11. Sketch with possible deformation modes of the injection hole (by *Bezuijen & van Tol*).

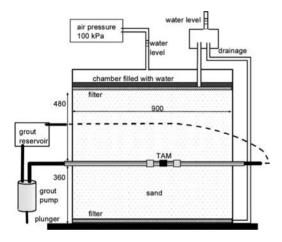


Figure 12. Schematic diagram of the experimental setup (by *Safar et al.*).

## 4 LOAD AND PRESSURE

Geotechnical aspects on lining pressure and pressure acting on TBM are summarized in Table 7.

Hashimoto et al. analyzed a series of observed earth pressure measured by PAD type earth pressure cell in soft clay, stiff clay and sand respectively. It was found that the distributions of earth pressure are very uniform in each kind of soils. In the very soft clay, a large portion of the overburden will act upon the lining, while in the stiff clay and the dense sand, the magnitude and distribution of earth pressure also depend largely on the backfill grouting, shown as Figures 13 & 14.

Talmo & Bezuijen presented a very interesting paper on the prediction of the ground pressure (lining pressure) based on the flow theory of backfill grout in combination with the time dependent consolidation of grout material. The measured results and the predicted results are shown in Figure 15. In their paper, they displayed the results that the lining pressure drops largely with time, shown as Figure 16. However, according to the research by *Hashimoto et al.* the lining pressure

Table 7. Geotechnical aspects on lining pressure and pressure acting on TBM.

Lining pressure for design	Lining pressure during construction and at long term, magnitude and distribution of pressure, effects of backfill grouting (grouting pressure and materials), soil types and ground condition
Longitudinal deformation and bending of tube	Backfill grouting, live load and dead load distribution, subsoil reaction, et al
Pressure on TBM	Backfill grouting pressure, jack force, slurry pressure, grease pressure at tail seal, earth pressure at a face, driving control of TBM, shape and rigidity of shield, subsoil reaction, tround deformation by pressure and load from TBM

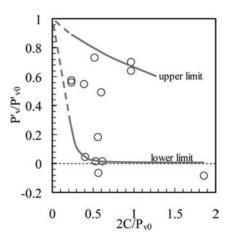


Figure 13.  $p'_{\nu}/p'_{\nu 0}$  vs.  $2C/p_{\nu 0}$  in clayey ground (by *Hashimoto et al.*).

drop is very small. The main differences are considered to be the grout material and injection methods.

Talmon et al. studied the longitudinal tube bending due to grout pressure. They carried out beam action calculation using input parameters of the bending moment by TBM jacks, transverse force by TBM, vertical grout pressure gradient behind TBM, loading diagram and unsupported length of tunnel lining in TBM, et al. The calculated result fits the observed ones to some extent, shown as Figure 17.

Bezuijen & Bakker described the interaction between the slurry from the face and the grout from the tail, see Figure 18. The pressure distribution along the longitudinal direction of TBM is calculated theoretically based on the pressure loss ( $\Delta P$ ) due to the flow. The calculated result is shown in Figure 19. They

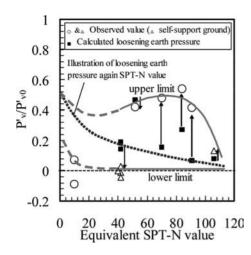


Figure 14.  $p'_{\nu}/p'_{\nu 0}$  vs. SPT-N in sandy ground (by *Hashimoto et al.*).

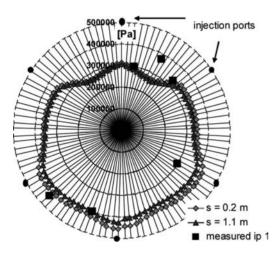


Figure 15. Meausred and calculated grout pressures (by *Talmo & Bezuijen*).

found that  $\Delta P$  depends on the shear stress of the grout along the TBM ( $\tau_r$ ), gap between TBM and ground, and length increment along the TBM.

Song & Zhou did a research work on the earth pressure distribution of excavation chamber in EPB tunneling. According to their work, the total supporting pressure can be composed by two parts: (1) Earth supporting pressure  $P_E$  in working chamber; (2) Cutter head plane supporting pressure  $P_P$ . The authors proposed an estimation method of earth pressure ratio

 $EPSR = P_E/(P_E + P_P)$ 

based on the empirical relation among cutter head torque, trapezoidal shape of pressure distribution,

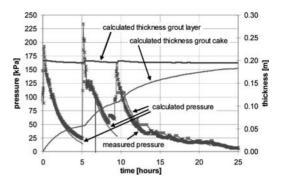


Figure 16. Drops of lining pressure with time (by Talmo & Bezuijen).

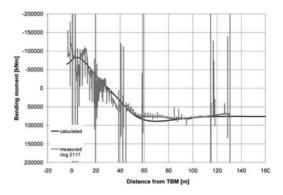


Figure 17. Measured and calculated bending moments compared (Groene Hart Tunnel, the Netherlands) (by *Talmon et al.*).

opining ratio of cutter face. They found that *EPSR* in clay is larger than that in cobble and sand.

## 5 CONDITIONING ADDITIVES FOR EPB

Required properties of conditioned soil for EPB are summarized in Table 8. Until now, there are many types of conditioning additives have been utilized in practice, including slurry, foam, polymer, water (for clayey ground), cellulose, sodium alginate, et al.

There are many data in the world especially in Japan for this subject, but these data has not been summarized in general. In Session 2, there are two papers by *Hajialiue-Bonal et al.* concerning condition additives for EPB. The two papers described the following results from laboratory test for foam and conditioned sandy by foam:

- 1 Polymer type foam shows a good stability;
- 2 With some combination, foam/sand mixtures have high compressibility;

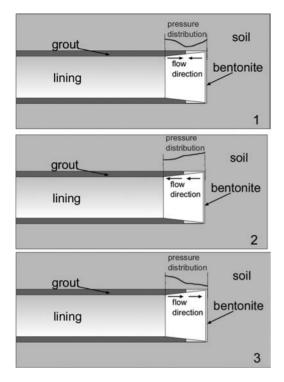


Figure 18. Possible flow directions and sketched pressure distributions along the TBM.

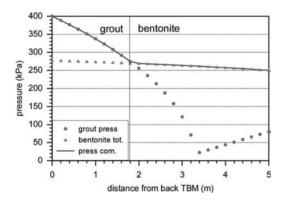


Figure 19. Pressures and joint width along a TBM.

- 3 The soil conditioning by foam cause decrease of shear strength  $(c, \Phi)$ ;
- 4 When Foam Expansion Ratio is 10 < FER < 18, the change of strength are negligible.

#### 6 CONCLUSIONS

The papers in this session provide abundant case studies to advance our knowledge of tunnel constructions,

Table 8. Required properties of conditioned soil for EPB.

High flowability	Low shear strength, reduction of cutting torque and wear of cutter bits, stable and precise monitoring of earth pressure in working chamber
Impermeability	Prevention of water inflow and piping
High compressibility	Reducing earth pressure fluctuation during excavation
Uniformity	Prevention of water inflow and piping, precise monitoring of earth pressure

and many new technologies developed in recent years were introduced. This is particularly important, because through case study, we can understand the advantage, disadvantage, applicable field, and feasibility of the new construction methods and technologies.

Some papers introduced new monitoring and measuring technologies. This is also an important aspect for tunnel construction. On-time and accurate monitoring and measuring can make tunnel construction work quicker, safer, and more economical. Of course, abundance of measuring data help us analyze and understand the mechanism and the essence of the interaction between tunnel structure and ground.

Prediction and theoretical analysis was concerned in some papers. In general, prediction and analysis results were compared with observed results to verify their validity. But we should pay attention to the limitation and applicable field of these methods.

#### REFERENCES

- Bakker K.J. & Bezuijen A. 10 years of bored tunnelling in the Netherlands: Part I geotechnical issures.
- Bakker K.J. & Bezuijen A. 10 years of bored tunnelling in the Netherlands: Part II structureal issures.
- Bezuijen A. & Bakker K.J. The influence of flow around a TBM machine.
- Bezuijen A. & van Tol A.F. Mechanisms that determine between fracture and compaction grouting in sand.
- Chong H., Li T. & Yan J. The double-o-tube shield tunnel in Shanghai soil.
- Deng Z.G. & Zhang Q.H. Research of non-motor vehicle -rail transit-tube interchanging transport system pattern.
- Fillibeck J. & Vogt. N. Shotcrete excavations for the Munich subway – Comparison of different methods of face support in settlement sensitive areas.
- Gafar K., Soga K., Bezuijen A., Sanders M.P.M. & van Tol A.F. Fracturing of sand in compensation grouting.
- Guatteri G., Koshima A., Lopes R., Ravaglia A. & Pieroni M.R. Historical cases and use of horizontal jet grouting solutions with 360° distribution and frontal septum to consolidate very weak and saturated soils.
- Hajialilue-Bonab M., Ahmadi-adli M., Sabetamal H. & Katebi H. The effects of sample dimension and gradation on shear strength parameters of conditioned soils in EPBM.

- Hajialilue-Bonab M., Sabetamal H., Katebi H. & Ahmadiadli. M. Experimental study on compressibility behavior of foamed sandy soil.
- Hashimoto T., Ye G.L., Nagaya J., Konda T. & Ma X.F. Study on earth pressure acting upon shield tunnel lining in clayey and sandy ground based on field monitoring.
- Hu X.D. & Pi A. Frozen soil properties for cross passage construction in Shanghai Yangtze River Tunnel.
- Kuzma K. The influence of engineering-geological conditions on the construction of radioactive waste dump.
- Li Z.X., Han X. & Wang K.S. Critical ventilation velocity in large cross-section road tunnel fire.
- Sfriso A. O. Metro tunnels in Buenos Aires: Design and onstruction procedures 1998–2007.
- Song T.T. & Zhou S.H. Study on the Earth Pressure Distribution of Excavation Chamber in EPB tunneling.
- Talmon A.M. & Bezuijen A. Backfill grouting research at Groene Hart Tunnel.
- Talmon A.M., Bezuijen A. & Hoefsloot F.J. Longitudinal tube bending due to grout pressures.

## Physical and numerical modelling

## P.L.R. Pang

Geotechnical Engineering Office, Civil Engineering and Development Department, Government of the Hong Kong Special Administrative Region, HKSAR

ABSTRACT: This General Report has been prepared based on a review of twenty papers submitted to the session on "Physical and numerical modelling" related to geotechnical aspects of underground construction. The papers cover a wide range of model feature types in different materials. The problems studied include ground/tunnel face stability, ground/tunnel deformation and earth pressures, ground/tunnel-structure interaction, seismic behaviour, and vehicle fires in a road tunnel. This report highlights and discusses the approaches used in modelling and presents the key findings. Some remarks are given at the end on the objectives of modelling and the work of TC28.

## 1 INTRODUCTION

20 papers have been submitted to this session (Table 1). Three of the papers are joint contributions from authors of two countries.

The papers cover a wide range of model feature types (Table 2). These include tunnels in clay, sand, aluminum rods (modelling a granular mass), layered soils, as well as tunnels in soft or weak rock. There is a paper on modelling of deep excavations with steppedtwin retaining walls, and a paper on vehicle fires in a road tunnel.

#### 2 PHYSICAL MODELLING

Eleven papers present results of physical modelling (Table 3). These include six models at 1 g, a photoe-lastic model and four centrifuge models.

Table 1. Geographic distribution of the papers.

Country	Papers
China	5
China/France	1
Denmark	1
France	1
Italy	1
Italy/UK	1
Japan	2
Japan/UK	1
Korea	4
The Netherlands	1
UK	2
	20

For the three models that use aluminum rods, the tests were carried out at 1 g. Numerical modelling was also carried out to compare with the results of the 1 g tests.

The sand model at 1 g was prepared by compaction of the sand using a plate vibrator. The compaction could have created locked-in compaction stresses on the model braced wall and the adjacent tunnel thus influencing the model test results. This was not discussed in the paper.

Two papers present results of modelling of rock tunnel problems using 1 g tests. One used barite powder, sand and plaster mixed with water, and the other used concrete bricks to model the soft rock. There was some discussion on the modelling laws in the papers. While the conclusions on the qualitative behaviour seem reasonable, and are not unexpected, it is not sure if the quantitative results are valid at prototype scale where discontinuities in the rock and the higher stress levels could influence the magnitude of the deformations.

Table 2. Feature types covered in the papers.

Feature type	Papers
Tunnels in:	
(a) Clay	5
(b) Sand	3
(c) Aluminum rods/crushed glass	4
(d) Layered soils	3
(e) Soft/weak rock	3
Deep excavation (aluminum rods)	1
Vehicle fires in road tunnel	1
	20

Table 3. Papers on physical modelling.

Technique and scale	2D/3D	Materials used	Papers
Laboratory 1 g model	2D	Aluminum rods	3
(scales: 1/10,		Sand	1
1/19, 1/80)		Barite powder/ sand/ plaster	1
		Concrete bricks	1
Photoelastic models	2D	Crushed glass	1
Centrifuge models	3D	Clay	1
(75 g, 100 g, 160 g)		Sand	2
		Sand overlying clay	1

There are four papers on centrifuge tests. One of the papers is to study the effects of pile loading on an existing tunnel in an overconsolidated clay, two are on centrifuge tests where dry sand was used to construct the models for studying the interaction mechanisms, and one on tunnelling in an overconsolidated clay overlain by sand under the water table.

## 3 NUMERICAL MODELLING

18 papers present numerical modelling results (Table 4). Different numerical modelling techniques were used.

14 out of these 18 papers used either 2D or 3D codes based on the finite difference method (FDM) or the finite element method (FEM). Some of the codes, e.g. CRISP, FLAC and PLAXIS, are well established codes and the 2D versions are commonly used in current engineering practice. In the analyses, the soil was modelled either as a linear elastic or an elastic-perfectly plastic material with the Mohr-Coulomb or Drucker Prager failure criterion. Where a comparison was made, the elastoplastic model performed better than the elastic model.

In one paper, a slope stability analysis program SLOPE/W based on the limit equilibrium method was used to compute the factors of safety of a clay slope. The results were compared with the results of FLAC and PLAXIS which used the strength reduction method. However, no information is given on the theoretical method used (a few options are available in SLOPE/W such as Janbu, Bishop and Morgenstern & Price) and the choice of slip surfaces, which could affect the computed safety factors. Also, no information is given on what the slope deformation and the soil shear strain were, when the soil strength is reduced for the factor of safety to approach unity.

A visco-elastic model adopting a nonlinear relationship between the normalized shear modulus (and damping ratio) and the shear strain amplitude was used for a 1D ground dynamic shear response analysis. The code EERA was used for the analysis, the objective of this study was to "calibrate" a linear visco-elastic, effective stress based, constitutive model for use in coupled 2D dynamic analyses using the finite element program PLAXIS. The viscous damping was accounted for using the Rayleigh formulation (Woodward & Griffiths, 1996).

The subloading tij finite element model (developed by Nakai & Hinokio (2004)) was used in two cases to provide results for comparing with physical modelling at 1 g which used aluminum rods in the model tests. The tij model takes into account the influence of the intermediate principal stress by introducing a modified stress tij. Also, the subloading concept (proposed by Hashiguchi (1980)) is adopted to model the influence of soil density. Five of the seven parameters in the tij model are the same as those in the Cam-clay model, with one more parameter added to describe the influence of soil density and confining pressure, and another parameter added to characterize the shape of the yield surface. Laboratory biaxial tests were carried out to compare the stress-strain curves obtained from the finite element program FEMtij-2D. In the biaxial tests, shearing of the aluminum rods, which had low friction angles, induced dilatant behaviour. The match between the biaxial tests and the finite element analysis results appears reasonable but this is up to a shear strain level of about 1-2% only (Figure 1).

The Distinct Element code UDEC was used in one case to compare with the results of large-scale model tests carried out using concrete bricks to model rock. However, the paper does not indicate how the discontinuities in the rock were modelled. For the other two papers on tunnels in rock, the numerical simulations were carried out using finite element codes adopting an elastoplastic rock model with the Drucker-Prager failure criterion. It seems that the need for modelling the discontinuities that may be present in the rock was not considered. It is not too clear from the two papers how the rock parameters were determined for the continuum models and the field prototypes.

Results obtained from closed form solutions derived using upper bound limit analysis were presented in two of the papers, for comparison with the results of centrifuge modelling and numerical modelling respectively.

The Fire Dynamics Simulator code incorporating a large eddy simulation model was used to carry out computational fluid dynamics modelling. The objective of this work was to study the heat release rates from vehicle fires in a road tunnel of 15 m in diameter. The computed results were compared with an empirical equation. This indicates that the empirical equation requires improvement for the case of small fires in road tunnels with a large cross section.

Table 4.	Papers	on	numerical	modelling.

Constitutive law	Modelling	Program	Papers
Linear elastic	2D FEM	PLAXIS	2
Nonlinear visco-elastic	1D shear	EERA	1
Elasto-plastic	2D FEM	CRISP, Msc.MARC	2
(Mohr Coulomb)	2D FDM	FLAC	1
``````````````````````````````````````	3D FEM	MIDAS-GTS	1
	3D FDM	FLAC3D	3
Elasto-plastic (Drucker Prager)	3D FEM	MARC	2
Elasto-plastic (Cam clay $+ 2$ parameters)	2D FEM	FEMij-2D	2
Distinct element	2D DEM	UDEC	1
Rigid-plastic	Limit analysis	Closed form solution	2
Large eddy simulation	CFD	Fire Dynamics Simulator	1

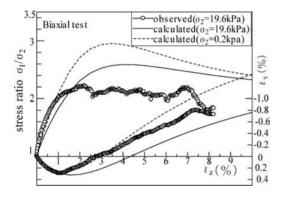


Figure 1. Stress-strain-dilatancy relation.

## 4 PROBLEMS STUDIED

The problems studied as reported in the papers include:

- 1. ground/tunnel face stability (5 papers),
- 2. ground/tunnel deformation and earth pressures (8 papers),
- 3. ground/tunnel-structure interaction (5 papers),
- 4. seismic behaviour (1 paper), and
- 5. vehicle fires in a road tunnel (1 paper).

A brief review of selected papers is given below.

### 4.1 Ground/tunnel face stability

The subject of face stability is a very important one. If the face pressure applied is too low, there could be a collapse or excessive ground settlement, and if the face pressure is too high, there could be a blow-out failure or excessive ground heave.

A number of researchers have studied this problem (e.g. Anagnostou & Kovári, 1994). The following papers have added to the knowledge base.

Li et al investigated the failure of a large slurry shield-driven tunnel using upper bound limit analysis and numerical modelling. The study is for the 15.43 m diameter Shanghai Yangtze River Tunnel constructed in soft clay. A shallow ground cover section, with a ground cover to tunnel diameter (C/D) ratio of 0.7, was selected for the study. Undrained conditions were assumed in the modelling. A multi-block failure mechanism with a uniform face pressure (suggested by Soubra, 2002) was used for the limit analysis. FLAC3D was used for the numerical modelling (which adopted an elastic-perfectly plastic constitutive model with a Mohr-Coulomb failure criterion). The results of the upper bound limit analysis and the 3D numerical modelling showed that partial blow-out failure of the upper part of the tunnel face occurs when the slurry pressure is large, whereas global collapse of the whole tunnel face occurs when the slurry pressure is small (Figure 2).

The authors noted that the difference between the slurry pressure and earth pressure at the crown and invert for a large diameter slurry TBM tunnel can be large and this could have a significant effect on the failure mechanism and the critical slurry pressure. The failure mechanisms and the critical slurry pressures at the tunnel axis level obtained from the limit analysis and the numerical modelling agree well with each other (Figure 3).

Caporaletti et al reviewed the past research on tunnel stability in undrained conditions (Davis, et al, 1980; Kimura & Mair, 1981; Sloan & Assadi, 1992), in drained conditions (Atkinson & Potts, 1977) and in layered ground (Grant & Taylor, 2000). They conducted centrifuge tests to investigate the stability of a circular tunnel in layered ground, with clay overlain by a medium dense sandy layer, below the water table. The C/D ratio of the tunnel was 2.38. The clay was consolidated from a slurry, to give an overconsolidation ratio ranging between 1.4 and 2.8 with depth. All tests

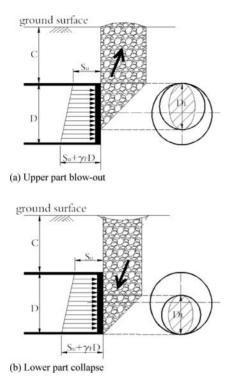


Figure 2. Two kinds of partial failure mechanisms.

were carried out at 160 g. The condition of tunnel collapse was taken as volume loss greater than 20%. In the centrifuge tests the mechanism of failure for the layered ground involved a wide area of soil both in sand and in clay, with pseudo-vertical settlements at the sand-clay interface (Figure 4).

It was found that the contribution to stability due to friction acting within the upper sand layer represented a significant contribution. A significant overestimate of the tunnel support pressures to prevent collapse might result if the theoretical solutions obtained for homogenous clays are used with the sand layer treated as a surcharge. The authors proposed a new failure mechanism which provided an upper bound to the experimental data obtained (Figure 5). It would be interesting to examine whether the proposed mechanism is applicable for the case of a loose sand layer.

Date et al carried out a series of centrifuge tests at 75 g to investigate the ground deformation patterns during excavation of tunnels in dry sand. The C/D ratio of the model tunnels was one, and some of the models incorporated reinforcements. The ground deformation was found to be small even when the face pressure was reduced to half the initial pressure of 100 kPa, but once movement started upon further reduction of the face pressure it increased sharply leading to "instantaneous" collapse (i.e. a brittle failure).

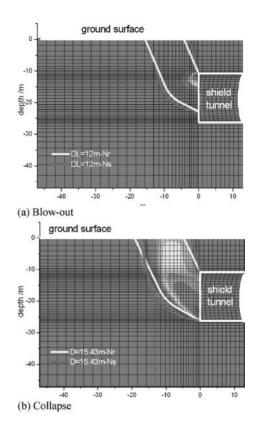


Figure 3. Comparison of failure mechanisms of Case 2 (velocity contour for FLAC<sup>3D</sup> analysis).

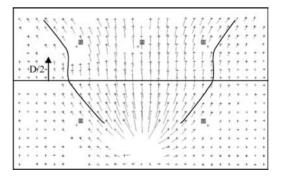


Figure 4. Mechanism of failure from centrifuge tests (VL  $\cong$  20%).

The model tests without reinforcement collapsed at a support pressure which agrees with the centrifuge test results of Chambon & Corté (1994). The study found that introduction of face bolts and forepoling yielded different tunnel collapse mechanisms, which depended on the density of the face bolts and forepoling bolts. Surprisingly, the reinforcements contributed

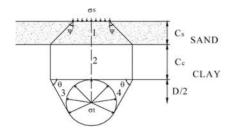


Figure 5. Mechanism of failure for layered ground.

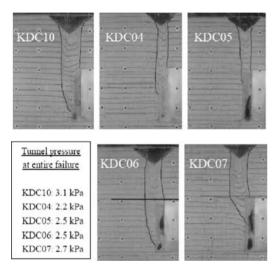


Figure 6. Tunnel failure patterns on the longitudinal section.

to only a slight reduction in the support pressure required to keep the tunnel face stable, compared to the case without reinforcement. The face bolts installed stiffened the ground ahead of the face and were found to be able to reduce the face extrusion. The forepoling divided the ground around the tunnel face into two zones, with the outer zone forming an arch comprising the forepoling bolts. The geometries of the collapse mechanisms are similar to those observed by other researchers for tunnels in sands, e.g. as reported by Chambon & Corté (1994) and Mair & Taylor (1997). They all involve a narrow "chimney", propagating almost vertically from the tunnel up to the ground surface (Figure 6).

FLAC3D analyses were also carried out. The Mohr-Coulomb soil model with strain softening/hardening was found to give a better match to the centrifuge data than the Mohr-Coulomb model without strain softening/hardening. The deformation pattern obtained from the analysis for a model reinforced with face bolts was similar to that of the centrifuge test but the magnitude was smaller. The authors recommended to study further the effect of mesh shape and the effect of changes

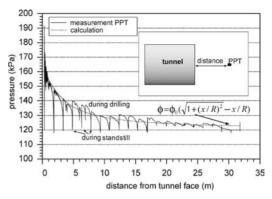


Figure 7. Measured excess pore pressure in front of a slurry shield and approximation.

in soil-bolt interaction properties upon excavation in the numerical analyses.

The information on failure mechanisms presented in the above papers is interesting and useful. There is recent improvement in understanding of the groundtunnelling interaction processes associated with influence of grouting pressures, removal of the filter cake and the pore pressures generated during the advance of a slurry TBM (Figure 7). This was achieved through field measurements obtained during construction (Bezuijen & Talmon, 2008). Further data and study in this area will no doubt augment the results of existing laboratory and analytical modelling, which have not accounted for such processes. Further understanding of the processes could help to evaluate the need to refine the calculation models and design methods for estimation of face pressures required to prevent collapse and blow-out.

# 4.2 Ground/tunnel deformation and earth pressures

A number of papers in this session present results of modelling to study the ground deformation and earth pressures around a tunnel.

Shahin et al developed a new circular tunnel apparatus and conducted 1 g model tests to examine the ground movements induced by tunnelling and the earth pressures around the tunnels. Aluminum rods were used to model a granular soil mass. The surface settlement was measured using a laser type displacement transducer with an accuracy of 0.01 mm, and photographs were taken during the experiments which were later used as input for the assessment of the ground movements using the Particle Image Velocimetry technique (White et al, 2003). To compare with the model test results, numerical simulations were carried out using 2D finite element analyses under plane strain and drained conditions. The computer program FEMtij-2D was used. The initial stresses

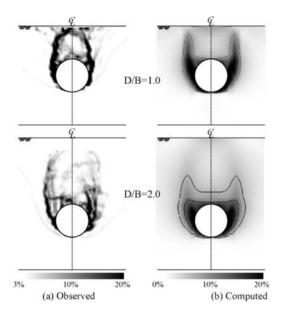


Figure 8. Distribution of shear strain: tunnel invert is fixed.

applied correspond to the self-weight condition. Two C/D ratios, viz. 1 and 2, were examined. The effects of full face excavation (with the centre of the excavation kept fixed) and top drift excavation (with the invert of the tunnel kept fixed) were also studied. The surface settlement and earth pressures around the tunnel were found to be significantly influenced by the displacement at the tunnel crown for the same overburden and same volume loss. The volume loss was less significant compared to the crown drift in the case of the shallow tunnel. The full face excavation case produced a wider shear deformation region than that for the case of top drift excavation (Figure 8). The use of an elastoplastic soil model produced better match with the model test surface settlement profile than an elastic soil model. The distribution of earth pressures around the tunnel depended on the excavation pattern. The authors indicated that the numerical simulations were generally in good agreement with the model test results. However, it is no clear whether the tij finite element model is capable of describing the behaviour of tunnels constructed in real soils especially in soils which exhibit contractile behaviour.

Liang et al studied the effects of soil stratification on tunnelling-induced ground movements. 3D analyses were carried out using the computer program FLAC3D. The behaviour of the 2.47 m diameter Thunder Bay sewer tunnel in Canada, constructed using a TBM with segmental concrete lining, in soft to firm clays with silt and sand seams, was simulated. The C/D ratio of the tunnel was 3.8. The soil strata were divided into four sub-layers for the purpose of the analyses.

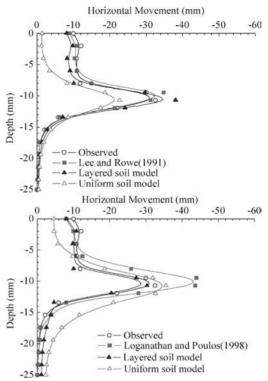
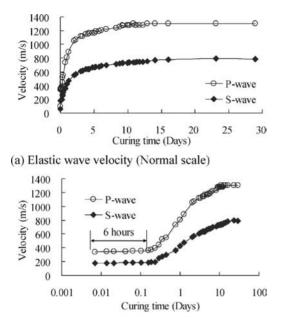


Figure 9. Lateral displacement 15 m behind the tunnel face.

The ground surface settlement, lateral displacement profile at 15 m behind the tunnel face and the subsurface settlement with depth above the tunnel axis from the analyses were compared with the analysis results obtained by Lee & Rowe (1991) using the FEM3D program (also based on an elastoplastic soil model). They were also compared with the field data reported by Belshaw & Palmer (1978). Additional comparisons were carried out with the analytical solution given by Loganathan & Poulos (1998). The study showed that the elastoplastic soil model could simulate the deformation profiles better than those based on the elastic model. The results of the elastoplastic soil model indicated that soil stratification had little effects on the ground surface settlement but significantly influenced the lateral displacement and subsurface settlement profiles (Figure 9). This was different from the elastic soil model which predicted that soil stratification had significant effects in all cases. This is an interesting case history of benchmarking a 3D computer program using data from a past project, illustrating the value of documenting good data and making it available for research.

Song et al studied the time-dependent behaviour of soft ground tunnels constructed using steel reinforcements grouted into the ground ahead of a tunnel



(b) Elastic wave velocity (Log scale)

Figure 10. Time-dependent characteristics of elastic wave velocities of a sand-cement mixture.

(a technique which the authors called the "reinforced protective umbrella" method). Laboratory direct shear tests and P and S wave velocity tests (using piezoelectric bender elements) were carried out to determine the strength and stiffness of the sand-cement mixture at different curing times. The test results showed that the sand-cement mixture gained significant increases in stiffness after about 6 hours whereas the apparent cohesion increased to about 2 MPa after 7 days (Figure 10).

3D finite element analyses were carried out using a computer program MIDAS-GTS (2005) to simulate the behaviour of such a tunnel. The tunnel is 18.8 m wide and 10.4 m high, at 15 m below ground. It was constructed in weathered rock, using 12 m long steel pipes as reinforcement. The water table was at ground surface. The analyses incorporated the time-dependent material properties of the sand-cement mixture. The excavation rate was taken as 0.75 m per day. The study concluded that use of the 2-3 days strength and stiffness parameters was adequate for predicting the time-dependent deformation behaviour, for practical design purposes, provided that there is sufficient overlap between the reinforcements. No comparison with any field performance monitoring results was however presented.

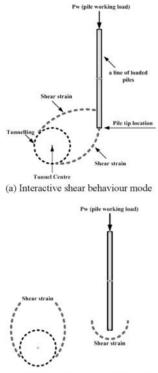
Lee et al studied the behavior of a 2-arch rock tunnel using a large-scale test machine (6 m wide  $\times$  6.5 m high). The model tests (1/19 scale) were conducted at 1 g. The rock was modelled using concrete bricks. The

tests showed that the ground displacements induced by tunnelling were mainly within a zone of 0.25D from the tunnel, where D is the tunnel width. Horizontal displacements of more than 40% and vertical displacements more than 20% of the total displacements occurred during excavation of the pilot tunnel. The authors suggested that the stability of the 2-arch tunnel could be dominated by the stability of the pilot tunnel excavation and that the rock bolt length should be longer than 0.25D. Displacements obtained from UDEC analyses were presented. While these showed the same pattern, details of the analyses were not given. Based on the limited measurements obtained. the authors suggested that the rock load acting on the centre pillar of the 2-arch tunnel may be taken to be 0.15W for preliminary design, where W is the centreto-centre distance between the left and right tunnels. when the RMR of the rock mass is more than 60. No numerical analyses were carried out. More research was recommended to confirm the proposed empirical relationship. It would be useful to examine the influence of rock discontinuities and the effect of rock block size relative to the tunnel diameter.

#### 4.3 Ground/tunnel-structure interaction

Broere & Dijkstra investigated the influence of tunnel volume loss on piles using the photoelastic technique. 2D plane strain model tests were conducted to examine the tunnel-pile interaction. Crushed glass (a photoelastic material) was used to model the soil. The effects of volume loss were simulated by making the tunnel diameter contract vertically. From the tests, it was found that significant stress changes occurred close to the pile tips. The tests with a volume loss of 0.6%showed a clear influence of the volume loss on the stresses near the pile tips up to one tunnel diameter away. The study suggested that the influence zone for displacement piles with both end bearing and skin friction, might be slightly larger than for bored piles with end bearing alone. The authors indicated that further field observations, model testing and numerical modelling are required to determine the influence zone.

Lee & Yoo studied the ground shear strain patterns developed around a tunnel and the existing piles nearby due to tunnel construction. Small-scale laboratory model tests at 1 g were conducted. Aluminum rods were used to model the soil mass and the piles embedded in it. A tunnel diameter reduction system capable of achieving a tunnel volume loss of up to 20% was specially developed. The strained controlled tests carried out using this system resulted in ground shear strains which were captured by close range photogrammetry. 3D numerical analyses were also carried out using the finite element program CRISP. Comparison between the physical model tests and the finite element analyses showed good agreement in terms of shear strain patterns. Based on the maximum shear



(b) No interactive shear behaviour mode

Figure 11. Schematic illustration of shear strain modes for pile-soil-tunnelling interaction.

strain contours, two distinct shear strain patterns were observed, viz. with and without tunnel-pile interaction (Figure 11). The boundary between these two modes of behaviour depended on the location of the pile tip from the tunnel and the magnitude of the tunnel volume loss. The authors suggested that this boundary might serve as a useful guide in the planning the tunnel alignment in areas where piles are present. It may be worthwhile to compare the results given in these papers with the findings of Jacobsz et al (2004; 2005) from centrifuge tests and Selemetas et al (2005) from field tests.

Yao et al studied the effects of loading of bored piles on existing tunnels. Centrifuge model tests were carried out at 100 g. The model tunnel was formed in firm to stiff clay consolidated from a slurry. The tunnel lining deformation, pore pressures in the clay, pile load applied, pile settlements and tunnel face pressures were monitored while the pile loading was being applied. Two C/D ratios, viz. 2 and 3, were studied. The tests examined the behaviour after pile construction. The influence of pile excavation was not considered. In the tests the pile base was set at two different positions: tunnel crown and invert level. The rate of loading was designed to create undrained conditions. Preliminary analysis of the results indicated that the pile settlement had a linear relationship with increase in applied load when the load exceeds half the designed ultimate load. The tunnel centre always moved downwards and away from the pile. Increasing the pile-tunnel clear spacing reduced the deformation of the tunnel lining. The long pile had more effect on the tunnel lining than the short pile regardless of the C/D ratio. The tunnel crown was always subject to significant movement due to pile loading.

Marshall & Mair investigated the soil-structure interaction mechanisms resulting from tunnel construction beneath buried pipelines using centrifuge modelling. The study aimed to validate visually the interaction mechanisms that account for pipeline behaviour. Particle Image Velocimetry was used to measure displacements for characterising the soilstructure interaction. The model tests were carried out at 75 g, using sand prepared to a relative density of 90%. The C/D ratio of the tunnel was 2.4. The study showed that estimation of the tunnel volume loss (defined as change in tunnel volume divided by the original total tunnel volume) using soil displacement data was not simple for sands. This was due to the uncertainty on the extent of the dilation and contractile behaviour of the sand around the tunnel. The soil volume loss (defined as the volume calculated by integrating the soil settlement profile and dividing by the original total tunnel volume) was not always the same as the tunnel volume loss. The magnitude of the former calculated at the ground surface can be greater or less than the latter. The centrifuge pipeline test illustrated that a gap formed below the pipeline at a tunnel volume loss of between 1 and 2%. The gap grew as the tunnel volume loss increased. The bending moments induced in the pipe increased from the onset of tunnel volume loss but did not appear to be sensitive to the growth of the gap height (Figure 12).

Lee & Kim studied the behaviour of a braced excavation in sand adjacent to a tunnel using large-scale (1/10 scale) model tests at 1 g. The braced wall was subjected to preloading to limit the wall deflections during the ground excavation. The tunnel was at a distance of half the tunnel diameter from the braced wall. The sand was prepared to a relative density of 56%. 2D numerical analyses were carried out using the finite element program PLAXIS. It is not clear what constitutive model was used for the sand. The study found that if the wall deflections were significantly reduced by preloading, the stability of the adjacent tunnel would greatly increase. The maximum bending moment and shear force in the tunnel lining decreased due to the preloading. The ground surface settlement also decreased as a result of preloading. The wall deflection profiles from the model tests agreed well with the numerical analysis results. It is noted that the

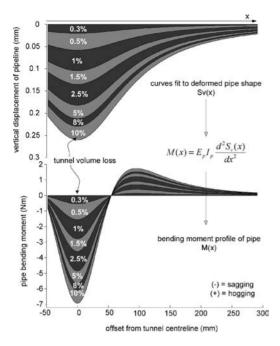


Figure 12. Derivation of bending moments from deformed shape of pipeline (from PIV data).

sand was compacted to construct the models. However, it is not clear what initial soil stresses were used in the numerical analyses. Also, no information is given on whether the wall installation and excavation sequence in the analyses matched those in the model tests.

## 5 CONCLUDING REMARKS

In the papers submitted to this session, the modelling objectives are generally not explicitly stated but they probably include one or more of the following objectives:

- To observe/understand collapse mechanisms
- To observe/understand deformation patterns and interactive behaviour
- To assess/verify the usefulness or accuracy of theoretical solutions, software or empirical rules against laboratory (1 g or Ng) model test data
- Same, but against field measurement data
- To benchmark theoretical solutions or software
- To predict field performance

Other than to gain knowledge and to understand the problem, an important goal of the modelling research should be to provide useful and reliable tools or to enhance the existing tools for prediction of field performance, for use in engineering practice. In this regard, some of the papers have contributed to this goal. TC28 has recently set up two working groups, one on databases on underground works and another on preparing guidelines for comparing field or physical modelling with numerical simulations. The first initiative will be useful for modellers in that good quality data will be archived systematically for easy reference and retrieval, physical modellers could use the information to plan their research and check their model test results against others' work for benchmarking purpose, and numerical modellers could use the data to check the reliability and limitations of the existing theoretical closed form solutions and numerical codes. The second initiative will be useful for those who are carrying out modelling to understand real behaviour or to validate numerical codes.

One of the databases on underground works could include failures observed in model tests and failures in actual projects. Data in the latter category are more difficult to obtain unless there is a forensic investigation and the information is subsequently made publicly available. Information on failures would be invaluable to provide lessons learnt, for calibration of design methods and for providing insights for risk management. If this is to proceed, then there may be merit to collect data on the size of the failure influence zones for different ground and geometrical conditions, and also the time for any cavity created at depth to migrate to the ground surface. Such information is potentially useful for risk management, in particular, for preparing monitoring plans, planning of risk mitigation measures, and preparation of emergency preparedness and contingency plans.

Another database will be on monitoring results. For such databases, the monitoring data should be accompanied by the necessary data on ground and groundwater conditions, the way the soil and rock parameters was measured and interpreted, the method of wall installation, information on ground treatment and the sequence of construction. Such data would allow numerical modellers to check the capabilities and limitations of the existing computer programs. From a practising engineer's point of view, it is often not practical to use overly sophisticated software requiring multiple parameters to characterize the soils for design. This is because of the cost, time and difficulties in obtaining high quality ground investigation data, the uncertainties associated with modelling the ground and the hydrogeological conditions (including the boundary conditions), the effort needed to model the range of design situations and to carry out sensitivity analyses, the need for having relatively simple tools for undertaking design reviews in a timely manner during construction, the difficulties in incorporating effectively the wall installation and ground treatment effects in the analyses, and the lack of competent personnel in the use of sophisticated codes and checking of the computed results from such codes. There is a lack of systematic comparison on the results obtained from sophisticated software with those from less sophisticated ones. The availability of good quality monitoring data and benchmarking of the existing numerical codes using good quality monitoring data could help to addresses some of these issues.

TC2 on physical modelling in geotechnics has similar initiatives on databases (see http://www.tc2.civil. uwa.edu.au). Cross committee communication will create synergy.

## ACKNOWLEDGEMENTS

The author is grateful to Paul Wu for his assistance in preparing this General Report.

## LIST OF PAPERS REVIEWED

- Boldini, D. & Amorosi, A. Tunnel behaviour under seismic loads: analysis by means of uncoupled and coupled approaches.
- Broere, W. & Dijkstra, J. Investigating the influence of tunnel volume loss on piles using photoelastic techniques.
- Caporaletti, P., Burghignoli, A., Scarpelli, G. & Taylor, R.N. Assessment of tunnel stability in layered ground.
- Date, K., Mair, R.J. & Soga, K. Reinforcing effects of forepoling and facebolts in tunneling.
- Du, J.H. & Huang, H.W. Mechanical behavior of closely spaced tunnels – laboratory model tests and FEM analyses.
- Idris, J., Verdel, T. & Alhieb, M. Stability analysis of masonry of an old tunnel by numerical modelling and experimental design.
- Iwata, N. Shahin, H.M., Zhang, F., Nakai, T., Niinomi, M. & Geraldni, Y.D.S. Excavation with stepped-twin retaining wall: model tests and numerical simulations.
- Kasper, T. & Jackson, P.G. Stability of an underwater trench in marine clay under ocean wave impact.
- Lee, S.D., Jeong, K.H., Yang, J.W. & Choi, J.H. A study on behavior of 2-arch tunnel by a large model experiment.
- Lee, S.D. & Kim, I. Behavior of tunnel due to adjacent ground excavation under the influence of pre-loading on braced wall.
- Lee, Y.J. & Yoo, C.S. Two distinctive shear strain modes for pile-soil-tunnelling interaction in a granular mass.
- Li, Y., Zhang, Z.X., Emeriault, F. & Kastner, R. Stability analysis of large slurry shield-driven tunnel in soft clay.
- Liang, F.Y., Yao, G.S. & Li, J.P. Effects of soil stratification on the tunneling-induced ground movements.
- Marshall, A.M. & Mair, R.J. Centrifuge modeling to investigate soil-structure interaction mechanisms resulting from tunnel construction beneath buried pipelines.
- Shahin, H.M., Nakai, T., Zhang, F., Kikumoto, M., Tabata, Y. & Nakahara, E. Ground movement and earth pressure due to circular tunneling: model tests and numerical simulations.
- Song, K.I., Kim, J. & Cho, G.C. Analysis of pre-reinforced zone in tunnel considering the time-dependent performance.
- Wang, K.S., Han, X. & Li, Z.X. Vault temperature of vehicle fires in large cross-section road tunnel.

- Wang, X.M., Huang, H.W. & Xie, X.Y. Effects of different bench length on the deformation of surrounding rock by FEM.
- Yao, J., Taylor, R.N. & McNamara, A. The effects of loaded bored piles on existing tunnels.
- You, G.M. 3D FEM analysis on ground displacement induced by curved pipe-jacking construction.

## REFERENCES

- Anagnostou, G. & Kovári, K. 1994. The face stability of slurry-shield driven tunnels. *Tunnelling and Underground Space Technology*. 9(2), 165–174.
- Atkinson, J.H. & Potts, D.M. 1977. Stability of a shallow circular tunnel in cohesionless soil. *Géotechnique*, 27(2), 203–215.
- Belshaw, D.J. & Palmer, J.H.L. 1978. Results of a program of instrumentation involving a precast segmented concretelined tunnel in clay. *Canadian Geotechnical Journal*, 15, 573–583.
- Bezuijen, A. & Talmon, A.M. 2008. Processes around a TBM. Keynote Lecture. Pre-print Volume of the Proceedings of the 6th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, Shanghai.
- Chambon, P. & Corté, J.F. 1994. Shallow tunnels in cohesionless soil: Stability of tunnel face. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 120(7), 1148–1165.
- Davis, E.H., Gunn, M.J., Mair, R.J. & Seneviratne, N. 1980. The stability of shallow tunnels and underground openings in cohesive material. *Géotechnique*, 30(4), 397–416.
- Grant, R.J. & Taylor, R.N. 2000. Stability of tunnels in clay with overlying layers of coarse grained soil. *Proceedings* of GeoEng2000. Melbourne, Australia.
- Hashiguchi, K. 1980. Constitutive equation of elastoplastic materials with elasto-plastic transition. *Journal of Applied Mechanics, ASME*, 102(2), 266–272.
- Jacobsz, S.W. Standing, J.R., Mair, Soga, K., Hagiwara, T. & Sugiyama, T. 2004. Centrifuge modelling of tunnelling near driven piles. *Soils and Foundations*, 44(1), 51–58.
- Jacobsz, S.W., Bowers, K.H. and Moss, N.A. 2005. The effects of tunnelling on pile structures on the CTRL. Preprint Volume of the Proceedings of the 5th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, Amsterdam.
- Kimura, T. & Mair, R.J: 1981. Centrifugal testing of model tunnels in soft clay. Proceedings of 10th International Conference on Soil Mechanics and Foundation Engineering. Stockholm, 1, 319–322.
- Lee, K.M. & Rowe, R.K. 1991. An analysis of threedimensional ground movements: the Thunder Bay tunnel. *Canadian Geotechnical Journal*, 28, 25–41.
- Lee, K.M., Rowe, R.K. & Lo, K.Y. 1992. Subsidence owing to tunnelling. I. Estimating the gap parameter. *Canadian Geotechnical Journal*, 29, 929–940.
- Loganathan, N. & Poulos, H.G. 1998. Analytical prediction for tunnelling-induced ground movements in clays. *Journal of Geotechnical and Geoenvironmental Engineeing*, *ASCE*, 124(9), 846–856.
- Mair, R.J. & Taylor, R.N. 1997. Theme lecture: Bored tunnelling in the urban environment. *Proceedings of 19th*

International Conference on Soil Mechanics and Foundation Engineering, Hamburg, 2353–2384.

- MIDAS-GTS. 2005. Geotechnical & Tunnel Analysis System, MIDAS Information Technology Co., Ltd.
- Nakai, T. & Hinokio, M. 2004. A simple elastoplastic model for normally and over consolidated soils with unified material parameters. *Soils and Foundations*, 44(2), 53–70.
- Selematas, D., Standing, J.R., & Mair, R.J. 2005. The response of full-scale piles to tunnelling. Pre-print Volume of the Proceedings of the 5th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, Amsterdam.
- Sloan, S.W. & Assadi, A. 1992. Stability of shallow tunnels in soft ground. *Predictive Soil Mechanics*, Thomas Telford, London, 1993, 644–662.

- Soubra, A. H. 2002. Kinematical approach to the face stability analysis of shallow circular tunnels. *Proceedings of 8th International Symposium on Plasticity*, British Columbia, Canada, 443–445.
- Wang, J.N. 1993. Seismic Design of Tunnels: A State-ofthe-art Approach. Monograph 7, Parsons, Brinckerhoff, Quade & Diuglas Inc., New York.
- White, D., Take, A. Bolton, M.D. 2003. Soil deformation measurement using particle image velocimetry (PIV) and photogrammetry. *Géotechnique*, 53(7), 619–631.
- Woodward, P.K. & Griffiths, D.V. 1996. Influence of viscous damping in the dynamic analysis of an earth dam using simple constitutive models. *Computers and Geotechnics* 19(3), 245–263.

## Case histories

## A. Sfriso

Department of Estabilidad, University of Buenos Aires, Argentina

ABSTRACT: Twenty papers accepted for publication under IS-Shanghai 2008 Session 3 on Case Histories are classified and summarized. Papers deal with open pit excavations, ground improvement, tunnels, monitoring systems and impact on surroundings, most of them related to projects performed in challenging urban environments. It has been found that different authors follow different approaches when reporting case histories, mainly with respect to the quantitative description of ground conditions and behavior. While this can be attributed to different scientific and professional schools, it is judged that a higher degree of consistency and completeness of the basic information is required for a better usability of the informed data. To contribute to this goal, a short guideline on reporting case histories is proposed.

## 1 INTRODUCTION

IS-Shanghai 2008 became a major opportunity to share recent experience related to underground construction in soft ground. In Session 3, devoted to case histories, many projects reflecting advances in geotechnical engineering related to challenging urban conditions were discussed.

Eight countries contributed a total of twenty papers to this session: eleven from China, one from Japan, two from Korea, one from Singapore, one from Taiwan, one from Italy, two from France and one from Germany.

In the following sections, the twenty papers are classified and summarized. The purpose of this classification is to guide the interested reader to specific information that might be useful for his/her research. Papers are grouped as follows: i) seven papers dealing with open pit excavations; ii) four papers dealing with NATM and drill&blast tunnels; iii) five papers dealing with TBMs and shield tunnels; and iv) four papers dealing with monitoring systems and the evaluation of the impact of under-ground projects on surroundings.

Almost all kind of difficult ground conditions due to existing infrastructure and space constraints were described. For instance, papers dealing with TBM tunnelling describe crossing beneath a shield tunnel and a railway line, across the foundations of a highway bridge and above existing tunnels of an operating metro line. Open pit excavation projects are not simpler, showing the challenges that urban construction poses to geoengineering. Two facts became evident during the revision process, as follows: i) ground conditions are described in widely different ways and with highly varying degree of completeness; and ii) ground and structure behavior are characterized by some "representative" numbers selected with ample liberty. While the reported data is very valuable, some effort must be done to fully exploit it's usability.

It is remarkable that seven out of twenty papers deal with either recent or on-going underground projects in Shanghai, thus reflecting the impresive rate of infrastructure development of the city. A large amount of information is provided with respect to Shanghai soils, including laboratory and field tests and extensive reporting of ground behavior during construction. It is desirable that this valuable information be further analyzed by researchers to produce a consistent and complete set of material parameters for Shanghai soils, as the raw data provided by the papers does not allow for a complete understanding of soil conditions and soil behavior.

In each section, the list of papers is listed in a table and a brief description is presented for each paper. The description merely states the type of project, geology conditions where known and the description of a few selected contributions. These contributions can be of any type, from an overall description of construction processes to a quantitative measure of ground behavior or detailed monitoring data. The writer recommends the reading of all papers, as the valuable information contained there is only superficially grasped by the short descriptions that follow in this report.

Table 1. Papers on open pit excavations.

Author	Project
Hsiung &	Three excav. 20 m deep in Kaohsiung,
Chuay	Taiwan
Konda et al	Eleven braced excavations at Osaka, Japan
Liu, D. et al	18 m deep propped excavation in Shanghai
Liu, G. et al	21 m deep propped excavation in Shanghai
Liu, T. et al	40 m deep propped excavation in Shanghai
Mei et al	6 m deep propped excavation in Shanghai
Osborne et al	JMM ground treatment in Singapore

#### 2 OPEN PIT EXCAVATIONS

The list of papers dealing with open pit excavations is given in Table 1.

## 2.1 Hsiung and Chuay. Observed behaviour of deep excavations in sand

The behavior of three excavations in Kaohsiung, Taiwan, is described. The excavations are approx. 20 m deep, supported by propped diaphragm walls 1.0 m thick and 36 m long, and excavated in medium dense silty sand with clay layers ( $N_{SPT}$ : 5–30). The water table is reported at 3 m to 6 m below ground surface.

The maximum lateral wall displacement  $\delta_{h\,max}$  and surface settlement  $\delta_{v\,max}$  are reported. Values are normalized by the effective height of the excavation  $H_e$  for comparison among the three projects. It is observed that  $\delta_{h\,max}/H_e$  falls in the range 0.03% to 0.3% and that  $\delta_{v\,max}$  is about one half of  $\delta_{h\,max}$ . The effect of the construction sequence and remedial effects to reduce surface settlements are discussed.

## 2.2 Konda et al. Measurement of ground deformations behind braced excavations

The paper reports surface and wall deformations of braced support systems used at eleven sites of the Osaka Subway L8 project in Japan. While details of the support systems are not informed, the geotechnical conditions are reported to vary widely among the sites analyzed, from gravels to soft clays. Wall deformation and surface settlements are described by area indices as shown in Fig. 1. It is concluded that the ground settlement area  $A_s$  is about 20%–30% of the wall deflection area  $A_\delta$  for excavations approx. 21 m deep, but can be much higher if consolidation settlements occur.

## 2.3 Liu, D. et al. Research on the effect of buried channels to the differential settlement of building

The paper deals with the impact of a deep excavation on adyacent structures in Shanghai, China. The

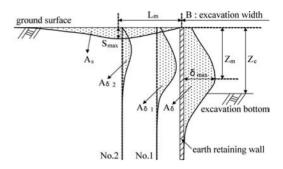


Figure 1. Symbol definition (Konda et al 2008).

Table 2. Description of the soils (Liu, D. et al 2008).

					Shear p	ars
Name	Bottom level [m]	ω %	γ kN/m <sup>3</sup>	e -	c[kPa]	φ[°]
Fill	2.93	_	_	_	_	_
Clay	0.33	34.6	18.2	0.99	21	17.5
Silty clay	-3.87	43.0	17.3	1.21	13	17.0
Silty clay	-11.87	49.1	16.8	1.39	14	11.0
Clay	-14.87	38.9	17.6	1.12	16	14.0
Silty clay	-21.37	34.9	17.9	1.02	15	18.5
Sandy silt	-35.87	32.2	18.0	0.94	4	29.0
Silty sand		26.3	18.8	0.77	1	32.0

excavation is 18 m deep, supported by a diaphragm wall 0.8 m thick and 26 m long with steel supports, excavated in Shanghai soft clays. Ground conditions are described as shown in Table 2. The water table is assumed to be 1 m below ground level.

The maximum lateral wall displacement  $\delta_{h max}$  is reported to be 60 mm, or 0.33% of the excavation height. Extensive analysis of the settlement behavior of an adyacent building is reported in the paper, with emphasis on the non-uniform settlement rate during the excavation stage. While the complexity of the geological conditions is assessed, no data on the compression and permeability parameters of the soft clay layers is given and the consolidation process is not discussed, despite the fact that the reported settlement of the building was up to 125 mm.

## 2.4 Liu, G. et al. Performance of a deep excavation in soft clay

The deformation behavior a 21 m deep excavation in Shanghai, China, is described. The geologic profile is the quaternary soft alluvial and marine clay deposit typical of Shanghai City. The authors brief on the geology and present Fig. 2 that allows for a first understanding of the site conditions. GWL is reported to be 1 m below ground surface.

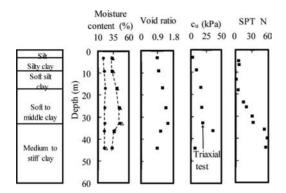


Figure 2. Soil profile and parameters in Shanghai (Liu, G. et al 2008).

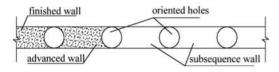


Figure 3. The "two-drill-one grab" method for the construction of the diaphragm wall (Lui, T. et al 2008).

The support system is a  $0.8\,m\times40\,m$  propped diaphragm wall with base compensation grouting and prestressed struts. The maximum lateral wall displacement  $\delta_{h\,max}$  is reported to be 55 mm, or 0.32% of the excavation height. This result is compared with other measured values in Shanghai and other sites having rather similar soil conditions.

The effect of the stiffness of multi-propped support systems is analysed and the three dimensional behavior of the excavation is assessed. It is concluded that a corner effect exists that reduces the lateral wall displacement corner-to-center ratio  $\delta_{h\,max\,(corner)}/\delta_{h\,max\,(center)}$  to about 0.39–0.74.

### 2.5 Liu, T. et al. The construction and field monitoring of a deep excavation in soft soils

The paper describes the construction procedure of a very large and deep excavation performed in Shanghai clays. The excavation was 263 m long, 23 m wide and 38 m–41 m deep, supported by a 1.2 m thick and 65 m long multi-propped diaphragm wall.

The deep diaphragm wall construction procedure is described in detail, including the so called "two-drillone grab" construction method shown in Fig. 3 and the employment of a counterweight to better clean the last panel's lateral surface before pouring the next panel, as shown in Fig. 4.

Jet grouting was extensively employed to improve soil conditions in the passive zone. Reported inclinomenter data shows wall behavior along the

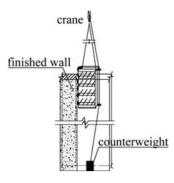


Figure 4. Procedure used to clean the lateral surface of the last panel (Lui, T. et al 2008).

construction stages. The max lateral wall deflection was  $\delta_{h max} = 50$  mm, or 0.12% of the total height of the excavation. Surface settlements are reported but not associated by the authors to a consolidation process. Moreover, neither a set of compression parameters nor an analysis of compression behavior of Shanghai clays is reported.

## 2.6 Mei et al. Excavation entirely on subway tunnels in the central area of the People's Square

The design and construction of a shallow excavation 6 m deep in Shanghai, China, is described. The particular challenge of this project was that the bottom of the excavation was placed 3 m above existing tunnels. A support system consisting in a soil-cement pile wall 3.2 m thick with directional jet grouting was designed and passive tension piles were provided to control uplift.

## 2.7 Osborne et al. The benefits of hybrid ground treatment in significantly reducing wall movement: a Singapore case history

The paper reports the first major use in Singapore of a hybrid ground improvement procedure called Jet Mechanical Mixing (JMM). JMM is a large diameter deep mixing method that forms a central core of mixed soil combined with a jet-grouted outer annulus. A schematic diagram of the drilling tool is shown in Fig. 5.

The system was employed in the Nicoll Highway Station Project. The excavation was 27 m deep, supported by a 1.5 m thick and 51 m long diaphragm wall. The JMM was used to make a base plug of improved soil 7 m thick below the excavation. Ground conditions include fill, fluvial sand, fluvial clay and normally consolidated marine clay. The authors concluded that the ground improvement technique employed reduced the lateral wall displacements  $\delta_{h max}$  by a factor of three.

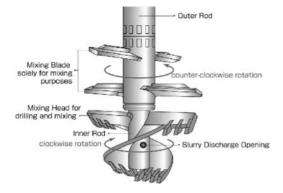


Figure 5. Schematic diagram of the drilling tool showing the mixing arm of the JMM machine (Osborne et al 2008).

Table 3. Comparison of wall deformation data.

Author	Soil	H <sub>e</sub> m	$\delta_{hmax}/H_e$ %
Hsiung & Chuay	Sand	20	0.03-0.30
Konda et al	Varies	21	0.10-0.24
Liu, D. et al	Clay	18	0.33
Liu, G. et al	Clay	20	0.32
Liu, T. et al	Clay	40	0.12
Osborne et al	Clay	27	0.09

#### 2.8 Comparison between wall deformation data

Table 3 lists the lateral wall displacement  $\delta_{h\,max}$  as a fraction of the excavation height  $H_e$  for the different projects and construction procedures described. No correlation can be observed between  $H_e$  and  $\delta_{h\,max}$ , confirming the well-known fact that wall deflection heavily depends on the particular construction procedure, to the extent that it might be concluded that diaphragm walls and construction procedures are designed to accomplish lateral wall deflections that balance the performance requirements of engineers and clients.

## 3 NATM/DRILL&BLAST TUNNELS

The list of papers dealing with NATM and drill&blast tunnels is given in Table 4.

3.1 Eclaircy-Caudron. Displacements and stresses induced by a et al tunnel excavation: case of Bois de Peu (France)

The paper describes the ground response during the construction of the two twin drill&blast tunnels in Bois de Peu, France. The tunnels have a cross section area of  $130 \text{ m}^2$ , a length of 520 m, and were excavated through

Table 4. Papers on drill&blast and NATM tunnels.

Author	Description
Eclaircy- Caudron et al	Drill&blast tunnel in Bois de Peu, Fr.
Guiloux et al Quick et al Yoo et al	Drill&blast tunnel in Morocco NATM tunnel in Mainz, Germany Subsidence due to water drawdown

clays and soft rocks under 8 m to 140 m of overburen. The support system was formed by shotcrete, steel ribs and radial bolts. Unfavourable ground conditions in the clay soils demanded the use of a structural invert, forepoling and face bolting.

An interactive design and construction procedure was employed, where monitoring data was used in an adaptive design process. The paper reports the construction sequence, the use of monitoring information to adjust design, and extensive data on face displacements measured at four instrumented sections. It is shown that extrusion extended one diameter ahead of the tunnel face and that high face extrusion proved to be a good indicator of poor ground performance and risk of face failure.

## 3.2 Guiloux et al. Case history on a railway tunnel in soft rock (Morocco)

The construction process of the Ras R'Mel tunnel in Morocco is described. The tunnel is 2.6 km long and has a cross section of  $60 \text{ m}^2$ . It was excavated through weak flysch under 50 m to 150 m overburden by drill&blast method. The support system consists in 23 cm of shotcrete and steel ribs. A particular feature of the construction procedure is the use of a formwork to reduce shotcrete loss, as shown in Fig. 6. Stress-to-UCS ratios up to 3.5 and convergences up to 300 mmwere reported, values higher than usual for drill&blast tunnels in rock.

# 3.3 Quick et al. Challenging urban tunnelling projects in soft soil conditions

The design and construction of the New Mainz Tunnel in Germany is presented. New Mainz Tunnel runs parallel to Old Mainz Tunnel, built in 1884, with a clearance of 4 m to 50 meters. The tunnel is 1250 m long, with a cross section of  $140 \text{ m}^2$  and runs below buildings with 10 m to 23 m overburden. Soils are marly clays, silts and sand, and the support system is a complex combination of bolting, umbrellas, face bolting and reinforced shotcrete. Ground improvement techniques employed at some sections to reduce surface settlements are described. It is reported that a reduction of settlements from 11 cm to 1.5 cm–2.5 cm was achieved by ground improvement.



Figure 6. Construction of Ras R'Mel tunnel (Guiloux et al 2008).

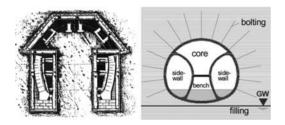


Figure 7. Construction procedure for Old and New Mainz Tunnels (Quick et al 2008).

It is very interesting to note the differences and similitudes in construction procedures used in two similar tunnels separated in time by one century, as shown in Fig. 7.

## 3.4 Yoo et al. Characteristics of tunneling-induced ground settlement in groundwater drawdown environment

The paper studies the effect on surface settlements of groundwater drawdown due to tunnel construction. The case analyzed consists in a  $70 \text{ m}^2$  tunnel excavated through weathered granite with 20 m to 30 m overburden formed by fill, alluvium and weathered rock. The support system consisted in pre-grouting, pipe umbrellas, rock bolts and shotcrete.

Water drawdown produced surface settlements that started approximately six diameters ahead of tunnel face and that stabilized six diameters behind it. The problem was analyzed by a parametric study using a 2D finite element model with Mohr-Coulomb constitutive model. It was concluded that surface settlements due to tunnel construction have different patterns when ground-water drawdown is present, when compared to the normal case.

It must be noted that surface settlements due to groundwater drawdown are a well-known problem

Table 5. Papers on shield tunnels and TBMs.

Author	Description
Antiga& Chiorboli	EPB tunnels in Milano, Italy
Gong&Zhou Wang et al Wong et al Xu et al	Tunnel beneath railway line in Shanghai Crossing below existing tunnel in Shanghai Crossing above existing tunnel in HKSAR Crossing bridge foundations, Shanghai

of geotechnical engineering that is accounted for by consolidation theory and that is simulated with constitutive equations that account for inelastic compression. The Mohr-Coulomb constitutive model reported to be used in the model, however, neither simulates inelastic compression nor includes compression parameters.

## 4 SHIELD TUNNELS AND TBMS

The list of papers dealing with shield tunnels and TBMs is given in Table 5.

4.1 Antiga and Chiorboli. Tunnel face stability and settlement control using earth pressure balance shield in cohesionless soil

The paper analyzes and compares two case histories of EPB tunnels driven in Milano, Italy. Both tunnels were excavated through 50 m to 60 m of medium-dense to dense alluvial sands and gravels.

The authors provide a comprehensive summary of factors affecting subsidence of shield tunnels in sands. They conclude that a high advance rate produces less volume loss and reduces subsidence and show that EPB face pressure is poorly correlated to surface settlements but depends on technological aspects of backfilling operations.

# 4.2 Gong and Zhou. Shield tunneling beneath existing railway line in soft ground

The design and construction of the Metro Line 11 tunnel running below the Hu-Ning railway line in Shanghai, China is described. The tunnel was driven through Shanghai soft clays below 11 m overburden. Water level is reported to be 1 m below ground surface.

The tunnel has a cross section of  $30 \text{ m}^2$  and is supported by a segmental lining 35 cm thick. Extensive soil improvement, including jet-piles and grouting, was performed to control surface settlements. It is reported that settlements in the improved sectors were 85% lower than those of the unimproved sectors. Fig. 8 shows the longitudinal irregularity of the tracks after the tunnel was driven below the railway.

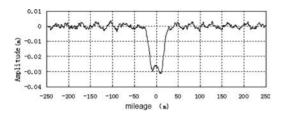


Figure 8. Longitudinal irregularity of the tracks by tunneling-induced deformation (Gong and Zhou 2008).

## 4.3 Wang et al. Supervision and protection of Shanghai Mass Rapid Line 4 shield tunneling across the adjacent operating metro line

The paper reports the design, construction and monitoring procedure of the crossing of Mass Rapid Line 4 (MRL4) shield tunnels below existing L2 Metro tunnels in Shanghai, China. MRL4 tunnels have a cross section of  $32 \text{ m}^2$  and were driven in Shanghai soft clays only one meter beneath L2 tunnels at a small crossing angle. No information is provided with respect to soil parameters and support systems of both the existing and new tunnels.

Ground control measures taken to reduce displacements in the existing tunnels are described. It is remarkable that, despite the short distance between the new and existing tunnels, control measures did not include ground improvement due to lack of surface space.

Extensive monitoring data was generated during the operation, and some of it is summarized in the paper. It was found that a strict control of shield deviation, careful tail grouting and a slow advance rate aided in controlling L2 tunnel displacements. Shield tail grouting is reported to have influenced settlements some ten to fifteen meters above and behind the grouting stages. As a general conclusion, authors recommend very slow advance rates to minimize tunnel induced subsidence.

## 4.4 Wong et al. Kowloon Southern Link – TBM crossing over MTR Tsuen Wan Line tunnels in HKSAR

The paper describes the construction of Mass Transit Railway (MTR) Crossing. MTR Crossing is the point where the new Kowloon S. Link twin tunnels cross (2 m above) the existing MTR tunnels in Hong Kong. Kowloon S. Link tunnels have a cross section of  $51 \text{ m}^2$  and were driven through decomposed and sound granite under 6.8 m overburden by a shield-slurry TBM with an horizontal clearance of 900 mm. Water table is reported to be 2.5 m below ground level.

No restrictions to service of MTR tunnels were allowed, and therefore a series of ground improvement

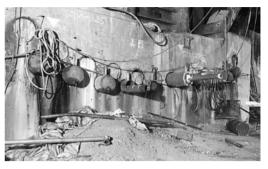


Figure 9. Horizontal umbrella placed between the new and existing tunnels (Wong et al 2008).

and ground control measures had to be undertaken. These measures included extensive jet-grouting of the whole area and the installation of a physical barrier made by an umbrella of horizontal pipe piles placed below the new tunnel as shown in Fig. 9. While a sophisticated monitoring system is reported to have been installed, there is no information of ground or tunnel behavior during the construction of MTR Crossing.

## 4.5 Xu et al. Application of pile underpinning technology on shield machine crossing through pile foundations of road bridge

The paper deals with the design problem of a  $39 \text{ m}^2$  EPB tunnel hiting 14 piles of a bridge foundation. The tunnel belongs to Metro Line 10 in Shanghai, China. Soil conditions are described as fill, organic soil, clayley silts and clays. Two underpinning schemes are proposed in the paper: i) the bridge's foundations be reinforced before eliminating the existing piles; and ii) the existing piles be replaced after foundation reinforcement. It is unclear whether the project is completed or not.

## 5 MONITORING AND IMPACT TO SURROUNDINGS

The list of papers dealing with monitoring systems and impact to surroundings is given in Table 6.

## 5.1 Kim et al. Environmental problems of groundwater around the longest expressway tunnel in Korea

A 3D hydrogeologic model that simulates the impact of Inje tunnel on grounwater level is presented. Inje tunnel is in fact two 14.5 m wide by 11 km long drill&blast twin tunnels, claimed by the authors to form the longest expressway in Korea. The geologic

Table 6. Papers on monitoring systems and impact to sourroundings.

Author	Description
Kim et al	Hydrogeological model for Inje Tunnel, Korea
0	Description of deformation monitoring systems
Qiu et al	Monitoring system applied at Beijing
	Subway L1
Zhao et al	Math. model of settlement induced lining stress

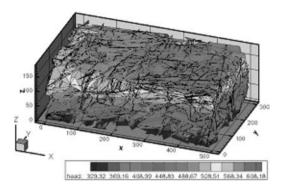


Figure 10. 3D model of fracture network and tunnel for steady state simulation of groundwater flow (Kim et al 2008).

profiles is composed by methamorphic rocks with some superficial debris.

A MODFLOW continuous model was implemented for the far field simulation of groundwater flow, while a MAFIC discontinuous model was developed for the near field model including the effect of grouting on water flow, as shown in Fig. 10. The conclusion is that grouting might reduce groundwater inflow to the tunnel by 53% to  $3.6 \text{ m}^3/\text{h/km}$ , and that the expected drawdown might be reduced by 65% to about 0.6–1.1 meters.

## 5.2 Liu and Wang. Deformation monitoring during construction of subway tunnels in soft ground

The objectives, methods and required precision for open field and urban tunnel ground monitoring are discussed in this paper. Tunnel Profile Scanners are introduced. These are two digital cameras mounted on a rigid frame that produce a stereoscopic digital image of the tunnel surface.

A high amount of low-precision displacement data is recorded, with an estimated error of 5 mm. Statistical analysis of this data, however, is reported to be useful as a monitoring tool. An automatic deformation device using an advanced Geodetic Monitoring Software, capable of managing high precision monitoring data, is also described.

## 5.3 *Qiu et al. 3D deformation monitoring of subway tunnel*

The paper describes the application of LIDAR technology to Beijing Subway L1 tunnel. LIDAR technology allows for a rate of 3D data acquisition of 100.000 points/sec by 3D laser scanning. With this technology, expensive reflecting prisms do not need to be used and can be replaced by reflection sheets placed on the tunnel walls. NURBS (non uniform rational B splines) technology is used to interpolate the obtained data, and a mathematical model is developed for the analysis of the information. An example is given where a differential settlement of 0.29 mm could be measured using this technology.

## 5.4 Zhao et al. Effect of long-term settlement on longitudinal mechanical performance of tunnel in soft soil

The paper presents a structural model for the induced longitudinal stress developed in a segmental tunnel due to nonuniform settlements and apply the theory to a case history.

As reported by the authors, an unspecified highway shield tunnel 30 yr old, settled/heaved up to 30 mm in the last 10 years. The mathematical model was used to evaluate the structural performance of the tunnel based on longitudinal curvature radius R. It is concluded that R < 27300 m may induce leakage; stresses induced by R < 18800 m may fail segments; bolt yielding should be expected for R < 15000 m; and that tensile failure would probably occur for R < 302 m.

## 6 SUMMARY OF CASE HISTORIES RELATED TO PROJECTS IN SHANGHAI

Seven out of the twenty case histories presented at Session 3 are related to challenging underground projects in Shanghai, China. This is an unique opportunity to advance in technology and to calibrate design procedures for soft soils with valuable experimental evidence.

However, it must be noted that no complete and consistent description of Shanghai soils has been found among all papers. There is almost no information on basic index parameters like liquid limit, compression or recompression indexes and apparent OCR due to ageing.

The shear strength parameters as reported by the different authors are listed in Table 7 and can be used as an example to further illustrate the observed lack of information. While parameters listed in Table 7 are

Table 7. Strength parameters reported for Shanghai clays.

Author	Depth: 10-15 m		Depth: 20–25 m		
	c kPa	ф °	c kPa	ф °	
Gong&Zhou	8	24.0	45	15.0	
Liu, D. et al	14	11.0	15	18.5	
Liu, G. et al		Fig	. 2.		
Liu, T. et al	7	32.0	43	15.5	
Mei et al		no c	lata		
Wang et al	no data				
Xu et al	no data				

not – and cannot be – either undrained or effective stress parameters, none of the authors explained what these parameters actually mean.

Shanghai clays are normally consolidated clays with some degree of ageing. Strength of these soils is universally characterized by undrained shear strength  $s_u$ , either determined by in situ testing or lab testing. While it might be argued that there are many undrained shear strengths for a given clay due to stresspath dependency of shear strength, it must be accepted that the geotechnical community would appreciate any reported value of shear strength. The overall lack of a complete description of Shanghai clays highlights the value of Fig. 2.

Drained shear strength parameters are probably fewer in quantity, though also presumed to be widely available, given the impressive pace of city growth and the excellent degree of geoengineering involved. Unfortunately, no clear discussion on the critical state friction angle of Shanghai soils was found among all papers.

# 7 SHORT, DRAFT GUIDELINE ON REPORTING CASE HISTORIES

After the experience of reviewing papers submitted to Session 3 to write this report on Case Histories, the writer believes that a short and draft guideline on reporting case histories for projects dealing with soft soils might be useful. The guideline neither intends to be complete nor definitive, as it is only based on the information searched but not found in the papers during the process of writing this report.

- 1. Name of the project and location.
- 2. State of the project by the time of submittal of the paper: design phase, under construction, or completed.
- Basic information on geometry: i) for tunnels, length, area and a picture showing the cross section with main dimensions; ii) for excavations, type,

dimensions, structural description of the support system and a picture showing the support system and soil profile.

- 4. A description of geological/geotechnical ground conditions and water table.
- 5. A comprehensive set of clearly defined soil parameters. Ideally, SPT and/or CPT profiles should be included. Both total stress and effective stress shear parameters should be indicated, either as measured or estimated values. If other strength parameters are also reported, their meaning should be fully explained. For problems involving large subsidence or other compression-driven phenomena, compression parameters and material permeability should also be indicated.
- 6. A brief description of the construction process.
- 7. Description of ground behavior and unexpected changes in ground conditions during construction activities.
- 8. Monitoring information when available, or a statement otherwise. Some amount of basic raw data should be included to better understand and use some derived parameters like  $A_{\delta}$ , see Fig. 1. Fig 8 is a good example of information relevant to the subject being discussed.
- 9. For unusal equipments or construction procedures, some figures/pictures that better explain the idea, see Figs. 3 to 7 and 9.
- 10. For non conventional calculations and models, an illustrative picture, see Fig. 10.

In all cases, the source and degree of confidence of the provided information should be assessed.

A good example of reporting a case history can be found in a paper by Shao and Macari (Shao and Macari 2008), selected because it deals with a deep excavation in Shanghai clays. Twenty three key parameters identify each of the six main layers that form Shanghai soils profile, including water content and void ratio, classification data, shear and compression parameters, permeability and SPT values (Shao and Macari 2008).

# 8 CONCLUSIONS

Session 3 of IS-Shanghai 2008 became an excellent opportunity to share experience related to underground construction in soft ground in challenging urban conditions.

Twenty papers from eight countries, dealing with open pit excavations, NATM and drill&blast tunnels, TBMs and shield tunnels, and monitoring systems were classified and summarized in this report. While the reported data is very valuable, some effort must be done to fully exploit it's usability because no consistent procedure was followed by the authors to report ground conditions and ground behavior during construction activities. A large amount of information was provided with respect to Shanghai soils, including laboratory, field tests and ground behavior during construction. Lack of definition of the reported parameters is judged to maje the interpretation of the reported information not easy. To allow for a better consistency and completeness of reported data, a short draft guideline on reporting case histories is proposed.

# ACKNOWLEDGEMENTS

The writer wishes to acknowedge the authors of the summarized papers for sharing valuable information with the geo-community and the organizing comittee for inviting him to deliver this general report.

#### REFERENCES

- Antiga, A. and Chiorboli, M. 2008. Tunnel face stability and settlement control using earth pressure balance shield in cohesionless soil. IS-039.
- Eclaircy-Caudron, S., Dias, D. and Kastner, R. 2008. Displacements and stresses induced by a tunnel excavation: case of Bois de Peu (France). IS-107.
- Gong, Q. and Zhou, S. 2008. Shield tunneling beneath existing railway line in soft ground. IS-013.
- Guiloux, A., Le Bissonnais, H., Marlinge, J., Thiebault, H., Ryckaert, J., Viel, G., Lanquette, F., Erridaoui, A. and Hu, M. 2008. Case history on a railway tunnel in soft rock (Morocco). IS-367.
- Hsiung, B. and Chuay, H. 2008. Observed behaviour of deep excavations in sand. IS-005.
- Kim, S., Yang, H. and Yoon, S. 2008. Environmental problems of groundwater around the longest expressway tunnel in Korea. IS-087.
- Konda, T., Ota, H., Yanagawa, T. and Hashimoto, A. 2008. Measurements of ground deformations behind braced excavations. IS-337.

- Liu, D., Wang, R. and Liu, G. 2008. Research on the effect of buried channels to the differential settlement of building. IS-118.
- Liu, G., Jiang, J. and Ng, C. 2008. Performance of a deep excavation in soft clay. IS-082.
- Liu, S. and Wang, Z. 2008. Deformation monitoring during construction of subway tunnels in soft ground. IS-120.
- Liu, T., Liu, G. and Ng, C. 2008. The construction and field monitoring of a deep excavation in soft soils. IS-029.
- Mei, Y., Jiang, X., Zhu, Y. and Qiao, H. 2008. Excavation entirely on subway tunnels in the central area of the People's Square. IS-140.
- Osborne, N., Ng, C. and Cheah, C. The benefits of hybrid ground treatment in significantly reducing wall movement: a Singapore case history. IS-378.
- Qiu, D., Zhou, K., Ding, Y., Liang, Q. and Yang, S. 2008. 3D deformation monitoring of subway tunnel. IS-151.
- Quick, H., Michael, J., Meissner, S. and Arslan, U. 2008. Challenging urban tunnelling projects in soft soil conditions. IS-358.
- Shao, Y. and Macari, E. 2008. Information feedback analysis in deep excavations. ASCE Int. Jou. Geom. Vol. 8, 1, 91–103.
- Wang, R., Cai, Y. and Liu, J. 2008. Supervision and protection of Shanghai Mass Rapid Line 4 shield tunneling across the adjacent operating metro line. IS-033.
- Wong, K., Ng, N., Leung, L. and Chan, Y. 2008. Kowloon Southern Link – TBM crossing over MTR Tsuen Wan Line tunnels in HKSAR. IS-370.
- Xu, Q., Ma, X. and Ma, Z. 2008. Application of pile underpinning technology on shield machine crossing through pile foundations of road bridge. IS-326.
- Yoo, C., Kim, S. and Lee, Y. 2008. Characteristics of tunneling-induced ground settlement in groundwater drawdown environment. IS-329.
- Zhao, H., Liu, X., Yuan, Y. and Chi, Y. 2008. Effect of longterm settlement on longitudinal mechanical performance of tunnel in soft soil. IS-199.

*Theme 1: Analysis and numerical modeling of deep excavations* 

# Optimization design of composite soil-nailing in loess excavation

# G.M. Chang

Urban Construction and Environment Engineering Department, West Anhui University, Lu'An, Anhui, P.R. China School of Civil Engineering, Chang'An University, Xi'An, Shanxi, P.R. China

ABSTRACT: The loess excavation has its unique characteristics compared with the others due to its structural property and collapsibility. In order to acquire the work mechanism and design methods of composite soil-nailing in loess excavation, a reasonable finite element analysis model is selected. The optimization design methods are introduced based on the results of finite element analysis conducted to determine the regularity of deformation, the safety factor and the endogen force of the structure along with the change of design variable. Finally, the optimization design methods are validated contrasted with the data measured in an actual project.

# 1 INTRODUCTION

Composite soil-nailing combined soil nails with other forms of supporting measures has avoided the soil-nailing technology from excessive dependence on the soil and expanded its application field. Among the different kinds of composite soil-nailing forms, the anchor composite soil-nailing support method is widely applied for its powerful location adaptability, easy construction, low cost and reducing the pit deformation remarkably. However, its work mechanism and design method, especially the Loess Pit anchor composite soil-nailing research, fall behind the project practice by far. In the first instance, this paper aims at studying nail design parameter selection in plain soilnailing on the premise of maintaining soil-nailing total length, and then replacing a anchor for a soil nail to research the parameter value of anchor composite soilnailing structure and optimization design under the circumstances of maintaining plain soil-nailing design parameters a more optimal value.

# 2 PARAMETER ANALYSIS

The overall stability and working performance of excavation supporting is closely related to the design parameters. Understanding and grasping the relations of the overall stability safety factor with the change of these design variables, particularly this kind of sensitivity degree that variety, have special and important meaning for guiding engineering practice.

# 2.1 Hypothesis

To simplify the calculations, we make the following assumptions when carry on the numerical analysis to the composite soil-nailing numerical analysis:

- 1 Composite soil-nailing problems are plane strain problems;
- 2 Soil-nailing and assistance reinforcement materials are elastic materials;
- 3 The soil is presumed as the elastic-plastic material.

# 2.2 Computation diagram and parameter of material

# 2.2.1 Computation diagram

Engineering experience shows that the influence of excavation width is about 3 to 4 times of the excavation depth, influence depth is about 2 to 3 times of excavation depth. The case assumes that the excavation depth is 9.5 m, the total length of the finite element model is 45 m, the total height is 25 m and the slope gradient is 1:0.1 (Fig. 1).

# 2.2.2 Boundary conditions and loads

On the left and right boundary of the model, we set the X-direction displacement to zero and allow the Y direction deformation; the X and Y direction displacement of the bottom boundary are zero; the top is a free surface. Initial stress field is gravity stress field; the value of Gravitational Acceleration is 9.8 m/s<sup>2</sup>. Since composite soil-nailing is usually constructed after precipitation, the impact of groundwater is not considered.

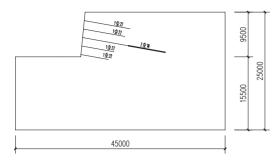


Figure 1. Finite element analysis model (Anchor at middle).

### 2.2.3 Material parameters

This research was completed against the loess excavations and the soil parameters were provided from a engineering investigation report in Xi'an City. Because the soil distributes in certain scope are uneven, it is discommodious to carry on the Numerical Calculation and take the soil strength average value of each level. Soil nails and anchors take the form of the commonly used procedure in Xi'an: Soil-nailing 110 mm diameter bored, steel bar 1  $\phi$  22; the anchor hole diameter 150 mm, steel bar  $2\phi$  18, surface 100 for C25 thick concrete, reinforced with distribution steel bar network. Soil nail and anchor were made of steel bars that wrapped with cement slurry composition. Slurry tightly wrapped the external part of steel bars, and occluded with the soil in dogtooth. In order to simulate the mechanical behavior of soil-nailing and anchors correctly and simplify finite element analysis process, we regard the steel bar and the cement paste body as a kind of compound material. Materials geometric and mechanical parameters, as follows:

Soil: c = 30 kpa,  $\varphi = 18^{\circ}$ , gravity  $\gamma = 18$  KN/m<sup>3</sup>, deformation modulus  $E_0 = 1.8 \times 10^7$  Pa, Poisson's ratio  $\mu = 0.3$ ; Soil-nailing: diameter 0.11 m, sectional area is 0.0094985 m<sup>2</sup>, moment of inertia  $1.8324 \times 10^{-6}$  m<sup>4</sup>, equivalent modulus of elasticity  $E_{eq} = 2 \times 10^{10}$  Pa, Poisson's ratio  $\mu = 0.3$ ;

Anchor: the sectional area of free segment is  $5.0868 \times 10^{-4} \text{ m}^2$ , moment of inertia is  $1.030077 \times 10^{-8} \text{ m}^4$ , elastic modulus  $E_s = 2 \times 10^{-11}$  Pa, the sectional area of anchorage segment is  $0.0176625 \times 10^{-4} \text{ m}^2$ , moment of inertia is  $2.483789 \times 10^{-5} \text{ m}^4$ , equivalent elastic modulus  $E_{eq} = 2.03 \times 10^{10}$  Pa. Poisson's ratio  $\mu = 0.3$ ; Surface (unit length) : The sectional area is  $0.1 \text{ m}^2$ , moment of inertia is  $8.33333 \times 10^{-5} \text{ m}^4$ . Equivalent elastic modulus  $E_{eq} = 2.1 \times 10^{10}$  Pa, Poisson's ratio  $\mu = 0.3$ .

Contact surface friction: According to the research of reference (Chen, 2000, Wang, 1997), the soil-nail contact surface and the soil-anchor contact surface friction value is 60 kPa.

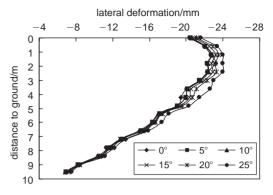


Figure 2. Relations of soil-nailing angle and pit displacement.

Table 1. Relations of soil-nailing angle and safety factor.

Soil-nailing angle	0	5	10	15	20	25
Safety factor	1.593	1.623	1.654	1.615	1.568	1.53

### 2.3 Soil-nailing support

#### 2.3.1 *The angle of soil-nailing*

Concerning with the construction method, the angle of soil-nailing has great influence on the pit displacement, the safety factor and the surface subsidence. Taking the total length of soil-nailing is 40 m, establishing five rows of soil-nailing, taking the soil-nailing level and the vertical spacing takes 1.8 m, the first row of soil-nailing depth of burying is 1.8 m. Dividing five steps excavates, the first step of excavation depth is 2.3 m, and the other step of cutting depth is 1.8 m each. Soil-nailing obliquities are calculated by inclination of  $0^\circ$ ,  $5^\circ$ ,  $10^\circ$ ,  $15^\circ$ ,  $20^\circ$  and  $25^\circ$  respectively.

Figure 2 and Table 1 show that the horizontal displacement is gradually increasing and changing at an increasingly rapid pace as soil-nailing angle from 0 degrees to 25 changes gradually. When the pit design requires strict control of the horizontal displacement, they should use a smaller angle. Safety factor in soil-nailing angle reduces 10 degrees at the largest and declines rapidly with the angle increases after 10 degrees.

On the other hand, Soil-nailing angle is related with construction methods and soil-nailing construction usually adopt the self grouting methods, in the hope of soil-nailing has more inclination to make the cement grout fill soil-nailing holes under the weight easily.

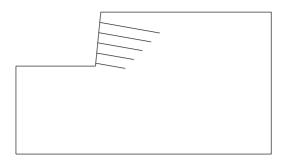


Figure 3. Long at upper row and short at lower row.

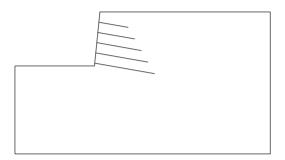


Figure 4. Long at middle row and short at upper and lower row.

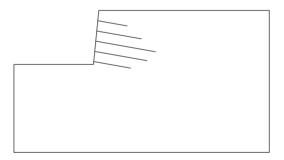


Figure 5. Short at upper row and long at lower row.

So after considering the pit displacement, the safety factor and construction factors, the angle should be about 10–15 degrees.

#### 2.3.2 Scheme of soil-nailing layout

Other researchers do more about the schemes of soilnailing layout (Hu & Song, 1997, Li & Zhang, 1999). But their studies focus more on comparison between long at upper row and short at lower row scheme (longshort scheme) and short at upper row and long at lower row scheme (short-long scheme). But in practice, we often use long at mid row and short at upper and lower row scheme (mid-long scheme), especially when the soil-nailing is used with anchor together (Figs. 3–5).

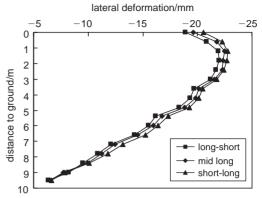


Figure 6. Relations of soil-nailing layout and displacement.

Table 2. Relations of scheme of soil-nailing layout and safety factor.

Scheme of layout	Long-short	Short-long	Mid-long
Safety factor	1.478	1.447	1.376

Selecting nail angle is 10 degrees then calculates and analyzes on three different layouts, we get the results Figure 6 and Table 2 below.

From Figure 6 and Table 2 we can find that there are smallest displacement and largest safety factor when using long-short scheme. On the contrary, there are largest displacement and smallest safety factor when using short-long scheme. As the same conclusion with our forerunners, displacement and safety factor that use short-long short scheme are between the other two modes and we can see that when using plain soil-nailing support a long-short scheme should be adopted.

# 2.4 Anchored soil-nailing support

At present, the application of prestressed anchor in composite soil-nailing design is often used empirically and there has not a determinate calculative method to set the anchor position, the length of anchorage, prestressed value.

Based on previous studies, we chose the nails angle of 10 degrees and long-short layout scheme to research the anchored soil-nailing support.

# 2.4.1 Location of prestressed anchor

After replacing the 1st and 3rd rows and the fifth row of soil nails with anchor, let us study the influence of blot location on the pit's level displacement and safety factors. Diagrams shows in Figures 7–8, 1, and the results are shown in Figure 11 and Table 2 below.

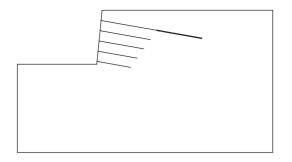


Figure 7. Anchor at upper row.

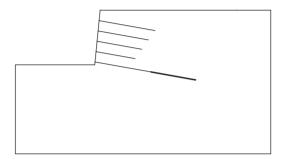


Figure 8. Anchor at lower row.

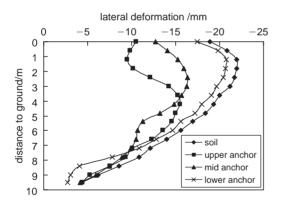


Figure 9. Relations of anchor locations and displacement.

Table 3. Relations of anchor locations and safety factor.

Anchor locations	Upper	Middle	Lower
Safety factor	1.496	1.546	1.511

Figure 9 and Table 3 show that add prestressed anchor into soil-nailing can significantly reduce the maximum horizontal displacement, especially in and near the anchor location. In addition, compared with

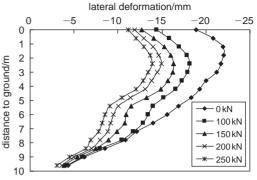


Figure 10. Relations of prestressing and displacement.

Table 4. Relations of prestressing and safety factor.

Prestressing value/kN	0	100	150	200	250
Safety factor	1.546	1.597	1.617	1.624	1.628

plain soil-nailing, it can significantly reduce the level of surface displacement through adding prestressed anchor, particularly the top-anchor scheme and bottom-anchor scheme have the most obviously effect on the surface of the horizontal displacement control. Anchor locations also affect the safety factor. It has the biggest safety factor when anchor at the central pit.

Therefore, to control the pit deformation, the angle of anchor would favor the upper-anchor scheme; the mid-anchor scheme is the most beneficial to improve the safety factor. However, in the engineering practice, because excavations concentrate more and more on urban areas and the anchor may into the pit slope outside more distance, there may affect anchor construction for the adjacent buildings when used the top anchor scheme.

# 2.4.2 Level of prestressing value

Used prestressed anchor replace with the 3rd soilnail, taking the anchor free segment length is 10 m, anchorage segment length is 8 m, prestressing value is 0 kN, 150 kN, 200 kN and 250 kN respectively for calculating. The results calculated from the horizontal displacement and the safety factors are shown in Figure 10 and Table 4.

Figure 10 and Table 4 show that the impact of prestressing value on the horizontal displacement is greater. When the magnitude of prestressing is 100 kN, horizontal displacement decreases more than plain soil-nailing from the maximum displacement of 23.1 mm to 17.6 mm lower. If prestressing value

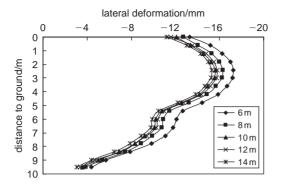


Figure 11. Relations of anchorage segment length and displacement.

 Table 5.
 Relations of anchorage segment length and safety factor.

Anchorage length/m	6	8	10	12	14
Safety factor	1.586	1.617	1.629	1.637	1.641

increases to 150 kN, the maximum horizontal displacement will decrease to 15.3 mm. If the prestressing value is more than 150 kN, such as 200 kN, 250 kN, horizontal displacement will continue to decline, but the reduction range is insignificant. As the value of the prestressing increases, the Pit safety factor will gradually increase in a limited extent, which means the value of prestressing has no significant impact on safety factor.

# 2.4.3 The length of anchorage segment

Selecting a middle-anchor scheme, the prestressing value is 150 kN, taking the length of anchorage for 6 m and 8 m, 10 m, 12 m, 14 m to calculate, the results are shown in Figure 11 and Table 5.

Figure 11 and Table 5 show that the horizontal displacement of Pit gradually diminishes as the anchorage length increases, but the reduction is modest. Pit safety factor would increase as the length of anchor increased either, but not markedly.

# 3 ANCHOR AND SOIL-NAILING WORKING TOGETHER MECHANISM

Plain soil-nailing support is a passive support system and soil-nailing would have a role only when the soil generates sufficient deformation. Anchor belongs to the initiative support system and through prestress to control soil deformation. Anchored composite soil-nailing is a special kind of support, which between plain soil-nailing and prestressed anchoring support. It has the advantages of both the plain soilnailing support and the prestressed anchor support simultaneously.

#### 3.1 Anchor and soil-nailing working together

It is at the initiative stressful condition as anchor support construction completed because of the existence of prestressing. As a result of the anchor prestressing reaction, the soil is caused to be at the pressed condition, reduced soil lateral deformation. On the other hand, anchor is wrapped in the cement paste, and adhibited with cement paste. Because of the holes, pores and crannies existed in soil; the cement and the soil assume the zigzag linking. After anchor tension deformation, there will have a shear stress due to elastic deformation and retraction in the anchorsoil interface, which direction on the soil deformation under shear stress is the contrary. It reduces soil internal tensile stress, and will also limit the deformation of soil. The axial force of soil-nailing is related with the deformation of the earth. Because stress reduces the soil deformation, soil-nailing internal force is reduced more remarkably than plain soil-nailing .The closer the anchor approaches the soil nails, the more the axial force decreases. Therefore, the role of restrictions pit deformation is the base of anchor and soil-nailing working together.

# 3.2 Anchor contribution to resistance moment

When the Pit Slope in the event of damage, the slip surface have too much plastic deformation to make slide mass along for destruction under sliding. Generally, the sliding moment entirely depends on the depth of excavation and the soil gravity. For a certain pit, the soil depth and its weight usually are constant and its sliding moment can be seen as a constant. Meanwhile resistance moment is provided by the undisturbed soil, shear strength, soil nails and anchor.

The contribution of soil nails performance lies in three main aspects: soil-nailing presence gives the slip surface place to the post-transfer slip, improves the sliding area and increases the friction of slip surface. Uplift role of the soil-nailing that outside the slip surface, and the bending resistance role of soil-nailing.

The contribution of anchor main features (Chang, 2007):

- 1 The anchor's anchorage is long in general and extends into the steady soil mass in central slip away from the excavation surface to provide a stronger uplift capacity.
- 2 The anchor's prestressing makes slide and stability soil mass tightly squeezed each other to improve the friction resistance to sliding and increase the resistance moment.

3 When the slip surface crosses the anchorage segment, the anchor resistance to bending has some contribution, but the contribution is weak in general.

# 3.3 The impact of prestressing to soil-nailing axis force

There are many studies about the impact of prestressing to soil-nailing axis force and the conclusions are the same. Generally, the prestressing will significantly reduce soil-nailing forces and the greater the value of prestressing, the smaller the soil-nailing internal force. And moreover, the closer the anchor, the greater the internal force reduction (Zhang & Liu, 2002, Zhen et al. 2005).

# 4 DESIGN OPTIMIZATION

- Soil-nail design. Recommending taking the soilnailing's long-short layout and it is advisable to select 10–15 degrees.
- 2. Loess has strong structure, which should be used as much as possible.
- 3. Anchor should be installed close to the central vertical part of the pit, which can achieve a higher safety factor and restrict the pit deformation.
- 4. It does not provide a greater safety factor and better control deformation even the anchor length is too long. So it had better be about 8–12 m.
- 5. Since the anchor prestressed reaction limits soil lateral deformation, reduces the lateral displacement of composite soil-nailing retaining and axial force of the-soil-nailing near the anchor. On the premise of meeting deformation control request, we can shorten the length that several soil-nailing top of the pit appropriately in long-short scheme, long-short-long scheme while forming to reduce project cost. However, in order not to reduce safety factor, it is not recommended to shorten the length of soil nails in the lower side.
- 6. The fixing on the prestressed value must be according to the soil shear strength values, it will be about 100 kN to 200 kN as well. Due to the prestressed value has no obvious influence on project cost, we can incline safety to choose the values greater. But too much prestressed has no significant impact on pit retaining performance.

# 5 ENGINEERING ANALYSIS

# 5.1 Project overview

A project in Xi'an, the excavation depth is 11.0 m, both the Pit's length and width are about 100 m. In the east

Table 6. Site layer structure and geotechnical characteristics.

Soil class	Thickness/ (m)	Unit weight (kN/m3)	Cohesion/ kPa	Angle of internal friction/°
Miscellaneous	0.90	18.20	25.00	18.00
LoessQ3 <sup>2EOL</sup>	6.60	16.20	28.00	18.00
LoessQ3 <sup>2EO</sup> L	1.70	18.00	26.80	18.10
Ancient soilQ3lal	3.40	18.90	32.20	17.60
LoessQ3 <sup>al+PL</sup>	8.00	19.50	20.00	18.00

by north from the pit 6.3 m is a seven-storey masonry structure residential buildings, the lime soil foundation depth is about 3 m.On the south-east there is a 18-storey high-rise building with one-storey basement which depth is 6 m, reinforced concrete pile foundations are 36 m long, from Pit 9.10 m, and the adjacent side of the project pit used soil-nailing in the construction; north of the seven-storey residential building masonry structure, and roughly parallel to pit edges, buildings length is 42 m and the width 13 m, the nearest to excavation is 5.3 m; the west side of the South and an adjacent hotel podium which is a two storeys building with an underground layer from the edge of pit 4 m, framework and infrastructure end elevation is -7.13 m; the south side is close to a main road, there are water and gas pipelines under the sidewalks.

# 5.2 Engineering geological conditions

According to geotechnical engineering investigation report that the project site geomorphic units belong to the Loess beam-swamp landscape. Proposed site layer structure and geotechnical characteristics are in range of 30.0 m deep in Table 5.

# 5.3 Retaining design

The design of anchored composite soil-nailing adopts the methods proposed in part 4 of this paper, which is the excavation depth was 11.0 m and the slope was 1:0.1. The basic design parameters were shown in Table 6. There are six layers of soil nails and the layout is cinquefoil. We also set a prestressed anchor at the depth of -6.0 m to reduce the lateral displacement and ensure the high-rise buildings in safety and stability. Using two 18 mm diameter grade 60 bars in anchor (10 m free, 8 m anchorage); Using one 22 mm diameter grade 60 bar in soil-nailing with 1.5 m spacing and inclination of 15 degrees, prestressing value is 150 kN. (Fig. 12).

# 5.4 Monitoring results

This project was constructed since March 20, 2006 and lasted 75 days. Monitoring results showed the greatest

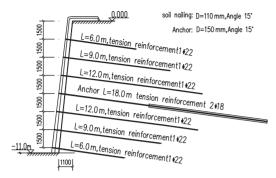


Figure 12. Composite soil-nailing support diagram.

lateral displacement occurred in the east central pit and the largest displacement was 16 mm. It was no excessive lateral deformation and earth surface subsidence and surrounding buildings were no greater settlement.

# 6 CONCLUSIONS

In this paper, the author analyzed the parameter sensitivity of composite soil-nailing in loess excavation using the finite element method. The optimization design methods are introduced based on the results of finite element analysis. The writer believes that anchor should be installed in the pit in the central vertical, which can achieve a higher safety factor and restrict the pit deformation obviously.

We can shorten the length that several soil-nailing top of the pit appropriately in long-short scheme, midlong scheme while forming to reduce project cost. Anchor length should not be too long and prestressing also should not be too large. Practical project proved that this optimization method in loess pit is applicable.

# ACKNOWLEDGMENT

This paper was supported by Youth Foundation of Anhui Education Committee (No. 2007jql181), and supported by Youth Foundation of West Anhui University (No. wxzq2006018).

#### REFERENCES

- Chang, G.M. 2007. Study on the Application of Composite soil-nailing in Loess Excavations. Xi'An: Chang'An University
- Chen, Z.Y. 2000. The application of soil-nailing in excavations. Beijing: China Architecture & Building Press
- Hu, K.G. & Song, Q.G. 1997. Nonlinear analysis of action mechanism of soil-nailing wall. *Industrial Construction* 27(11): 10–13
- Li, S.H. & Zhang, Y.J. 1999. Numerical simulations by 2D FEM in process of excavation and supporting of deep foundation ditch. *Chinese Journal of Rock Mechanics and Engineering* 18(3):342–345
- Wang, B.Y. 1997. Design of soil-nailing. Geotechnical Engineering Technique (4): 30–41
- Zhang, F, Liu, Z.C. & Chen, G.G. 2002. The mechanical working mechanism research on the united supporting of prestressed soil anchor and soil-nailing. *Rock and Soil Mechanics* 23(3): 292–296
- Zheng, Z.H., et al. 2005. In-situ testing study on retaining miscellaneous fill slope by using compound soil-nailing. *Chinese Journal of Rock Mechanics and Engineering* 24(5): 898–904

# Three-dimensional finite element analysis of diaphragm walls for top-down construction

# J. Hsi, H. Zhang & T. Kokubun

SMEC Australia Pty Ltd, Sydney, NSW, Australia

ABSTRACT: The Tugun Bypass Tunnel in Gold Coast, Australia was constructed using diaphragm walls with the top-down cut-and-cover method to allow simultaneous construction of an airport runway extension above the tunnel, whilst excavation of the tunnel continued underneath. The tunnel was built in an environment of high groundwater table and deep deposits of alluvial and estuarine soils with the toes of the walls founded in soil deposits. There was a potential risk for differential settlements between the diaphragm wall panels, caused by the runway fill placed over the tunnel roof during excavation. Three-dimensional numerical modelling was undertaken to predict the differential settlements of the tunnel with considerations of varying subsurface profile, staged excavation and dewatering, non-uniform loading and complex soil-structure interaction. Field instrumentation and monitoring was implemented to confirm numerical predictions.

# 1 INTRODUCTION

The 7 km long Tugun Bypass forms part of the Pacific Highway, and connects south-east Queensland to northern New South Wales, Australia. One of the key features of the project was a tunnel of about 334 m in length (Ch5588 to Ch5922.4), constructed below the proposed runway extension of the Gold Coast Airport. Figure 1 presents the project route plan showing the locality of the project.



Figure 1. Project route plan.

As the tunnel was to be constructed in the proximity of the airport runway, there was a strict height restriction for the construction activities. Low headroom plant and equipment were chosen to construct the diaphragm walls for the cut and cover tunnel. As the construction of the runway extension coincided with the tunnel construction, the top-down construction method was adopted.

The subsurface of the tunnel site comprised mainly alluvial and estuarine soils up to depths of about 35 m underlain by weathered rock of Neranleigh Fernvale formation. To minimize construction costs, the diaphragm walls were founded in soil deposits which were subjected to settlement under the loading from the runway extension.

Excessive differential settlement of the diaphragm walls could overstress the tunnel structure and affect the tunnel serviceability. Detailed numerical modelling was carried out using the finite element package PLAXIS 3D Foundation (Version 1.6) where the spatial subsurface variation and non-uniform loading patterns could be taken into consideration.

Instrumentation and monitoring were undertaken to demonstrate the field performance, which was then compared with the numerical predictions.

# 2 SITE GEOLOGY

The tunnel was situated in a flood plain which was subjected to periodical flooding. The geology of the site comprised Neranleigh Fernvale Beds overlain by Cenozoic estuarine and coastal deposits. These

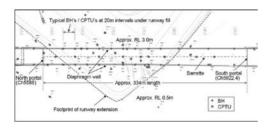


Figure 2. Site investigation plan.

deposits were up to 35 m in thickness, comprising river gravels, sands and clays, and flood plain and tidal delta muds and silts. At the tunnel location, the subsurface horizons consisted of dune sands, "Coffee Rock" (local term given to cemented silty sands), estuarine interbedded clays and sands, and residual soils derived from the weathered bedrock. Groundwater was slightly saline due to the close proximity to the ocean. The water table was influenced by both tidal movements and rainfall events recharging Cobaki Broadwater. Due to low-lying ground surfaces, potentials existed for acid sulphate soils.

# 3 GEOTECHNICAL MODEL

As the subsurface conditions varied spatially along the length and width of the tunnel, extensive site investigations using boreholes (BH) and piezocones (CPTU) were undertaken at the wall and barrette locations. Within the footprint of the runway extension, the investigations were done at a spacing of approximately 20 m intervals. The plan of the site investigation is shown in Figure 2.

The geotechnical model of the site included subsurface stratigraphy and geotechnical parameters. The subsurface was divided into discrete soil units, classified according to material type and consistency or density and is summarized as follows (top down):

- Top soil thin skinned (<1 m) comprising peaty sandy organic topsoil, having loose consistency. The ground surface was marshy and generally untrafficable;
- Dune sands a sequence of generally loose to very loose sands of up to about 8 to 10 m in thickness, fine to medium grained sands;
- "Coffee Rock" (CR) a sequence of medium dense to very dense cemented silty sands of about 7 to 10 m in thickness with occasional loose consistency;
- Estuarine a sequence of about 15 m thickness comprising shell fragments, sand and silty sand, clay and sandy clay, silt and clayey silt, clayey silty sand and gravels. Relative density varied from very loose to dense, and consistency varied from firm to very stiff;

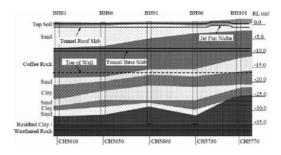


Figure 3. Subsurface profile.

Table 1. Typical soil profile and key geotechnical parameters.

RL (m)	Soil	φ' (deg)	k (m/day)	E <sup>ref</sup> <sub>50</sub> & E <sup>ref</sup> <sub>oed</sub> (MPa)	E <sup>ref</sup> (MPa)
0.5	Sand (VL)	30	1.0	10	30
-4.0	CR (MD)	32	0.1	50	150
-11.2	CR (D)	34	0.1	80	240
-13.5	Sand (L)	32	1.0	30	90
-17.5	Clay (St)	28	$1 \times 10^{4}$	10	30
-21.1	Sand (L)	32	1.0	30	90
-23.0	Clay (F)	24	$1 \times 10^{-4}$	7	21
-28.6	Clay (VSt)	29	$1 \times 10^{-4}$	25	75
-30.8	Bedrock	-	-	-	-

Note: RL (reduced level) is at top of each layer; VL is very loose; L is lose; MD is medium dense; D is dense; F is firm; St is stiff; VSt is very stiff;  $\phi'$  is drained friction angle; k is permeability;  $E_{50}^{ref}$  is secant Young's modulus at a reference pressure of 100 kPa;  $E_{oed}^{ref}$  is tangent Young's modulus for primary odometer loading at a reference pressure of 100 kPa; and  $E_{ur}^{ref}$  is unloading/reloading Young's modulus at a reference pressure of 100 kPa. Refer to PLAXIS manual for Hardening Soil (HS) model.

- Residual soil comprising clay and silty clay with some sands, and with residual fragments of extremely weathered and extremely low strength interbedded argillite and greywacke of the Neranleigh Fernvale Beds. The thickness ranged between about 1 m and 6 m;
- Bedrock comprising extremely weathered to moderately weathered and extremely low to low strength interbedded argillite and greywacke, having an irregular contact with the overlain residual material at a depth of approximately 30 to 35 m.

The subsurface profile based on the boreholes along the centre line of the tunnel is presented in Figure 3. The geotechnical parameters for each of the units were determined from interpretation of the field and laboratory test results, and based on local experience. The typical soil profile and key geotechnical parameters assumed are shown in Table 1. The ground surface level was approximately at RL 0.5 m and the groundwater table was at the surface.

# 4 ISSUES AND CONSTRAINTS

Construction of a tunnel in soft ground at shallow depths is conventionally undertaken using the "cut and cover" method. However, to allow for construction of the runway extension that occurred concurrently with the tunnel excavation, the top-down construction method had to be adopted. Diaphragm walls and cast in situ tunnel roof slabs had been chosen to facilitate the construction requirements and time constraints. Figure 2 shows the footprint of the runway extension oblique to the tunnel alignment.

Following the handover of ground surface, up to 2–3 m of fill for the airport runway extension was placed above the tunnel roof. Loads acting over the entire width of the roof slabs were transferred directly to the diaphragm walls and the barrettes. The site investigations revealed presence of estuarine deposits consisting of loose materials below the toe of the walls. Therefore, there was a potential for the tunnel to settle during excavation. One of the critical issues was the differential settlements between the walls and the central barrettes, and along the walls. These differential settlements could potentially induce significant stresses in the roof structures and in the walls.

Other issues in relation to the tunnel construction are listed below:

- "Obstacle Limitation Surface (OLS)" applied at both ends of the runway to provide safe airspace for approaching aircrafts. This required all construction activities to be undertaken within a headroom of as low as 8 m. Use of cranes or heavy-lifting equipment was only allowed outside the airport operating hours;
- High groundwater level due to its close proximity to the sea and Cobaki Broadwater. The groundwater was practically at the ground surface level. Reliable dewatering system was essential during excavation;
- Environmental requirements strict environmental controls were enforced such that drawdown of the groundwater table outside the diaphragm walls was insignificant. Also, all acidic sulphate soils excavated from the tunnel had to be dried and neutralized with lime prior to placement as fill in embankments.

# 5 CONSTRUCTION METHOD

Suitable construction methods were chosen to address the issues and constraints mentioned above. In order to adhere to the OLS requirements, special low headroom hydraulic grab (Leibherr HS852HD) and 2.8 m wide trench cutter (CBC25) were used. The guide walls were

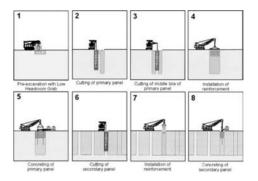


Figure 4. Diaphragm wall construction sequence (courtesy of Bauer/Piling Contractors).

built first followed by construction of the 6 m wide primary panels (Steps 1 to 5 of Figure 4) and 2.8 m wide secondary panels (Steps 6 to 8). The open trench was supported by mixture of bentonite slurry, when the cutter undertook full excavation (Steps 2 to 3 and 6). A steel reinforcement cage was lowered when the panel was excavated to full depth (Steps 4 and 7). Concreting of the panels was then achieved by the tremie method (Steps 5 and 8). Figure 4 presents the construction sequence of the diaphragm wall.

Following completion of the diaphragm walls and barrettes, dewatering and excavation commenced inside the walls. Excavation was initially undertaken to depths of up to about RL -2 m to allow for construction of the roof slab. Water-tight membrane was installed as part of the water-proofing system. When the roof slab was completed, it was backfilled and the site was cleared for handover to the Gold Coast Airport. These activities commenced in April 2006 after environmental approvals were granted, and were completed by November 2006 which was the scheduled date of handover of the site surface. Excavation below the runway extension continued through to January 2007, and the remaining construction of the tunnel continued.

# 6 STRUCTURAL DETAILS

The tunnel structure consisted of diaphragm walls and barrettes located at the centre of the tunnel. The diaphragm walls were 1 m in thickness, and extended from the Northern Portal (Ch5588) to the Southern Portal (Ch5922.4). The walls were installed to the depth of RL -17 m, from the top of the guide wall at RL 2 m. The internal width between the diaphragm walls ranged from about 25.7 m at the northern portal to 28 m at the southern portal. Barrettes were 0.8 m thick and 2.8 m wide with a clear spacing of 2.8 m throughout the central axis of the tunnel, extending to RL -17 m in depth. These structures had a 100 year design life, using N-grade reinforcing

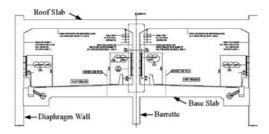


Figure 5. Typical tunnel cross section.

steels and 50 MPa high strength concrete. There were no mechanical 'joints' at the interface of the primary and secondary panels in the longitudinal direction. However, the barrettes and the diaphragm walls were rigidly connected to the 1 m thick roof slab. There were three jet fan niches where the roof slab was slightly elevated. The base slab was also 1 m thick with a founding level ranging from RL -5.5 m to RL -9.5 m. Figure 5 shows the typical cross section of the tunnel.

# 7 DESIGN CONSIDERATIONS

Geotechnical design of the tunnel was required to satisfy the following three key issues:

- Excavation support during construction the diaphragm wall structures were designed to ensure stability of the excavation. Issues included structural design of the walls, base heave, hydraulic uplift, piping, and liquefaction;
- Long term stability of the tunnel buoyancy of the tunnel when the groundwater table was close to the surface;
- Serviceability assessment due to settlement of the tunnel during construction – the tunnel was subjected to loading from airport runway fill which resulted in settlements. The influences of differential settlements on structural capacity were assessed.

# 8 NUMERICAL MODELLING

### 8.1 Two-dimensional numerical modelling

Design of the tunnel was initially undertaken using the finite element software PLAXIS (Version 8.4) at selected sections. This numerical package was used to analyze two-dimensional plane-strain problems involving complex soil-structure interaction for the design of the structural members. Structural beam elements were used to simulate the diaphragm walls. Global factor of safety during each of the construction stages was calculated based on the  $c'-\varphi'$  reduction method to ensure the minimum FoS was achieved. The software allowed modelling of construction sequence, changing groundwater levels, and varying subsurface conditions across the width of the tunnel.

#### 8.2 Three-dimensional numerical modelling

A three-dimensional numerical modelling package, PLAXIS 3D Foundation (Version 1.6), was employed to predict the settlements of the tunnel caused by runway fill loading and excavation. Due to the limitation of the program, settlement analyses were undertaken in sections, each of approximately 40 to 60 m in length. The major advantages of the 3D modelling were as follows:

- Ability to model the physical dimensions of the wall and barrette structures. This improved the accuracy of settlement prediction, as it accounted for longitudinal stiffness of the tunnel which assisted in load redistribution and toe resistance of the structures;
- Ability to simulate 3D load distribution where the runway fill was placed oblique to the longitudinal axis of the tunnel;
- Ability to model 3D subsurface profile based on probe holes at discrete locations;
- Ability to simulate dewatering within the tunnel excavation area.

The Hardening Soil (HS) model was considered most appropriate to simulate soil behaviour in an open excavation. The HS model took into account unloading and reloading behavior and irreversible plastic strains of soil. The HS stiffness parameters were defined with respect to a reference pressure of 100 kPa. The key parameters included  $E_{50}^{ref}$ ,  $E_{oed}^{ref}$ , and  $E_{ur}^{ref}$  as shown in Table 1. The published data indicate the ratio of  $E_{0ed}^{ref}$  to  $E_{50}^{ref}$  is about 0.7 to 1.4 and the ratio of  $E_{0ed}^{ref}$ , varies from 2 to 4. The analysis adopted  $E_{50}^{ref} = E_{oed}^{ref}$ , and  $E_{ur}^{ref} = 3E_{50}^{ref}$ .

Presented here is a 41.2 m long section of the tunnel between Ch5728.8 and Ch5770. This section of the tunnel was of the "deepest" location of the tunnel, beneath the thickest layer of the runway fill, and underlain by sloping bedrock level and changing clay thickness. A jet fan niche of approximately 12 m long also lied within the centre of this section which had also been incorporated in the model. Within this chainage range, there were seven boreholes. Due to the capacity of the program, four representative boreholes, which were evenly distributed spatially, were selected for the analysis. The assumed subsurface profiles are shown in Table 2.

#### 8.3 Assumptions of analysis

The construction sequence was considered in the analysis to simulate the load transfer from the runway fill to the diaphragm walls. The assumed construction sequence is described below:

1. Application of loads exerted on the virgin ground from the working platform built to RL 2 m (for construction of the guide walls) and construction load of 10 kPa;

Table 2. Subsurface profiles.

Borehole Location Chainage	#1 LHS 5730	#2 RHS 5737	#3 LHS 5757	#4 Centre 5768	Soil type
Unit	RL at to	op of eac	(density/ consistency)		
1	0.5	0.5	0.5	0.5	Sand (VL)
2	-4.8	-3.4	-5.0	-2.8	CR (MD)
3	-10.8	-14.4	-9.8	-9.7	CR (D)
4	-13.8	-15.9	-12.6	-11.7	Sand (L)
5	-17.8	-17.1	-17.5	-17.5	Clay (St)
6	-24.3	-21.8	-20.8	-17.5	Sand (L)
7	-27.3	-25.1	-21.8	-18.0	Clay (F)
8	-33.3	-30.8	-25.0	-25.2	Clay (VSt)
9	-36.3	-32.0	-27.4	-27.4	Bedrock

Note: LHS is left hand side of tunnel facing increasing chainage direction; RHS is right hand side of tunnel; and Centre is centre line of tunnel.

- 2. Installation of diaphragm walls and barrettes to RL -17 m;
- 3. Removal of the working platform, and application of 10 kPa construction load on surface;
- 4. Dewatering and excavation to underside of the roof slab;
- 5. Installation of the roof slab (and jet fan niche), and backfill to existing ground surface;
- Placement of runway fill to design heights (simulated as pressures) with 10 kPa live load above the runway;
- Staged dewatering and excavation within the diaphragm walls to underside of the base slab;
- 8. Casting of the base slab and completion of the tunnel structure;
- 9. Return of the groundwater table to the ground surface and removal of 10 kPa surface loads.

The settlement assessment was undertaken at stage 7, which was considered most critical with maximum excavation under full runway loading.

The assumed levels within the modeled chainage range are summarized in Table 3.

# 8.4 Results of analysis

The deformed mesh of the 3D finite element analysis under the full runway loading and at the final stage of the excavation is shown in Figure 6. The predicted settlement profiles at the top of the roof slab along the diaphragm walls and barrettes prior to casting of the base slab are presented in Figure 7.

The predicted settlement of the tunnel during excavation was about 45 mm on the LHS, 43 mm on the RHS, and 35 mm along the central barrettes. The maximum differential settlement was predicted to be 12 mm between the walls and the barrettes. To allow

Table 3. Assumed geometry during construction.

	5728.8 to 5737.6	5737.6 to 5743.6	5743.6 to 5755.2	5755.2 to 5761.2	5761.2 to 5770.0
Chainage range					
Feature	RL (m)				
Natural Ground Level	0.5	0.5	0.5	0.5	0.5
Top of Roof Slab	-0.8	-0.25	+0.4	-0.25	-0.8
Bottom of Roof Slab	-1.8	-1.25	-0.6	-1.25	-1.8
Initial Excavation	-2.8	-2.8	-2.8	-2.8	-2.8
Initial Dewatering	-3.8	-3.8	-3.8	-3.8	-3.8
Intermediate Excavation	-6.0	-6.0	-6.0	-6.0	-6.0
Intermediate Dewatering	-7.0	-7.0	-7.0	-7.0	-7.0
Top of Base Slab	-8.4	-8.4	-8.4	-8.4	-8.4
Bottom of Base Slab	-9.4	-9.4	-9.4	-9.4	-9.4
Final Excavation	-9.7	-9.7	-9.7	-9.7	-9.7
Final Dewatering	-11.7	-11.7	-11.7	-11.7	-11.7
Toe of Diaphragm Wall	-17.0	-17.0	-17.0	-17.0	-17.0

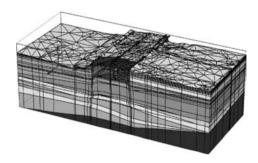


Figure 6. Deformed 3D finite element mesh.

for uncertainties, the tunnel was designed for a maximum differential settlement of 25 mm. The structural analysis showed that the longitudinal in-plane stiffness of the tunnel would smooth out differential settlements along the tunnel alignment, with the presence of the jet fan niche and variability of the subsurface conditions.

# 9 FIELD PERFORMANCE

The performance of tunnel during construction was assessed based on the field monitoring results. This was a means to confirm that the structural integrity of the diaphragm walls and barrettes were not adversely affected by differential settlements. Three instrumentation arrays were set up at Ch5655, Ch5718, and Ch5770 corresponding to locations of the runway fill (see Figure 8).

Each array consisted of three settlement plates placed above the LHS and RHS diaphragm walls

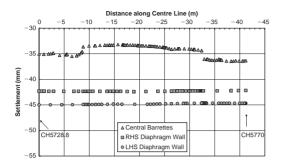


Figure 7. Predicted settlement profiles at top of roof.

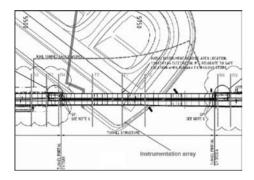


Figure 8. Plan of instrumentation arrays.

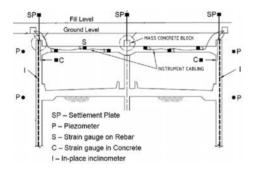


Figure 9. Typical instrumentation section.

and the central barrettes (see Figure 9). These were installed prior to runway fill placement and excavation of the tunnel in order to capture all construction induced movements. In addition to the settlement plates, survey targets were also installed at inner walls to the tunnel to record tunnel movement during excavation. This information had to be calibrated against the settlement plate measurements as the initial tunnel movement record was not available.

Figure 10 shows a summary of construction activities, recorded settlements, and the predicted settlements at diaphragm wall and barrette locations at Ch5718. The settlement prediction adopted here is

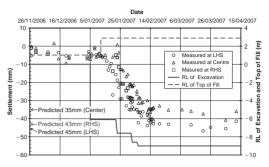


Figure 10. Settlement monitoring results at Ch5718.

the result of analysis between Ch5728.8 and Ch5770. Monitoring commenced at the beginning of November 2006. Excavation of the tunnel commenced in mid December 2006 from the Northern Portal at Ch5588. The excavation process reached Ch5718 in early January. Placement of runway fill above CH5718 followed in mid January, which had resulted in visible settlements of the tunnel. The settlements appeared to have ceased after the excavation reached final depth in mid February. The monitoring data showed that the field performance of the tunnel was consistent with the predictions obtained from the PLAXIS 3D Foundation modelling. Maximum differential settlements between the barrettes and the diaphragm walls were less than 25 mm at all stages of construction.

# 10 CONCLUSIONS

The Tugun Bypass tunnel had to be constructed under many strict constraints in a challenging geotechnical environment. Extensive site investigations were undertaken to better characterize ground conditions and reduce risks of geotechnical uncertainties. The top-down construction method was adopted to allow extension of the airport runway to occur simultaneously during tunnel construction. The additional loads from the runway fill induced settlements of the tunnel during construction. Settlement analysis of the tunnel using 3D numerical modelling techniques had been undertaken. The differential settlements of the tunnel were successfully predicted. The performance of the tunnel was monitored during construction and the field measurements were consistent with the numerical predictions.

# REFERENCES

PLAXIS 2D, Version 8.4, PLAXIS BV Netherlands, 2006.PLAXIS 3D Foundation, Version 1.6, PLAXIS BV Netherlands, 2006.

# Numerical evaluation of dewatering effect on deep excavation in soft clay

# L. Li

Tianjin Institute of Ubran Construction, Tianjin, P.R. China Tianjin Key Laboratory of Soft Soil Characteristics and Engineering Environment, Tianjin, P.R. China

# M. Yang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Tongji University, Shanghai city, P.R. China

ABSTRACT: This paper describes the application of FLAC3D analysis for modeling a top-down construction of a four-story (33.7 m depth), underground transformer substation in downtown of Shanghai city. There is unconfined aquifer and confined aquifer on the site of this project and drainage by desiccation in the foundation pit is adopted. The effective stress methods of analysis incorporate excavation and dewatering of the foundation pit for real-time simulation of construction activities. The results not considering dewatering are compared with the result considering dewatering, including wall deflections, basal heave, surface settlement. The analysis of considering leakage of the wall and leakage between confined aquifer is provided also. The analysis shows that although the difference is small in soft clay due to the low permeability of the soil, dewatering enhance the deformation of the foundation pit and the foundation pit is inclined to be not security if dewatering is not considered, the effect of leakage of the wall can be obvious on the surface settlement.

# 1 INTRODUCTION

For deep excavation in congested urban environments, designers are particularly interested in making reliable prediction of the magnitudes of movements in the surrounding soil (Peck 1969, Clough et al. 1989, O'Rourke 1981) and then estimating the effects of these movements on adjacent structures and facilities (Burland & Wroth 1974, Boscardin & Cording 1989). In principle these prediction can be achieved using powerful numerical methods such as finite element analyses, but there is difficulties in achieving reliable analytical predictions of soil deformations which can be attributed to a variety of factors including dewatering.

Dewatering is necessary in the excavation under the ground water level in soft clay, which provide a dry environment for the excavation and is benefit for the slope stability and reducing the harm induced by the groundwater, but dewatering have important effect on the behaviour of the foundation pit and the surrounding soils, seepage induced by dewatering from outside to the inside of the foundation pit have effect on the stability and the deformation of the foundation pit and surface settlement for the vertical consolidation by the underground-water drawdown out of the foundation pit, so effective stress methods of analysis distinct from vast majority of analyses relies on total stress methods of analysis and real-time simulation of coupling between ground-water flow (pore pressure)and soil deformations is adopted in this numerical analysis considering dewatering. In this paper, considering dewatering means considering the difference between in and out of the foundation pit in the simulating, and not considering dewatering means not considering the difference.

This paper describes the application of FLAC3D program for predicting soil deformations and ground water flow associated with the top-down construction of a columniform underground transformer substation in downtown of Shanghai city. The model incorporates a number of advanced features of analysis including: (1) A FLAC3D model which can consider interaction between support system of the foundation pit and the surrounding soil; (2) Dewatering and excavation are all considered in the numerical analysis at the same time; (3) The drawdown of the ground water and the surface settlement due to dewatering in the foundation pit; (4) The surface settlement induced by the leakage of the diaphragm wall. Site stratigraphy, material properties, and initial ground-water condition are all selected using information provided prior to construction, the simulation of the construction sequence is based on the scheme.

$\begin{array}{c} \hline -2.0 \text{m} \\ \hline -3.2 \text{m} \\ \hline -3.2 \text{m} \\ \hline -10.5 \text{m} \\ \hline -10.5 \text{m} \\ \hline -21.3 \text{m} \\ \hline -21.3 \text{m} \\ \hline -26.5 \text{m} \\ \hline -31.6 \text{m} \\ \hline \hline -37.5 \text{m} \\ \hline \hline -45.3 \text{m} \end{array}$	(1) (3) (4) $(5)_{1-1}$ $(6)_{1-2}$ $(6)_{1}$ $(7)_{1}$ $(7)_{2}$	Roof slab Temporary bracing Floor slab Floor slab Temporary bracing Floor slab Temporary bracing Floor slab Temporary bracing Temporary bracing Temporary bracing	$ \begin{array}{c} \hline \nabla -2.0m \\ \hline \hline \nabla -7.0m \\ \hline \hline \nabla -11.5m \\ \hline \hline \nabla -16.5m \\ \hline \hline \nabla -22.0m \\ \hline \hline \nabla -26.5m \\ \hline \hline \nabla -31.0m \\ \hline \hline \hline \nabla -33.7m \\ \end{array} $
▽ -60.3m	(8) <sub>1</sub>	- Z	
▽ -73.4m	(8) <sub>2</sub>		
√ -77.4m	83		
<u> </u>	(9) <sub>1</sub>		
<u> </u>	(9) <sub>2</sub>		

Figure 1. Soil profile and the location of the wall and bracing.

# 2 GENERAL SITUATION OF THE PROJECT

# 2.1 Project description

The columniform underground transformer substation of a four floor and 33.7 m high underground structure occupies a plan area of 13000 square meter (interior diameter is 130 m) in the downtown of Shanghai city and is bounded by buildings and viaduct and lots of underground pipeline.

The underground transformer substation design uses a cast in situ, reinforced concrete, diaphragm wall (1.2 m thick) extending down into the elevation -57.5 m (With respect to the Shanghai City Base datum), the circular wall is braced internally by the floor slabs and 3 temporary annular bracing (Figure 1), which are in turn supported by the interior columns (steel and reinforced concrete & angel iron lattice) founded on the bearing pile (bored filling pile, the pile tip at depths  $-80 \sim -90$  m below ground level. Both the diaphragm wall and interior columns are installed prior to excavation using slurry trench methods. The roof and four floor levels are cast in sequence from the top-down by excavating the soil from beneath the most recently constructed slab. During excavation, dewatering is accomplished using drainage by desiccation.

# 2.2 Engineering geological and groundwater conditions

Table 1 shows an "averaged" profile of subsurface stratigraphy interpreted from borings conducted at the site. It should be noted that the borings logs actually

Table 1. Input parameters in Mohr-coulomb model.

	Е	$\mu$	С	$\varphi$	D
Stratum	MPa		kPa	(°)	(°)
1	1.2	0.36	0.32	22.5	0
2	9.3	0.36	25.9	17.4	0
3	8.9	0.37	5.1	21.2	0
4	8.2	0.39	8.0	19.7	0
<b>5</b> <sub>1</sub>	11.4	0.38	13.4	15.7	0
<b>5</b> <sub>2</sub>	14.9	0.37	27.1	15.8	0
6	26.8	0.34	42.7	13.7	0
$\overline{\mathcal{O}}_1$	44.9	0.32	5.0	31.2	0
$\overline{\mathcal{O}}_2$	80.2	0.31	0.0	33.0	0
8 1	16.4	0.34	21.2	23.1	0
8 <sup>2</sup>	28.3	0.33	16.9	24.1	0
83	72.2	0.33	22.1	20.3	0
<b>9</b> <sub>1</sub>	99.2	0.31	0.0	35.0	5
<b>9</b> <sub>2</sub>	133.4	0.29	0.0	37.0	7

Table 2. Porosity and coefficient of permeability.

Stratum	Soil name	Porosity	Coefficient of permeability		
			Vertical m/s	Horizontal m/s	
1	Artifical soil	0.56	_	_	
2	Silty clay	0.49	$2.5  imes 10^{-9}$	$5.5  imes 10^{-7}$	
3	Mucky silt	0.57	$1.7 \times 10^{-8}$	$3.5 \times 10^{-6}$	
4	Mucky silt	0.58	$7.2 \times 10^{-9}$	$8.1 \times 10^{-8}$	
<b>5</b> <sub>1-1</sub>	Clay	0.52	$3.9 \times 10^{-9}$	$4.1  imes 10^{-8}$	
$(5)_{1-2}$	Silty clay	0.51	$4.0 \times 10^{-8}$	$3.2 \times 10^{-8}$	
<b>6</b> <sub>1</sub>	Silty clay	0.43	$5.8 \times 10^{-9}$	$4.1 \times 10^{-8}$	
$\overline{\mathcal{O}}_1$	Silty silt	0.46	$2.6 \times 10^{-6}$	$2.6 \times 10^{-5}$	
$\overline{\mathcal{O}}_{2}$	Silt sand	0.44	$5.4  imes 10^{-6}$	$3.8 \times 10^{-5}$	
<b>8</b> <sub>1</sub>	Silt clay	0.51	$8.2 \times 10^{-9}$	$4.0 \times 10^{-6}$	
<b>8</b> <sub>2</sub>	Silt sand	0.50	$6.2 \times 10^{-8}$	$2.8 \times 10^{-6}$	
<b>8</b> <sub>3</sub>	Silt sand	0.47	$1.7 \times 10^{-6}$	$3.0  imes 10^{-4}$	
9 <sub>1</sub>	Medium sand	0.37	$5.7  imes 10^{-6}$	$3.0  imes 10^{-4}$	
$9_2$	Coarse sand	0.35	$7.9  imes 10^{-6}$	$3.0 \times 10^{-4}$	

show significant variations in the thickness of the individual strata across the site, The assumption of an average profile is consistent with the limited area and uncertainties in engineering properties of individual strata. The material parameter of the layers is in table 1.

There is unconfined aquifer and confined aquifer in the site. The ground water level of the unconfined aquifer is  $1 \sim 2$  m under the ground and the 6th layer is relative impervious layer. The confined aquifer is divided into the 1st confined aquifer and the 2nd confined aquifer by the  $\gamma_1$   $\circledast_2$  layer. The 1st confined aquifer lies in the  $\mathfrak{D}_1$  and  $\mathfrak{D}_2$  layer and the 2nd confined aquifer lie in the  $\circledast_3$  and  $\circledast$  layer, there may be some relationship between the 1st and 2nd confined aquifer. The porosity and the coefficient of permeability of the layers is in table 2.

# 2.3 FLAC3D model description

The three dimensional numerical analysis program FLAC3D is developed by Itasca Consulting Group, Inc. The groundwater flow model may be coupled to the stress model. The finite element model extends far beneath the excavation (to 100 m depth) and laterally a distance of 200 m beyond the perimeter wall where soil displacements, due to the simulation of underground transformer substation construction, are negligible. Constitutive modeling of soil behavior and selection of input parameters represent a major source of uncertainty in finite element analysis. The soil constitutive model use Mohr-coulomb failure criterion and in order to model realistically the depth variations in properties the elastic shear and bulk modulus are assumed to be proportional to the mean effective confining stress. The unloading modulus of the soil in the foundation pit is trinal loading's and the modulus of the soil under the ultimate base of excavation adopt mixed modulus because of tension pile. The parameters of the soil are shown in table 1.

The soil and diaphragm wall adopt 8 nodes solid element and the permanent floor slab adopted shell element and the temporary annular bracing adopted beam element, and interface is adopted in the joint of the diaphragm and soil. Elastic model is used for the diaphragm wall and the young's modulus is  $2.3 \times 10^4$  MPa, and the Poisson's ratio is 0.167. Elastic model is also used for floor slab and temporary annular bracing and the parameter of them is consistent with the scheme. The FLAC3D model of this project is in Figure 2.

Boundaries condition is summarized as follows: (1) The underside displacement of the model is zero, the horizontal displacement of the side of the model is zero, the upper surface is free; (2) The underside of the model is impervious boundary, the side of the model is pervious boundary and the pore pressure is fixed, the pore pressure of the upper surface is fixed as zero in the formation of the initial stress field and hydrostatic pressure and is free during dewatering and excavation. Dewatering in the foundation pit is simulated by controlling the saturation and pore pressure at specific locations in the element model.

# 2.4 Construction sequence

Based on the actual record of site activities and the sequence of events occurs in the finite element simulation, this process is simulated by 17 stages. Each "stage" in the analysis represents a distinct change in either the geometry, boundary conditions or time elapsed between events. The first stage is the formation of the initial stress field and hydrostatic pressure and after every dewatering and excavation (including adding bracing) is a stage. The numerical simulation assumes that the construction of the diaphragm wall



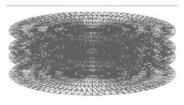
(a) 1/4 model (prior to excavation)



(b) Diaphragm wall (solid element)

1		-	LILL
	AW NN		

(c) Column and bearing pile and temporary bracing



(d) Floor slab (shell element)

Figure 2. FLAC3D model of the project.

has no effect on the surrounding soil (i.e., the wall is "wished-in place") and does not consider the installation of load-bearing elements used to support the internal column.

# 3 RESULT AND ANALYSIS

# 3.1 Analysis of the seepage field

Due to unconfined aquifer and confined aquifer existing in the site, there is difficulty in simulating them at the same time. The confined aquifer is not taken into

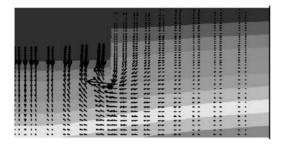


Figure 3. The neural pressure contour and flow vector.

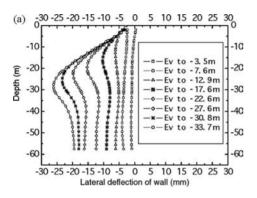


Figure 4(a). Lateral deflection of wall.

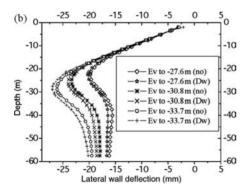


Figure 4(b). Contrast of the wall deflection.

account when analyzing the seepage of the unconfined aquifer, which have no influence on the phreatic line. The neutral pressure contour and the flow vector when excavating to -33.7 m is as Figure 3. It shows that the drawdown of the phreatic water is small and the depression cone is not obvious due to the low permeability of the soft clay.

# 3.2 Analysis of the lateral deflection of wall

Figure 4 (a) is the computing result of the lateral deflections of the diaphragm wall along the depth at every

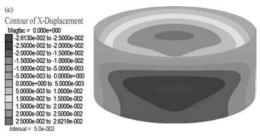


Figure 4(c). Contour of the lateral deflection.

excavation stage. The maximum lateral deflection of the wall is 23.3 mm, the ratio of the maximum lateral deflection to the end excavation depth is 0.07%, the top-down and the high rigidity of the slab is the reason for the small ratio. The location of the lateral deflection is a little higher than the excavation face and falling with the excavating and the location of the maximum lateral deflection is at -28.1 m. Figure 4(b) is the contrast of the lateral deflection considering dewatering and not in the last 3 excavation stage. The maximum lateral deflection considering dewatering is 27.7 mm and higher than the result not considering dewatering. The difference mostly happen in the middle and underside wall and is increasing with the excavation depth for the seepage force is concentrated in the middle and underside wall and the hydraulic head is increasing with the excavation depth. Figure 4(c) is the contour of lateral displacement of the diaphragm wall.

### 3.3 Analysis of the basal heave formation

The basal heave is including the elastic rebounding and the local plastic failure and the deep-seated plastic failure. The elastic rebound is because of the unloading, the local plastic failure is that the soil near the wall yield, and the deep-seated plastic failure is that the soil in the bottom is short of bearing power. When the excavation depth is little, the elastic rebound is the most, the maximum heave lies in the center, with the excavation depth increasing the heave in the circumference preponderate over the heave in the center because of the failure in the circumference, Figure 5(a) is the curve of the basal heave, Figure 5(b) is the contour of the basal heave when excavating to -33.7 m. Figure 5(c) is the contrast of the basal heave result considering dewatering and not in the last two excavation stages, it shows the basal heave considering dewatering is high than the not and the difference is increasing with the excavation depth .

# 3.4 Analysis of the surface settlement

Figure 6(a) shows the surface settlement after the every excavation stage. The surface settlement and

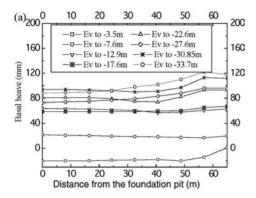


Figure 5(a). The deformation of the basal heave.

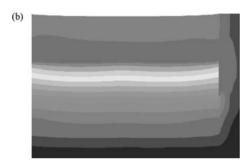


Figure 5(b). Contour of the basal heave.

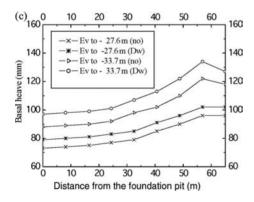


Figure 5(c). Contrast of the deformation of the basal heave.

the distance between the maximum surface settlement and the foundation pit is increasing with the excavation depth. The maximum settlement is 15.7 mm and the maximum distance is 44 m when excavating to -33.7 m depth. The diaphragm wall move up and raise the soil near the foundation pit because of the unloading, which has a effect on the location of the maximum surface settlement.

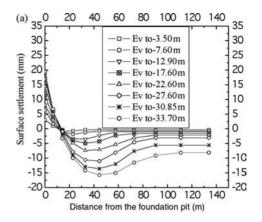


Figure 6(a). Surface settlement at stages.



Figure 6(b). Contour of the surface settlement.

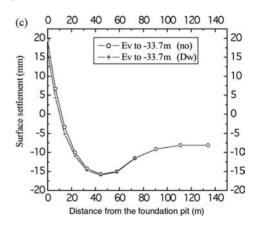


Figure 6(c). Contrast of the surface settlement.

Figure 6(b) is the contour of the surface settlement. Figure 6(c) shows the contrast of the surface settlement considering dewatering and not. It shows that the magnitude and range of the surface settlement induced by the dewatering is small for the reason of the low permeability of the soil and the small descent of groundwater level induced by dewatering. The location of maximum surface settlement induced by dewatering is closer to the foundation pit than by excavation and the

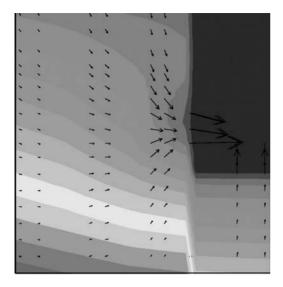


Figure 7. Leakage in the 6th layer.

maximum surface settlement considering dewatering is 15.9 mm.

# 3.5 Analysis of leakage

By the excavation experience in Shanghai city, the leakage of diaphragm wall often occur when the depth beyond 28 m and the probability increase with the excavation depth. The main cause is that the bad joint of the wall and the dimension error in construction and the distortion of the diaphragm wall. The leakage will induce the decline of the ground water level and the additional surface settlement, so the effect of the leakage have to be taken into account.

If the leakage occurs in the sixth layer, Figure. 7 shows the neutral pressure and the flow velocity vector, by the contour of the neutral pressure, only the groundwater level near the foundation pit descend little, so the leakage in the 6th layer have little effect on the settlement around the foundation pit because of the low permeability and small discharge.

If the leakage occurs in the 1st confined aquifer which of the water pressure is  $5 \sim 6$  m under the ground, the water pressure of the 1st confined aquifer descend clearly and the surface settlement is distinct for the permeability of the 1st confined aquifer is high and the discharge is much.

The 2nd confined aquifer can leak to interior of the 1st confined aquifer for the fall of the neutral pressure in the 1st confined aquifer due to dewatering and excavating in  $\overline{\mathbb{O}}_1$  layer. The numerical analysis shows leakage occurs, but the discharge is little, so the leakage have little effect on the settlement for permeability of the of the  $\circledast_1$  gray clay is low (8.21 × 10<sup>-9</sup> m/s) and the thickness of  $\circledast_1$  layer reach to 15 m.

# 4 CONCLUSIONS

This paper introduce the FLAC3D model for simulating the top-down construction of an underground transformer substation at Shanghai city. Results considering dewatering are compared with the result not considering including wall deflection, basal heave, and surface settlement, the effect of leakage is also analyzed. The main conclusions of this study are as follows:

- Dewatering by desiccation in the foundation pit have effect on the behavior of the foundation pit, the seepage could enhance the wall deflection and the deformation of the basal heave, the drawdown of the groundwater level outside of the foundation pit could result in the vertical consolidation and add the surface settlement. The foundation pit is inclined to be not security if dewatering is not considered.
- The wall deflection and basal heave due to the seepage is little because of the low permeability of the soil in soft clay. The drawdown of the groundwater level due to dewatering is little, which result in little surface settlement for the same reason above.
- The leakage of the wall have important effect on the surface settlement, which is a problem needed to solve.

# REFERENCES

- Andrew J. Whittle, Youssef M. A. Hashash & Robert V. Whitman. 1993. Analysis of deep excavation in boston, *Journal of Geotechnical Engineering*, 119(1): 69–90.
- Chang-Yu, Ou, Tzong-Shiann Wu & Hsii-Sheng Hsieh, 1996. Analysis of deep excavation with column tye of ground improvement in soft clay. *Journal of Geotechnical Engineering* 122(9): 709–716.
- Itasca Consulting Group, Inc. 2002.6. FLAC3D (Fast Lagrangian Analysis of Continua in 3D Dimensions) User Manuals, Version 2.1. Minneapolis, Minnesota.
- Itasca Consulting Group, Inc.2005.5. FLAC(Fast Lagrangian Analysis of Continua )User Manuals, Version 5.0. Minneapolis, Minnesota.
- Jacob Bear, 1983. *Dynamics of Fluids in Porous Media*. Beijing: China Architecture & Building Press.
- Lin Li, 2007. Studies on the behavior of deep excavation and surroundings due to dewatering effect, Ph.D, Thesis, University of Tongji, Shanghai, China.
- Lin Li & MinYang, 2007. The analysis of deformation characteristics of the deep excavation in soft clay. *China Civil Engineering Journal*, 40(4): 66–72.
- Sunil S. Kishnani & Ronaldo I.Borja, 1993. Seepage and soilstructure interaction effects in braced excavtion. *Journal* of Geotechnical Engineering, 119(5): 912–927.
- Youssef M.A. Hashash & Andrew J Whittle, 1996. Ground movement prediction for deep excavations in soft clay. *Journal of Geotechnical Engineering* 122(6): 474–486.
- Yuqi Li, 2005. Studies on the behavior of foundation pit with excavation considering seepage, Ph.D, Thesis ,University of Zhejiang, Hangzhou, China.

# Analysis of the factors influencing foundation pit deformations

Y.Q. Li

Department of Civil Engineering, Shanghai University, Shanghai, P.R. China

K.H. Xie

Institute of Geotechnical Engineering, Zhejiang University, Hangzhou, P.R. China

J. Zhou & X.L. Kong

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: Due to the complexity of excavation and groundwater seepage, the behavior of foundation pits is not yet well understood. In this paper, based on three-dimensional (3D) Biot's consolidation theory and nonlinear Duncan-Chang's model, finite element equations considering the coupling of groundwater seepage and soil skeleton deformation during excavation are deduced and a corresponding three-dimensional finite element program is developed. Using the program, the influence of soil permeability, rigidity and tiers of supports, rigidity of retaining wall and construction period of excavation on ground surface settlement, wall horizontal displacement and pit base heave are analyzed in detail. Some useful conclusions are drawn by analyzing the influence of these factors on the excavation deformations, which are very significant for guiding design and construction of excavations.

# 1 INTRODUCTION

In urban areas, more and more underground space is utilized with the fast development of city construction, and thus a lot of excavation engineering appears. However, the pit deformations induced by excavation greatly influence the safety of not only the pit itself but also the buildings and municipal facilities around it. Therefore, study of the behavior of foundation pits has received much attention. Whittle et al. (1993) described the application of a finite element analysis for modelling the top-down construction of a seven-storey, underground parking garage at Post Office Square in Boston. The results demonstrated that reliable and consistent predictions of soil deformations and groundwater flow can be achieved by advanced methods of analysis without recourse to parametric iteration, but emphasized the need for adequate characterization of engineering properties for the entire soil profile. Vaziri (1996) described a simple, efficient and practical numerical model for analysis of cantilevered and strutted flexible retaining walls. The model had incorporated a variety of features that affected the performance of the retaining walls in the field such as installation and removal of struts, application of surcharge, changes in groundwater table, changes in soil properties and simulation of staged excavations. The model can be used effectively to perform a broad suite of parametric studies in the design stage and also as a reliable tool for predicting performance. Ou et al. (1996) further proposed a nonlinear, 3D finite element technique for deep excavation analysis. The technique as well as the analytical procedures for modeling the excavation processes were coded into a computer program, and the accuracy of the program was assessed. The case of an irregularly-shaped excavation with field measurements of wall deflection was studied and the results showed close agreement with field measurements. Zdravkovic et al. (2005) studied the effect of excavation on the surrounding areas and provided a detailed assessment of wall and ground movements.

There have been a few studies on the influencing factors of foundation pit deformations. In this paper, 3D consolidation finite element equations are derived, and the corresponding finite element program is developed. Some useful conclusions are drawn by analyzing the influence of factors such as soil permeability, rigidity and tiers of supports, rigidity of retaining wall and construction period of excavation on the pit deformations, which are beneficial to optimisation of excavation design.

# 2 FINITE ELEMENT EQUATIONS

Based on Biot's 3D consolidation finite element equations (Xie & Zhou 2002), and considering groundwater seepage induced by the water head difference between the inside and outside of a pit, the finite element equations of excavation are as follows:

$$\begin{bmatrix} \begin{bmatrix} \mathbf{K}_{eij} \end{bmatrix} & \begin{bmatrix} \mathbf{K}_{cij} \end{bmatrix} \\ \begin{bmatrix} \mathbf{K}_{eji} \end{bmatrix} & -\theta \Delta t \mathbf{K}_{sij} \end{bmatrix} \begin{bmatrix} \Delta u_i \\ \Delta v_i \\ \\ \Delta w_i \\ P_{i(n+1)} \end{bmatrix} = \begin{bmatrix} \Delta R'_{xi} \\ \Delta R'_{yi} \\ \\ \Delta R'_{zi} \\ \Delta R'_{pi} \end{bmatrix}$$
  
(*i*, *j* = 1, 2, ..., 8) (1)

where  $\theta$  is an integral constant;  $\Delta t$  is the time increment;  $[\mathbf{K}_{eij}]$  and  $[\mathbf{K}_{cij}]$  are respectively the submatrices of the stiffness matrix and the coupling matrix;  $K_{sij}$  is an element of seepage matrix;  $\Delta u_i$ ,  $\Delta v_i$  and  $\Delta w_i$  are the displacement increments of element node i;  $P_{i(n+1)}$  is the soil water potential of element node i at  $t = t_{n+1}$ ;  $\Delta R'_{xi} = \Delta R_{xi} + [\mathbf{K}_{cij}]P_{i(n)}$ ,  $\Delta R'_{yi} = \Delta R_{yi} + [\mathbf{K}_{cij}]P_{i(n)}$ ,  $\Delta R'_{zi} = \Delta R_{zi} + [\mathbf{K}_{cij}]P_{i(n)}$ , and  $\Delta R'_{pi} = \Delta R_{pi} - \theta \Delta t K_{sij}P_{i(n)}$ ,  $\Delta R_{xi}$ ,  $\Delta R_{yi}$  and  $\Delta R_{zi}$ are the equivalent load increments of element node i, and  $\Delta R_{pi}$  is the equivalent water runoff increment of element node i,  $P_{i(n)}$  is the soil water potential of element node i at  $t = t_n$ .

The soil water potential of a saturated soil can be expressed using the following equation when the solute potential of the soil is neglected:

$$P = p + \gamma_{\rm w} z \tag{2}$$

where the spatial coordinate z is upwards positive; P is soil water potential of saturated soil; p is the sum of the pressure potential and the load potential, i.e. the total pore water pressure; and  $\gamma_w z$  is the gravity potential.

# 3 ANALYSIS OF THE INFLUENCING FACTORS OF PIT DEFORMATIONS

In order to analyze the parametric influence on the pit deformations, a 3D consolidation finite element program is developed on the basis of the finite element equations derived. Using a numerical example given below, the main factors influencing the pit deformations such as soil permeability, rigidity and tiers of supports, rigidity of retaining wall and construction period of excavation are analyzed respectively.

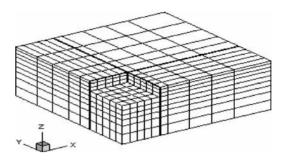


Figure 1. Mesh of finite elements.

Table 1.	Duncan-Chang	model	parameters
of soil.	-		-

Parameters	Values	
K	150	
n	0.7	
$R_{f}$	0.85	
$R_f$ c'	15 kPa	
arphi'	35°	
F	0.15	
G	0.35	
D	3.5	
Kur	300	

#### 3.1 *Reference case numerical example*

The excavated length, width and depth of the foundation pit in a certain homogenous and isotropic stratum of soft soil are 60 m, 50 m and 8 m respectively. The soil's vertical and horizontal permeability coefficients are both  $2.0 \times 10^{-6}$  cm/s and the effective unit weight of the soil is 9.0 kN/m<sup>3</sup>. The retaining wall is 0.6 m thick and embedded 16 m deep in soft soil. Reinforced concrete supports are installed at different excavation stages and the horizontal spacing between supports along the pit's long side (i.e. y-direction) and short side (i.e. x-direction) is 6 m and 5 m respectively in every tier.

In order to minimize the boundary effects and improve the computational efficiency, the calculation domains in x-, y- and z-direction are 100 m, 100 m and 40 m respectively in consideration of the symmetry about the pit centerline. The finite element mesh of the soil mass and retaining wall are shown in Figure 1.

All soil units are discretized using eight-node hexahedral isoparametric elements, modelled using the nonlinear Duncan-Chang model with parameters listed in Table 1, where c' and  $\varphi'$  are the effective cohesion and the effective friction angle of the soil respectively,  $R_f$  is the failure ratio, and K, n, F, G, D and  $K_{ur}$  are some parameters determined by tests. The retaining wall adopts Wilson non-harmony

elements, modelled as a linear elastic model, whose modulus of elasticity and Poisson's ratio are 25 GPa and 0.167 respectively. A row of 0.1 m thick interfaces, connecting the soil mass and the retaining wall is at the two sides of retaining wall, adopting 3D thin interface elements derived from Yin's rigid plastic model (Yin et al. 1995) with the outer friction angle =  $1.0^{\circ}$  and cohesion = 0.5 kPa, and its other model parameters are the same as those of the soil mass elements. The supports are modelled using a linear elastic model and spatial bar elements, with  $0.6 \text{ m} \times 0.6 \text{ m}$  cross section, whose elasticity modulus is 23 GPa.

The excavation involves three stages. The detailed description of the staged excavation of the pit is as follows:

- 1. Stage 1: 2.0 m excavation depth without supports for four days, and four days' excavation intermission for installing supports at the next excavation stage. The z value is -1.5 m for the first tier of supports and -2.0 m for the corresponding excavation level below the supports.
- 2. Stage 2: 3.0 m excavation depth (excavation to 5.0 m deep) with a tier of supports in six days, and six days' excavation intermission for installing the next tier of supports. The z value is -4.5 m for the second tier of supports and -5.0 m for the corresponding excavation level below the supports.
- 3. Stage 3: 3.0 m excavation depth (full excavation to 8.0 m deep) with two tiers of supports in eight days, and twenty days' excavation intermission for casting the pit base concrete.

# 3.2 Influencing factors

### 3.2.1 Soil permeability

In this section, the influence of soil permeability on the pit deformations at the y=0 section after the third excavation stage is studied. The soil permeability for the reference case is  $2.0 \times 10^{-6}$  cm/s. Four more analyses are carried out for soil permeability of  $2.0 \times 10^{-5}$  cm/s,  $5.0 \times 10^{-6}$  cm/s,  $5.0 \times 10^{-7}$  cm/s and  $2.0 \times 10^{-7}$  cm/s. Figures 2–4 show the influence of soil permeability represented by permeability coefficient k on the wall horizontal displacement, ground settlement and pit base heave. With the soil permeability increasing, the vertical effective stresses outside the foundation pit also increase, but those beneath the pit base decrease, so ground settlement and pit base heave increase, which are shown in Figures 3-4. For the wall horizontal displacement, with the soil permeability increasing, the horizontal effective stresses inside and outside the pit both increase, and the wall horizontal displacement decreases as a result of the greater influence of lateral pressures acting on the wall inside the pit, which can be seen in Figure 2.

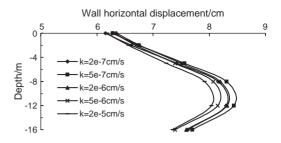


Figure 2. Influence of soil permeability on wall displacement.

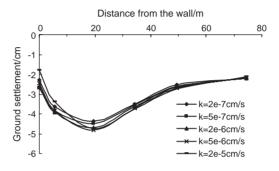


Figure 3. Influence of soil permeability on ground settlement.

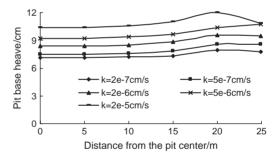


Figure 4. Influence of soil permeability on pit base heave.

#### 3.2.2 Rigidity of supports

The influence of support rigidity on wall horizontal displacement, ground settlement and pit base heave at the y = 0 section after the third excavation stage are shown in Figures 5–7. When the support rigidity becomes larger, the retaining wall movement is more restricted, so the wall horizontal displacement is smaller. However, the influence of support rigidity on ground settlement and pit base heave is relatively insignificant.

# 3.2.3 Tiers of supports

The influence of support tiers on the pit deformations at the y=0 section after the third excavation

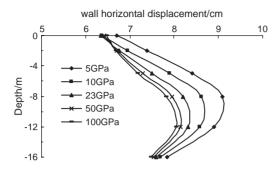


Figure 5. Influence of support rigidity on wall displacement.

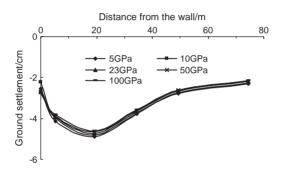


Figure 6. Influence of support rigidity on ground settlement.

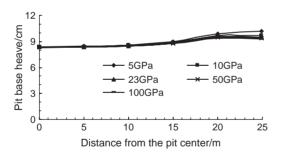


Figure 7. Influence of support rigidity on pit base heave.

stage is studied. The reference case has two tiers of support. Three more analyses are carried out: no support, one tier at 2.0 m excavation depth and one tier at 5.0 m excavation depth. Figures 8–10 show the influence of support tiers on wall horizontal displacement, ground settlement and pit base heave respectively. The deformations of the foundation pit during excavation with no support are the largest, and they evidently are smaller with adding support tiers. Tiers of support also influence the pit deformations, which for two-tier supports are less than those with one-tier. In addition,

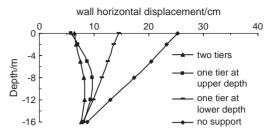


Figure 8. Influence of support tiers on wall displacement.

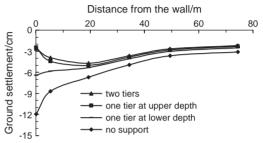


Figure 9. Influence of support tiers on ground settlement.

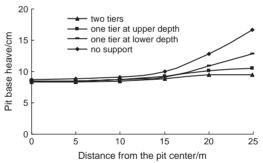


Figure 10. Influence of support tiers on pit base heave.

the position of the supports also greatly influences the pit deformations. The deformations with supports installed at a higher level are less than those with supports installed at a lower level under the same conditions, so the former approach is more effective for controlling the pit deformations.

# 3.2.4 Rigidity of retaining wall

Figures 11–13 show the influence of rigidity of the retaining wall on wall horizontal displacement, ground settlement and pit base heave at the y = 0 section after the third excavation stage. The wall horizontal displacement will obviously decrease with an increase in the rigidity of the retaining wall. However, the

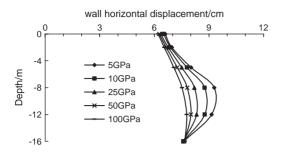


Figure 11. Influence of rigidity of retaining wall on wall displacement.

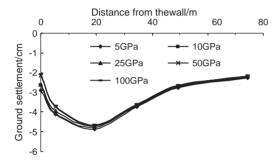


Figure 12. Influence of rigidity of retaining wall on ground settlement.

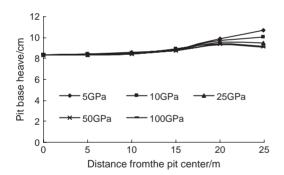


Figure 13. Influence of rigidity of retaining wall on pit base heave.

influence of rigidity of retaining wall on ground settlement and pit base heave is not significant. Therefore, increasing the rigidity of the retaining wall can effectively reduce the wall horizontal displacement and is beneficial to the safety of excavations.

### 3.2.5 Construction period

The construction period includes the excavation period and intermissions at all excavation stages, which is 48 d in the reference case. Four more analyses were carried out for construction period of 24 d, 36 d, 60 d and 72 d.

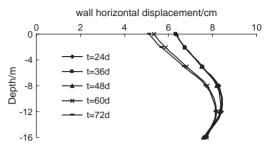


Figure 14. Influence of construction period of excavation on wall displacement.

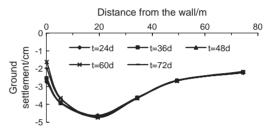


Figure 15. Influence of construction period of excavation on ground settlement.

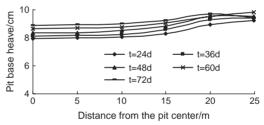


Figure 16. Influence of construction period of excavation on pit base heave.

The influence of construction period on wall horizontal displacement, ground settlement and pit base heave at the y = 0 section after the third excavation stage is shown in Figures 14–16. On the one hand, with the construction period increasing, the excess pore water pressures have a longer time to dissipate, and the soil strata can achieve a higher degree of consolidation, gaining higher strength and stiffness, thus the wall horizontal displacement decreases to a certain extent. On the other hand, the pit base heave increases with an increase in construction period. The influence of construction period on ground settlement is relatively insignificant.

# 4 CONCLUSION

Based on Biot's consolidation theory, finite element equations were deduced and a computer program was developed. The influence of the key parameters such as soil permeability, rigidity and tiers of supports, rigidity of the retaining wall, and the construction period on pit deformations is studied using the finite element program. The study and the results reported in this paper are helpful to guide excavation engineering.

# ACKNOWLEDGEMENTS

This research project was supported by the China Postdoctoral Science Foundation (No. 20060400672) and Innovation Fund of Shanghai University, China.

# REFERENCES

- Ou, C.Y., Chiou, D.C. & Wu, T.S. 1996. Three-dimensional finite element analysis of deep excavation. *Journal of Geotechnical Engineering*, ASCE 122(5):337–345.
- Vaziri, H.H. 1996. A simple numerical model for analysis of propped embedded retaining walls. *International Journal* of Solids Structures 33(16):2357–2376.
- Whittle, A.J., Hashash, Y.M.A. & Whitman, R.V. 1993. Analysis of deep excavation in Boston. *Journal of Geotechnical Engineering*, ASCE 119(1):69–90.
- Xie, K.H. & Zhou, J. 2002. *Theory and Application of Finite Element Analysis in Geotechnical Engineering*. Science Press, Beijing.
- Yin, Z.Z., Zhu, H. & Xu, G.H. 1995. A study of deformation in the interface between soil and concrete. *Computers and Geotechnics* 17:75–92.
- Zdravkovic, L., Potts, D.M. & St John, H.D. 2005. Modelling of a 3D excavation in finite element analysis. *Geotechnique* 55(7):497–513.

# Construction monitoring and numerical simulation of an excavation with SMW retaining structure

Z.H. Li & H.W. Huang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: The soil mixing wall (SMW) retaining structure is applied in two long strip excavations in Shanghai. Firstly, the bearing and deforming mechanism of SMW is analyzed in brief. The structural analysis method of SMW is discussed. Secondly, based on the in-situ excavating construction procedures, the construction steps of excavating and supporting are simulated in the numerical calculation with the method of Fast Lagrangian Analysis of Continua 3D. There are two cases simulated in numerical calculation, case 1 is the normal case in which the supports are installed timely, and case 2 is a case in which the supports are not installed timely because of some reasons. Then, the deformation of the retaining structure, the horizontal displacement at the top of SMW and the axial forces of steel pipe supports are analyzed and compared with the actual observation data in two cases. A good agreement can be found between the calculation results and observation data. It can be seen that in case 1 the excavating is in the permissible range. In case 2, however, the excavation is in danger of instability and some measures should be taken to protect the SMW retaining structure from failure.

# 1 INTRODUCTION

The composite structure with H-shaped steel and deep cemented-soil piles is called SMW method. This method can be applied in cohesive soil, sandy soil and sandy gravel layers. It has been widely accepted in China, which is mainly applied to deep excavations in soft soils of eastern and southern China.

This paper studies the bearing and deforming mechanism of SMW. Based on the in-situ excavation construction procedures of the engineering example, the construction steps of excavating and supporting are simulated in the numerical simulation method with FLAC3D. Because the excavations are too long, the method of excavating is similar to tunnel excavating, which is from one side to another. And there are two cases simulated, one is the normal case in which the supports are installed timely. However, in middle of March 2007 during excavating, because of the bad weather and some other reasons, the supports were not installed timely so that the excavation was in danger of collapse. Therefore, another case is a case in which the supports are not installed timely.

Through comparing the actual observation data in-situ and the calculation results, some useful conclusions are achieved.

# 2 THE PROJECT GENERAL SITUATION AND MONITORING SCHEME

# 2.1 The project general situation

The underground channels project contains two long strip excavations, the eastern and western excavations, which are similar to each other. The length is both 820.5 m, and the width is 16.5 m. The excavation depth is between 6.645 m  $\sim$  8.039 m. It is clear around the construction site.

The foundation soil layers belong to Quaternary Pleistocene-Holocene deposit, including cohesive soils, silt and sandy soils which distribute in planes. The physico-mechanical parameters of soils in site are shown on Table 1.

The  $\varphi$ 650 SMW method is applied and steel pipe supports are installed. The SMW retaining structure is 17 m long in depth, which is inserted with  $500 \times 200 \times 10 \times 16$  H-shaped steels. In order to facilitate the earthwork excavating, two steel pipe support tiers including the upper and the lower supports are installed considering the characteristic of the excavations. The upper steel supports locate 1.0 m below the ground surface. The lower steel supports locate 2.5 m above the excavation bottom. The sizes of the steel

Table 1. Physico-mechanical parameters of soils in site.

The soil layers	Depth (m)	γ (kN/m <sup>3</sup> )	C (KPa)	φ (°)	E (MPa)	μ
Silty clay and clayey silt	3	17.8	10.0	23.0	4.38	0.3
Sandy silt	2	18.2	4.0	29.0	4.78	0.33
Silty clay	2	17.3	10.0	21.5	3.84	0.35
Sandy silt and silty clay	2	17.6	3.0	25.5	5.08	0.3
Silty clay	10	16.2	11.0	11.0	13.62	0.3
Clay	11	17.3	13.0	12.0	17.0	0.3

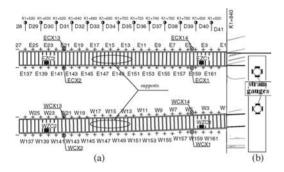


Figure 1. General layout of monitoring points: (a) The south section of the excavations; (b) The strain gauges welded around the support.

supports are all  $\varphi$ 609 × 12. The distance between the adjacent supports is 5 m.

### 2.2 The monitoring scheme

Around the two excavations, the monitoring points for the horizontal displacement and settlement at the top of the retaining structure are located about every 10 m. And they are numbered using E and W, in which E denotes the eastern excavation, W denotes the west excavation. There are all 324 monitoring points for the top horizontal displacement and settlement of the SMW in two excavations, as shown in Figure 1(a). Because the excavations are so long that the south section is given only.

14 inclinometer tubes for lateral deformation of SMW are set in every excavation. They are located in the same distance about 100m symmetrically. The location of every monitoring point is shown in Figure 1(a), in which ECX denotes the inclinometer tube in the eastern excavation and WCX denotes the inclinometer tube in the western one.

In every excavation there are 6 pairs of monitoring points for axial forces of supports, and every pair has two points including the upper and the lower supports. As shown in Figure 1(a), for example, EZC1 includes the upper EZC1 and the lower EZC1 supports. Therefore, there are 12 monitoring points for every excavation, EZC denotes the axial force monitoring points in the eastern excavation, and WZC denotes the western one. The distance between each pair monitoring points is about 120 m. The steel strain gauges are welded around the steel pipe supports as shown in Figure 1(b).

# 3 BEARING AND DEFORMING MECHANISM OF SMW

The cemented soil material that is produced generally has a higher strength, lower permeability, and lower compressibility than the native soil. Therefore, the SMW method can make it possible to form waterpreventing and earth-retaining walls quickly by mixing earth collected at a construction site with cement slurry. The rigidity of the earth retaining walls was further enhanced by forming a compound earth-retaining wall with H-shaped steel materials welded with studs that act as stress material arranged within the improved soil walls. And under the suitable conditions, the H-shaped steels can be recycled.

Stress-strain characteristics of SMW are extremely complex during the course of the pit excavation. The curves of H-shaped steel strain are under the linear elastic scope, but cemented-soil is nonlinear response, and the rigidity changes of composite structure mainly by the cemented-soil. It is commonly considered that the H-shaped steels bear all the lateral water and earth pressure and the cement deep mixing piles are used to prevent water. However, it is testified through experiments that cement soil can enhance the H-shaped steels to reduce the deformation. In addition, the cement soil can also have confinement effect to prevent the H-shaped steels instability. The composite flexural stiffness is 20% greater than only H-shaped steels. The stiffness enhancing coefficient can denote the degree of stiffness enhancing as follows:

$$\alpha = \frac{E_{cs}I_{cs}}{E_sI_s} \tag{1}$$

where  $E_{cs}$  and  $E_s$  are the elastic modulus of cement deep mixing pile with H-shaped steel and the elastic modulus of H-shaped steel, respectively;  $I_{cs}$  and  $I_s$  are the inertia moment of cement deep mixing pile with H-shaped steel and the inertia moment of H-shaped steel.

In this numerical calculation, the cement deep mixing pile with H-shaped steel is equivalent to diaphragm wall and the influence of stiffness enhancing coefficient  $\alpha$  is considered. According to the principle that the stiffness is equal to each other, the equation is given by

$$E_{cs}I_{cs} = \frac{1}{12}E(W+t)h^{3}$$
 (2)

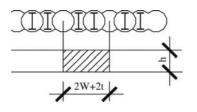


Figure 2. The stiffness equivalence between SMW and diaphragm wall.

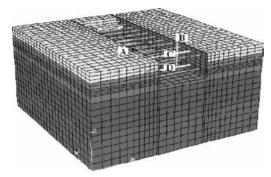


Figure 3. Elements of model in case 1.

Based on equation (1), thus

$$h = \sqrt[3]{\frac{12\alpha E_s I_s}{E(W+t)}}$$
(3)

Here,  $\alpha$  is considered as 1.2. The equivalent thickness of diaphragm wall in this numerical calculation is h = 0.65 m and the Young modulus is E = 12.6 GPa.The equivalent figure from SMW with H-shaped steels to diaphragm wall in this project is shown as Figure 2 shown.

The interfaces are installed to simulate the interface characteristics between the retaining structure and the soils. In FLAC3D, Interfaces have the properties of friction, cohesion, dilation, normal and shear stiffness, and tensile and shear bond strength, which are characterized by Coulomb sliding and/or tensile and shear bonding. In this computation, the equivalent diaphragm wall is considered as elasticity.

# 4 CALCULATION CASE

# 4.1 Case 1

Because there are two long strip excavations and they are similar, one part of the eastern excavation is chosen to be simulated. The model size is 60 m in extent, 60 m in breadth and 30 m in height. The model is shown in Figure 3.

The earthworks soils for excavating are divided into 3 layers. The first layer is from 0.0 m to -2.0 m, the

second layer is from -2.0 m to -6.5 m, the third is from -6.5 m to -8.0 m. There are upper and lower two supports installed, the upper supports are located at -1.0 m and the lower supports are at -6.5 m. And the distance between two adjacent supports in y direction is 5 m. Therefore, the length of soils excavated in every layer is 5 m in y direction in every excavating step. Because the excavation is too long, the excavating method is similar to tunnel excavating method which is from one side to another. The construction procedure of excavating and supporting is divided into lots of steps, as follows:

- The construction of SMW.
- The first layer is excavated 5 m in y direction and the first upper steel support is installed.
- The first and second layers are excavated 5 m in y direction and the second upper and the first lower supports are installed.
- All the three layers are excavated 5 m in y direction, and the third upper and the second lower supports are installed.
- Do this until the earthworks excavation is completed.

The whole procedure of excavating steps and installing supports is simulated by 3D numerical method. There are 11 excavating and supporting steps except the construction of SMW.

In this numerical simulation calculation, the mechanical soil behavior is modeled with Mohr-Column model and the supporting structures are considered as elastic model. The interfaces are installed between SMW and soils. The top of the model, at z = 30 m, is a free surface. The base of the model, at z = 0 m, is fixed in the z-direction, and roller boundaries are imposed on the sides of the model, at x = 0 m, x = 60 m, and y = 0 m, y = 60 m.

# 4.2 Case 2

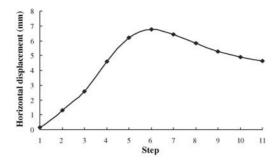
In middle of March 2007, because of the bad weather and other reasons, the steel pipe supports were not installed in time. There were about 30 m in length without supports from the excavating face to the lattermost supports for a long time. Meanwhile, according to the in-situ measurements, there was a sharp increment in horizontal displacement of the soil mixing wall. This case is simulated to analyze the influence.

In this case, the material properties and boundary conditions are same to case 1. The excavating and supporting procedures are same, too.

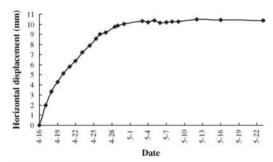
### 5 RESULTS

# 5.1 Case 1

In order to analyze the calculation results conveniently, some key points are set in the model, as shown in



(a) The calculation results.

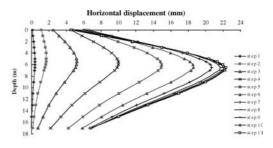


(b) The observation data.

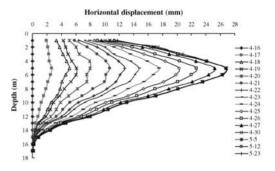
Figure 4. The horizontal displacement at point A in case 1.

Figure 3. Firstly, the horizontal displacement at the top of SMW is analyzed. The curve of horizontal displacements at point A is shown in Figure 4(a). It can be seen that the horizontal displacement increases before step 6 but then decreases in the following steps. The maximum value is 6.76 mm at step 6 and the ultimate value is 4.63 mm. According to the excavating steps in calculation, when the excavating face exceeds point A about 15m, the value of the horizontal displacement begins to decline. The actual observation data for point A is shown in Figure 4(b). It can be seen that the actual values are bigger than the calculation results. The curve is monotone increasing by steps and tends to be constant after having reached a certain level. Its ultimate value is 10.4 mm. The calculation result of the horizontal displacement at point A is much less than the observation value.

The curves of calculation results with excavating steps and the observation data with date for the horizontal displacement of the retaining structure in line B are shown in Figure 5. According to the calculation results, its maximum horizontal displacement of the SMW occurs at the point of 6.5 m depth, and its value is 22.35 mm. As shown in Figure 5(a), when the excavating face reaches the line B at step 3, the deformation of SMW increases dramatically. When it has passed away from line B about 20 m, the deformation increases



(a) The calculation results.

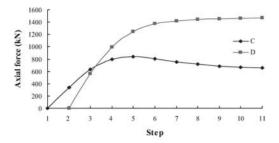


(b) The observation data.

Figure 5. The horizontal displacement of the SMW in line B in case 1.

slowly. Figure 5(b) shows the observation curve of horizontal displacement of SMW in line B with the date. A rather good agreement can be found between (a) and (b). According to the observation data, the maximum horizontal displacement in line B occurred at point of 5.0 m depth and its value is 26.8 mm, which is greater than the maximum calculation value by 4.45 mm. With the excavating face advancing, the deformation increment is becoming smaller. From the actual observation data and calculation results, it can be seen that the horizontal displacement of the SMW mainly occurred during the period of excavating surface passing this line.

The curves of axial forces of the steel pipe supports C and D with excavating steps are shown in Figure 6(a). The final axial force of the upper support C is 657.60 kN, and the lower support D is 1467.7 kN in calculation results. As shown in Figure 6(b), the observation data is greater than the calculation results. The maximum axial force of the upper support C is 995.83 kN, and the lower support D is 1575.7 kN. Both C and D have an ascending firstly and then declining process with lapse of time in actual observation. This is because the foundation mat bore a part of soil pressure with its pouring and strengthening. However, the procedure of pouring and strengthening of foundation mat is not simulated in numerical calculation. So there



(a) The calculation results.

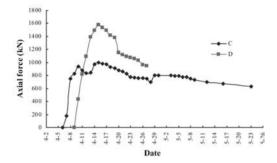
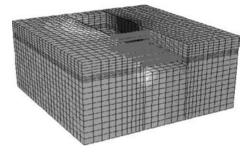


Figure 7. Elements of model in case 2.



(b) The observation data.

Figure 6. The axial force of supports C and D in case 1.

is no declining trend of axial forces. But in the early stage, the trend and shape of calculation results curve with excavating steps and observation data curve with date is agreed generally. As shown in Figure 6(b), on 28 April the lower support D was removed, therefore, the axial force of the upper support C had a significant rise by 102.5 kN. However, in calculation this procedure is not simulated.

Through analysis, it can be seen the values of observation data are greater than the calculation results universally. The main discrepancy between calculation and measure can be explained that the physico-mechanical parameters of soils are not accurate enough. However, in reality, this area was less stiff than initially planned. According to the numerical simulation and the actual observation data, the excavation is stable if the steel pipe supports can be installed in time. The numerical results and actual data of axial forces are lower than the alarm values. The displacement due to excavating is in the permissible range.

# 5.2 Case 2

In case 2, the numerical calculation model is shown in Figure 7. Figure 8 shows the horizontal displacement of the model and the axial forces of steel pipe supports at last step. It can be seen the maximum horizontal displacement at the top of SMW occurs at point

Figure 8. The horizontal displacement and the axial forces of steel pipe supports in case 2.

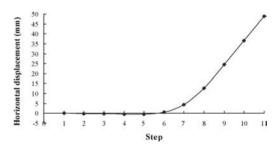


Figure 9. The horizontal displacement of point E in case 2.

E with 48.96 mm, as shown in Figure 7. The curve of horizontal displacements at point E when excavating from step 1 to step 11 is shown in Figure 9. It can be seen the displacement at point E is nearly zero before step 6 until the excavating face passes point E. With the excavating face advancing after step 6, the displacement increases significantly. According to calculation results, the maximum horizontal displacement of SMW occurs at the point of 3 m depth, which is in line F, and its value is 50.44 mm. As shown in Figure 10, the horizontal displacement of SMW in line F develops slightly until the excavating face passes this line at step 6. Because the supports are not installed near this line, the deformation of SMW develops more rapidly. In actual observation, there are three points of which horizontal displacements in line F are greater than the alarm value with 50 mm. The axial forces

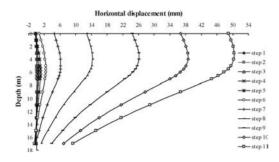


Figure 10. The horizontal displacement of the SMW in line F in case 2.

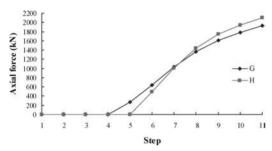


Figure 11. The axial forces of supports G and H in case 2.

of lastly installed supports G and H are much larger because of without installation of subsequent supports. The axial force of support G is 1924.50 kN, and H is 2095.60 kN. The curves of the axial forces of supports G and H with the excavating steps are shown in Figure 11. According to calculation results, the axial forces of G and H are much greater than the adjacent supports by 903.75 kN for upper support and 546.00 kN for lower support. Compared with Figure 6(a), the magnitude is much greater. The axial force of G exceeds the alarm value with the upper supports for 1500kN and H exceeds the alarm value of lower supports for 2000 kN.

There are two inclinometer tubes ECX2 and ECX13 for lateral deformation of SMW around this site. According to the observation data of 19 March and 20 March, the velocity of horizontal displacement exceeded the alarm value of 3 mm/d for two days between the depth of  $5 \text{ m} \sim 8 \text{ m}$  at ECX13. The velocity values were 4.18 mm/d, 3.97 mm/d, 3.64 mm/d and 3.20 mm/d when the depths are 5 m, 6 m, 7 m and 8 m respectively on 19 March. Moreover, the velocity values were 5.18 mm/d, 5.66 mm/d, 4.98 mm/d and 3.44 mm/d in the next day. Meanwhile, the velocities of horizontal displacement were greater than 3 mm/d in the depth from 1 m to 12 m at ECX2 on 19 March, which were over 10 mm/d within 3 m from the top of SMW.

According to the results of numerical simulation and actual observation data, it was possible to collapse for this excavation because the supports were not installed in time. The risk was existent so the corresponding measures should be taken. After being alarmed, the construction team stopped excavating and installed the supports speedily. It's turned out that the measures are very effective according to the subsequent observed data.

#### 6 CONCLUSIONS

In this paper, firstly, the bearing and deforming mechanism of SMW is analyzed in brief; secondly, the construction monitoring scheme is introduced; thirdly, a 3D numerical simulation of this long stripe excavation is described, including all the components of the project (the SMW construction, stepped excavation and supports installation); then the numerical results are compared with the actual data in-situ observation.

- The 3D numerical method can simulate the whole excavation construction very well. A good agreement can be found between the numerical results and the actual observation data except for some small deviations.
- The excavation is stable and the displacement due to excavating is in the permissible range if the steel pipe supports are stalled timely. However, because of bad weather and other reasons the steel pipe supports are not installed in time and without supports for a long time, such as case 2, the excavation is in danger of collapse.
- According to the in-situ observation data, the construction team can take corresponding measures to protect the excavation away from some undesirable events and risks.

#### REFERENCES

- Commend, S. Geiser, F. & Crisinel, J. 2004. Numerical simulation of earthworks and retaining system for a large excavation. Advances in Engineering Software Vol. 35: 669–678.
- Li, J.C. Zhang, Z.Y. & Xu, Q 2005. Study on threedimension numerical simulation of deformation of the deep-foundation pit with excavation. *Journal of Nanjing University of Technology* 27(3): 1–7.
- Liu, H.Y. et al 2006. Numerical analysis on excavation safety of deep foundation engineering. *Chinese Journal* of Geotechnical Engineering 28 (Supp): 1441–1444.
- Liu, J.G. & Zeng, Y.W. 2006. Application of FLAC3D to simulation of foundation excavation and support. *Rock* and Soil Mechanics 27(3): 505–508.
- Liu, J.H. & Hou, X.Y. 1997. The Handbook of Foundation Engineering, Beijing: China Architecture & Building Press. 569–572.
- Zhang, P & Liu, R.H. 2000. The Application of SMW Method in Foundation Pit. *Chinese Journal of Rock Mechanics and Engineering* 19(Supp): 1104–1107.

### A simplified spatial methodology of earth pressure against retaining piles of pile-row retaining structure

#### Y.L. Lin

Geotechnical Research Institute, Hohai University, Nanjing, P.R. China Key Laboratory for Geotechnical Engineering of Ministry of Water Resource, Hohai University, Nanjing, P.R. China

#### X.X. Li

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: Pile-row retaining structure is widely adopted in the excavation of deep foundation pit. In the supporting system, retaining piles are the main bearing members. It is extremely important to obtain the magnitude and distribution of earth pressure against retaining piles. Based on the mode of failure, a new methodology is proposed to evaluate the earth pressure against retaining piles of pile-row retaining structure. In the proposed method, both spatial effect and intermediate principal stress effect are considered. Finally, the methodology is applied to practice engineering. It is demonstrated that the strength theory has more influence on earth pressure and the potential strength of filling materials is sufficiently developed under the guidance of the united strength theory.

#### 1 INTRODUCTION

The pressure against the back of a retaining structure caused by backfill and surcharge on the ground surface is a classical problem of soil mechanics. It is influenced by retaining structure types, movement mode, stiffness and contact conditions between soil and structure (Fang & Ishibashi, 1986; Harrop-Willrams, 1989; Zhou,1990; Fang et al., 1994; Wang, 2000; Pal & Salgado, 2003). In addition, the distortion of soil mass has a certain effect on the earth pressure. Before the soil achieves breakage, the magnitude of earth pressure cannot be determined. Even if it reaches limit state, the earth pressure cannot also be calculated because inner soil mass cannot synchronously arrive at limit equilibrium state. So the reliable parameter of soil cannot be acquired. Thus, to apply in practice expediently, it is usually assumed the soil is on ideal failure state.

In the excavation of deep foundation pit pile-row retaining structure is widely adopted. In the supporting system, retaining piles are the main bearing members. It is very important to obtain the magnitude and distribution of earth pressure against retaining piles. Because of failure mechanism of soil behind piles, the influence of interaction between retaining structure and soil on earth pressure can't be achieved accurately according to classical earth pressure theory. So earth pressure should be taken as the spatial problem rather than plane problem. Mohr-Coulomb strength theory is usually introduced into the computation analysis and the influence of intermediate principal stress is omitted. However, plenty of experiments reveal the soil strength varies with the intermediate principal stress (Yu, 2004), which is quite different from what has been depicted in the conventional Mohr-Coulomb theory. The unified strength theory is a system of yield and failure criteria of material sunder complex stresses. It has a clear physics and mechanics background, a unified mathematical model, and a simple and explicit criterion which includes all independent stress components and simple material parameters (Yu, 2002).

In this study, a new methodology based on the plastics limit analysis is proposed to evaluate the earth pressure against retaining piles of pile-row retaining structure based on the mode of failure. In the proposed method, both spatial effect and intermediate principal stress effect are considered. The solution of the equation is obtained, giving a theoretical result for the earth pressure on retaining piles.

#### 2 UNIFIED STRENGTH THEORY

Based on a twin-shear element and the multiple slip mechanism, Yu and He (1992) established the unified strength theory. It has a unified model and simple

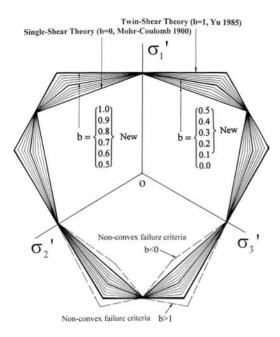


Figure 1. Varieties of the unified strength theory on deviatoric plane (Yu, 2004).

unified mathematical expression that is suitable for various materials (Yu, 1994). The unified strength theory covers all the regions from the lower bound to upper bound, as shown in Figure 1. The unified strength theory considers the different contributions of all stress components acting on the stress element to the yield or failure of materials.

The mathematical modeling is expressed as follows (Yu et al., 2002):

When 
$$\sigma_2 \leq \frac{\sigma_1 + \alpha \sigma_3}{1 + \alpha}$$
  
 $F = \sigma_1 - \frac{\alpha}{1 + b} (b\sigma_2 + \sigma_3) = \sigma_t = \alpha \sigma_c$  (1)

When 
$$\sigma_2 \ge \frac{\sigma_1 + \alpha \sigma_3}{1 + \alpha}$$
  
 $F' = \frac{1}{1 + b} (b\sigma_2 + \sigma_1) - \alpha \sigma_3 = \sigma_t = \alpha \sigma_c$  (2)

If it is prescribed that press stress is positive, Equation (1) and Equation (2) can be rewritten as

When 
$$\sigma_2 \le \frac{\sigma_1 + \sigma_3}{2} - \frac{\sigma_1 - \sigma_3}{2} \sin \varphi_0$$
  
 $F = \sigma_1 (1 - \sin \varphi_0) - \frac{b\sigma_2 + \sigma_3}{1 + b} (1 + \sin \varphi_0) = 2c_0 \cos \varphi_0$  (3)

When 
$$\sigma_1 \ge \sigma_2 \ge \frac{\sigma_1 + \sigma_3}{2} - \frac{\sigma_1 - \sigma_3}{2} \sin \varphi_0$$
  
 $F = \frac{b\sigma_2 + \sigma_1}{1 + b} (1 - \sin \varphi_0) - \sigma_3 (1 + \sin \varphi_0) = 2c_0 \cos \varphi_0$  (4)

where  $c_0 = \text{cohesion}$ ,  $\phi_0 = \text{friction}$  angle, b = unified strength parameter that reflects the influences of the intermediate principal stress on the yielding of the material ( $0 \le b \le 1$ ),  $\sigma_t$  and  $\sigma_c$  are uniaxial tensile strength and compressive strength, respectively, and  $\alpha$  is tensile-compressive strength ratio.

Introducing Lode parameter  $\mu_{\sigma}$ , thus

$$\sigma_2 = \frac{\sigma_1 + \sigma_3}{2} + \frac{\mu_{\sigma}(\sigma_1 - \sigma_3)}{2} \tag{5}$$

Substituting Equation (5) into Equation (3) and (4), letting

$$\sigma_1 = \frac{1 + \sin \varphi_i}{1 - \sin \varphi_i} \sigma_3 + \frac{2c_i \cos \varphi_i}{1 - \sin \varphi_i}$$
(6)

The unified cohesion  $c_t$  and the unified friction angle  $\varphi_t$  can be defined as

when 
$$\mu_{\sigma} \le -\sin \varphi_{0}$$
  

$$\sin \varphi_{t} = \frac{2(1+b)\sin \varphi_{0}}{2+b(1-\mu_{\sigma})-b(1+\mu_{\sigma})\sin \varphi_{0}}$$

$$c_{t} = \frac{2(1+b)c_{0}\cos \varphi_{0}\cot\left(45^{\circ}+\frac{\varphi_{t}}{2}\right)}{2+b(1-\mu_{\sigma})-(2+3b+b\mu_{\sigma})\sin \varphi_{0}}$$
(7)

When  $\mu_{\sigma} \ge -\sin \varphi_0$ 

\*\*\*\*\*

$$\sin \varphi_{t} = \frac{2(1+b)\sin \varphi_{0}}{2+b(1+\mu_{\sigma})+b(1-\mu_{\sigma})\sin \varphi_{0}}$$

$$c_{t} = \frac{2(1+b)c_{0}\cos \varphi_{0}}{(2+b+b\mu_{\sigma})(1-\sin \varphi_{0})\tan\left(45^{\circ}+\frac{\varphi_{t}}{2}\right)}$$
(8)

According to Mohr circularity of stress state at a point, the unified expression of shear strength can be obtained

$$\tau = \sigma \tan \varphi_t + c_t \tag{9}$$

#### 3 FAILURE MODE OF SOIL HEHIND PILES

For pile-row retaining structure, the arching effect in the retaining soil mass occurs (Hu et al., 2000). It is a stress redistribution process by which stress is transferred around a region of the soil mass, which then becomes subject to lower stresses. So the earth pressure acting on piles is enhanced, while the earth pressure on soil around piles is depressed. The smaller is pile space, the stronger is soil arching effect. And it is more propitious to the stability of foundation pit.

#### 3.1 Simple shear failure mode

Figure 2 shows the failure mode A homogeneous foundation pit of depth *H* and the net space *B* is considered.

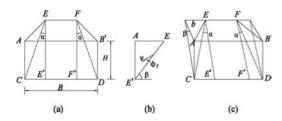


Figure 2. Simple shear failure mode of soil mass between piles.

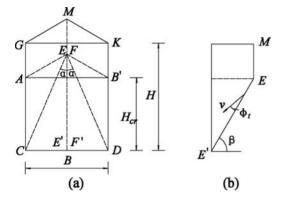


Figure 3. Rip-shear failure mode of soil mass between piles.

The soil wedge is assumed to be rigid and slide alone a planar surface. The critical inclination of failure plane is expressed as  $\beta$ .

When  $B - 2b \ge 0$ , simple shear failure mode occurs, letting

$$b = H \tan \alpha / \sin \beta \tag{10}$$

According to Equation (10), the failure condition of simple shear failure mode can be obtained as

$$H \le H_{cr} = B\sin\beta/2\tan\alpha \tag{11}$$

where  $H_{cr}$  = critical height of failure mode.

#### 3.2 Rip-shear failure mode

Rip-shear failure mode will arise when  $H > H_{cr}$ , as shown in Figure 3. The soil mass behind piles is divided into two portions from the critical height  $H_{cr}$ . Above the height  $H_{cr}$ , the soil is rip failure, and it is shear failure below the height  $H_{cr}$ .

#### 4 CALCULATION OF ACTIVE EARTH PRESSURE

#### 4.1 Earth pressure of simple shear failure mode

Figure 4(a) shows the mechanism of simple shear failure. The single pile endures the earth pressure of the

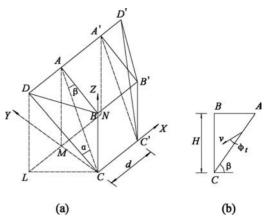


Figure 4. Schematic for earth pressure of simple shear failure.

region soil of block *BCDB'C'D'*. It is assumed that the soil is perfectly plastic and their deformation is governed by the associative flow rule. Then, Kinematical admissibility requires the velocity jump vector, v, be inclined to the velocity discontinuity at angle  $\varphi_t$ , as presented in Figure 4(b).

The Cartesian coordinates system established for the present analysis is shown in Figure 4(a). The point *C* is the origin of the Cartesian coordinates system, and plane *XY* is level plane. Section *AC* (or *A'C'*) is slippage tangent of fracture surface *BCC'B'*, and it is inclined to the velocity discontinuity at angle  $\varphi_t$ . Slimily, the velocity of fracture surface *BCD* and *B'C'D'* is at an angle  $\varphi_t$  to the slippage tangent *DC* and *D'C'*. In the plane *BCD*, according to the directional derivative of the velocity *v*, the angle  $\alpha$  can be obtained as

$$\alpha = a \tan\left(\frac{DA}{AC}\right) = a \tan\left(\frac{\cos\beta\sin\varphi_t}{\sqrt{\cos^2(\beta - \varphi_t) - \sin^2\varphi_t}}\right)$$
(12)

The rate of work of soil weight can be calculated as the work rate of block ABC-A'B'C' plus the work rates for blocks C-BAD and C'-B'A'D'. Consequently, this work rate takes the form

$$\dot{W}_{soil} = \frac{1}{2} \mathcal{H}^2 v f_1 \tag{13}$$

where  $\gamma =$  the soil unit weight, d = the diameter of pile,  $f_1$  is a function, it can be determined as

$$f_1 = \left(d + \frac{2H}{3}\frac{\tan\alpha}{\sin\beta}\right)\cot\beta\sin(\beta - \varphi_t)$$

Due to the homogeneous soil masses being rigid, the internal energy is only dissipated along the sliding surface. The work dissipation rate can be calculated as the work dissipation rate of block CC'DD' plus the work dissipation rates for blocks DBC and D'B'C'. Consequently, this dissipation work rate can be calculated as

$$\dot{W}_{int} = c_1 H v f_2 \tag{14}$$

where

$$f_2 = \cos \varphi_t \left[ \frac{d \sin \beta + H \tan \alpha}{\sin^2 \beta} + \frac{H}{\sin \beta} \left( \cos^2 \beta + \tan^2 \alpha \right)^{\frac{1}{2}} \right]$$

Since soil-pile interface can be considered as velocity discontinuity rather than stress characteristic, ideal plastic model is not applicable to the interface material. The relative movement between soil and pile, which depends on the interface characteristic sand the property of the adjacent soil, is not always of purely frictional sliding. If it is assumed that the total horizontal earth pressure induced by soil mass is  $P_{au}$ . The friction angle of soil-pile interface is  $\delta$ . Two conditions are considered in the following.

1 Smooth pile ( $\delta < \varphi_t$ )

The external work rate contributed by the resultant horizontal earth pressure  $P_{au}$  is

$$\dot{W}_p = -P_{au}v\cos(\beta - \varphi_t) \tag{15}$$

The work dissipation rate along pile surface is

$$\dot{W}_e = P_{au} v \tan \delta \sin(\beta - \varphi_t) \tag{16}$$

Equating the rate of internal energy dissipation to the rate of the external work, we can obtain

$$P_{au} = \frac{1}{\cos(\beta - \varphi_t) + \tan \delta \sin(\beta - \varphi_t)} \cdot \left(\frac{1}{2} \gamma H^2 f_1 - c_t H f_2\right)$$
(17)

2 Rough pile ( $\delta \ge \varphi_t$ )

The external work rate contributed by the resultant horizontal earth pressure  $P_{au}$  is

$$\dot{W}_{p} = -P_{au}v \frac{\cos(\beta - 2\varphi_{t})}{\cos\varphi_{t}}$$
(18)

The work dissipation rate along pile surface is

$$\dot{W}_{c} = \frac{\pi}{2} c_{t} dHv \sin(\beta - \varphi_{t})$$
(19)

Similarly, equating the rate of internal energy dissipation to the rate of the external work, the resultant horizontal earth pressure can be obtained

$$P_{au} = \frac{\cos\varphi_t}{\cos(\beta - 2\varphi_t)} \left(\frac{1}{2}\gamma H^2 f_1 - c_t H f_2 - \frac{\pi}{2} c_t dH \sin(\beta - \varphi_t)\right)$$
(20)

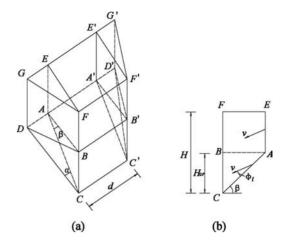


Figure 5. Schematic for earth pressure of rip-shear failure.

#### 4.2 Earth pressure of rip-shear failure mode

The rip-shear failure mechanism for the present analysis is shown in Figure 5. The earth pressure acting on single pile is induced by the region soil mass FCDG-G'D'C'F'. Similar to the earth pressure of simple shear failure mode, the rate of work due to the soil mass weight can be expressed as

$$\dot{W}_{soil} = WV \sin(\beta - \varphi_t) = \gamma v f_1' \tag{21}$$

where

$$f_1' = \left[\frac{1}{4}H\left(S^2 - d^2\right) - \frac{1}{24}\left(S - d\right)^2\left(2S + d\right)\frac{\sin\beta}{\tan\alpha}\right] - \frac{\cos\beta\sin(\beta - \varphi_i)}{\tan\alpha}$$

where S = the center distance of two adjacent piles.

For the rigid material considered, the internal energy is only dissipated along the sliding surface and the interface surface of soil-pile. The rate of energy dissipation along the sliding surface can be expressed as

$$\dot{W}_{int} = c_i v (f_2^{'} + f_3^{'})$$
 (22)

where

$$C = \left(\cos^2\beta + \tan^2\alpha\right)^{\frac{1}{2}}$$
$$f'_2 = \frac{\cot\alpha\cos\varphi_i}{4} \left[ \left(S^2 - d^2\right) + C\left(S - d\right)^2 \sin\beta\cot\alpha \right]$$
$$f'_3 = \frac{\cos(\beta - \varphi_i)}{2} \left( H - \frac{S - d}{2} \frac{\sin\beta}{\tan\alpha} \right) \left[ C\left(S - d\right) \cot\alpha + S \right]$$

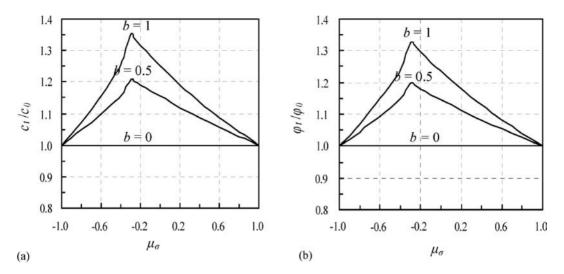


Figure 6. Relations between the unified strength parameters and the soil strength.

where

$$C = \left(\cos^{2} \beta + \tan^{2} \alpha\right)^{\frac{1}{2}}$$

$$f'_{2} = \frac{\cot \alpha \cos \varphi_{t}}{4} \left[ \left(S^{2} - d^{2}\right) + C\left(S - d\right)^{2} \sin \beta \cot \alpha \right]$$

$$f'_{3} = \frac{\cos(\beta - \varphi_{t})}{2} \left( H - \frac{S - d}{2} \frac{\sin \beta}{\tan \alpha} \right) \left[ C\left(S - d\right) \cot \alpha + S \right]$$

The rate dissipation along the interface surface of soil-pile is similar to the Equation (16) and (19). For Smooth pile  $(\delta < \phi_t)$ , according to the energy conversation law, we can obtain

$$P_{au} = \frac{[f_1' - c_i(f_2' + f_3')]}{\cos(\beta - \varphi_i) + \tan\delta\sin(\beta - \varphi_i)}$$
(23)

Similarly, for rough pile ( $\delta \ge \phi_t$ ), the total horizontal earth pressure  $P_{au}$  can be derived as

$$P_{au} = \frac{\cos \varphi_t}{\cos(\beta - 2\varphi_t)} \left[ \mathcal{J}_1' - c_t \left( f_2' + f_3' \right) - \frac{\pi}{2} c_t dH \sin(\beta - \varphi_t) \right]$$
(24)

Accordingly, the corresponding resultant  $P_a$  acting on the pile is

$$P_a = \frac{P_{au}}{\cos \delta} \tag{25}$$

The magnitude of active earth pressure can be obtained from Equation (25). Obviously, for a given example, the resultant  $P_a$  is only determined by failure angle  $\beta$ . By taking the first derivatives of Equation

(25) with respect to  $\beta$ , and equating it to zero and solving it, we can obtain the critical angle  $\beta_{cr}$ . Substituting  $\beta_{cr}$  into Equation (25), we have the maximal upper-bound for the active earth pressure.

#### 5 RESULTS AND DISCUSSION

To evaluate the validity of the proposed method, a pilerow retaining structure without anchor is analyzed. Numerical results are presented and compared.

# 5.1 Effects of the unified strength parameters on soil parameters

The influences of the unified strength parameters on soil mass strength are represented in Figure 6 for  $\varphi_0 = 15^\circ$ . It is observed that the soil mass strength varies with the variety in the value of the unified strength *b*. The soil mass parameters of  $c_t$  and  $\varphi_t$  are piece-wise functions and they achieve extremum when  $\mu_{\sigma} = -\sin\varphi_0$ .

### 5.2 Effects of the unified strength parameters on failure mode

Figure 7 presents the effects of the unified strength parameters on critical height  $H_{cr}$  for  $\gamma = 17 \text{ kN/m}^3$ ,  $\varphi_0 = 15^\circ$ , H = 10 m. As a whole, the critical height  $H_{cr}$  decreases with the increase in the value of *b*. Figure 7(a) and Figure 7(b) also show the influence of *S* and pile diameter *d*. For  $c_0 = 0$ , the critical height increases with the increase in the value of *S*, while it decreases with the increase of *d*. It is also clear

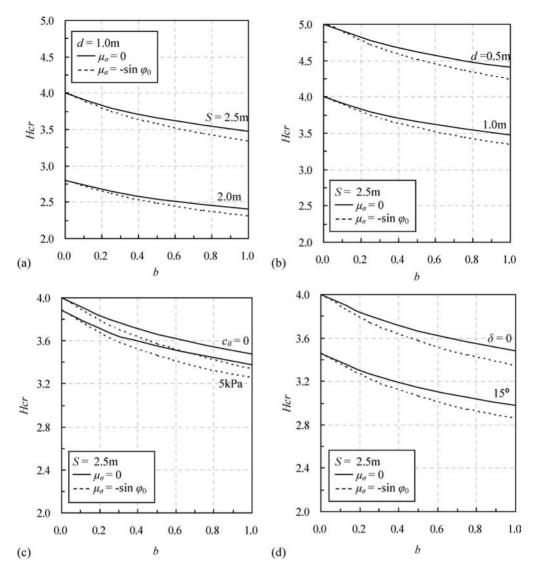


Figure 7. Influence of the unified strength parameters on critical height  $H_{cr}$ .

that under the same conditions  $H_{cr}$  decreases with the increase of  $c_0$  or  $\delta$  in Figure 7(c) and Figure 7(d).

 Table 1.
 Effects of the unified strength parameters on active earth pressure.

# 5.3 *Effects of the unified strength parameters on active earth pressure*

Based on the unified strength theory, the values of active earth pressure is shown in Table 1 for  $\gamma = 17 \text{ kN/m}^3$ ,  $\varphi_0 = 15^\circ$ , H = 10 m, d = 1.0 m,  $c_0 = 0$ . The proposed formula can be degenerated into the expressions induced by Mohr-Coulomb strength theory. From the table, it is found that the strength

0	b = 0		b = 0.5		b = 1	
δ Deg	(1)	(2)	(1)	(2)	(1)	(2)
0	483.6	483.6	414.9	396.3	378.8	350.0
15	364.1	364.1	305.4	378.8	275.5	252.0

\*b = 0 is simplified to Mohr-Coulomb theory; b = 1 is twin shear theory. Situation (1) represents  $\mu_{\sigma} = 0$ , and situation (2) is  $\mu_{\sigma} = -\sin\varphi_0$ . theory has more prodigious influences on the active pressure. The resultant earth pressure  $P_a$  decreases with the increase in unified strength parameter *b*. For  $\mu_{\sigma} = 0$ , when *b* changes from 0 to 1, the resultant earth pressure  $P_a$  decreases by about 21.7%. Similarly, the earth pressure  $P_a$  is also influenced by the Lode parameter  $\mu_{\sigma}$ .

#### 6 CONCLUSIONS

The estimation of active earth pressures acting on retaining piles is very important in geotechnical design. However, unlike the assumption used in the analysis of Coulomb, which generally calculates earth pressure according to plane strain, the earth pressure behind the piles should be taken as the spatial problem. This is due to arching effects in the retaining soil, which result from the frictional resistance between the piles and the soil.

In this paper, a new methodology is proposed to evaluate the earth pressure against retaining piles of pile-row retaining structure. The advantage of the proposed method lies in the fact that both spatial effect and intermediate principal stress effect are considered. It is indicated that the strength theory has more influence on earth pressure and the potential strength of filling materials is sufficiently developed under the guidance of the united strength theory. But the methodology requires experiment or field verification.

#### REFERENCES

Fang, Y. & Ishibashi, I. 1986. Static earth pressures with various wall movements. *Journal of Geotechnical Engineering*, ASCE, 112(3):317–333.

- Fang, Y.S., Chen, T.J. & Wu, B.F. 1994. Passive earth pressures with various walls movements. *Journal of Geotechnical Engineering*, ASCE, 128(8):651–659.
- Harrop-Willrams, K.O. 1989. Geostatic wall pressures. Journal of Geotechnical Engineering, ASCE, 115(9):1321– 1325.
- Hu, M.Y., Xia, Y.C. & Gao, Q.Q. 2000. Calculation principle of earth pressure against retaining piles of pile-row retaining structure. *Chinese Journal of Rock Mechanics* and Engineering, 19(3):376–379.
- Palk, K.H. & Salgado, R. 2003. Estimation of active earth pressure against rigid retaining walls considering arching effects. *Geotechnique*, 53(7):643–653.
- Wang, Y.Z. 2000. Distribution of earth pressure on a retaining wall. *Geotechnique*, 50(1):83–88.
- Yu, M.H. 2004. Unified strength theory and its applications. Berlin: Springer.
- Yu, M.H. 2002. Advance in strength theory of materials under complex state in the 20th Century. *Applied Mechanics Reviews*, 53(3):159–218.
- Yu, M.H. 1994. Unified strength theory for geomaterials and its applications. *Chinese Journal of Geotechnical Engineering*, 14(2):1–10.
- Yu, M.H., He, L.N. & Liu, C.Y. 1992. Generalized twin shear stress yield criterion and its generalization. *Chinese Science Bulletin*, 37(24):2085–2089.
- Yu, M.H., Zan, Y.W. & Zhao, J. 2002. A unified strength criterion for rock material. *International Journal of Rock Mechanics and Mining Sciences*, 39:975–989.
- Zhou, Y.Y. & Ren, M.L. 1990. Experimental study of the active earth pressure on rigid retaining wall. *Chinese Journal of Geotechnical Engineering*, 12(2):19–26.

# Consideration of design method for braced excavation based on monitoring results

#### H. Ota, H. Ito & T. Yanagawa

Osaka Municipal Transportation Bureau, Osaka, Japan

A. Hashimoto *Kotsu Service Co., Ltd., Osaka, Japan* 

T. Hashimoto & T. Konda Geo-Research Institute, Osaka, Japan

ABSTRACT: A comparison between observed data and design value of earth retaining wall deflection due to braced excavation was carried out in soft and sensitive clay ground of some construction sites of Osaka Subway Line No.8. The beam-spring model was employed in the braced design method, and it was taken into account the characteristics of the Osaka soft ground, and there was good agreement between the observed data and design value in past results. According to this comparison, the observed wall deflection was larger than the designed one in some construction sites consisted of the soft and sensitive clay layer with 10 to 20 m thickness. In this paper, the measuring process of the horizontal coefficient of subgrade reaction  $k_h$  in the excavation side of soft clay layer is discussed. As the value of  $k_h$  became small depended on the wall deflection, the new knowledge has been employed on the design method. It is found that the calculation with the revised design method agree well with the monitoring data.

#### 1 INTRODUCTION

In densely populated city, it is necessary to use the underground space highly and effectively for the development of city. It is believed that the demand for much deeper underground excavation will increase gradually. Therefore an applicable design method is demanded for deep, safe and economical excavation. Osaka Municipal Transportation Bureau (OMTB) suggested an original design method for braced excavation based on the characteristics of the Osaka ground and subway constructions. At each construction site (elevens stations and railway depot) where open cut method was adopted in Osaka Subway Line No.8, braced excavation design based on this original design guideline was carried out, and the observational method was also utilized effectively.

In this paper, some comparisons between observed data and design value of earth retaining wall deflection due to braced excavation have been carried out on soft and sensitive clay ground of two construction sites of Osaka Subway Line No.8. The evaluation method for design has been described based on the ground properties.

#### 2 CHARACTERISTICS OF THE MODIFIED BEAM SPRING MODEL

East and West sides of Osaka ground are consistent with the Holocene layers (soft clay and loose sand), but Pleistocene layers exist around the ground surface of Uemachi plateau. The water levels are high in unconfined and confined aquifers, also the permeability of these aquifers are large.

The beam-spring model for the braced excavation, which is indicated in "Standard Specifications for Tunnel [Open Cut Tunnel]" published by the Japan Society of Civil Engineers (JSCE, 2006), is frequently implemented as a widely usable method in Japan. However, since the result of the prediction of wall deflection and strut force are not always consistent with the observation data, OMTB proposed the modified beam spring design method (OMTB, 1993) (Kishio *et al.*, 1997) which can consider "the characteristics of Osaka ground" and "the conditions of subway construction". The outline of the OMTB model is shown in Figure 1.

# 2.1 Active lateral pressure above the excavation bottom

Because there are some possibilities of gap between the braced wall and back ground, the active earth pressure is estimated by Rankine-Resal's equation with the water pressure. In sandy layer, the water pressure is assumed as hydrostatic pressure. In clayey layer, the water head of the upper sandy layer is extended in this clayey layer.

### 2.2 Active lateral pressure below the excavation bottom

In the case of sufficient penetration depth of braced wall, the wall deflection near the tip is small and the lateral pressure is kept as the initial condition. So, if the active lateral pressure is defined basically only by the limit equilibrium theory, there are some cases which the wall deflection is overestimated by giving much lateral pressure. Therefore, the active lateral pressure which is deeper than the bottom layer of excavation is gradually decreasing in the area of triangle formed from the lateral pressure at the bottom of the excavation to the tip of wall.

### 2.3 *Resisting lateral pressure of the excavated ground*

Resisting lateral pressure of the excavated ground is the multiplication of the coefficient of horizontal

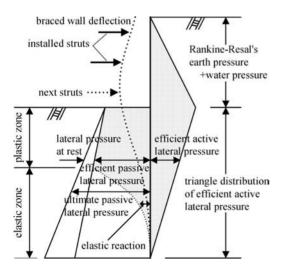


Figure 1. The concept of the modified beam spring model (presented by OMTB, 1993).

subgrade reaction and the wall deflection. However, this value should not exceed the coefficient of the passive lateral pressure which is the subtraction from limit passive lateral pressure defined by Coulomb's equation to the lateral pressure at rest.

#### 2.4 Water pressure in clayey ground

Since it is difficult to distinguish the water pressure from the lateral pressure in clayey ground, lateral pressure is often identified as the integration of soil and water. On the other hand, it is considered that soil is separated from water in modified beam-spring model. Because the pore water pressure acts on the braced wall hydrostatically, the water path is possibly formed between the wall and the back ground due to braced excavation.

For these reasons, the effective stress method is adopted in both sandy layer and clayey layer. The groundwater table in clayey ground is taken on the higher gravitational pressure distribution by comparison between the upper water-bearing layer and down side water-bearing layer.

#### 2.5 Supported effect of covering plates

Because the effect of depressing the wall deflection is recognized when the covering plates are constructed in the same direction as struts, the supported effect of covering plates is considered by reducing 10% of the spring-beam coefficient.

### 2.6 Horizontal coefficient of subgrade reaction of excavated side

The coefficient of subgrade reaction  $k_h$  used in the JSCE model is taken into consideration the geometrical effect related to the difference of loaded width based on some plate loading test results performed near the ground surface. However, the lateral pressure acts on the horizontal direction against the earth retaining wall, because the wall is installed to the vertical direction in subway construction site. Therefore, it is not always appropriate to apply the coefficient of subgrade reaction used in the JSCE model to braced excavation design directly. So in the OMTB model, the coefficient of subgrade reaction is expressed as equation (1) and (2) (Yanagida *et al.*, 1981) empirically.

sandy layer: 
$$k_h (MN/m^3) = 10/16 \times N$$
 (1)  
(N : standard penetration test N-value)

clayey layer:  $k_h$  (MN/m<sup>3</sup>) =  $\alpha \times q_u$  ( $\alpha = 1/20$ ) (2) ( $q_u$  (kN/m<sup>2</sup>) : unconfined compressive strength)

#### 3 COMPARISONS BETWEEN OBSERVATION AND DESIGN OF BRACED WALL DEFLECTION

The comparison between observed data and design value of earth retaining wall deflection due to braced excavation was carried out in soft and sensitive clay ground of No.8 Line construction sites.

In general, the design value can estimate the observed data appropriately in most construction sites. But in some sensitive and soft alluvial clay layer accumulate from 10 m to 20 m thick, observed data are larger than the design value due to braced excavation. The causes for these phenomena are described as follows.

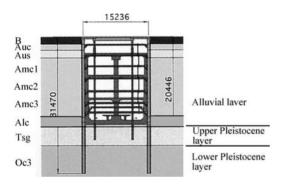


Figure 2. Cross section (A site).

Table 1. Earth retaining wall and each strut (A site).

#### 3.1 A-site

The layer of this A-site ground consists of the alluvial layer, upper lower Pleistocene from the ground surface.

The uniformity coefficient of this fine sand Aus is small, and is called the first water-bearing layer. The alluvial clayey layer Auc is sensitive (*N*-value = 0 to 3, liquid limit  $I_L = 0.4$  to 1.0, cohesion c = 20 to 100 kN/m<sup>2</sup>), and is typical soft ground in this construction site. On the other hand, under the alluvial layer, the upper Pleistocene sandy and gravel layer Tsg (the second water-bearing layer), the lower Pleistocene clay layer Oc3 (Osaka Group, Marine Clay Ma3, c = about 400 kN/m<sup>2</sup>) and the lower Pleistocene sandy layer Os3 (Osaka Group, the third water-bearing layer, *N*value > 60) are deposited continuously to horizontal direction.

The cross section of A-site is shown in Figure 2, the wall and struts conditions are shown in Table 1 and the soil parameters are shown in Table 2. In this construction site, the seepage control method was adopted by extending the earth retaining wall to the low permeable layer Oc3, excavation width is 16.2 m, the final excavation depth is GL-21.5 m and the penetration depth is 4.8 m.

Figure 3 shows the comparison between observed data and design value of the earth retaining wall deflection. The observed wall deflections in east and west sides are symmetric till the 4th step. It is confirmed that the design value can estimates the actual phenomenon adequately. However, since the 5th step, the observed wall deflection of west side was larger than the east side, which can be seen from the results of the 8th step

Soil r	nixing wall (H-st	eel) condition						
Size (mm)		Pitcl S(m	-	Length L(m)	<i>EI</i> (kN·m <sup>2</sup> /m)	)	Area $A(m^2)$	
$H-588 \times 300 \times 12 \times 20$		0 0.60		27.25	399000		0.01925	
Excav	vation condition	Strut condition	on					
Deptl	1	Depth		<i>a</i> :		a	<b>D</b> 1	
Step	(GL-m)	Stage	(GL-m)	Size (mm)		Span L(m)	Pitch S(m)	Area $A(m^2)$
0th	1.51	Cover beam	0.51	H-588 × 300	$\times$ 12 $\times$ 20	16.25	2.00	0.01925
1st	4.50	1st	3.50	$H-300 \times 300$	$\times 10 \times 15$	14.76	2.50	0.01048
2nd	7.00	2nd	6.00	$H-300 \times 300$	$\times 10 \times 15$	14.86	2.50	0.01048
3rd	8.70	3rd	7.70	$H-300 \times 300$	$\times 10 \times 15$	14.66	2.50	0.01048
4th	11.20	4th	10.20	$H-300 \times 300$	$\times 10 \times 15$	14.76	2.50	0.01048
5th	13.90	5th	12.90	H-350 × 350	$\times$ 12 $\times$ 19	14.46	2.50	0.01549
6th	16.45	6th	15.45	$H-350 \times 350$	$\times$ 12 $\times$ 19	14.46	2.50	0.01549
7th	19.05	7th	18.05	$H-350 \times 350$	$\times$ 12 $\times$ 19	14.46	2.50	0.01549
8th	21.55	_	_	_		_	_	_

in Figure 3. It can be assumed that the cause of this phenomenon was the difference of construction process and developmental pattern of creep deformation. Moreover, the observed data exceeded the design value at the west side. It was considered that this disparity occurred for the reason that the plastic zone under the excavation bottom expanded in the Amc and Tsg layer from the 5th step drastically. In addition, it was confirmed that the stress in the wall was controlled within the allowable stress.

In the excavation stage at the Amc layer, the calculation result considering the 75% stress reduction under the 5 m from the excavation bottom was shown together in Figure 3. During an excavation in cohesive

Soil layer	Bottom depth (GL-m)	N-value	Cohision c (kN/m <sup>2</sup> )	Internal friction angle $\phi$ (°)	<i>E</i> <sub>50</sub> (MN/m <sup>2</sup> )
В	1.8	2	0	19.3	_
Auc	4.9	4	42	0	4.1
Aus	8.3	2	0	19.3	_
Amc1	13.8	0	29	0	4.7
Amc2	16.8	1	60	0	6.9
Amc3	19.4	4	91	0	15.9
Alc	21.8	6	108	0	15.9
Tsg	25.4	26	0	32.7	_
Oc3	31.6	14	360	0	83.6

Table 2. Soil parameters (A site).

soil, if an excavation stage takes a long time, the suction of the subgrade soil will disappear due to swelling caused by the water infiltration from continuous rainfall, which leads to reduction in strength (Hashimoto *et al.*, 1997). In conjunction with this arrangement, the ultimate passive lateral pressure and coefficient of subgrade reaction were reduced. This phenomenon was verified by the consolidation with un-drained triaxial compression test, in which shear strength reduced to 70% after the suction was disappear completely and after measuring the water pressure and suction. In short, it is proved that there is a possibility that this phenomenon may occur (Kato *et al.*, 2006).

In the 8th step, the calculation result considering the stress reduction exceeded the design value which could explain the observed data appropriately to some extent. However, under the bottom of the excavation, especially in Tsg layer, the tendency that the design value and calculation result exceeded the observed data. The wall deflection distribution was different between the observed data and design value and calculation result. It would appear that one of the reasons for these tendencies is the deformation at the bottom of the wall towards the excavation side.

#### 3.2 B-site

The layer of this B-site ground constitutes the alluvial layer, upper lower Pleistocene from the

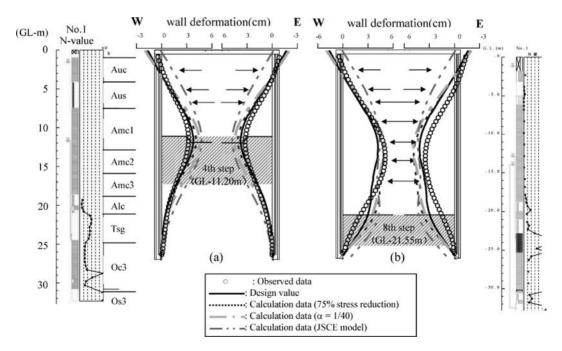


Figure 3. Comparison between observation and design value of braced wall deflection (B site, (a) : the 4th step, (b) : the 8th step).

ground surface. Especially, this construction site was located in the Neyagawa lowland, and it is peculiar that the very soft and sensitive alluvial clay layer (*N*-value  $\cong$  0, liquid limit  $I_L = 0.6$  to 1.5, cohesion c = 30 to  $100 \text{ kN/m}^2$ ), which is specific for the East side of Osaka Plain, deposited with 15 to 20 m thickness. The upper Pleistocene sandy and gravel layer Ts & Tg and the lower Pleistocene sandy layer Os3 (Osaka Group) constitute the second waterbearing layer under the alluvial layer.

The cross section of B-site is shown in Figure 4, the wall and struts conditions are shown in Table 3 and the soil parameters are shown in Table 4.

In this construction site, the seepage control method was adopted by extending the earth retaining wall to the low permeable layer Oc7 (about GL-42 m), too. The bottom depth of Soil Mixing Wall (H-steel) was extended to the Os8. The excavation width is 17.2 m,

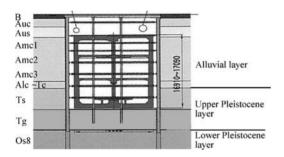


Figure 4. Cross section (B site).

the final excavation depth is about GL-22 m and the penetration depth is about 5 m.

Figure 5 shows the comparison between the observed data and design value of the earth retaining wall deflection in the 4th and 8th excavation steps (Oota *et al.*, 2007). The wall deflection distribution mode was similar both side in the 4th excavation step. However the amount of the observed wall deflection was two times of the design value. Moreover, in the 8th excavation step, the wall deflection distribution mode was different in both and observed data exceed the design value. In addition, it was confirmed that the stress in the wall was controlled under the allowable stress.

Table 4. Soil parameters (B si
--------------------------------

Soil layer	Bottom depth (GL-m)	N-value	Cohision c (kN/m <sup>2</sup> )	Internal friction angle $\phi$ (°)	<i>E</i> <sub>50</sub> (MN/m <sup>2</sup> )
В	0.8	2	0	19.9	_
Auc	2.0	0	27	0	2.2
Aus	4.0	2	0	19.9	_
Amc1	8.0	0	42	0	2.2
Amc2	13.0	0	63	0	5.5
Amc3	16.0	0	76	0	7.4
Alc	19.0	3	73	0	5.6
Tc	20.8	7	129	0	_
Ts	23.3	42	0	37.5	_
Tg	26.0	45	0	38.2	_
Os8	39.1	60	0	41.8	-

Table 3. Earth retaining wall and each strut (B site)	Table 3.	Earth retaining	wall and	each strut	(B site).
-------------------------------------------------------	----------	-----------------	----------	------------	-----------

Soil n	nixing wall (H-st	eel) condition						
Size (mm)			Length <i>L</i> (m)	<i>EI</i> (kN·m <sup>2</sup> /m)	Area A(m <sup>2</sup> )			
H-588	$8 \times 300 \times 12 \times 20$	0 0.60	27.52	399000	0.01925			
Excav	vation condition	Strut conditi	on					
Depth	1	Depth		Size		Sman	Ditab	A #0.0
Step	(GL-m)	Stage	(GL-m)			Span L(m)	Pitch S(m)	Area $A(m^2)$
0th	1.42	Cover beam	0.42	H-488 × 30	$0 \times 11 \times 18$	17.15	2.00	0.01592
1st	2.81	1st	1.81	$H-300 \times 30$	$0 \times 10 \times 15$	16.55	2.59	0.01048
2nd	5.96	2nd	4.96	H-350 × 35	$0 \times 12 \times 19$	16.45	2.59	0.01549
3rd	8.26	3rd	7.26	H-350 × 35	$0 \times 12 \times 19$	16.45	2.59	0.01549
4th	11.51	4th	10.51	H-350 × 35	$0 \times 12 \times 19$	16.45	2.59	0.01549
5th	14.51	5th	13.51	H-350 × 35	$0 \times 12 \times 19$	16.45	2.59	0.01549
6th	17.21	6th	16.21	$H-400 \times 40$	$0 \times 13 \times 21$	16.35	2.59	0.01977
7th	19.61	7th	18.61	$H-400 \times 40$	$0 \times 13 \times 21$	16.35	2.59	0.01977
8th	21.70	-	_	-		_	-	-

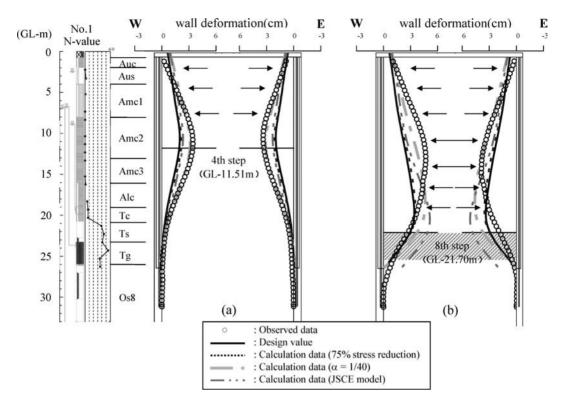


Figure 5. Comparison between observation and design value of braced wall deflection (B site, (a) : 4th step, (b) : 8th step).

As the ground condition under the bottom of the excavation is considered as the plastic zone in the calculation using JSCE model, the bottom of the wall deformed towards the excavation side in a larger value and the wall deflection distribution had different phenomenon compared to the observation. In the excavation stage at the Amc layer, the calculation considering the 75% stress reduction under the 5 m from the excavation bottom was shown together in Figure 5. Unlike the comparison result in A-site, this calculation was similar to the design because wall deflections around the bottom of the excavation are in the plastic zone. It was impossible to explain the observed phenomenon adequately used by some calculation model.

The horizontal coefficient of subgrade reaction  $k_h$  of clayey ground for excavation side in the OMTB model is determined by equation (2). This setting method of  $k_h$  was the empirical equation based on many observed data in the case that the wall deflection was about 3 cm (Yanagida *et al.*, 1981). This reference bring up the problem that  $k_h$  is tend to decrease due to the increase of the wall deflection.

In the actual construction site, as  $k_h$  is depended on the ground mechanical characteristics and some boundary conditions and so on, it is known that  $k_h$  changes every second due to braced excavation. For example,  $k_h$  is determined as solid line by the wall deflection function taking into consideration the nonlinearity (Japan Road Association, 1986).

The inverse analysis based on some earth retaining monitoring results was carried out to estimate the value of  $k_h$ . Modified Pawell Method was employed for the inverse analysis. It is possible to obtain the optimized solution stably on the many unknown parameter problem (Kishio, *et al.*, 1995). Input values for the inverse analysis are earth retaining wall deflection (angle of inclination) and axial force of struts, and output values are lateral pressure on the earth retaining wall and  $k_h$ .

Figure 6 shows the inverse analysis results based on the monitoring data in Osaka Subway Line No.8 touched to the Kishio, *et al.*, 1997. The vertical scale is the ratio of the estimation value  $k_h$  by the inverse analysis to design value of  $k_{h0}$ . In other words,  $k_h/k_{h0} = 1$ means the inverse analysis results and design value are the same.

In the case that the wall deflection was about 1 cm, the relation between  $k_h$  and  $k_{h0}$  was about the same in both past actual results and Line No.8 results. In short, the applicability of  $k_h$  in the design is reasonable.

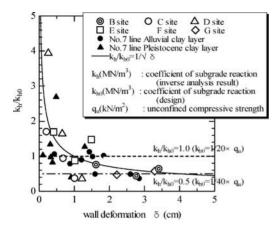


Figure 6. Dependence for brace wall deformation of  $k_h$  (touch in Kishio, *et al.*, 1997).

However in the case that the wall deflection was about 2 to 4 cm, inverse analysis results  $k_h$  are smaller than the design value  $k_{h0}$ , the ration  $k_h/k_{h0}$  decreased to about 0.5.

The relation of  $\alpha = 1/40$  was presumed on the assumption that  $k_h$  decrease due to the increase of the wall deflection. Figure 5 shows the recalculation results under this relation.

In the case of the 4th excavation step, the relation between observation and recalculation was in good agreement. In the case of the 8th excavation step, the wall deflection distribution mode between observation and recalculation was still different, but the maximum amount of wall deflection was similar. It is believed that the cause of differences in the wall deflection distribution mode is the limit explained by the beam spring model in design.

In accordance with these estimations results, it is preferable to consider the  $k_h$  setting method carefully as equation (3) with considering the traditional design idea in the case of earth retaining design in the soft and sensitive clayey layer, which *N*-value is about 0 to 2, with thick layer (about 10 to 20 m).

$$k_h(MN/m^3) = \alpha \times q_u (\alpha = 1/20 \text{ to } 1/40)$$
 (3)

#### 4 CONCLUSIONS

The results are shown as follows;

 At the A-site, the observed wall deflection in the east and west side are symmetric till the 4th step, and it is confirmed that the design value estimates the actual phenomenon adequately. However since the 5th step, the observed data exceeded the design value. It was assumed that the plastic zone expanded drastically to the penetration area in design.

- 2. It was possible that the calculation result considering the 75% stress reduction under the 5 m from the bottom of the excavation explained the observed data to some extent. However, under the bottom of the excavation, the tendency that the design value and calculation result exceeded the observed data. It was considered that one of the reasons for these tendencies is the deformation at the bottom of wall towards the excavation side.
- 3. At the B-site, the wall deflection distribution was similar between the observation and design in the 4th excavation step. However the observed wall deflection is two times of the design. In the 8th step, the wall deflection distribution mode was different in both, and observed wall deflection exceeded the design value.
- 4. Owing that  $\alpha = 1/40$  was presumed on the assumption that  $k_h$  decrease due to the increase of the wall deflection, the relation between observation and recalculation was in good agreement in the case of the 4th excavation step. In the 8th excavation step, the wall deflection distribution mode in both was still different, but the maximum wall deflection was close.
- 5. It is recommended that the  $k_h$  setting method carefully as equation (3) with considering the traditional design idea in the case of earth retaining design in the soft and sensitive clayey layer, which *N*-value is about 0 to 2, with thick layer (about 10 to 20 m).

#### REFERENCES

- Hashimoto, T., J. Nagaya, J. Kishio and T. Shiotani : Investigation of Strength Degrading due to Swelling of the Ground in Excavation, *Proc. of the Int. Conf. on Foundation Failures*, pp.393–398, 1997.
- Japan Road Association: Design Guideline for Underground Multipurpose duct, 1986 (in Japanese).
- JSCE : Standard Specifications for Tunneling-2006, Cut and Cover Tunnels, pp.142–181, 2006 (in Japanese).
- Kato, S., T. Konda and H. Shinkai: Effect of Suction Reduction Caused by Wetting Process on Shear Strength Characteristics Under Low Confining Pressure, *JSCE Journals* C, Vol.62, No.2, pp.471–487, 2006 (in Japanese).
- Kishio, T., N. Nakai, H. Arimoto, T. Konda and K. Takami : Inverse analysis example of Braced Wall used by Modified Pawell Method, Proc. of the 50th JSCE Annual Meeting, III-520, pp.1040–1041, 1995 (in Japanese).
- Kishio, T., H. Oota, T. Hashimoto, T. Konda, E. Saito and N. Kobayashi : Estimation of Lateral Pressure and Coefficient of Subgrade Reaction during Excavation Work in Osaka, *JSCE Journals*, No.560, VI-34, pp.107–116, 1997 (in Japanese).

- Kishio, T., H. Oota, T. Hashimoto and T. Konda : Some Aspects of Designing Earth Pressures for Braced Wall under the Bottom of Excavation, *Tsuchi-to-Kiso JGS*, Vol.45, No.10, pp.20–22, 1997 (in Japanese).
- Oota, H., H. Ito, T. Yanagawa, T. Konda and T. Hashimoto: Consideration of Design Method for Bracing Excavation Based on Monitoring Results, *Proc. of the 42nd JNCSFE*, 2007 (in Japanese).
- Osaka Municipal Transportation Bureau : Design Guideline for temporary structure (draft), 1993 (in Japanese).
- Yanagida, S., T. Watanabe, I. Yamaguchi, H. Nakamura and S. Mizutani : The Study of Lateral Earth Pressure for Earth Retaining Design (Part II), *Proc. of the 16th JNCSFE*, 1449–1452, 1981 (in Japanese).

### Ground movements in station excavations of Bangkok first MRT

#### N. Phienwej

Asian Institute of Technology, Bangkok, Thailand

ABSTRACT: The characteristics of movements of diaphragm wall and ground in the excavation of 18 stations of the first Bangkok underground MRT line were evaluated. Three modes of deflected shapes of the walls were observed at different excavation depths, namely cantilever mode, braced modes with bulge in soft clay and bulge in stiff clay. The ratio of maximum lateral wall deflection developing with excavation depth and the ratio of ground surface settlement to excavation and the normalized zone of ground surface settlement varied with the mode of wall deflection. Back-calculation of undrained soil moduli for different soil layers were made from wall movement data of three selected stations using the 2-D linear elasto-plastic FEM analyses. The modulus values, which were higher than those commonly obtained from conventional triaxial tests, can be used as guideline for future works in Bangkok.

#### 1 INTRODUCTION

Deep excavation by means of strutted concrete diaphragm walls is often used for construction of multi-level building basements in Bangkok soft soil. It is superior to the excavation with steel sheet piles for control of ground movement to avoid damages to adjacent structures. Prior to the construction of the first Bangkok MRT project, few studies were made on the characteristics of the ground movement and its prediction (e.g. Phienwej and Gan, 2003 and Teparaksa et al, 1999. etc.). However, it was the implementation of the first Bangkok MRT subway line that provided systematic and comprehensive monitoring data on the excavation of station boxes that allowed indepth evaluation on the characteristics of the wall and ground movements associated with deep excavation in Bangkok subsoil conditions using strutted diaphragm walls. The project involved the deepest ever excavation made in Bangkok to date. The excavations of all station boxes were fully instrumented. Monitoring data from 18 station excavations were compiled, summarized and interpreted.

The construction of the first MRT underground project in Bangkok, the Mass Rapid Transit Initial System Project (MRT ISP Blue Line) was started in 1998. Prior to that there were public doubts on technical viability of the construction and operation safety of underground MRT in Bangkok soft soil. That pessimistic outlook led to a call for an in-depth investigation on the application and performance of the excavation method and support systems to be integrated in the excavation in soft and subsiding Bangkok ground. The contracts made its compulsory that full instrumentation program be implemented during excavation for design verification and safety assurance. Evaluation on the actual performance at sites was performed to confirm the sufficiency in the design of the support systems for the MRT station excavations. A comprehensive study was made on the aspect of the wall and ground movements (Hooi, 2003) and the salient points from the study are reported herein.

#### 2 PROJECT DESCRIPTION

The ISP Blue Line is the first underground MRT line constructed in Bangkok. It comprises 22-km-long twin single-track tunnels, 18 stations and a depot. The horizontal alignment mainly follows the right of way of city roads. Construction of the underground structures was implemented under two fast track design-built contracts, each having approximately the same amounts of work. The South Contract involved construction of a twin bored tunnels from the inter-city railway terminal at Hua Lamphong eastwards for 5 km beneath the busy Rama IV road to the Queen Sirikit National Convention Center, then 4.5 km north beneath the narrow business Asoke road, and Ratchadaphisek road ending on the surface near the depot. Works of the North Contract continued for 4.5 km north along Ratchadaphisek road to Lad Phrao road then turned west to Chatuchak Park and finally terminated beneath the Bang Sue

railway station. The 18 stations of the project are as follows:

South Contract	North Contract		
1. Hua Lamphong Station	1. Thiam Ruam Mit Station		
2. Sam Yan Station	2. Pracharat Bumphen Station		
3. Silom Station	3. Sutthisan Station		
4. Lumphini Station	4. Ratchada Station		
5. Bon Kai Station	5. Lad Phrao Station		
6. Sirikit Centre Station	6. Phahonyothin Station		
7. Sukhumvit Station	7. Mo Chit Station		
8. Phetchaburi Station	8. Kamphaeng Phet Station		
9. Rama IX Station	9. Bang Su Station		

#### 3 GROUND CONDITIONS

Bangkok is situated on the southern part of the low lying Chao Phraya plain, which extends north from the coast line at the Gulf of Thailand up to approximately 350 km and spans east-westward up to 150 km. The flat topography plain is covered with a thick marine clay layer, which overlies a very thick series of alluvial deposits of alternating stiff to hard clay and dense to very dense sand to gravel. The thick soft clay layer generally extends from the ground surface to a depth of 12 to 15 m. The soft clay which is known as "Bangkok soft clay" has high water content (70–120%), high plasticity, low strength and high compressibility.

The shallow subsoil of the upper 35 m zone is relatively uniform and generally consists of layers of soft to medium clay, stiff to hard clay and sand. Below this shallow zone, alternating layers of stiff to hard clay and dense sand exist to a great depth. Bedrock is found at depths more than 450 m. The typical subsoil profile for the first 50 m depth is listed as follows:

- Made Ground
- Bangkok Soft Clay
- First Stiff Clay
- Medium Dense Clayey Sand, Sandy Clay and Silty Clay
- Very Stiff Sandy Clay/First Bangkok Sand
- Second Hard Clay
- Second Bangkok Sand

Maconochie (2001) summarized the general soil profile and properties at the Bangkok MRT ISP Blue Line. The variation in soil profile along the alignment was observed primarily in the Very Stiff Sandy Clay/First Bangkok Sand layer and the soils immediately below it. Figure 1 show the soil profile along the MRT alignment and at the stations.

Deep well pumping in Bangkok and its environs has reduced the pore water pressures in the sand layers by approximately 23 m from the original hydrostatic profile. The groundwater pumping has also created regional subsidence throughout Bangkok metropolis. At the locations of the project, a perched water table is typically encountered in Made Ground. Below this horizon, hydrostatic conditions are generally found to a depth of approximately 8 m to 10 m depending on the location and thickness of First and Second Sand layers. Typically, the upper few metres of sand layers underlying First Stiff Clay and Very Stiff Clay have been dewatered.

#### 4 STATION CONSTRUCTION

The MRT stations had following features:

- Typically, three levels of structure, with a centre platform that is fed by stairs and escalators between two lines of columns down the middle of the station.
- Up to 230 m long and approximately 25 m wide, excavated up to a depth of 25 m to 30 m below the ground surface.
- The perimeters were of diaphragm walls, 1.0 m thick and 30 m to 35 m deep and solid in-situ reinforced concrete slabs, typically 0.9 m thick for the roof slabs, 0.7 m thick for the intermediate slabs and 1.75 m thick for the base slabs, which were used as the excavation support system and permanent structures later on.
- There were three stacked stations, at Samyan, Silom and Lumphini Stations, due to space constraints caused by the existence of the foundation piles of the long road flyover and a water transmission tunnel at these busy intersections.
- There was a side platform station at Bang Su Station, with two levels only, to accommodate the track alignment for future elevated extension of the line to the north.
- The remaining stations were constructed as Centre Island Platform stations.
- Pracharat Bamphen and Sutthisan Stations incorporated intersection road underpasses on the roofs of the stations.
- Silom, Lad Phrao and Phahonyothin Stations were excavated underneath foundation piles of existing road flyovers and thus, the station structures and foundation were designed to support the flyovers via cross-beam and underpinning bored piles.

The top down construction technique was adopted for all station box excavations with diaphragm walls and concrete slabs as the excavation support system, which was later utilized as the permanent structures of the stations. The designs were made with an aid of FEM analyses. The excavation depths and the toe depth of the diaphragm walls of all station excavations

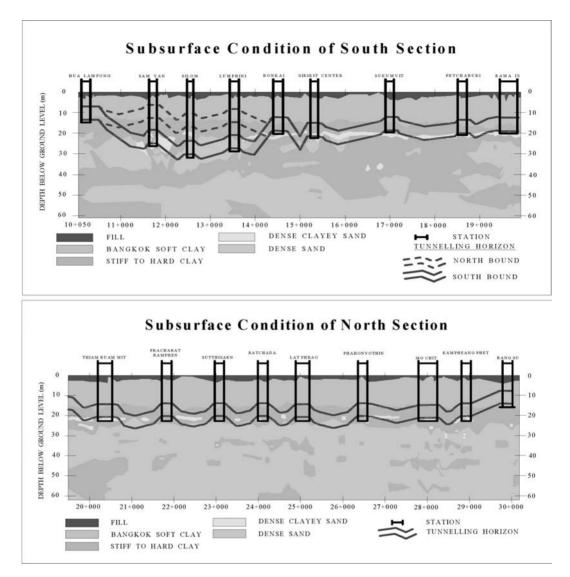


Figure 1. Soil profile along MRT alignment and at the stations.

are summarized in Fig. 2. It should be noted that the ratio of the depth of the wall embedment to the depth of excavation was significantly different between the two contracts, primarily due to the difference in the design criteria adopted by the two different designers.

#### 5 INSTRUMENTATION DATA

The measurement data from inclinometers and surface settlement points were compiled and interpreted to evaluate the overall performance of the station excavations in Bangkok subsoil using diaphragm walls. The data were screened to preclude movements not expressly related to the excavation and support installation, such as diaphragm wall construction, temporary decking works and the initial 2.3 m excavation that involved driving sheet piles, backfilling and extracting the sheet piles subsequently. A detailed study of the instrumentation data obtained was undertaken by comparing observed ground movement among the station excavations. Factors that may result in such patterns of data was examined and deduced, such as:

- Station box configuration and dimension
- Construction sequences

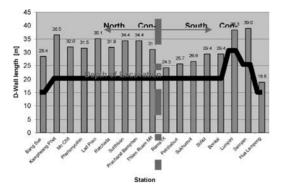


Figure 2. Depth of excavation and toe depth of D-wall.

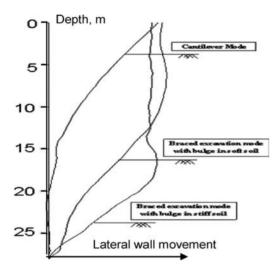


Figure 3. Modes of wall deflection.

- Variation in soil profile and properties
- Temporary works or presence of structures adjacent to excavation box

#### 5.1 Lateral wall movement

In general, most inclinometers all station excavations showed that the cantilever mode was the most predominant of wall deflection shape at the initial excavation stage, while the braced excavation mode developed in the subsequent stages as the excavations were deepened. Figure 3 shows the three modes of deflected shape of wall movement, which occurred at different excavation depths. The cantilever mode was most common during the first excavation stage. The braced excavation mode with bulge in soft soil prevailed at the second and third excavation stages. This mode continued to dominate the pattern of lateral movements for North Contract, but data of South Contract exhibit that the braced mode with bulge in the underlying

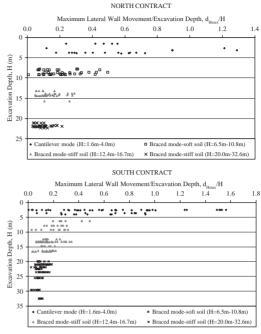


Figure 4. Normalized maximum wall deflection versus excavation depth.

stiff soil layers was more predominant. This different behavior may be attributed to the difference in soil profiles. In addition, South Contract had three deep station excavations of vertically stacked platforms.

The maximum lateral wall deflection of the 18 station excavations were in the range of 10-47 mm. The monitoring data also showed that there were significant variations in the shape and magnitude of the lateral wall movement within some of the station excavations. The variation may be attributed to a number of factors including the variation in soil profile and properties over the plan area of the station, adjacent temporary surface works, and confinement from road pavements and buried utility structures. The area of the excavation was quite large (about 25 m wide and 200 m long). For the three stations adopted for the detailed analysis in this study, the variations in the maximum lateral wall movements were as follow: 28-38 mm for the deepest Silom Station, 15 to 40 mm for Sirikit Station, and 18-27 for Thiam Ruam Mit Station. Figure 4 summarizes the ranges of the normalized maximum lateral wall movement with excavation depth ( $\delta_{Hmax}/H$ ) versus excavation depths recorded by all inclinometers at the 18 stations. The plots show that in the first stage cantilever mode excavation the value of  $\delta_{\text{Hmax}}/\text{H}$  was as high as 1.60. The maximum value of  $\delta_{Hmax}/H$  decreased with the increase in the excavation depth in the braced modes of the

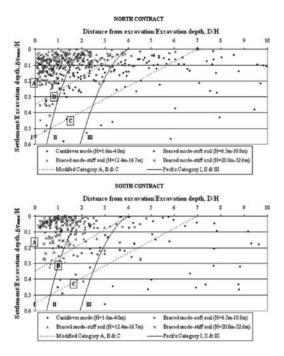


Figure 5. Normalized settlement versus distance from wall.

walls. The decrease in the value with depth was due to the change in soil condition at the excavation bottom from soft clay to stiff clays as the excavation depth increased. When the excavation depth reached stiffer soils,  $\delta_{Hmax}/H$  decreased and the range of the value became narrow. When the excavation bottom was still in the soft clay layer, the maximum value of  $\delta_{Hmax}/H$ was smaller than 0.5. The value was smaller than 0.2 when the excavations were deeper, in stiff clays.

#### 5.2 Ground surface settlement

The maximum ground surface settlement was observed at 58 mm at Bon Kai Station and 75 mm at Pracharat Bumphen, in South and North Contracts, respectively. Figure 5 shows plot of the ratio of maximum settlement to excavation depth ( $\delta_{vmax}$  /H) versus the distance from excavation normalized by the excavation depth (D/H). For shallow excavations under cantilever mode of wall deflection, the zone of ground settlement may extend up to D/H of 7 to 10. As for the excavation depths while the wall deflection was under the braced mode with bulge in soft clay layer, the  $\delta_{\text{vmax}}/\text{H}$  may extend up to D/H of 7. For deeper excavations while the wall was deflected under the mode of bulge in stiff clay layer, the zone of ground settlement may extend up to a distance of D/H of 4 and  $\delta_{\text{vmax}}/\text{H}$  value may be as high as 0.35 Under the braced mode in soft clay,  $\delta_{vmax}/H$  value may reach 0.55. For shallow excavation under the cantilever mode the value of  $\delta_{\text{vmax}}/\text{H}$  can be higher. The characteristics of ground surface settlement behind the excavation with diaphragm walls can be categorized according to the mode of wall deflection of excavation depth. The zones of ground settlement for the three modes of wall deflection in Bangkok soil are marked in figure. In addition, the three categories of ground movement in braced wall excavation for flexible walls (sheet piles or soldier piles) suggested by Peck (1969) are also shown in the plot. The level of ground settlement in excavation with diaphragm wall in Bangkok soft soil is much smaller than that predicted by Peck's chart for flexible wall. However, the influence zones of ground settlement were wider than those suggested by Peck's. This finding can be used as a general guideline for prediction ground surface settlement from deep excavation with diaphragm walls in Bangkok subsoil condition.

#### 6 PREDICTION OF MOVEMENTS

For the design of the MRT station excavations of both contracts, FEM analysis were made to determine ground movement and forces on the diaphragm walls and bracings. The analyses mainly utilized linear elasto-plastic Mohr-Coulomb soil parameters. The parameters were obtained from the soil investigation and testing program made for each station excavation. Triaxial compression tests as well as pressuremeter tests were conducted to determine the values of soil modulus for design analysis. The instrumentation data provided valuable information to evaluate the appropriateness of the soil model and the soil parameters used in the design calculation. In this study, monitoring data from three representative station excavations, i.e. Silom, Sirikit and Thiam Ruam Mit Stations, were examined in details and suitable soil parameters were back-calculated using a continuum FEM analysis. Computer code PLAXIS 2D was adopted in the study. Effective stress strength parameters were adopted in the undrained analysis. The drawdown phenomenon of the piezometric levels was considered in the simulation.

Silom Station was the deepest excavation in Bangkok to date (32.6 m deep), with a vertically stacked platforms thus it had four levels of slab below the roof. The station was designed to underpin the existing flyover roadway running over the station length. A dense sand layer of the first Bangkok Sand was encountered from depth 8.5 m above the final excavation level. Hence, the excavation required dewatering. The diaphragm wall was toed into the Second Sand layer.

Sirikit Station had the typical configuration of the centre island platform with three levels below the roof slab. The first stage and final excavation depths of 3.65 m and 23.6 m respectively, which were similar to majority of other stations.

Thiam Ruam Mit Station was selected because the soil profile was slightly different from those found in the first two stations. The area had a thicker First Stiff Clay layer with lenses of clayey sands. In addition, the first stage excavation was very shallow with roof slab was only at 1.8 m depth.

The back-calculation using the lateral ground movement data from the excavations of the three stations suggested the suitable undrained soil modulus parameters as follows.

- Soft and Medium Clay :  $E_u = 500 C_u \text{ kN/m}^2$
- First Stiff Clay :  $E_u = 700 N_{60} kN/m^2$
- Clayey Sand and Silty/ :  $E_u = 900 N_{60} kN/m^2$ Sandy Clay
- $\begin{array}{ll} \mbox{ Second Hard Clay} & : E_u = 1600 \ N_{60} \ kN/m^2 \\ \mbox{ Third Hard Clay} & : E_u = 2500 \ N_{60} \ kN/m^2 \end{array}$

where  $C_u$  is the corrected field vane shear strength and  $N_{60}$  is the corrected SPT "N" value according to Liao and Whitman (1986).

These back-calculated values of soil modulus are higher than those commonly obtained from conventional laboratory triaxial tests. It reflects the modulus values at low strain level which would be the dominating response of soil in the excavation problem (Mair, 1993). The soils would be mainly under unloading condition of stresses.

In similar early studies, Phienwej and Gan (2003) and Teparaksa (1999) both proposed the same modulus parameter of the soft clay as  $E_u = 500c_u$ . While for stiff clay, the value of  $E_u = 1200C_u$  and  $2000C_u$ were suggested, respectively. Based on the relationship of  $C_u = 0.6N_{60}$  kN/m<sup>2</sup> typically used for Bangkok subsoil, the parameters are equivalent to  $E_u = 720N_{60}$ and  $1200N_{60}$  kN/m<sup>2</sup>, respectively. The back-calculated values from this study were comparable to those suggested by Phienwej and Gan (2003).

#### 7 CONCLUSIONS

The following conclusions can be drawn from the study of the wall and ground movements in the excavation of the stations of the first Bangkok MRT underground.

- Three modes of deflected shapes of the wall movement were observed at different ranges of excavation depth. Mode 1: Cantilever mode (H = 1.6 m-4.0 m), Mode 2: Braced mode with bulge in soft clay layer (H = 6.5 m-11 m), and Mode 3: Braced mode with bulge in stiff soil (H = 12.4 m - 32.6 m).

- The maximum lateral wall movement ( $\delta_{Hmax}$ ) was smaller than 47 mm in both contracts. The normalized wall deflection,  $\delta_{Hmax}$ /H in the cantilever mode of movement was as high as 1.60, while it was reduced to no more than 0.60 and 0.40 in the latter stages of excavation when the wall deflection developed in the braced mode with bulge in soft clay and braced mode with bulge in stiff soil, respectively.
- The maximum ground surface settlement ( $\delta_{Vmax}$ ) was 58 mm for South Contract and 75 mm in North Contract. The normalized maximum ground settlement with excavation depth,  $\delta_{vmax}$ /H, was smaller than 0.55 and 0.35 for Modes 2 and Mode 3 wall deflection, respectively. The normalized distance from excavation of the zone of ground settlement, D/H, varied from 7.0 and 4.0 for the two modes of wall deflection. In the initial excavation stage of cantilever mode, the values of both normalized settlement and distance of ground movement were higher that those in the braced modes.
- Back-calculation of soil moduli of different soil layers using monitoring data from three selected stations showed higher values than those commonly obtained for conventional laboratory tests. The values are: Soft and Medium Clay:  $E_u = 500c_u$ , First Stiff Clay  $E_u = 700 N_{60} \text{ kN/m}^2$ , Clayey Sand and Silty/Sandy Clay  $E_u = 900 N_{60} \text{ kN/m}^2$ , Second Hard Clay  $E_u = 1600 N_{60} \text{ kN/m}^2$ , Third Hard Clay  $E_u = 2500 N_{60} \text{ kN/m}^2$

#### REFERENCES

- Hooi, K.Y. 2003. Ground Movements Associated with Station Excavations of the First Bangkok MRT Subway. Master Thesis, Asian Institute of Technology, Bangkok.
- Liao, S. & Whitman, R.V. 1986. Overburden Correction Factor for SPT in Sand. Journal of Geotechnical Engineering. *American Society of Civil Engineers* 112(3): 373–377.
- Mcconochie, D. 2001. Geotechnical Completion Report MRTA Chaloem Ratchamongkhon Line. A CSC Report submitted to the MRTA.
- Mair, R. 1993. Developments in geotechnical engineering research: application to tunnels and deep excavations. Unwin Memorial Lecture 1992, *Proceedings of Institution* of Civil Engineers, Civil Engineering, 93, Feb: 27–41
- Phienwej, N. & Gan, C.H. 2003. Characteristics of Ground Movements in Deep Excavations with Concrete Diaphragm Walls in Bangkok Soils and their Prediction. *Journal of The Southeast Asian Geotechnical Society* 34(3): 167–175.
- Teparaksa, W., Thasnanipan, N. & Tanseng, P. 1999. Analysis of Lateral Wall Movement for Deep Braced Excavation in Bangkok. *Proc. of AIT 40th Anniversary Conference*, AIT, Bangkok, Thailand.

### Numerical modelling and experimental measurements for a retaining wall of a deep excavation in Bucharest, Romania

#### H. Popa, A. Marcu & L. Batali

Technical University of Civil Engineering, Geotechnical and Foundations Department, Romania

ABSTRACT: Although civil engineers dispose of various calculation methods for retaining structures, none of them have definitely imposed itself, each one bringing its own benefits or limitations. Finite Element Method (FEM) offers the benefits of complex models allowing taking into account the majority of soil – structure characteristic parameters. However, the experience shows that the differences between the experimental and the calculation results are often quite important. The paper presents the case history of a diaphragm wall for a deep basement of a new building in centre of Bucharest. Nearby the new building there is an ancient cathedral – historic monument. The deep excavation is also neighboring at different distances with another existing buildings and a heavy trafficked road. All these conditions led to choose the "top-down" technology in execution of the basement. The numerical results obtained by FEM are compared with the measurements recorded during the construction. The differences between the obtained values (displacements) are comprised between 15% and 75%, depending on the enclosure sides. The main factors leading to these differences are the soil parameters.

#### 1 INTRODUCTION

The effect of deep excavations on neighboring structures can become important and therefore special measures have to be taken in the design and monitoring of retaining walls.

Calculation of such structures must be based on methods taking into account the soil – structure interaction and, with this respect good soil knowledge is indispensable.

Paper presents the case of a diaphragm wall for a deep basement in the very centre of Bucharest. The basement is developed on 4 underground levels and needs an excavation of about 15 m deep. The groundwater level is at about 6 m depth. The ground is composed of alluviums layers comprising medium soft silty clays, as well as fine to coarse medium dense sand. Near the new construction, at about 6 m distance, there is an ancient cathedral, classified as historical monument. As well, the pit has on another side some buildings, while on the other two sides it is delimited by a heavy traffic road. All these conditions led to choose the "top - down" technology for building the infrastructure, in order to have minimum deformations and displacements of the wall, so that the integrity of the neighboring buildings not being affected.

#### 2 WORK AND SITE DESCRIPTION

#### 2.1 Site and geometrical characteristics

The new building is located at the centre of Bucharest, next to the Romano – Catholic Cathedral St. Joseph. Figure 1 presents a photo of the site.

As it can be seen on the photo, on the Western side the pit is very close to the St. Joseph Cathedral (about



Figure 1. Location of the deep excavation.

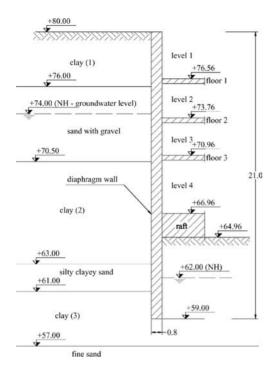


Figure 2. Cross section through the diaphragm wall.

6 m distance), on the Southern side there is a building complex at about 10 m distance, while on the two other sides there is a public road with heavy traffic.

Figure 2 shows a cross section through the buildings' infrastructure, presenting also the ground lithology.

The diaphragm wall is made of panels 80 cm thick and 21 m deep. The embedment depth has been established considering the wall stability and excavation bottom imperviousness. Dewatering has been performed only inside the enclosure, the groundwater level outside being left unchanged to avoid undesirable settlements of the ground around the pit due to dewatering, which would be in addition to the inherent settlements due to excavation inside the enclosure and to the erection of the new structure, which could affect especially St. Joseph Cathedral.

#### 2.2 Geotechnical characteristics of the ground

The geotechnical parameters are specific for Bucharest area, which is characterized by alluvium soils. The triaxial tests performed with imposed stress path allow a direct determination of the shear strength parameters  $(\phi, c)$ , of the secant modulus (*E*) and of the earth coefficient at rest ( $k_0$ ) for the clayey layers; for the sands these parameters have been established based on SPT tests.

Table 1. Design values for the geotechnical parameters.

Stratum	Thickness, m	γ kN/m <sup>3</sup>	E, MPa	φ' (°)	c' kPa	<i>K</i> <sub>0</sub>
Clay (1)	4.00	19	25	25	30	0.7
Sand & gravel	5.50	20	40	38	0	0.4
Clay (2)	7.50	20	50	22	50	0.7
Silty clayey sand	2.00	20	75	28	20	0.5
Clay (3)	4.00	20	75	22	50	0.4
Fine sand	7.00	20	75	36	0	0.4

The ground lithology and the design values of the geotechnical parameters are shown in table 1 (level 0.00 m represents ground level).

The groundwater level is +74.0 m and, according to the site investigations, it can vary with  $\pm 1.00$  m. A second aquifer, confined, has been found between  $+63.0 \div +61.0$  m (within the sandy layer).

#### 3 NUMERICAL MODELLING

#### 3.1 Numerical model

Numerical modeling has been performed using 2-D FEM, the model having 1933 elements and 5866 nodes.

For the soil, a perfect elasto-plastic constitutive law has been used, with Mohr – Coulomb criteria, using the geotechnical parameters issued from laboratory and in situ tests.

#### 3.2 Calculation stages

Calculations have been organized in 6 stages, following the technological phases:

- phase 0 initialization of the stress state in the ground;
- phase 1 excavation down to the lower level of the first floor slab; execution of the first floor slab;
- phase 2 excavation below the first floor slab down to the lower level of the second floor slab and dewatering inside the enclosure; execution of the second floor slab;
- phase 3 excavation between floor slabs no. 2 and 3; execution of floor slab no. 3;
- phase 4 excavation below the third floor slab down to the final level (-15.00 m).

#### 3.3 Results

Figure 3 presents the evolution of the horizontal displacement of the wall as a function of the execution stages.

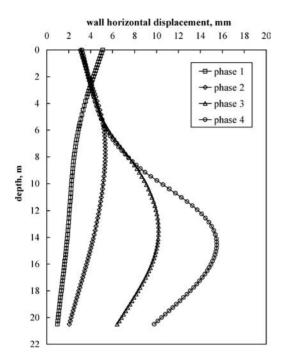


Figure 3. Horizontal displacements function of calculation phases.

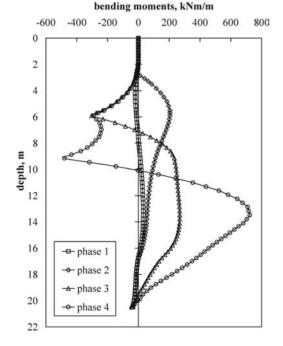


Figure 4. Bending moment function of calculation phases.

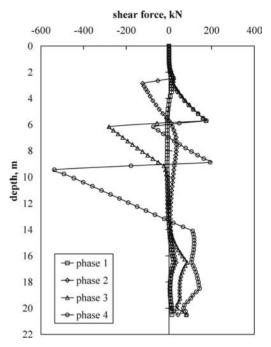


Figure 5. Shear force function of calculation phases.

Table 2. Floor slab reaction forces.

	Force, kN/ml				
	Floor 1	Floor 2	Floor 3		
Phase 1	_	_	_		
Phase 2	-142.2	_	_		
Phase 3	2.3	456.5	_		
Phase 4	6.7	249.2	729.2		

According to the calculations, the maximum horizontal displacements of the wall are of about 15 mm, at 15 m depth. Due to the "top-down" technology, the shape of the displacement curves shows greater values in the lower part of the wall. The displacements of the upper part are practically blocked by the already built floor slabs.

The upper maximum horizontal displacements are estimated to be of  $5 \div 6$  mm.

Bending moment and shear force graphs are shown figures 4 and 5, respectively.

The reaction forces on the basements' floor slabs are shown table 2.

#### 4 MEASUREMENTS

In order to record the influence the deep excavation and, moreover the whole new building, has on the

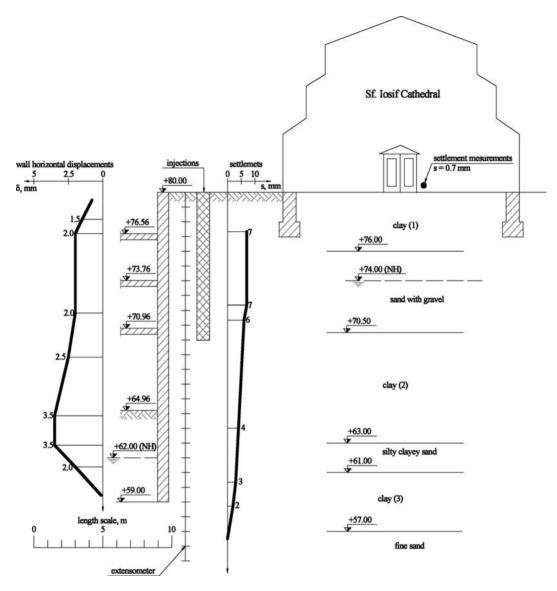


Figure 6. Experimental measurements.

neighbouring structures, a monitor of the displacements has been performed.

The retaining wall was equipped on each side with inclinometers for measuring the horizontal displacements and, thus, the walls' deformation.

Marks were installed on the surrounding buildings to monitor their settlements. As well, extensometers were installed, just behind the wall for measuring grounds' settlements.

Another monitoring concerned the groundwater level outside the enclosure. For this purpose, wells were drilled and equipped as piezometers, located on each side of the enclosure. The measurements showed that the dewatering works inside the enclosure didn't affect the groundwater level outside.

Figure 6 shows the monitoring equipment used for the St. Joseph Cathedral side.

It can be seen that, in order to reduce furthermore the risks of a negative influence of the retaining wall on the Cathedral, between these two first a stabilization wall was built using cement-based injections.

Figure 6 shows also the measured lateral displacements of the wall during the last excavation stage (stage 4). It can be seen that the shape of the displacement curve corresponds to the one obtained by numerical calculation. But the values are much less than the estimated ones.

So, at the upper part of the wall, the maximum displacements are of about 2 mm, representing about 50% of the estimated ones, while at the lower part the differences are more important, the measured values being only 25% of the calculated ones.

This important difference between calculations and measurements can be due also to the protection injection screen located on this side of the enclosure. On the other sides, where no such protection has been installed, the maximum walls' displacements were of about 13 mm, being quite closed to the estimated value (85%).

Anyway, estimations are still higher than the measured values for all excavation stages.

Concerning the ground settlement behind the retaining wall (at about 1 m distance), extensioneters showed a maximum value at ground level of about 7 mm. The settlement evolution versus the depth can also be seen in figure 6 and one can note that it becomes negligible at about 2-3 m below walls' toe.

Marks fixed on the St. Joseph Cathedral indicated a maximum settlement of 0.7 mm, its integrity not being endangered.

From this point of view it can also be noted the beneficial role of the injection screen, the difference of settlements on one side and the other of the screen being substantial.

Concerning the other neighboring buildings located at about 10 m distance, maximum settlements of about 3 mm were recorded, insignificant for their stability.

#### 5 CONCLUSIONS

Retaining structures imply complex soil – structure interaction phenomena. A correct estimation of their behavior is possible only by using numerical models, allowing a complex modeling of the system formed by the retaining wall, foundation ground and neighbouring buildings.

Even when such methods are used, the results can present significant differences from the real behavior. The reasons for these differences are many, among them:

- incertitude regarding the geotechnical parameters used for the calculations, especially when complex constitutive laws are used for the ground;
- difficulty in estimation of the initial stress state in the ground, taking into account its lithology, the presence of neighbouring structures, the execution of the retaining wall itself etc.;
- complexity of the numerical model itself, considering all implied parameters;
- three-dimensional behaviour of the retaining structure.

In order to obtain reliable results using numerical modeling it is important to calibrate and validate the model based on experimental measurements performed on similar structures and in similar site conditions. The experience in such modeling is also an important aspect.

#### REFERENCES

- Marcu, A., Popa, H., Marcu, D., Coman, M., Vasilescu, A. & Manole, D. 2007. Impact deep excavations on neighboring buildings, National Conference AICPS, 1 June 2007 (in Romanian).
- Marcu, A. & Popa, H. 2004. Calculations and measurements of deformations and displacements of a retaining wall for a deep excavation and of the neighboring structures. 10th National Conference of Soil Mechanics and Foundation Engineering, 16–18 September 2004, Bucharest, Romania, pp. 311–322 (in Romanian).
- Popa, H. 2002. Contributions to the study of the soil structure interaction in case of underground structures, PhD thesis, Technical University of Civil Engineering Bucharest, p. 311.

# 3D finite element analysis of a deep excavation and comparison with in situ measurements

#### H.F. Schweiger

Computational Geotechnics Group, Institute for Soil Mechanics and Foundation Engineering, Graz University of Technology, Austria

### F. Scharinger & R. Lüftenegger

GDP Consulting Engineers, Graz, Austria

ABSTRACT: The paper describes the analysis of a deep excavation project in clayey silt in Salzburg. The excavation was supported by a diaphragm wall, a jet grout panel and three levels of struts. Because of insufficient information on the material properties of the jet grout panel the stiffness of it was varied in a parametric study. The effect of taking into account the stiffness of a cracked diaphragm wall on the deformations was also investigated. In some of the 3D calculations a non-perfect contact between diaphragm wall and strut was simulated by means of a non-linear behaviour of the strut. The evaluation of the results and comparison with in situ measurements showed that analyses which took into account the reduced stiffness of the diaphragm wall due to cracking achieved the best agreement with the measurements. Furthermore settlements of buildings could be best reproduced by the three-dimensional model.

#### 1 INTRODUCTION

Soft subsoil deposits in Austria are mainly fresh water deposits, sedimented in the post-glacial lakes after the boulder periods. These deposits are known as lacustrine clays on the foothills of the Alps. One example for a widespread lacustrine clay deposit is the basin of Salzburg, where the city of Salzburg is situated on subsoil sediments, which partly show a thickness up to 70 m, called "Salzburger Seeton", which can be classified as clayey silt.

In the design stage of deep excavations in such problematic soils finite element calculations are a useful tool to obtain reasonably realistic predictions of deformations expected. In practical engineering 2D-models are still prevailing, but 3D-model become increasingly attractive. It will be shown, and this is the main purpose of this paper, that the best overall match with in situ measurements, in particular with respect to surface displacements behind the wall, is achieved with 3Dmodels. If only wall deflection is considered also 2D analyses show reasonable agreement. The mechanical behaviour of the soil is modelled with an elasto-plastic constitutive model, namely the Hardening Soil model as implemented in the finite element code Plaxis (Brinkgreve 2002). For the project the "class A" 2D analysis predicted the overall deformation behaviour with sufficient accuracy from a practical point of view, but a more detailed comparison with in situ measurements has been made after construction involving 3D finite element analyses. Furthermore some details with respect to the strutting have been changed during construction which have not been taken into account in the original analysis.

The input parameters for the constitutive model have been determined not solely from site investigations but also from previous experience of finite element analyses under similar conditions (see e.g. Schweiger & Breymann 2005).

In the following a brief description of the problem will be provided together with the material parameter used. The different assumptions with respect to modelling the diaphragm wall and the jet grout panel are discussed. Finally results from various 2D and 3D analyses are compared with in situ measurements of wall deflection and surface displacements.

# 2 PROBLEM DESCRIPTION AND MATERIAL PARAMETERS

#### 2.1 Project description

A cross section of the excavation with strut levels and final excavation depth is shown in Figure 1. In plan the excavation is roughly square, approximately

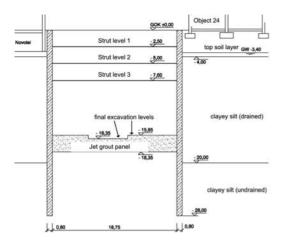


Figure 1. Cross section of excavation and strut levels.

 $19 \times 20$  m, which of course must raise doubts whether a 2D analysis is at all appropriate in this case. Attention is paid to the fact that a jet grout panel just below the final excavation level has been constructed to act as lateral support. This has been constructed before the start of the excavation and allowed excavation without installing a fourth strut level. Groundwater lowering inside the excavation was achieved by vacuum wells (commonly used in Salzburg) which extended below the excavation level in order to reduce uplift.

The construction sequence is closely reflected in the analysis. Starting from the initial stress state ( $K_0 = 0.55$  for all layers) and the loads of the foundations of the neighbouring buildings (80 kN/m<sup>2</sup> for the Novotel, 200 and 250 kN/m<sup>2</sup> for the strip footings of Object 24) the wall and jet grout panel have been introduced wish-in-place. Then excavation steps, groundwater changes and installation of struts have been modelled in a step by step analysis. Soil behaviour below -20 m is assumed to be undrained, above -20 m, due to the presence of thin sandy layers, as drained.

#### 2.2 Material parameters

The soil parameters used in the analysis for the top soil layer (0–4 m below surface) and the clayey silt are summarized in Tables 1 and 2. As mentioned previously, parameter determination is not only based on site investigations and laboratory experiments but also from experience of back analyses of other deep excavations in Salzburg. Therfeore soil parameters have not been varied in this study. In Table 1  $E_{50}$ ,  $E_{oed}$  and  $E_{ur}$  are the reference stiffness in primary loading (for deviatoric and oedometric stress paths) and unloading/reloading respectively.

The axial stiffness of the struts (Table 3) differs for the three levels, the material behaviour is assumed to

Table 1. Stiffness parameters for soil layers.

	E <sub>50</sub> (MPa)	E <sub>oed</sub> (MPa)	E <sub>ur</sub> (MPa)	m -	p <sub>ref</sub> (kPa)	ν <sub>ur</sub>
Soil layer	3	3	12	0.0	40	0.2
(0–4 m) Clayey silt	37.6	37.6	150.4	0.30	100	0.2

Table 2. Strength parameters for soil layers.

	c	φ	ψ
	(kPa)	(°)	(°)
Soil layer (0–4 m)	5	28	0
Clayey silt 1	30	26	0

Table 3. Axial stiffness of struts.

	EA (kN)	spacing (m)
Strut level 1	3.234E6	3
Strut level 2	1.067E7	3
Strut level 3	5.334E6	3

Table 4. Parameters for wall, jet grout panel and foundations.

	E (kN/m <sup>2</sup> )	ν _	R <sub>inter</sub>	UCS (N/mm <sup>2</sup> )
Diaphragm wall	2.9E7	0.2	0.7	18.8
Jet grout panel	5.0E5	0.2	0.7	2.25
Foundations	3.0E7	0.2	0.7	-

be linear elastic. Table 4 lists the basic set of parameters used for diaphragm wall, jet grout panel and the foundation structures of Novotel and Object 24. In the 2D analyses a Mohr-Coulomb failure criterion has been used for wall and jet grout panel whereas the cohesion was chosen in such a way to obtain the uniaxial compressive strength (UCS) as listed in Table 4, assuming  $\varphi' = 45^{\circ}$ . Tension cut-off was set to UCS/10. R<sub>inter</sub> denotes the reduction of soil strength to model wall friction. In the 3D analyses the wall was elastic and stiffness was either assumed to correspond to "uncracked conditions" or "cracked conditions". The stiffness properties of the jet grout panel have been varied because of the significant uncertainty in obtaining reliable values for the in situ stiffness of such panels.

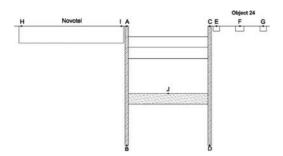


Figure 2. Location of points used for comparison.

#### 3 IN SITU MEASUREMENT PROGRAMME

The existence of structures in the close vicinity of the excavation required a careful observation of deformations during construction. Therefore, about 30 settlement gauges were installed to monitor the settlements outside the excavation, in particular of the adjacent buildings. In addition, four inclinometers were installed in the diaphragm walls in order to measure the horizontal deflection of the wall in all construction stages. Two of them were located approximately in the cross section chosen for the 2D analysis, i.e. along the centre line of the excavation. Figure 2 depicts the points chosen for the comparison of measurement and analysis for settlements.

#### 4 NUMERICAL MODELS

As mentioned previously 2D and 3D analyses have been performed using Plaxis 2D and Plaxis 3D Foundations. The 2D model consists of approximately 2,300 15-noded elements (Figure 3) and the 3D model of approximately 11,000 15-noded wedge elements (Figure 4). Lateral boundaries are fixed in horizontal direction and the bottom boundary in vertical and horizontal direction in both models. It can be seen that the 3D mesh is much coarser as compared to the 2D mesh but studies performed on the 3D model showed that a mesh with more than 20,000 elements resulted in only marginal differences in displacements. However, bending moments are more sensitive to discretisation and a stability analysis would certainly not yield correct results with the mesh adopted for the 3D analyses.

#### 5 RESULTS OF 2D MODEL

Four different analyses have been performed with the 2D model:

Variation 1 (V1): Wall and jet grout panel elastic with elastic properties according to Table 4.

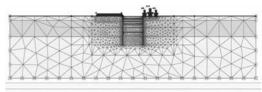


Figure 3. 2D finite element mesh.

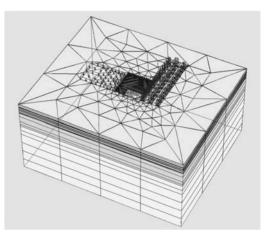


Figure 4. 3D finite element mesh.

Variation 2 (V2): Diaphragm wall modelled as elastic-perfectly plastic material with UCS as given in Table 4.

Variation 3 (V3): V2 and increase of stiffness of jet grout panel by a factor of 3.

Variation 4 (V4): V3 and increase of tension cut-off in diaphragm wall by a factor of 2.

Figures 5 and 6 compare the deflection of the wall for the final construction stage for all four analyses with the measurements obtained from the inclinometers.

It follows that the different assumptions made have little influence on the results in the upper part of the wall because in this part the deformations are governed by the struts. Results for the right wall compare well with measurements in the upper part, for the left wall this is not the case. For the lower part only V3 and V4 produce a reasonably match and it turned out that it is difficult to obtain the wall curvature as measured at the location of the jet grout panel.

Figures 7 to 9 show a comparison of calculated and measured vertical displacements at various points on the ground surface. The two sets of squares in each diagram represent pairs of settlement gauges which are in close distance to the points picked from the numerical analysis at various stages of construction (the dates are given within the diagram, the axis represents calculation steps, representing the progress of construction

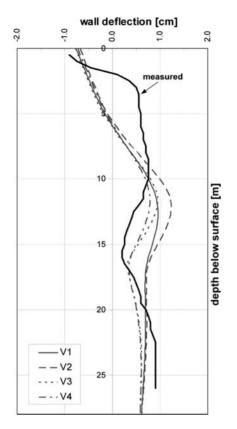


Figure 5. 2D analysis: wall deflection - left wall.

with time). Only for point H a reasonable agreement between calculation and measurements could be achieved, although one has to mention that absolute values are very small, with about 10 mm as maximum settlement. In point I the analysis predicts heave whereas settlements have been measured, but for point E settlements are overpredicted.

#### 6 RESULTS OF 3D MODEL

In this section results from 3D analyses are presented. These analyses have been performed because the geometry of the excavation (approximately quadratic in plan view) and also part of the bracing system (struts across the corners of the excavation) cannot be adequately represented in plane strain conditions. In the first series of analyses emphasis has been put on the stiffness of the diaphragm wall and 3 different calculations have been performed: the first assumed linear elastic behaviour for the wall with a stiffness assigned representing "stiffness I" (uncracked conditions), the second one assumed "stiffness II" (cracked conditions) and the third one introduced a non-linear behaviour by

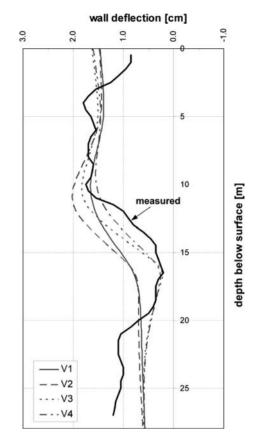


Figure 6. 2D analysis: wall deflection - right wall.

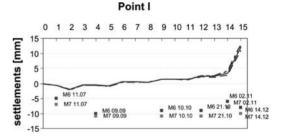


Figure 7. 2D analysis: surface displacements - Point I.

means of a pre-defined curve relating allowable bending moments to the curvature of the wall. In Figures 10 and 11 these are denoted with Z1, Z2 and non-linear respectively. It has been observed already in the 2D analyses that the assumption for the stiffness of the jet grouted panel has – as expected – a significant influence on the curvature of the wall. The inclinometer measurements indicate that the lower value – obtained from laboratory experiments – seems to underestimate the support in situ. This has been confirmed also from

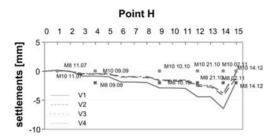


Figure 8. 2D analysis: surface displacements - Point H.

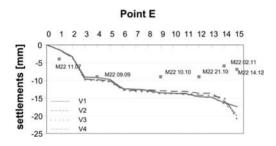


Figure 9. 2D analysis: surface displacements - Point E.

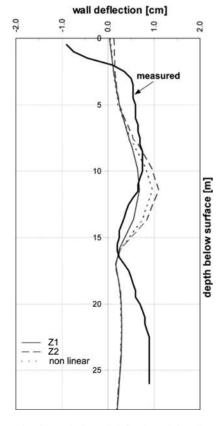


Figure 10. 3D analysis: wall deflection - left wall.

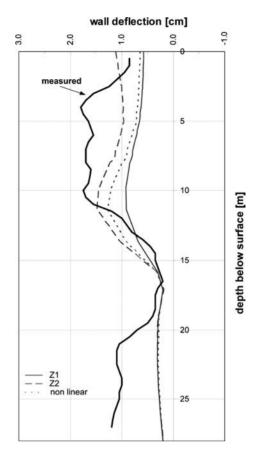


Figure 11. 3D analysis: wall deflection - right wall.

3D analyses and therefore only results assuming the high stiffness (1,500 MPa, as used in V3 and V4 of the 2D calculations) are presented in the following.

The comparison of horizontal displacements (Figures 10 and 11) clearly show the effect of varying the stiffness of the diaphragm wall in the unsupported zone whereas the assumption of "cracked stiffness" is closer to the measured curvature than the analysis with high wall stiffness, at least for the right wall. In the upper part the influence of varying wall stiffness is much less pronounced because the behaviour is dominated by the struts, however predicted horizontal displacements are less than measured. The non-linear model is, not surprisingly, between the two extreme cases.

Finally, after some discussion with the designer, an additional analysis was performed assuming a non-perfect connection of struts and wall, i.e. it was assumed that there is an imperfection before the full support of the strut can be mobilised. This has been achieved by a nonlinear model for the strut which

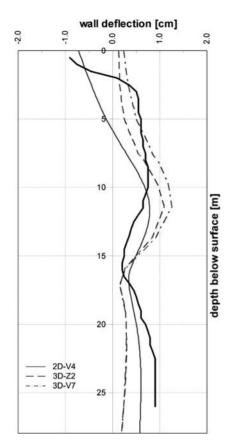


Figure 12. Comparison 2D-3D analysis - left wall.

results in a 0.25 mm/m "gap" to be closed before the full support is activated (this variation is denoted as V7 in the following diagrams). The consequence of this follows form Figures 12 and 13, in which results from the 2D analysis (Variation 4) are also plotted for comparison. For the left wall the curvature at the position of the grout panel is still not in full agreement with measurements but the upper part corresponds much better than in previous analyses. For the right wall the curvature and the upper part are now in reasonable good agreement with measurements (for the right wall the 2D analysis is also in good agreement). Figures 14 to 17 plot settlements at various observed points. It is immediately noticed that - in contrast to the 2D model - the 3D analysis predicts settlements also for Point I, although they are still slightly lower as compared to measured values. Point H corresponds in the sense that measured and calculated settlements are almost zero. Point E shows slightly higher settlements for later stages of construction than measured and the same holds for point G.

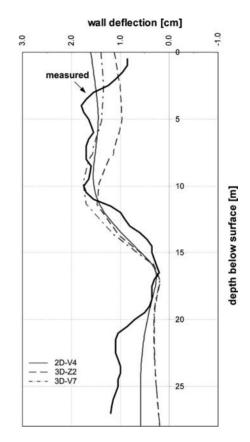


Figure 13. Comparison 2D-3D analysis - right wall.

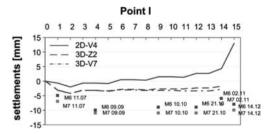


Figure 14. Comparison 2D-3D analysis: Point I.

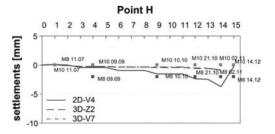


Figure 15. Comparison 2D-3D analysis: Point H.

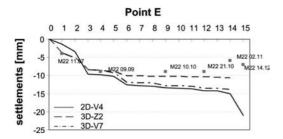


Figure 16. Comparison 2D-3D analysis: Point E.

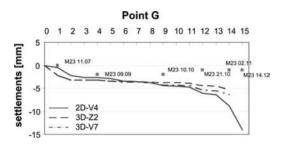


Figure 17. Comparison 2D-3D analysis: Point G.

#### 7 CONCLUSIONS

Results form 2D and 3D finite element analyses of a deep excavation have been compared to in situ measurements. The excavation is supported by a diaphragm wall, 3 rows of struts and a jet grout panel located just below the final excavation depth. In a parametric study the stiffness of the diaphragm wall and the jet grout panel have been varied. The study showed that a 2D analysis would reasonably predict wall deflections (in particular for the right wall) but if both walls and vertical displacements of all surface points are considered the 3D analysis produces a somewhat better overall agreement with the measurements.

#### REFERENCES

Brinkgreve, R.B.J. 2002. PLAXIS, Finite element code for soil and rock analyses, users manual. Rotterdam: Balkema.

Schweiger, H.F. & Breymann, H. 2005. FE-analysis of five deep excavations in lacustrine clay and comparison with in-situ measurements. In (K.J. Bakker, A. Bezuijen, W. Broere, E.A. Kwast, eds.), Proceedings 5th Int. Symp. Geotechnical Aspects of Underground Construction in Soft Ground, Amsterdam, June 15–17, 2005, Taylor & Francis/Balkema, Leiden, 887–892.

### The effect of deep excavation on surrounding ground and nearby structures

A. Siemińska-Lewandowska & M. Mitew-Czajewska

Warsaw University of Technology, Warsaw, Poland

ABSTRACT: In the paper problems related with the execution of 29 m deep excavation of Nowy Swiat Station (S11) of 2nd metro line in Warsaw are discussed. In the central section, Warsaw 2nd metro line runs below the center of the city (office and housing buildings and high traffic roads) as well as below Vistula river. This central section consists of 7 stations and 6 running tunnels – 6 km length in total. Running tunnels will be constructed using TBM, stations – cut and cover method. Deep excavation will be executed within diaphragm walls. The stability of the walls will be provided by several levels of slabs and struts. The analysis of settlements of ground surface, surrounding foundations and displacements of walls of the excavation have been made. Additionally, settlements of the surface were calculated above the TBM (running tunnels). Resulting values of settlements in both cases were compared and discussed.

#### 1 INTRODUCTION

Construction of 2nd Metro line in Warsaw is scheduled to begin in January 2008, announcement of design and build tender has been already published. In the central section, Warsaw 2nd Metro line crosses below the center of the city (office and housing buildings and high traffic roads) as well as below Vistula river. This central section consists of 7 stations and 6 running tunnels – 6 km length in total. Running tunnels will be constructed using TBM, stations – cut and cover method. In the paper problems related with the execution of 29 m deep excavation in Quarternary soils (silty sands, sands, clayey sands and Pliocene clays) are discussed. Within the excavation Nowy Swiat Station (S11) of the 2nd metro line will be built. The S11 station will be founded at the depth of 29 m below ground surface (b.g.s.) in the vicinity of so called "Warsaw Slope", where the denivelation (difference in ground surface levels) reaches 30 m (Figure 1).

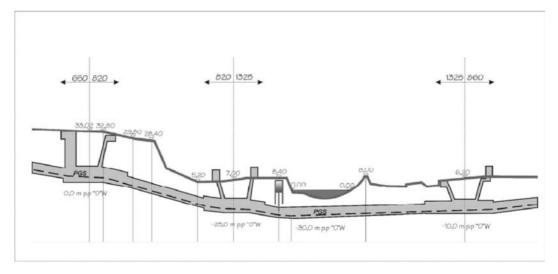


Figure 1. Longitudinal section of the central part of the 2nd Metro line in Warsaw.

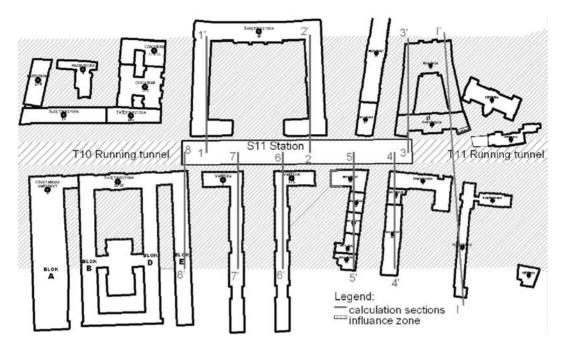


Figure 2. Longitudinal section of the central part of the 2nd Metro line in Warsaw.

The depth of the station is a consequence of sudden lowering of the tunnel from the upper slope level to the level below the bottom of the river with the consideration of appropriate soil cover resulting from TBM technology. Deep excavation will be executed within 100 cm thick diaphragm walls. The stability of the walls will be provided by several levels of slabs and struts. In the close vicinity of the excavation and above the tunnel there are many old buildings, such as:

- historic buildings built in XIXth century, partially destroyed during the 2nd World War and rebuilt after the war. Polish Academy of Sciences, Warsaw University and a Hospital are located there. These buildings are founded on spread foundations at a depth of 4,80 m b.g.s. Shortest distance between the excavation wall and foundation of the building amounts to 3 m;
- residential buildings constructed in 30. of XXth century, probably founded on piles. These buildings are located above the tunnel drilled using TBM;
- residential and office buildings constructed in 50. and 60. of XXth century on old pre-war foundations. These buildings are founded on spread foundations at a depth of 4,00 m b.g.s., 6 m apart from the excavation wall;
- masonry and concrete residential and office buildings constructed in 60. of XXth century, founded at a depth of 6,00 m b.g.s., 5 m apart from the excavation wall.

 Polish Central State Bank and the Ministry of Finance are located there.

Theses buildings are mostly masonry or reinforced concrete structures in good technical state. The majority of them is protected by the heritage conservator law. The location of the excavation of S11 Station, running tunnels and surrounding buildings is shown on Figure 2.

The analysis of settlements of ground surface, surrounding foundations and displacements of excavation walls have been made. Additionally, settlements of the surface were calculated above the TBM, T11 running tunnel (cross-section marked by green line). Figure 2 shows the location of all calculation cross-sections in the vicinity of S11 Station and T11 running tunnel.

#### 2 GEOLOGY

There are Quaternary and Tertiary soils in the area of the deep excavation of the station and running tunnels. According to the geotechnical investigations report, following geotechnical layers are distinguished:

- layer I uncontrolled fills 1,5–2 m thick, in some places up to 3 m;
- layer II moraine deposits reaching depth of 4 m b.g.s., consisting of medium and stiff sandy clays and clayey sands of Warta glaciation;

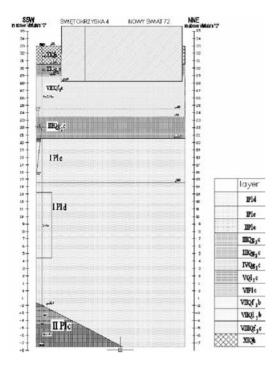


Figure 3. Calculation section N° 3.

- layer III medium sands and silty sands of Odra glaciation, to the depth of 10 m b.g.s.;
- layer IV moraine deposits reaching depth of 13 m b.g.s. consisting of medium and stiff sandy clays of Odra glaciation;
- layer V pliocen clays till the depth of 50 m b.g.s.

There are three levels of ground water table. Considering temporary stability of the bottom of the excavation, it was assumed that the water table would be lowered during construction of the station. Geotechnical conditions, distribution of soil layers and location of foundations are shown on Figure 3 and Figure 4. S11 station and T11 running tunnel are both located within the layer of stiff and very stiff Pliocene clays.

#### 3 DESCRIPTION OF THE DEEP EXCAVATION OF THE S11 STATION

It was designed that the deep excavation of S11 Station will be executed within 100 cm thick diaphragm walls, founded 10 m below the bottom of the excavation (that means the height of walls is 39 m). Due to the great depth of the excavation, amounting to 29 m, slab method of the excavation of the excavation was chosen in order to provide maximum safety of the construction works. The stability of diaphragm walls will be provided by 8 levels of 35 cm thick underground slabs. Vertical spacing of slabs is 3 m, which gives an opportunity to adopt underground surface for car parks and retail. Construction stages are considered as follows:

- execution of guide-walls, 1 m thick diaphragm walls and 1 m high reinforced conrete girt on the entire perimeter of the excavation,
- excavation till the depth of 2 m b.g.s., i.e. below the slab at level -1, execution of barrettes and temporary slab supports,
- construction of the slab at level -1, backfilling the excavation and allow traffic back,
- excavation till the depth of 5 m b.g.s., i.e. below the slab at level -2,
- construction of the slab at level -2,
- excavation till the depth of 8 m b.g.s., i.e. below the slab at level -3,
- construction of the slab at level -3,
- excavation till the depth of 11 m b.g.s., i.e. below the slab at level -4,
- construction of the slab at level -4,
- excavation till the depth of 14 m b.g.s., i.e. below the slab at level -5,
- construction of the slab at level -5,
- excavation till the depth of 17 m b.g.s., i.e. below the slab at level -6,
- construction of the slab at level -6,
- excavation till the depth of 20 m b.g.s., i.e. below the slab at level -7,
- construction of the slab at level -7,
- excavation till the depth of 23 m b.g.s., i.e. below the slab at level -8,
- construction of the slab at level -8,
- excavation till the depth of 26,5 m b.g.s.,
- installation of temporary struts at the depth of 26 m b.g.s.,
- final excavation till the depth of 29 m b.g.s.,
- construction of 1,5 m thick foundation slab,
- deinstallation of the temporary struts.

Calculations were made in 3 sections, chosen because of the vicinity of significant buildings.

Figure 3 presents example cross-section  $N^{\circ}$  3, located close to the beginning of the running tunnel (for the location of the section refer to Figure 2), showing geotechnical conditions and surcharges.

#### 4 DESCRIPTION OF THE T11 TUNNEL

Two versions of the tunnel structure has been considered: 1 tube including 2 tracks and 2 tubes, single track each.

The lining of the tunnel was assumed to be constructed of 40 cm thick segments. Following stages of the execution of the tunnel were modeled:

 initial stress including overburden and surcharges (buildings and traffic),  excavation of the tunnel and construction of the lining of the tunnel.

Figure 4 shows calculation cross-section  $N^{\circ}$  I-I (for the location of the section refer to Figure 2) including geotechnical conditions, tunnels (2 tubes) and location of existing buildings.

#### 5 CALCULATIONS

#### 5.1 Calculations of the excavation of S11 station

Finite element plain strain analysis were carried out using PLAXIS v. 8 software, Coulomb-Mohr constitutive soil model was chosen for modeling the soil body, diaphragm walls as well as slabs were modeled as 3-nodes, linear beam elements. Non-associated plastic flow law was considered. For modeling wall frictions Coulomb-Mohr low was used. Model dimensions are: 65 m (vertical), 100 m (horizontal), they were estimated taking into account polish regulations according to the range of influence zone of the excavation.

FEM model mesh, generated automatically, was built of 807 15-nodes triangle elements and 9773 nodes. For the purpose of the paper 3rd cross-section was chosen to be presented and discussed because of it's vicinity to the T11 running tunnel. Geotechnical conditions and location of existing buildings has been presented on Figure 3, FEM model is shown on Figure 5. Figure 6 presents maximum deformations of the model in the final construction stage. Maximum calculated lateral displacement of the diaphragm wall in section 3 amounts to 49,3 mm; maximum foundation displacement -30,6 mm.

Table 1 presents maximum calculated values of horizontal and vertical displacements of the wall as well as settlements of the surrounding buildings in 3 cross-sections chosen for calculation.

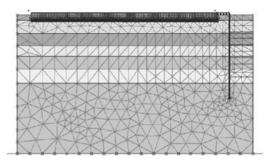


Figure 5. Numerical model - section 3 (PLAXIS).

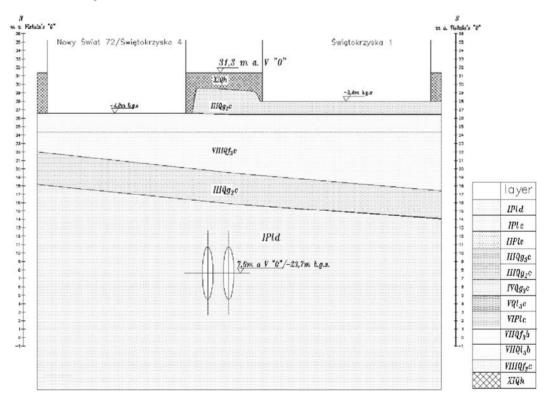


Figure 4. Calculation section No I-I (T11).

#### 5.2 Calculations of the T11 running tunnel

Place Finite element plain strain analysis were carried out using GEO4 TUNNEL software, Coulomb-Mohr constitutive soil model was chosen for modeling the soil body, tunnel lining was modeled using 3-nodes, linear beam elements. Non-associated plastic flow law was considered. For modeling wall frictions Coulomb-Mohr low was used.

Section I-I, 1 tube, 2 tracks:

model dimensions: 60 m (vertical) and 240 m (horizontal);

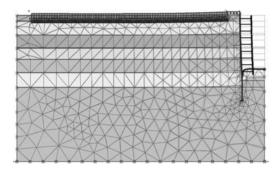


Figure 6. Final displacements - section 3 (PLAXIS).

Table 1. Results of calculations of deep excavation S11.

	Maximun displacem diaphragn	ients of	Maximum settlements of buildings U [mm]	
Section	Ux [mm]	Uy [mm]		
1-1 2-2 3-3	46,1 61,0 49,3	-32,8 -52,8 -53,3	24,5 35,2 30,6	

 FEM model mesh, generated automatically, was built of 7060 6-nodes triangle elements and 15011 nodes.

Section I-I, 2 tubes, single track each:

- model dimensions: 60 m (vertical) and 240 m (horizontal);
- FEM model mesh, generated automatically, was built of 8178 6-nodes triangle elements and 17284 nodes.

Geotechnical conditions, tunnels location (2 tubes, single track each case) and location of existing buildings has been presented on Figure 4, corresponding FEM model is shown on Figure 7. Figure 8 presents maximum deformations of that model.

Table 2 presents maximum calculated values of bending moments, and displacements of the lining as well as settlements of the surface and surrounding buildings.

#### 6 CONCLUSIONS

Taking into consideration results of analysis of the excavation of S11 Station as well as the results of T11 running tunnel calculations following conclusions are formed:

- In the vicinity of the 29 m deep excavation, which will be executed during the construction of S11 Metro Station estimated settlements of the surface and surrounding buildings amount to 24,5–35 mm.
- 2. Calculated settlements of the ground surface and surrounding buildings above the T11 running tunnel constructed by the means of TBM, taking into consideration both cases 1 two track tunnel and 2 single track tunnels are similar and amount to 37,5–37,8 mm.
- Theoretical values of settlements as well as displacements and forces in the structures were

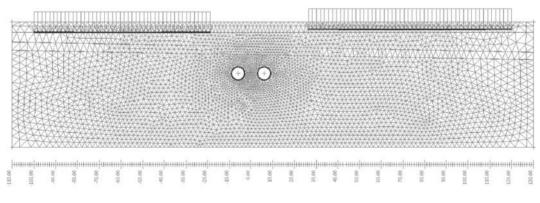


Figure 7. FEM model, T11 tunnel – 2 tubes, (GEO4 TUNNEL).

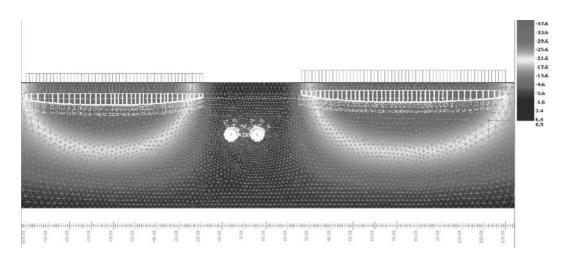


Figure 8. Maximum deformations of the model, T11 tunnel.

Table 2.	Results of	calculat	ions of 1	running 1	unnel T11.
----------	------------	----------	-----------	-----------	------------

	Finite elements method							
	Maximum bending m displaceme of tunnel l	oments, ents	Maximum surface settlements	Maximum settlements of buildings				
Type of tunnel	Mmax [kNm/m]	Umax [mm]	Umax [mm]	Umax [mm]				
1-1 (1 tunnel)	306,6	24,2	37,7	37,8				
(1 tunnel) 1-1 (2 tunnels)	290,2	8,1	37,5	37,6				

calculated considering that the value of the modulus of deformation of Pliocene clays, within which the structures are located, is E = 50 MPa. This value must be verified by means of in-situ tests and then the calculations will be adjusted.

4. Due to the expected differences in the values of settlements of the ground surface close to the deep excavation and above the tunnel further analysis of the case including 3D modeling of the contact of 2 types of tunnel structure (running tunnel and station) will be performed.

5. During the construction, the results of analysis described in the paper will be carefully verified and discussed.

#### REFERENCES

FINE Ltd. 2007. GEO4 User's manual. Prague: FINE Ltd.

- Geoteko Sp. z o.o. 2004. Evaluation of the technical state of buildings in the influence zone of 2nd metro line in Warsaw, section Rondo Daszyńskiego Station – Powiśle Station. Warsaw: Geoteko Sp. z o.o.
- Geoteko Sp. z o.o. 2004. Geotechnical and hydrological report for the construction of 2nd metro line in Warsaw, Nowy Świat Station. Warsaw: Geoteko Sp. z o.o., SGGW, Geoprojekt Sp. z o.o.
- Grodecki, W., Siemińska-Lewandowska, A. & Lejk, J. 2007. Second metro line in Warsaw – possibility and methods of realization, *Inzynieria i Budownictwo* 7-8/2007: 365–368.
- Kotlicki, W. & Wysokiński L. 2002. Protection of structures in the vicinity of deep excavations (376/2002). Warsaw: Building Research Institute.
- PLAXIS BV. 2005. *PLAXIS User's manual*. Roterdam: A. A. Balkema.
- Polish Committee of Standardisation. 2002. PN-EN 1538-2002 Execution of special geotechnical works. Diaphragm walls. Warsaw: Polish Committee of Standardisation.

# Multi-criteria procedure for the back-analysis of multi-supported retaining walls

#### J. Zghondi

Arcadis, Lyon, France and LGCIE, INSA-Lyon, France

## F. Emeriault & R. Kastner *LGCIE, INSA-Lyon, France*

ABSTRACT: A numerical back-analysis procedure for multi-supported deep excavations is proposed based on the optimization of several indicators, taking in account the forces in the struts and the differential pressures derived from the wall displacement. The evaluation of the procedure is performed on 1 g small scale laboratory experiments (Masrouri 1986) on semi-flexible retaining walls embedded in Schneebelli material. The proposed numerical procedure was applied on an excavation with 2 passives low stiffness struts. The resulting Hardening Soil Model parameters are further used to back-calculate the 14 different tested configurations. The results are compared with the classical methods, SubGrade Reaction Method, Finite Element analysis with Mohr Coulomb model with parameters proposed by Masrouri (1986) and with the back-analysis using Hardening Soil Model parameters based on triaxial tests results.

#### 1 INTRODUCTION

Numerical back-analysis of in situ monitoring results of multi-supported deep excavations is generally extremely complex (Hashash & Whittle 1996, Finno & Calvello 2005, Delattre 1999): soil characteristics can be heterogeneous or determined with a low degree of confidence, the different stages of the excavation can be difficult to reproduce in a 2D numerical approach, key mechanical parameters can be unknown (for example the actual stiffness of the strut-to-wall contact) and the number of measured quantities such as wall displacements, settlements and strut forces is generally too small to perform a comprehensive comparison between the actual behavior and the numerical results.

Thus the full validation of a back-analysis numerical procedure (including in particular the choice of the constitutive law and the determination of all the required parameters) is rarely directly possible on real case histories. Therefore, the numerical procedure proposed in this paper is validated on 1 g small scale laboratory experiments performed by Masrouri (1986) on semi-flexible retaining walls embedded in Schneebeli material (mixture of steel rods of different diameters representing in 2D the behavior of a cohesionless soil). The 14 considered experiments correspond to a retaining wall, whose length and mechanical properties are kept constant, supported by one or two levels of active or passive steel struts with various axial stiffness and prestressing. Even for such simple comprehensive laboratory experiments, usual design method like SubGrade Reaction Method (SGRM) or classical limit equilibrium methods do not capture all the observed behaviors and test results. It is thus necessary to propose a unified numerical procedure to back calculate with an acceptable degree of confidence, all the results of the excavations tests of Masrouri (1986).

#### 2 EXPERIMENTS AND COMPARISON WITH CLASSICAL/SGRM CALCULATIONS

The experiments correspond to small scale 2D models of flexible retaining walls (Figure 1).

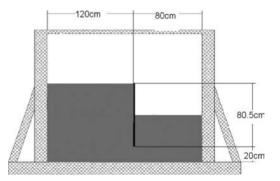


Figure 1. Masrouri's experimental set up.

	First strut		Second strut		
Experiments	Prestressed (kN/m.ml)	Stiffness (kN/m.ml)	Prestressed (kN/m.ml)	Stiffness (kN/m.ml)	
B1	2.133	83333	(not used)		
B2	3.025	83333	(not used)		
B3	0.208	816	(not used)		
B4	2.1	816	(not used)		
B5	3	816	(not used)		
B6	0.191	404	(not used)		
B7	2.016	404	(not used)		
B8	2.916	404	(not used)		
B10	0.383	83333	0.333	83333	
B11	1.330	83333	2.691	83333	
B12	0.366	816	0.400	816	
B13	2.075	816	2.700	816	
B14	1.733	816	4.041	816	

Table 1. Summary of Masrouri (1986) experiments.

Schneebelli 2D analogic soil was used: this material offer a good repeatability and enables to build a homogeneous 2D soil model with quick handling for the experiments. However Schneebeli materials have some inconvenients: the unit weight is close to  $6.5 \text{ kN/m}^3$ , the angle of friction is smaller than that of most of the soils (21°) and it only presents a dilatant behavior.

While maintaining the same geometrical and mechanical characteristics for the wall (EA = 1.2  $10^6$  kN/m and EI = 14.4 kN/m<sup>2</sup>/m), a wide range of configurations (1 or 2 struts, with different prestressing and stiffness) was considered, see Table 1. The phases used in all the excavations were planned as follows;

- 1. strut cases: 10 cm of excavation, installation of the first strut at -5 cm from the top and prestressing, then 3 excavations of 10 cm each till -40 cm, then several excavations of 5 cm until failure is obtained.
- 2. struts cases: the same procedure as for the 1 strut cases is followed until the excavation reaches -40 cm from the top, then the 2nd strut is installed at the level -25 cm and prestressed. The step-wise excavation (by increment of 5 cm) is resumed until failure occurs.

The strut to wall contact is hinged in order to prevent bending moment to be transmitted to the struts. For each excavation phase, the horizontal displacements of the top or bottom of the wall are measured. The curvature is also measured in 26 locations (both sides on the wall) allowing to determine with a reasonable accuracy the differential pressure acting on the wall from the top to the bottom (polynomial approximation performed by the Palpan program created by Boissier *et al.* 1978). Photographs were also taken to determine the displacement fields with a stereophotogrammetric technique.

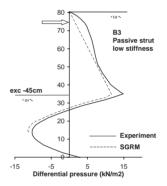


Figure 2. Differential pressure calculated with SGRM compared to experimental values: – Case B3 – low stiffness passive strut (–45 cm excavation level).

Load cells determine the forces on the cylindrical struts.

#### 2.1 Classical and SGRM calculations

The whole series of experiments were first compared with classical (modified Blum method) and SGRM calculations (Terzaghi 1955). The subgrade reaction modulus used in RIDO calculations (Fages 1996) is increasing with depth in the following manner:

 $K_h = K + K'\sigma_v$ . with K = 0,  $K_h = K'\sigma_v$ Classical methods calculation totally neglects the influence of the stiffness of the wall, construction steps, stiffness and prestressing on the struts, arching effect, etc. On the opposite SGRM explicitly considers the stiffness of the wall, the construction steps and the stiffness of the strut.

In the case of one passive strut with low stiffness (tests B3, B6), the SGRM method reproduces in an acceptable manner the differential pressures results (Figure 2).

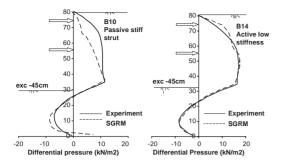


Figure 3. Calculated (SGRM Method) and measured differential pressure for B10 and B14 at -45 cm of excavation.

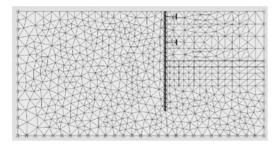


Figure 4. Mesh used for the Plaxis calculations of 2 struts supported walls.

With one stiff or active strut or in the case of excavations with 2 struts, both the SGRM and the limit equilibrium methods fail to reproduce the experiment differential pressures (Figure 3a - Test B10). In some cases, a good description of the pressure diagrams seems to be obtained (Figure 3b - Test B14). It actually results from two errors compensating each other: the over estimation of the pressure induced by prestressing and the lack of ability to reproduce the arching effect.

#### 2.2 Finite element back calculation with Mohr Coulomb model

It appears that the classical and SGRM methods do not, even in simple cases of excavation, accurately describe all the observed results. A finite element approach is therefore proposed.

The finite element calculations were performed with Plaxis V8.2, the model represents a vertical slice of Masrouri's experiment. The mesh is composed of triangular 15 nodes elements (Figure 4).

For these calculations, the same characteristics of the Schneebelli material were used (c = 0,  $\phi = 21^\circ$ ,  $\gamma = 65 \text{ kN/m}^3$ ), taking into account for the soil-wall interface an interface factor  $R_{int} = 0.55$  (related to the friction angle value noted by Masrouri).

Masrouri (1986) estimated the elastic modulus at 4500 kPa with an increment of 26830 kPa/m after

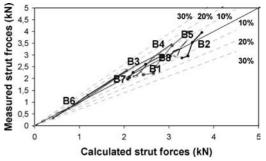


Figure 5. Strut forces for cases B1 to B8 with 1 strut – variation during the last 4 excavation phases before failure.

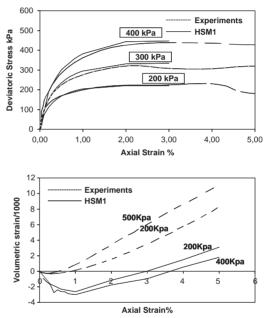


Figure 6. Simulation with HSM1 parameters of the biaxial tests with 200, 300 and 400 kPa.

0.6 m of depth (values back calculated on test B4 with 1 active strut of low stiffness).

In all the presented calculations the difference with the measured strut forces and wall displacements do not exceed 20% (see Figure 5 for the 1-strut cases B1 to B8).

A sensibility analysis is performed considering all the parameters of the Mohr Coulomb model. It appears that the differences between calculated and experimental results can not be satisfyingly reduced (especially in the cases with 2 struts). Therefore a more sophisticated constitutive law is required (Figure 6): the Hardening Soil model.

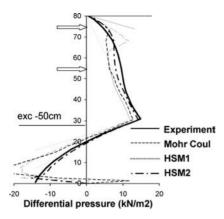


Figure 7. Differential pressure for test B12 and excavation at -50 cm: comparison of the experimental values with the results of SGRM, FE with Mohr Coulomb law and Hardening Soil Model (HSM1 and HSM2).

#### 2.3 FE back-calculation with Hardening soil model parameters based on triaxial tests

Hardening soil model (Shanz et al. 1999) can capture soil behavior in a very tractable manner. The values of the different parameters were first fitted on the biaxial tests results (under a 200 Kpa confining pressure) performed on  $20 \times 10$  cm samples by Kastner (1982). The 200 kPa confining stress is greater than the mean stress generally observed in Masrouri experiments, but in a first approach it was considered as more representative of the soil behavior than the biaxial tests using lower confining pressures: the size of the sample and the high value of the unit weight of Schneebelli material induces a non homogeneous stress state in the sample that could greatly affect the accuracy of the results. The obtained set of parameters noted HSM1 in the sequel provides a satisfying description of the biaxial test with 200 kPa or more confining stress (Figure 6).

This set of parameters was further used to back calculate the whole series of Masrouri's experiments (14 cases). It appears that these calculations do not reproduce well the test results in term of strut forces or differential pressures on the wall (Figure 7 represents only the results of differential pressures).

#### 3 PROPOSED BACK ANALYSIS PROCEDURE

The aim of this procedure is to find the proper set of parameters for the constitutive soil model (Mohr Coulomb or Hardening Soil models) considered in the Finite Element simulations. Considering one particular test configuration, the parameters will be first obtained from the minimization of indicators based on differential pressures and struts forces errors. The resulting set of parameters will then be confronted with the results of the biaxial tests and of 14

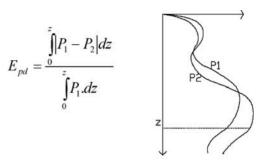


Figure 8. Differential pressure curves for indicator explication.

configurations considered experimentally by Masrouri (1986).

#### 3.1 Definition of indicators

In order to consider the main features of the retaining wall behaviour, two indicators were defined:  $E_{sm}$  is related to the strut forces and  $E_{pd}$  related to the differential pressure on the wall (linked to the displacement profile).

$$E_{sm} = \frac{E_{b1} + E_{b2}}{f_1' + f_2'} \tag{1}$$

where  $E_{b1} = f_1 - f'_1$  and  $E_{b2} = f_2 - f'_2$  and with  $f_{1(2)}$  and  $f'_{1(2)}$  the strut force calculated or measured in the 1st (2nd) strut level.

The  $E_{sm}$  indicator is based on the error of the sum of struts forces: therefore an error on the strut force  $f_1$  can be compensated by  $f_2$ .

The  $E_{pd}$  indicator (Figure 8) takes into account the absolute value of the difference between the measured and calculated differential pressures (respectively  $P_1$  and  $P_2$ ), divided by the integral of the measured differential pressure  $P_1$ . Integrals are calculated from the top of the wall to 10 cm below the final excavation level.

 $E_{pd}$  is the main indicator while the possible inaccuracy of the strut force measurement especially at the beginning of the excavation makes the  $E_{sm}$  indicator a second validation indicator.

#### 3.2 Parameter optimization for Mohr Coulomb model based on a 2 strut excavation test

The determination of a second set of parameters for Mohr Coulomb model is performed by fitting the final results of a 2 struts excavation case (excavation level -50 cm). The particular case of test B12 (corresponding to passive struts with low stiffness) was selected because it clearly appears that the SGRM fail

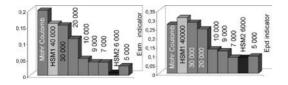


Figure 9. Variations of indicators  $E_{sm}$  and  $E_{pd}$  with  $E_{50}^{ref}$ .

to reproduce the differential pressure diagram obtained in such configuration.

Only one independent parameter was used for fitting the results, the elastic modulus E. The other parameters are kept constant (c,  $\phi$ ,  $\gamma$  ...). The final value of E is determined through the optimization of the indicators  $E_{sm}$  and  $E_{pd}$ . Actually, a linear variation of E with the depth is considered:

$$E = E_0 + \alpha z \tag{2}$$

Both the initial value of the elasticity  $E_0$  and its variation with depth  $\alpha$  were considered in the optimization procedure. It appeared that none of these parameters could be modified to improve the description of the experimental results, indicating that the Mohr Coulomb model is unable to reproduce important features of the soil behaviour involved in the global behaviour of the retaining wall.

## 3.3 HSM model optimized by fitting on a 2 strut excavation test

The determination of a second set of parameters for Hardening Soil Model (noted HSM2 in the sequel) is performed by fitting the final results of the same 2 struts excavation case (B12 excavation level -50 cm) as in section 3.2.

Only one parameter was used for fitting the results, the reference stiffness modulus  $E_{50}^{ref}$ . The other parameters are either constant (c,  $\phi$ ,  $\gamma$  ...) or keep a direct relationship with  $E_{50}^{ref}$ . For example:

$$E_{oed}^{ref} = 1.5 E_{50}^{ref}$$
 and  $E_{ur}^{ref} = 3 E_{50}^{ref}$  (3)

The final value of  $E_{50}^{ref}$  is determined through the optimization of the indicators  $E_{sm}$  and  $E_{pd}$ . Considering the accuracy of the experimental results (in particular the procedure leading to the differential pressure diagrams),  $E_{50}^{ref}$  is determined with a precision of 1000 kPa (Figure 9).

The value of  $E_{50}^{ref} = 6000 \text{ kPa}$  is chosen because it appears to minimize both indicators. HSM2 is further used to simulate the biaxial tests with low confining stress (50 and 100 kPa). Figure 10 shows a good agreement with the experimental results, even though the latter can be affected by the non-homogeneity of the initial stress state in 20 cm × 10 cm samples.

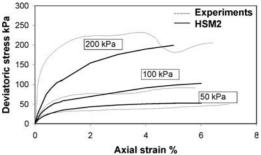


Figure 10. q- $\varepsilon_1$  biaxial curves: experimental results and calculated values with HSM2.

The proposed back analysis procedure shows its ability to verify or justify the HSM parameters that accurately describe the biaxial test results.

#### 3.4 Back calculation of the 14 configurations tested by Masrouri (1986)

The HSM2 set of parameters is now used to back calculate the 14 different tests (Table 1). Figures 11 and 12 present the differential pressure diagrams obtained with HSM2 and other methods and the ratios f/f' of the calculated to the measured strut forces respectively for tests B1 to B8 (1 strut) and tests B10 to B13 (2 struts).

The strut forces and the differential pressure are well represented compared to the classical methods, the SGRM method or Finite element analysis with a simpler constitutive model (Mohr Coulomb). The effect of the prestressing of the strut is well reproduced by that procedure (comparing B3 and B5 in Figure 11 or B10 and B11 in Figure 12), as well as the arching effect (case of B10 in Figure 12) and the influence of the strut stiffness (comparing B1 and B7 in Figure 11).

The proposed back analysis procedure and the resulting set of parameters HSM2 show their efficiency in all the configurations (unlike the SGRM where the arching effect and the prestressing on the struts are not well reproduced).

#### 3.5 Summary

Figure 13 presents a comparison of the values obtained for the selective indicator  $E_{pd}$  in 3 cases of excavation and for the SGRM, Mohr Coulomb, HSM1 and HSM2 models:

- B3 corresponds to a single passive and low stiffness strut
- B12 uses 2 passive low stiffness struts
- and B10 2 passive rigid struts.

In all of these 3 cases, it appears that the HSM2 set of parameters clearly minimizes the error between experimental and numerical results.

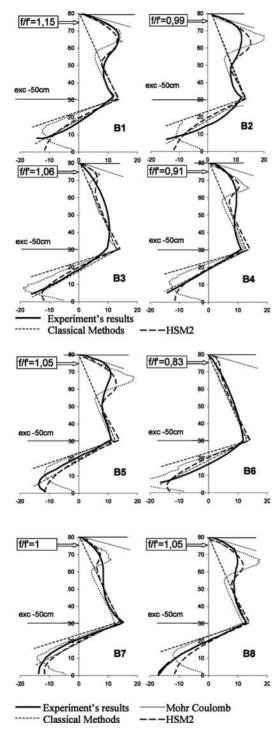


Figure 11. Differential pressure diagrams and strut forces obtained with HSM2 (excavations with 1 strut at -50 cm) compared with experiments, Mohr Coulomb, and classical methods results.

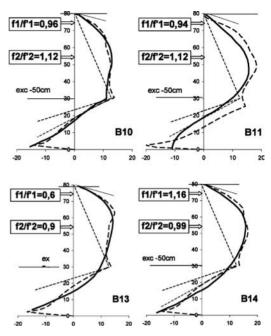


Figure 12. Differential pressure diagrams and strut forces obtained with HSM2 (excavations with 2 struts at -50 cm) compared with experiments and classical methods results.

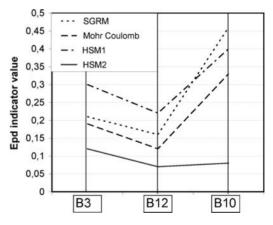


Figure 13.  $E_{pd}$  indicator calculated for tests B3, B12, B10 (excavation -50 cm), for the SGRM, HSM1, HSM2 and Mohr Coulomb calculation.

With the same constitutive model (HSM1 and HSM2) and with the same parameters except  $E_{50}^{ref}$ , the error is divided by 3 to 5.

#### 4 CONCLUSION

A comprehensive series of 14 small scale experiments on flexible retaining walls with different strut stiffness and prestressing is used to validate a numerical back calculation using the Hardening Soil Model. The final set of parameters HSM2 is fitted on one single test (B12 at the final excavation level). The proposed model is based on the simultaneous minimization of two indicators  $E_{sm}$  and  $E_{pd}$  respectively related to the strut forces and differential pressure diagram. The verification of the proposed back analysis procedure showed that the HSM2 model gives the most acceptable description of the differential pressures and forces on the struts in all of the 14 tested configurations compared to the SGRM method or a Finite Element approach with either Mohr Coulomb model or Hardening Soil model with parameters based on biaxial tests (HSM1).

Further developments will include the verification of the ability of proposed back-analysis procedure to determined the HSM parameters not only on  $E_{sm}$  and  $E_{pd}$  at the final excavation level but also on intermediate levels, for example the first excavation step after the installation and prestressing of the lower strut.

Despite the already mentioned difficulties, further validation of the procedure on well-instrumented excavation sites will also be tested.

#### REFERENCES

Boissier, et al. 1978. Détermination des moments et des pressions exercés sur un écran à partir de mesures inclinométrrique. Revue Canadienne de Géotechnique, 15, (4), 522–536. Brinkgreve, R.B.J. & Broere, W. 2004. Plaxis V8 manuel.

Delattre, L. 1999. Comportement des écrans de soutènement-Expérimentations et calculs. *PhD dissertation, ENPC*, *Paris, ENPC*, 498 p.

Fages, J. 1996 Rido - Users manual, RFL, Miribel, France.

- Finno, R. & Calvello, M. 2005. Supported excavations: observational method and inverse modeling. J. Geotech. Geoenv. Eng. ASCE, 131, (7): 826–836.
- Hashash, Y. M. A. & Whittle, A. J. 1996. Ground Movement Prediction for Deep Excavations in soft Clay. *Journal of Geotechnical Engineering*, 122(6): 474–486.
- Kastner, R. 1982. Excavation profonde en site urbain Problèmes liés à la mise hors d'eau- Dimensionnement des soutènements butonnés. PhD dissertation, INSA Lyon & Université Claude Bernard-Lyon 1, 409 p.
- Masrouri, F. 1986. Comportement Des Rideaux de soutènement Semi- Flexibles. PhD dissertation, INSA Lyon, France.
- Schanz, T., Vermeer, P.A., Bonnier, P.G., 1999. The Hardening-Soil Model: Formulation and verification. In: R.B.J. Brinkgreve, *Beyond 2000 in Computational Geotechnics*. Balkema, Rotterdam: 281–290.
- Terzaghi, K. 1955. Evaluation of coefficients of subgrade reaction, *Géotechnique*, 5: 297–326.

### Monitoring and modelling of riverside large deep excavation-induced ground movements in clays

#### D.M. Zhang & H.W. Huang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

#### W.Y. Bao

China State Construction Engineering Corporation (SH), Shanghai, P.R. China

ABSTRACT: The Riverside large deep excavation of Shanghai international passenger center was 800 m long and 100–150 m wide with the depth of 13 m. The south long side of the deep excavation was at a distance of 4.6 m from the parallel flood wall of Huangpu River. The north long side was 5 m away from a historic building. Problems resulted from the large deep excavation was the asymmetric ground movements along the long sides due to the complex surrounding condition and surface surcharge. The monitoring during the excavation provided numerous data to study the characteristics of the ground movement and earth pressure. The numerical modelling was also adopted aim to predict the ground movements.

#### 1 INTRODUCTION

The development of underground space along the bund of Huangpu River in Shanghai, China has resulted in excavations becoming progressively larger and closer to the River, where the groundwater table was just near the ground surface and a great number of underground works are within a few meters of the surface. The riverside excavations were all located close to the existing buildings, network and the city lifeline of flood wall. It has become a great challenge to protect these neighboring buildings and public utilities from damage during the deep excavation due to the complex geotechnical constraints and the small opening from the Huangpu River. The soils near the Huangpu River was usually weak with a very low strength and higher water content, which were a potential causes of the larger ground movement. Meanwhile, the complex and dense environments put forward a strict requirement on the ground movement controlling. It was difficult to determine the earth pressure acted on the retaining wall with any conventional earth pressure theory considering the small soil body left between retaining wall and flood wall. Besides, the retaining wall of riverside deep excavation was usually asymmetrically loaded with much higher earth pressure on one side, which was caused by great surface surcharge due to the existing buildings and the pile of the construction material. The stability of the deep excavation as a whole was worth considering to avoid any kinds of failure of the deep excavation and consequent damage on the environments.

However, there were few references for the construction of the large deep excavation because of the geotechnical condition and complex environment along the bund of Huangpu River. The deep excavation of Shanghai international passenger center (SIPC) was the largest and closest one to Huangpu River so far. The construction and the analysis method of the deep excavation of SIPC and the induced ground movement as well will be a useful and practical reference for the subsequent riverside large deep excavation.

#### 2 PROJECT OUTLINE AND SOIL CONDITIONS

#### 2.1 Project outline

The deep excavation of SIPC was 800 m long with the width of 100-150 m and the depth of 13 m. The large deep excavation was divided into two sub-excavations with the lengths of 480 m and 218 m respectively to reduce the risk of damage for the existing structure and the failure of deep excavation. The study presented in this paper was carried out based on the deep excavation with the length of 480 m. The sketch view of the project was illustrated in Figure 1. The space between the deep excavation and flood wall of Huangpu River was only

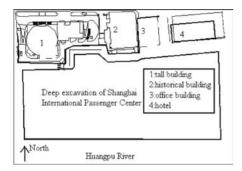


Figure 1. Sketch view of the project.

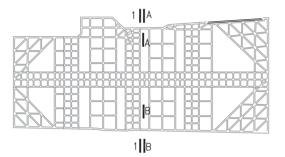


Figure 2. The plane view of strut arrangement.

4.6 m at south side. A number of existing buildings, including a historic building, were located along the north side of the deep excavation with a distance of about 5 m.

Bored piles supplemented by SMW piles wall were used as the retaining structure. The bored pile was 950 mm in diameter with a center-to-center space of 1150 mm. The effective length of the bored pile was 26 m and the embedment was adequate to provide sufficient passive earth pressure to keep the stability of the retaining wall. The SMW piles were 850 mm in diameter with an effective length of 20.8 m. The distance between SWM piles was 600 mm to guarantee the waterproof performance. The mix ratio of cement was as high as 20% for SMW piles. Three reinforced concrete struts were set at the depth of -0.9 m, -5.7 mand -9.6 m with the cross section of  $1250 \times 800 \text{ mm}$ for the first strut and of  $1200 \times 800$  mm for the second and third strut. The plane space of the strut was about 1.2 m and illustrated in Figure 2. The cross sections of the deep excavation were presented in Figure 3a and 3b.

The jet grouting belt of 4 m wide and about 4 m high was employed closely above the bottom of the deep excavation along the retaining wall. The grouting could significantly increase the capacity of the soil resistance for the retaining wall during the excavation. The bored pile was extended from 26 m to 27 m near

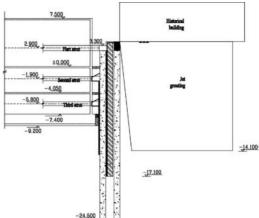


Figure 3a. A-A cross section of deep excavation.

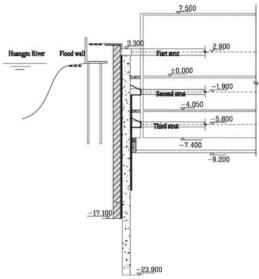


Figure 3b. B-B cross section of deep excavation.

the historic building to protect the building from crack and tilting. Besides, the isolation piles were specially designed to reduce to deep excavation-induced effect on the historic building.

#### 2.2 Soil conditions

The soil profile throughout the deep excavation comprises the mixed filling to a depth of 6.4 m, which contains many obstacles and made a lot of trouble for the deep excavation, underlain by silt, silty clay and mucky clay. The retaining wall including the waterproof wall was embedded in the silty clay. The detailed characteristics of the soils were presented in Table 1.

Table 1. Soils characteristics throughout the deep excavation.

Soil	Depth m	Water content %	Bulk density kN/m <sup>3</sup>	Compression modulus kPa	Cohesion kPa	Friction angle °
Mixed filling	6.38					
Silt	4.89	31.7	18.2	8170	8	28.5
Mucky clay with silt	4.96	40.5	17.5	4130	11	22
Mucky clay	7.17	49.7	16.6	2580	14	13
Silty clay 1	7.31	34.0	18.0	4570	17	17
Silty clay 2	4.05	33.3	17.8	8130	8	29
Silty clay 3	17.23	33.2	17.9	5240	17	23.5

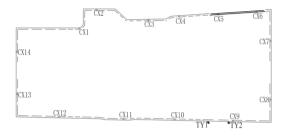


Figure 4. Monitoring layout of lateral displacements.

Table 2. Progress of excavation.

Date	corresponding excavation depth (m)
11/5/2005-11/11/2005	3
11/11/2005-12/2/2005	6.2
12/2/2005-1/3/2006	13
1/13/2006	completion of bottom plate

#### **3 MONITORING OF EXCAVATION**

As shown in Figure 4, 14 inclinometers, which were denoted by CX1 to CX14, were set into the retaining wall around the deep excavation. 2 earth pressure gauges denoted as TY1 and TY2 in Figure 4 were also installed close to the retaining wall to study the evolution of earth pressure of the small soil body between retaining wall and flood wall during excavation. The earth pressure gauges were installed every 5 m in the vertical overall 25 m.

The progress of excavation was presented in Table 2.

#### 3.1 Monitored lateral displacements

Figures 5–8 showed the lateral displacements at monitoring points of CX1, CX3, CX12 and CX10 corresponding to the studied excavation stages. These inclinometers were close to the center of the long side

lateral displacement/mm

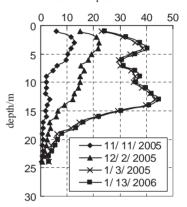


Figure 5. Lateral displacement at inclinometer CX1.

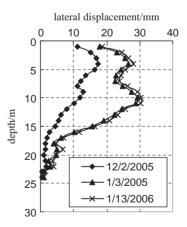


Figure 6. Lateral displacement at inclinometer CX12.

of the deep excavation and thus the readings were representative of the maximum displacement of the retaining wall. It could be found from figures 5–8 that the maximum lateral displacement was less than 60 mm during the whole excavation stage. Meanwhile,

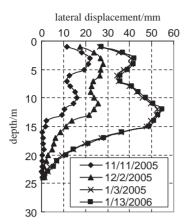


Figure 7. Lateral displacement at inclinometer CX3.

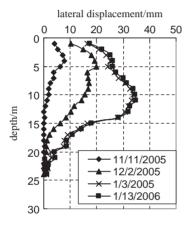


Figure 8. Lateral displacement at inclinometer CX10.

the lateral displacement exhibited asymmetric behavior along the two long sides of the deep excavation because of the following two reasons. Firstly, the earth pressure acted on the retaining piles was asymmetric because of the small soil body between Huangpu River and deep excavation. Secondly, the surface surcharge was asymmetric due to the existing buildings. Comparing the records of CX1 with CX12, CX3 with CX10, it could be found that lateral displacement of retaining wall was 15-29% smaller at the south side than north side. Unfortunately, the larger lateral displacement at north side would result in a potential damage to the neighboring historic building. Consequently, the jet grouting should be immediately carried out to improve the foundation of the historic building and it was proved to be an effective way to avoid the damage of crack and tilt of the building.

The recorded lateral movements at the top of retaining wall were presented in Figure 9. The five movement

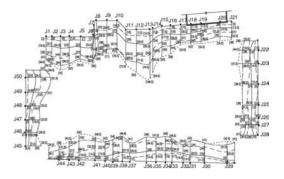


Figure 9. Distribution of lateral movement at the top of retaining wall.

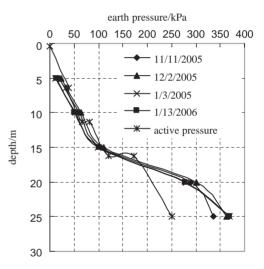


Figure 10a. Comparison of recorded earth pressure with calculated active earth pressure at TY1.

curves from outside to inside were corresponding to the excavation depth of 2.3 m, 6.2 m, 13 m, completion of bottom plate and completion of underground structure respectively. The maximum lateral movement at the top of retaining wall reached 106.5 mm at the north side, while it was only 50 mm at the south side, when the underground structure was completed. These phenomena also confirmed the influence of asymmetric earth pressure on the movement of the retaining wall.

#### 3.2 Evolution of earth pressure

The monitored earth pressure was illustrated in Figure 10a and 10b with calculated one. The calculated active earth pressure was obtained using Rankine earth pressure theory.

From figure 10a and figure 10b, it could be found that the monitored earth pressure at top 15 m was very

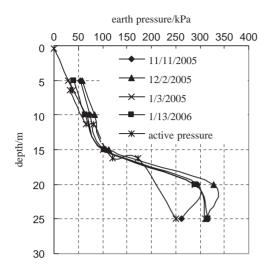


Figure 10b. Comparison of recorded earth pressure with calculated active earth pressure at TY1.

close to the active pressure for both monitored points. It became much larger than active pressure below 15 m. Two factors could be contributed to the distribution of the earth pressure. One was that the magnitude of lateral displacement of the retaining wall was larger in the top 15 m than that of below 15 m, and it could be verified from Figures 5–8. The other cause was that the small bulk of the soil body against the retaining wall at top 15 m. It could be found from Figure 3 that the soil body in the top 15 m was much smaller than in below 15 m. Both figure 10a and figure 10b implied that the soil body had a significant effect on the distribution of earth pressure against the retaining wall.

However, the earth pressure at north side of the deep excavation was not monitored. No comparison could be performed between the two sides.

#### 4 MODELLING OF DEEP EXCAVATION

2-D numerical modelling was carried out using FEM code of Plaxis v8 considering the narrow plane characteristic of the deep excavation. The cross section 1–1 shown in Figure 2 was adopted in FEM analysis because it was almost the center of the deep excavation and near the historic building as well.

#### 4.1 Numerical model

The overall width of deep exaction at cross section 1-1 was 100 m with excavation depth of 13 m. The width of the numerical model was 240 m, which was 18 times as wide as the depth of the excavation. The vertical dimension was 50 m, which was more than 3.5 times the depth of the excavation. The model dimension was

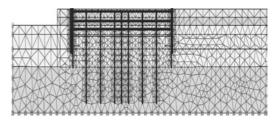


Figure 11. The FEM mesh.

Table 3. Parameters of retaining piles and ground improvement.

	Elastic modulus kPa	Poison's ratio
Retaining piles	$3.3 \times 10^{7}$	0.15
Strut	$3.0 \times 10^{7}$	0.15
Ground improvement	$1.4 \times 10^{5}$	0.20

large enough to lower the boundary effect. A linear elastic model was adopted for the ground improvement. The retaining wall as well as the strut was simplified as elastic beam in numerical modelling. The soils were simulated with Mohr-coulomb model. The numerical simulation was performed with 15-node isoparametric finite elements under the assumption of plane strain conditions. The FEM model was presented in Figure 11.

The boundary conditions in the numerical simulation contain the following two types, one was the displacement boundary condition, and the other was the drainage condition. A free displacement boundary condition was adopted at the ground surface. It was assumed that no horizontal nor vertical displacement taken place at the lower boundary, for it was beyond the influence of deep excavation. The lateral displacements at left - and right - hand boundary were both fixed as zero. The drainage condition at the ground surface was assumed to be free, hence the excess pore pressure was kept as zero along the ground surface; meanwhile the lower boundary as well as the left and right - hand boundary condition were considered to be kept as hydrostatic pore pressure during excavation. The initial effective stresses and hydrostatic pore pressure were calculated based on the weight of the soil and the underground water condition.

#### 4.2 Parameters used in numerical modelling

The parameters of retaining structure and ground improvement used in the numerical analysis were listed in Table 3. The soil parameters could be referenced as Table 1. The interface between retaining piles and soil was adopted and the interface parameters were determined according to Plaxis manual. The modulus

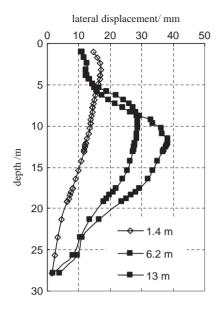
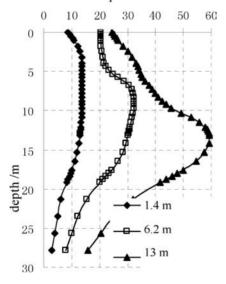


Figure 12. Evolution of lateral displacements at south side of deep excavation.



#### lateral displacement/mm

Figure 13. Evolution of lateral displacements at north side of deep excavation.

of resilience was adopted for soils. The modulus of resilience was obtained by back-analyzing the monitored lateral displacements of the first excavation progress shown in Table 2. It was found that the modulus of resilience was as 5 times high as the compression modulus for soils.

#### 4.3 Numerical modelling procedure

The excavation was modeled with 7 consecutive steps shown as following: STEP 1 was to determine the initial stress state due to the gravity of soils. STEP 2 was used to exert the loading of existing buildings on the surface with the magnitude of 60-70 kPa according to the type of the buildings. The movements induced in STEP 1 and 2 were reset to zero in the modelling. STEP 3 represented the construction of retaining wall and ground improvement, the surcharge of 20 kPa was also loaded at this step. STEP 4 meant the first excavation to 1.4 m deep and the construction of first strut. In STEP 5, the second strut was set after excavating to 6.2 m deep. Excavating to 10 m deep and the third strut was finished in STEP 6. The excavation was completed and bottom plate was constructed in STEP 7. Dewatering was considered during the excavation by changing water table.

#### 4.4 Calculated lateral displacements

Figures 12–13 presented the evolution of lateral displacements with excavation progress. The lateral displacements of south side of the retaining piles reached 28.8 mm, 38.4 mm respectively when excavated to the depth of 6.2 m and 13 m. They were smaller than those of north side of the retaining wall, which were 32 mm and 60 mm. It could be found that the maximum lateral displacement at north side was 1.5 times larger than that of south side of the retaining wall by comparing figure 12 and figure 13.

The comparison between calculated and monitored lateral displacement could be carried out because cross section 1-1 was coordinate with inclinometer CX3 and CX10. The monitored lateral displacements were 57.8 mm and 35 mm at monitored points CX3 and CX10 corresponding to the excavation depth of 13 m, while the accordingly calculated ones were 60 mm and 38.4 mm respectively. The agreement between calculation and monitoring implied the validation of the simulation procedure of FEM modelling with backanalysis on the modulus of resilience of soils. Besides, the consideration of main influential facts, such as surface surcharge due to existing loading and piling of construction material, the process of excavation, the supplemented techniques of dewatering and ground improvement, was essential in the FEM modelling to reasonably predict the behavior of the retaining piles.

#### 5 CONCLUSIONS

The lateral displacement of Riverside deep excavation with complex surrounding environment was studied with monitoring data and FEM modelling. The lateral displacements of the retaining wall were asymmetric because of the asymmetric earth pressure. The maximum lateral displacement at north side was almost 1.5 times as large as that of south side of the deep excavation. The earth pressure was close to the active earth pressure in top 15 m due to the large lateral displacement and small soil body against the retaining wall. The earth pressure was much larger than active pressure below 15 m. It was found the soil body bulk had a noticeable effect on the distribution of earth pressure against retaining wall.

#### REFERENCES

Chang, C. T. & Sun, C. W. et al. 2001. Response of a Taipei rapid transit system TRTS tunnel to adjacent excavation. *Tunnelling and Underground Space Technology* 16: 151–158.

- Yamagushi, I. & Yamazaki, I. et al. 1998. Study of ground tunnel interactions of four shield tunnels driven in close proximity, in relation to design and construction of parallel shield tunnels. *Tunnelling and Underground Space Technology* 13(3): 289–304.
- Zhang, D. M. & Huang, H. W. 2007. Ground movements and controlling measurements in deep excavation under asymmetric loading. *Proceeding of 10th national conference on soil mechanics. Chongqing*, 1–4 Novermber 2007 (in press). (in Chinese).

# GPS height application and gross error detection in foundation pit monitoring

#### H. Zhang

School of Safety and Resource Engineering, China University of Mining & Technology, Beijing, P.R. China

#### S.F. Xu

College of Architecture & Civil Engineering, Zhejiang University of Technology, Hangzhou, P.R. China

#### T.D. Lu

Department of Survey, East China University of Technology, Fuzhou, P.R. China

ABSTRACT: The author introduces a deformation monitoring model combined by traditional measuring technology and modern GPS measuring technology based on technical attribute of foundation pit deformation monitoring and author's experience of deep foundation pit construction project of underground tunnel in Lishui Road, Hangzhou city. When analyzing GPS height conversion, in order to improve GPS datum mark's reliability, one can use Dixon's test in GPS datum mark reliability test to find out height anomaly, thus provide convenience to search and delete marks with gross error. This test also improves deformation monitoring process's efficiency.

#### 1 BACKGROUND PROJECT INTRODUCTION

Lishui Road (from Huzhou Road to Qingfang Road) project is one of Hangzhou City government's "33929" engineering project. The tunnel of the project is composed by U-tanks and box culverts.  $1 + 568 \sim 1 + 638$ ,  $1+794 \sim 1+864$  are U-tanks. Each tank is 70 m long and the width of banks is summed to 22 m.  $1 + 638 \sim 1 + 794$  are box culverts. The sum of lengths of all box culverts reaches 156 m. Equally divided it into 4 parts, each box culvert is 38.25 m long and the width of all box culverts is summed to 21.4 m. Reinforced concrete piles with diameter of  $\Phi$  100 steel pipe were taken as support. They are 21 m long with concrete outside. The concrete piles with  $\Phi 60@30$ were used to keep dry from water. The depth of concrete pile is 10 m. They are connected to each other side by side. The steel in the shape of  $\mathcal{I}$  is used as the inside supports. The distance between two supports is 6 m wide. The depth of the foundation pit is 8 m. This foundation pit is level 2 foundation pit. The construction's  $\pm 0.000$  m level is equal to Huanghai height +4.125 m. The situation around site area is quiet complex, especially Jinghang Canal on the west side of site and ancient municipal heritage Gongcheng Bridge which is close to the nadir of underground lot, smallest distance is about 2 m.

#### 2 GPS HEIGHT APPLICATION

GPS positioning technology has advantages such as no need of keeping vision between measuring stations, not restrained by weather conditions, able to measuring the target's 3D displacement and highly automated. The accuracy of short distance deformation monitoring can reach minor millimeter level<sup>[1]</sup>, thus provides a new method for high-accuracy deformation monitoring of large construction and foundation pit. In Lishui Road project's case the visibility condition in foundation pit construction site is bad and most datum marks can't share vision, monitoring marks and datum marks are in different height, and also there is a across-river benchmark problem. To solve these problems above, this project take a monitoring plan using both modern and traditional measuring technology: using GPS technology to set up a 3D datum mark network, and using traditional measuring methods to monitor after the network is established<sup>[2]</sup>.

After adjusting GPS measuring results, the outcome height is geodetic height  $H_{GPS}$  relevant to WGS-84 ellipsoid. Since the benchmark height (normal height) is using in foundation pit engineering application, the geodetic height  $H_{GPS}$  should be transferred into normal height  $H_0$  in this project. The difference between normal and geodetic height is called height anomaly<sup>[3]</sup>:

$$\xi = H_{GPS} - H_0 \tag{1}$$

In solving GPS height anomaly, known marks height anomaly value's reliability is crucial to solving result accuracy. Because of restraint from site condition, it is impossible to have enough GPS marks meet benchmarks or taking benchmark co-measuring. So every single mark's height anomaly value will make considerable affect to calculating result accuracy, a mark with gross error height anomaly value could even lead to a totally useless result and complete failure. Thus the initial data should take a gross error test. During the test, the data is normally checked by geometric conditional closure, like triangle closure in triangle network or pole condition closure, which monitoring value must meet or by residual from adjustment error. Since gross error is hard to distinguish from limited error, this method is hard to discover small gross error. Also it is hard to find applicable geometric condition closure during GPS height transferring. To solve this problem, one can pick up some trustful geometric benchmark spot height and geodetic height in the GPS network to fit other benchmark height, or pick some spot separately to processing repetitive trail calculation, then obtain other measured geometric benchmarks' trail height with mathematic model from fit and using the equation below to obtain fit residual:

$$V_{i} = H_{i}' - H_{i} = \zeta_{i} - \zeta_{i}'$$
(2)

 $H'_i, \zeta'_i$  is trail height and trail height anomaly,  $H_i, \zeta_i$  is measured benchmark height and measured height anomaly. Then, one can use residual to process relevant spots' measured benchmark height gross error test, after carefully analysis of measured value with gross error, select enough reliable measured value to run fit again.

#### 3 HEIGHT ANOMALY GROSS ERROR TEST METHOD

According to Dixon Test <sup>[4]</sup>, assume there're a set of residual  $V_1, V_2, \dots, V_n$ , sort them from low to high, and get a sequence like below:

$$V_{(1)} \le V_{(2)} \le \dots \le V_{(n)}$$

Then we have:

$$r_{10} = \frac{V_{(n)} - V_{(n-1)}}{V_{(n)} - V_{(1)}} \quad r_{10}' = \frac{V_{(1)} - V_{(2)}}{V_{(1)} - V_{(n)}}$$

$$r_{11} = \frac{V_{(n)} - V_{(n-1)}}{V_{(n)} - V_{(2)}} \quad r_{11}' = \frac{V_{(1)} - V_{(2)}}{V_{(1)} - V_{(n-1)}}$$

$$r_{21} = \frac{V_{(n)} - V_{(n-2)}}{V_{(n)} - V_{(2)}} \quad r_{21}' = \frac{V_{(1)} - V_{(3)}}{V_{(1)} - V_{(n-1)}}$$

$$r_{22} = \frac{V_{(n)} - V_{(n-2)}}{V_{(n)} - V_{(3)}} \quad r_{22}' = \frac{V_{(1)} - V_{(3)}}{V_{(1)} - V_{(n-2)}}$$
(3)

If one from  $r_{10}$ ,  $r_{11}$ ,  $r_{21}$ ,  $r_{22}$  and  $r'_{10}$ ,  $r'_{11}$ ,  $r'_{21}$ ,  $r'_{22}$  is larger than critical value, then we can consider  $V_{(n)}$  or  $V_{(1)}$  as anomaly value. After analyzes the sensitivity of anomaly in r statistics test, Dixon claimed that when  $3 \le n \le 7$ , it is better to use  $r_{10}$  or  $r'_{10}$ ; when  $8 \le n \le 10$ , use  $r_{11}$  or  $r'_{11}$ ; when  $11 \le n \le 13$ , use  $r_{21}$  or  $r'_{21}$ ; when  $14 \le n \le 25$ , use  $r_{22}$  or  $r'_{22}$ .

It is natural to use different statistics depending on different *n*. When *n* is small, range estimation has a better efficiency, but while *n* become larger, range estimation's efficiency decrease accordingly. So when n is relevant large, use range  $V_{(n)} - V_2$  or  $V_{(n)} - V_{(3)}$  to estimate. Statistics  $r_{ij}$  or  $r'_{ij}$ 's critical value is given in  $r(n, \alpha)$  in reference<sup>[4]</sup>.  $\alpha$  is Type 1 probability, also called significance. Its value usually is 0.05 or 0.01.

When running the test, one can calculate and discriminate from both ends of residual sequence separately, until there is no gross error suspicion in both ends of the test.

#### 4 GROSS ERROR TEST EXAMPLE

In Lishui Road underground tunnel foundation pit construction project, the datum marks are the deformation monitoring datum control system. So they are usually built in the area outside and far from the construction site to maintain their stability. They should not be too far though for the consideration of having better monitoring accuracy and also for our convenience of work. Our monitoring network is divided into two levels. The first level of monitoring network is composed by the datum marks and working spots, measured once a week to maintain its stability. The second level of the network is set up by working spots and monitoring spots, using stable datum marks to verify working spots. Six datum marks were set up: four are at the east bank and other two are at west bank of the ancient Jinghang Canal. Using GPS to introduce the two datum marks at west bank of the canal to the canal's east in favor of monitoring network. The datum network is surveyed four times; following the official construction standards entitled "Global Positioning System for Urban Survey Technique standards" CJJ73-97. Three

Trimble 4600LS GPS single-frequency receivers were set up to receive the signal at the same time. The observation time lasted more than 90 min. Information from 5–10 satellites were efficiently received. The elevation angle of satellites is  $\geq 15$  degree and a break of 20 sec was set for every two observations. 12 base lines were observed and four of them are the repeat ones. Specific software provided by America supplier was employed to process the data and to carry out the effective solutions. The maximum of error is about  $\pm 5$  mm while the minimum is  $\pm 2$  mm. The observation results were further checked by time synchronized and unsynchronized circle. Datum height network monitoring data is fit from 4 spots and 16 sets of data of GPS benchmarks' geometric benchmark height residual.

Running Dixon test, first discriminate the largest residual  $V_{(16)}$ , since n = 16, so take  $r_{22}$  and  $r'_{22}$  as statistics.

$$r_{22} = \frac{V_{(16)} - V_{(14)}}{V_{(16)} - V_{(3)}} = \frac{0.018 - 0.011}{0.018 + 0.010} = 0.250$$

Using n = 16,  $\alpha = 0.05$  as argument, according to table<sup>[4]</sup>,  $r_0(16, 0.05) = 0.507$ , since  $r_{22} < r_0(16, 0.05)$ , the conclusion is the geometric benchmark height which  $V_{(16)}$  refers to doesn't have gross error. Discrimination of smallest residual  $V_{(1)}$ :

$$r_{22}' = \frac{V_{(1)} - V_{(3)}}{V_{(1)} - V_{(14)}} = \frac{-0.039 + 0.010}{-0.039 - 0.011} = 0.580$$

As  $r'_{22} > r_0(16, 0.05)$ , the conclusion is the geometric benchmark height which  $V_{(1)}$  refers to has gross error, should be eliminated.

After the elimination of residual  $V_{(1)}$ , the both ends test should be run again.

First discriminate the largest residual  $V'_{(15)}$ 

$$r_{22} = \frac{V_{(15)}' - V_{(13)}'}{V_{(15)}' - V_{(3)}'} = \frac{0.018 - 0.010}{0.018 + 0.005} = 0.348$$

Using n = 16,  $\alpha = 0.05$  as argument, according to table<sup>[4]</sup>,  $r_0(15, 0.05) = 0.525$ , since  $r_{22} < r_0(15, 0.05)$ , the conclusion is the geometric benchmark height which  $V'_{(15)}$  refers to doesn't have gross error. Then test the smallest residual $V'_{(1)}$ :

$$r_{22}' = \frac{V_{(1)}' - V_{(3)}'}{V_{(1)}' - V_{(13)}'} = \frac{-0.025 + 0.005}{-0.025 - 0.010} = 0.571$$

Since  $r'_{22} > r_0(15, 0.05) = 0.525$ , and  $r'_{22} < r_0$ (15, 0.01) = 0.616 it can be concluded that the geometric benchmark height which  $V'_{(1)}$  refers to doesn't have gross error.

#### 5 CONCLUSION

With GPS technology's widely application, people can simply and efficiently obtain horizontal accuracy of certain spot on minor millimeter level, but still can't obtain the spot's height on same accuracy level. So in order to extend GPS's superior ability in surveying 3D displacement, we should put our efforts on researching how to improve GPS survey accuracy of vertical displacement, thus it can match with survey accuracy of horizontal displacement. The reason why GPS has a low survey accuracy of vertical displacement is that though GPS could provide a high accuracy geodetic height, the lack of a geodic model with relevant accuracy lead to a serious accuracy decrease during transferring from GPS geodetic height to normal height. To seek the GPS height anomaly's value, the reliability of known spots' height anomaly value is critical to result's accuracy<sup>[5]</sup>, is the key to improve vertical deformation accuracy. To apply the GPS height survey in our project's foundation pit monitoring, the questions below should be considered:

- Height anomaly is unstable, it maybe smooth in small range or flat-contour region, where height datum network of foundation pit monitoring is often established, thus is easy to seek anomaly value; but it is very variant in wide range or complex contour region, possible to occur several value with high residual. So in order to improve reliability of gross error detection, when discriminated an anomaly value, one should analyze carefully before delete it.
- 2. In calculation of GPS height anomaly, the source of error is various; it could be surveying error of GPS geodetic height or GPS geodetic height difference, or error from geometric benchmark surveying. This problem directly leads to a difficulty of deciding error distribution pattern for height anomaly. Since test method usually run in a certain error distribution pattern, (e.g. Dixon test, requires residual is random sample from normal distribution) the credibility of using this test to run gross error detection is decreased in real application.
- 3. Many factors could affect GPS height component accuracy. Various measures should be taken to guarantee the accuracy in specific projects. Minimize the multipath effect when surveying with GPS in urban area, choosing geodetic type of GPS to monitoring datum during foundation pit construction period. Experience proved that using these methods not only avoid the restrictions brought to site conditions from conventional methods, but also improve working efficiency and assure construction quality.

GPS static relative positioning survey has tremendous practical significance to precise engineering survey. With the reasonable monitoring plan according to engineering condition and purpose, its accuracy could meet almost every requirements of precise engineering survey. It also has multiple advantages such as low cost, high efficiency and a high degree of automation. The application in Lishui Road foundation pit project is a useful experience.

#### REFERENCES

- Li, Z.H. & Huang, J.S. GPS measurement and date process. Wuhan: Wuhan University Press, 2005
- Lu, T.D. Zhou, S.J. Guan Y.L. Height anomaly Gross Error Test and Analysis in GPS Height Conversion. *Geotechni*cal Investigation & Surveying, 2004(4) 51–54
- Yang, J.T. Jiang Y.X. Zhou J. Analysis on Reliability and Accuracy of Subsidence Measurement with GPS Technique. *Journal of Geodesy and Geodynamics*, 2006(1) 70–75
- Zhang, H. & Gu, J.S. Deformation monitoring and data analysis of foundation in municipal engineering. *Journal of Zhejiang University of Technology*, 2003(5) 571–574
- Zhang, F.G. & Zhang J.Y. Statistical Distribution and Test of Survey Error. Beijing: China Measurement Press, 1991

### Study on deformation laws under the construction of semi-reverse method

J. Zhang, G.B. Liu & T. Liu

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering Tongji University, Shanghai, P.R. China

ABSTRACT: Taking a 24.09-m-deep foundation pit of Shanghai Metro Line 1 which uses the semi-reverse construction process of "three open excavating-one tunneling" as an example, through gathering and analyzing field monitoring data and making use of forward and back analysis methods, we found out deformation laws of foundation pit under the construction of semi-reverse method. The implementation results of this project indicated that the semi-reverse method is an effective way to improve rigidity of the exterior support, control the deformation of excavation, and ensure safety of the surrounding buildings and pipelines. Meanwhile, the results coincide essentially with time-space effect. The deformation of the excavation is closely correlative with excavation speed and exposure time. It provided some useful reference for the design of deep excavation in soft soil.

#### 1 INTRODUCTION

With the development of urban construction, more attentions have been paid to the utilization of underground space, and the construction technology level of foundation engineering has been improved continuously. At the existing construction process of deep foundation engineering, open excavation method is the most common construction method at present. Because it boasts many advantages, such as more construction operation surface, short period and less cost. However in the application of some deep excavations which have complicated adjacent environment, narrow operation space, and complicated geological conditions, open excavation method would cause great influence on the traffic flow. At the same time, the pollution of mud fluid, dust particle, acoustic noise, and vibration which caused in the construction would induce discommodity to the residents' life. Especially, open excavation method would go against with deformation control of pits, which would cause perimeter buildings and structures cracking, and bring great economic loss or unfavorable social influence. The complete reverse method has little effect on adjacent environment, but its speed of excavation is slow, construction technologic process is complicated, and the cost of pillar piles is high.

Combined with advantages of open excavation method and complete reverse method, semi-reverse construction method emerges as the time require, and has been used more and more widely in Shanghai deep foundation constructions. Taking a pit of Shanghai metro line 1, which uses semi-reverse construction method, as an example, through getting field monitoring data and setting up the finite element model, this paper has given an evaluation for the characteristics of semi-reverse method such as construction technology and deformation control laws, kindly expected to provide with a beneficial reference to those similar projects in future.

#### 2 ENGINEERING CASE

#### 2.1 General engineering situation

A railway station of Shanghai Rail Transit Line No. 10 (metro line 1) is situated at the intersection of South Xi Zang Road and Fu Xing Road, and "cross" transferred with metro line 8. The geographical position of this station is shown in Figure 1.

The station of metro line 1 is below that of line 8. The structure form of this subway station with three floors is two pillars and three spans, the outside dimension are 179.2 m (length)  $\times 23.8 \text{ m}$  (width). And the size of east and west end well is  $27.8 \text{ m} \times 16.1 \text{ m}$ , which bottom floor buried depth are 24.06 m, -24.09 m.

According to the requirements of waterproof design and construction plan, the whole railway station main body structure is divided into two construction region with eight parts. The subsection construction drawing of this station is shown in Figure 2. The west end well is the first construction part, which requires higher environment protection. This end well approaches the

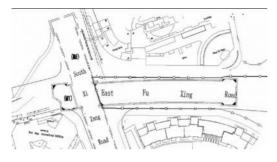


Figure 1. Geographical position of the station.



Figure 2. Subsection construction of the station.



Figure 3. Distribution of monitoring points in west end well.

International Squire (28 floors) and Shen Neng International Building (26 floors), and surrounding with lots of pipelines. According to the requirements of the first class environment protection specified for Shanghai subway station, the horizontal deformation of diaphragm wall should be  $\leq 1.4\%$ H (H is the depth of excavation), and the maximum settlement of perimeter ground surface should be  $\leq 1\%$ H. The excavation depth of the west end well is about 24.09 m. It adopts the underground diaphragm wall with width 1000 mm and depth 44 m. The brace system applies 1 piece concrete brace of 900 × 800 and 7 pieces steel tube brace of  $\Phi 609 \times 16$ . The distribution of monitoring points is shown in Figure 3.

#### 2.2 Geological condition

Basing on the geological prospecting data, the soils of engineering site are divided into 9 layers from up to bottom. They are **①**fill soil layer, **②**silt clay layer, ③mud-silt clay layer, ④muddy clay layer, ⑤<sub>1-1</sub>clay layer, ⑤<sub>1-2</sub>silt clay layer, ⑤<sub>3</sub>silt clay layer, ⑤<sub>4</sub>silt clay layer, ⑦<sub>2</sub>fine sand layer. Table 1 shows physical and mechanical characteristics of different soil layers. Figure 4 shows Geotechnical section of excavation.

Main hydrology condition of this station is as follows: the shallow groundwater field is phreatic aquifer, which mainly comes from infiltration of precipitation and seepage of surface water. The annual average water stage of Shanghai ranges from  $0.50 \text{ m} \sim 0.70 \text{ m}$ , and generally 0.5 m is chosen as design value.

In the report of geological prospecting data, the soil of  $\mathcal{D}_2$  fine sand layer is distributed in the site, whose buried depth is 44–46 m and confined water head is 10.5–11.0 m. Considering the worst factors, when the pit excavated to 24 m, the coefficient of upheaval in the bottom of the pit would not meet the requirement of safety factor, so it should be adopted measurement for decreasing confined water head.

#### 2.3 Construction procedure

Considering the actual factors such as construction period, traffic organization, underground pipelines and environment protection, the semi-reverse construction process of "three open excavating-one tunneling" was adopted in this project.

The detail of the process is as follows: Firstly excavate the soil to the fifth brace, and then construct the second median plate between the forth and fifth brace. With the top reinforced concrete brace, the reversed median plate and underground diaphragm wall formed a frame system. While the pavement maintenance of median plate has been finished, utilize two shield structure holes of the end well to dig the soil below the plate until the bottom plate finished.

The semi-reversed construction has brought lots of inconvenience to excavating and supporting of the pit under the second median plate. This inconvenience generally reflects at the narrow perpendicular channel and the complicated supports installation. Commonly, installation procedure of supports under the construction of semi-reversed method is that, divide the brace into several pieces, bring these pieces to the bottom one after another, and then assemble them together to the design elevation. For the process as it is mentioned, installing one straight brace generally needs 7hours, and installing one diagonal brace needs 10 hours, which far from the requirement of "time-space effect". "Time-space effect" requires that the excavation width should be no more than 6m, and the excavation plus supporting time should be no more than 24 h (excavation time-16 h; supporting time-8 h). The deformation should be control ineffectively, if the pit was not supported within such time. So based on the engineering traits, the project excavate the soil as soon as possible while in the open cut period. It uses open cut method to excavate the soil until 0.5 m below

	Buried depth	gravity	1	value of lidated shear	Compression module Es (MPa)	Permeability coefficients (m/s)		
No.	(m)	r (kN/m <sup>3</sup> )	C (kP	a)	ψ (°)	Kv	Kh	
1	2.31							
2	3.41	18.6	18	14	4.80			
3	9.21	17.5	11	16	3.51	1.97E-9~2.18E-9	2.08E-09	
4	19.81	16.7	13	12	2.27	1.17E-9~1.48E-9	1.33E-09	
<b>5</b> <sub>1-1</sub>	22.91	17.5	16	14	3.93	1.77E-9	1.77E-9	
(5) <sub>1-2</sub>	27.81	18.1	15	17.5	4.98	1.52E-9	1.52E-9	
53	42.11	18.2	15	19.5	5.08	1.60E-9~2.13E-9	1.87E-09	
54	44.71	19.7	39	15.0	8.00	9.3E-10~1.46E-9	1.20E-09	
$\mathcal{O}_2$		19.3	0	31	15.11	2.48E-8~2.57E-8	2.53E-08	

Table 1. Physical and mechanical characteristics of different soil layers.

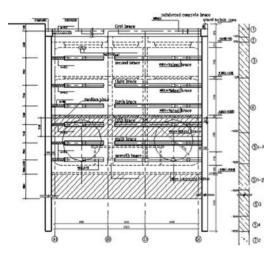


Figure 4. Geotechnical section of excavation.

the fifth brace (3 m below the second median plate). When the seventh soil was excavated, disassemble the supports form the location of the fifth brace and install them to the location of the seventh brace. So the installation of every brace just needs 0.5 hours, which should accelerate the construction speed as well as control the deformation of the pit effectively.

#### 3 FINITE ELEMENT MODELING

Combined with field monitoring data, the finite element software PLAXIS 8.2 (Brink Greve & Vermeer 1998) was used to compute the response of the soil around the excavation for effective analysis. The problem was simulated assuming plane-strain conditions and chosen half of the pit as the research subject. The side boundaries of the mesh (total size 90 m  $\times$  75 m) were established beyond the zone of influence of the settlements induced by the excavation (Caspe 1966; Hsieh & Ou 1998). The finite element mesh boundary conditions were set using horizontal restraints for the left and right boundaries and total restraints for the bottom boundary. The soil stratigraphy was assumed to be uniform across the site. Those soils with similar properties would be combined by weighted similarity method. So six soil layers were compartmentalized for the calculation simplify.

The soil model used to characterize the clays in the PLAXIS simulation of the excavation is the hardeningsoil (H-S) model (Schanz et al. 1999).

This effective stress model is formulated within the framework of elastoplasticity. Plastic strains are calculated assuming multisurface yield criteria. Isotropic hardening is assumed for both shear and volumetric strains. The flow rule is nonassociative for frictional shear hardening and associative for the volumetric cap. The initial values of the basic H-S input parameters for the soil layers are referenced as Table 1 and calibrated by inverse analysis.

The linear spring-layer model is adopted to simulate the braces; the plate element model is adopted to simulate the underground diaphragm wall, reversed median plate and bottom plate. Considering the buildings around the pit,  $50 \text{ kN/m}^2$  overload is applied for calculation.

Figure 5 shows the calculation model. Table 2 shows 11 calculation phases and the construction stages used in the finite element simulations. PLAXIS employs a penalty formulation so that undrained conditions can be explicitly modeled. Because there was a long time interval between Phase 6 and Phase 7, the displacements are due to partially drained conditions. So consolidation should be considered in this stage. Other stages which not noted as "consolidation" in Table 2 were modeled as undrained and the excess pore water pressures were computed relative to some steady-state value (1m) that changes with dredge line level.

Identification	Phase no.	Calculation	Stages
Initial equilibrium	0	Plastic	construction
Set up diaphragm wall and apply overload	1	Plastic	construction
Excavate the first soil and support the first brace	2	Plastic	construction
Excavate the second soil and support the second brace	3	Plastic	construction
Excavate the third soil and support the third brace	4	Plastic	construction
Excavate the forth soil and support the forth brace	5	Plastic	construction
Excavate the fifth soil and support the fifth brace	6	Plastic	construction
Construct median plate	7	Plastic	construction and consolidation
Excavate the sixth soil and support the sixth brace	8	Plastic	construction
Excavate the last soil	9	Plastic	construction
Construct bottom plate	10	Plastic	construction

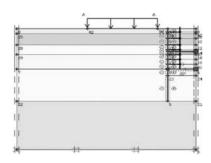


Figure 5. Calculation model.

#### 4 COMPARISON OF FIELD DATA WITH CALCULATION RESULTS OF FINITE ELEMENT SOFTWARE

In order to study the deformation laws under the construction of semi-reversed method, lots of field monitoring data of west end well have been finished from the second brace has been supported to the roof plate has been finished. Choose the inclination survey point CX3 and the settlement points J6-1, J6-2, J6-3, J6-4, J6-5 which have the same cross section with CX3 as representative points. Combined with calculation results of finite element software, it could be got detailed analysis.

#### 4.1 Inclination deformation of underground diaphragm wall

In the construction process of foundation pit, the displacement curve of inclination point CX3 at different depth which changed with the working condition is shown in Figure 6.

From Figure 6, it could be found that the maximum inclination displacement of diaphragm wall is only 33.69 mm when the bottom plate has been poured, which is satisfied with the requirement of Class 1

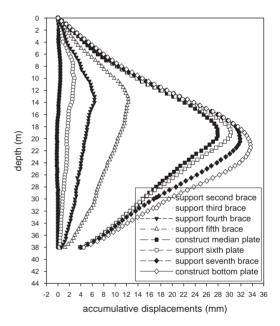


Figure 6. Displacement curve of CX3 at different depth which changed with the working condition.

environment protection. Based on the experiences of Shanghai underground works these years, with the similar excavation depth, excavation size, geological conditions and peripheral circumstance, if the foundation pit adopts open cut method, the deformation value could not be controlled at so small range.

Calculation results of finite element software and field data were compared from the time that fifth braces had been installed, which is shown in Figure 7. In the figure, dashed line represents the calculation value, and solid line with circle represents measured value. Table 3 shows the specific comparative value.

The shape of calculation curve was in good agreement with the measured curve, and the maximum value of calculation deformation was in accordance with field data while the bottom plate has been constructed. The results show that finite element method can correctly reflect excavation deformation regularity. It shows that the diaphragm wall engendered comparative larger deformation within the period from the fifth brace supported to the median plate constructed. This is because the discrepancy of the two work conditions lasts as long as 20 days. Though soils weren't excavated, exposure time for the foundation pit with braces was comparative long. The excavation face is situated in @muddy clay layer which has very strong flow property, and the permeability of the soil is relatively large  $(\cong 1.77 \times 10^{-9} \text{ m/s})$ . For above reasons the diaphragm wall engendered larger deformation. In the finite element calculation, consolidation has been considered, so it could correctly reflect the actual deformation.

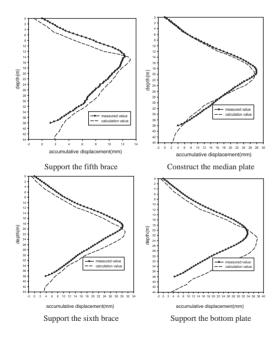


Figure 7. Measured versus computed horizontal displacements.

In order to analyze the relationship between deformation of diaphragm wall and time, we chose the department of the maximum deformation for filed data CX3 (CX3-43 point whose depth is 21.5 m) as a key point. The variations of deformation with time for this point in the whole excavation construction are inspected as Figure 8 shows.

From Figure 8, we can find that though the diaphragm wall engendered large deformation from the fifth brace supported to the median plate supported, in the process of concrete maintenance, the deformation stopped to grow and it even had a little falling, and also when the soil under the reversed media plate was excavated, the deformation rate is smaller than that of previous. With the top reinforced concrete supports, the reversed median plate and underground diaphragm wall could be formed as a frame system. which constrained the spreading of soil deformation. The reduction of deformation rate in this phase has released the comparatively large deformation, which engendered as a result of soil creep in forward phase. This is beneficial for the reduction of foundation deformation and assurance of pit stability.

#### 4.2 Ground settlement

Ground settlement points J6-1, J6-2, J6-3, J6-4, J6-5 are in the same section with the inclination point CX3, which is distributed with the distance of 3 m for every point from the edge of pit. Figure 9 shows the field settlement curves in the process of construction.

From Figure 9, it shows that ground settlement increased with excavation depth. When the bottom plate has been finished, the maximum settlement is only 7.1 mm. The soil presented a little uplift at 6 m from the edge of excavation. With reference to the actual engineering project, high pressure jet grouting was used in the end well for the stability of shield access to tunnel. It might be the reason that caused the soil uplifting.

#### 4.3 Building settlement and pipeline settlement

To March 2007, while the roof plate has been finished, the maximum building settlement was only -3.3 mm, which is at F05 point. The variations of deformation

Table 3. Specific values of measured versus computed horizontal displacements.

Project database		Fifth brace	Median plate	Sixth brace	Seventh brace	Bottom plate
Completion time		06-11-16	06-12-6	06-12-29	07-1-1	07-1-15
Measured Value	Maximum value	12.33	27.94	30.25	31.85	33.69
	Depth	14	19	19	21	21.5
Calculation value	Maximum value	13.19	27.32	31.05	35.35	37.68
	Depth	15	20	21	21	22

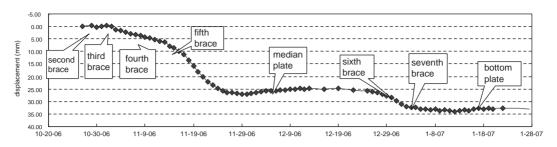


Figure 8. Variations of displacement with time for CX3-43 in the whole excavation construction.

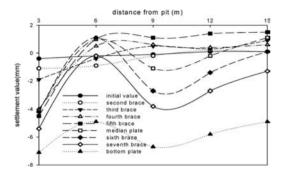


Figure 9. Field settlements in the process of construction.

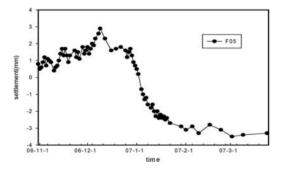


Figure 10. Variations of deformation with time for F05.

with time for F05 are shown in Figure 10. As a result of high pressure jet grouting, in the former phases of construction the vertical deformation of F05 presented an uplifting trend. It was not until January 2007 that the soil deformation fell back. This was one of the main reasons for the so small accumulative building settlement.

The conditions of pipeline settlement were as follows: the maximum deformation point of gas pipe was M01, whose accumulative settlement was -8.6 mm; the maximum deformation point of water supply pipe was S02, whose accumulative settlement was -8.4 mm; the maximum deformation point of rain pipe is Y01, whose accumulative settlement

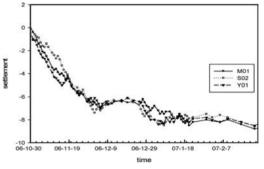


Figure 11. Time-history curves of M01, S02 and Y01.

was -8.2 mm. Choosing pressure pipe point M01, S02 and non-pressure pipe point Y01 as key point, whose time-history curves is shown in Figure 11.

The settlement trends of these pipelines were uniform, and these time-history curve shapes were similar to that of inclination point in Figure 8. From December 6th to December 29th, when the median plate has been maintained, the deformation value of pipelines also presented a stable period. In 1969, Peck put forward stratum compensation theory, which indicated that the shapes and the enclosed area of lateral deformation curves caused by foundation pit excavation are similar to that of ground settlement curves. From Figure 11, it can be found that this similarity changed uniformly with time, that it is to say the ground settlement changed with the lateral deformation at any time, which is favorable for the environment protection.

#### 5 CONCLUSION

 Adopting semi-reverse construction method in metro foundation pit could control the deformation of pit effectively, and decrease the influence of excavation construction on its surrounding environment. Semi-reverse construction method owns a deep foundation support technology with practical value and brilliant prospects, which would be further developed and applied in rail transit construction.

- 2. The reversed median plate and underground diaphragm wall formed a frame system. In the process of median plate maintenance, the deformation of soil behind retaining wall was stable. When the plate maintenance finished and the soil excavated, the deformation rate was smaller than those engineering works which adopted open-cut method of the same conditions.
- 3. In the process of reversed median plate supporting, a long period was needed for reinforcement assemble and scaffold erection. As a result of soil flow property, larger deformation may be generated at this period.
- 4. As the excavation is in clay, longer times of construction may result in partial drainage as well. Consolidation of the soil should be considered in finite element calculation.
- 5. The shapes of lateral deformation curves caused by foundation pit excavation are similar to that

of ground settlement. This similarity changed uniformly together with time pass. It is conjectured that the ground settlement changed with the lateral deformation shape at any moment.

#### REFERENCES

- Brinkgreve, R.B.J. & Vermeer, P.A. 1998. Finite element code for soil and rock Analysis. PLAXIS 7.0 manual, Balkema, Rotterdam, The Netherlands.
- Liu, J.H. & Hou, X.Y. 1997. Foundation engineering manual. Beijing: China Construction Industry Press.
- Richard, J., Finno, M. & Michele, C. 2005. Supported excavations: observational method and inverse modeling. *Jour*nal of geotechnical and geoenvironmental engineering 10.1061:826–836.
- Zhao, G.W. & Guo, H.B. 2006. Application of semi-reversed construction method to rail transit construction. *Building Construction* 28(10):815–818.

# Comparison of theory and test on excavation causing the variation of soilmass strength

#### J. Zhou & J.Q. Wang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering Tongji University, Shanghai, P.R. China

#### L. Cong

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China

ABSTRACT: In view of the excavation unloading characteristic, the variation of soilmass strength is studied through the theoretical deduction and the test analysis. Baseed on the Hvorslev's real strength theory, the strength ratio of the unloading soil and the normal compressed soil considering the pore-water pressure is deduced and the test simulating excavation is carried out. Through comparing data of the theory and test, the soilmass is caused to be at the overconsolidated state, and the soil microstructure is damaged, then the soilmass strength is reduced in the unloading process. The analysis result of theory and test are helpful to the further understanding of the effect of unloading in excavation on the variation of the soilmass strength, which are very significant for avoiding project accidents.

#### 1 INTRODUCTION

With the rapid and remarkable development of city construction, an increasingly large number of the exploitation of the underground spaces have emerged, such as high-story building, the underground market and underground garage etc., which need to excavate for building foundation. The excavation, including the influence of the soilmass's engineering property and the variation of the environment characteristic, have been systematically studied by numerous scholars. Rutledge (1944) summarized the soil sample disturbance to the influence of the unconfined compression strength and the initial tangential modulus in stress and strain curve, and the result showed that the initial tangential modulus of the remoulded soil sample is smaller than the one of the undisturbed value by about 20%, some were only even 3%~4%. On resonant column test, Drnevich & Massarsch (1979) discovered that even if the soil sample suffered from the small disturbance, its initial tangential modulus also obviously reduced. Broms (1980) pointed out that the soil sample disturbance in the brittle soil to the stress and strain curve's influence was much bigger than in the plastic soil. Zeng (1995) studied the subway double lines shield tunnel construction to the influence of the surface, the buildings and the underground pipelines, and analyzed the tunnel interval to the influence of stress and the displacement of surrounding soilmass. Zeng & Pan (1988) studied stress path to the influence of the undrained strength in excavation. Wei (1987) has studied the relation of the excavation unloading and passive soil pressure.

Several examples of the collapse of foundation pits in the past had very serious consequences, which urged the people to study the design and construction of foundation pit deeply. At present the maintenance structure of foundation pit is designed and calculated by using the elastic foundation beam law or the elastoplasticity finite element method. The routine-test parameters generally were adopted as the computation parameters, which had not really considered the excavating and unloading to the influence of soilmass strength. The variation of soilmass strength after excavating and unloading is studied through the analysis of theory and test in this paper. The result of the study indicates that the unloading in excavation has influence on the variation of soilmass strength, which can be of some help to avoid project accidents.

# 2 THEORETICAL DEDUCTION OF SOILMASS STRENGTH UNDER UNLOADING

After excavation, the surrounding soil can be seen as the overconsolidated soil layer, and Wei (1987) deduced the undrained strength of the excavation unloading soft clay according to the Hvorslev (1960) real strength theory, which was the same formula that Mayne (1980) obtained the undrained strength of overconsolidated clay soil according to the statistics of a large number of test data. After the excavation, in fact, the effective stress of bottom soil layer of foundation pit is in unceasingly developing and changing process, rather than the static overconsolidated state that above formula derives. In this process, because of unloading, the negative pore water pressure dissipates slowly, and effective stress decreases gradually, and eventually stops at the overconsolidated state. According to the Hvorslev real strength theory, the undrained strength of the soils after excavation and unloading is deduced.

The Hvorslev strength formula is as follows:

$$\tau = ptg\varphi_e + c_e \tag{1}$$

where  $c_e = \xi \cdot p_e$ ; In normal consolidated soil,  $p_e$  is equal to the current effective stress (drained shear strength) or the consolidation pressure (undrained shear strength); In overconsolidated soil,  $p_e$  is the consolidation pressure that the test specimen failure's porosity ratio corresponds in the normal pressure dense curve.  $tg\varphi_e$  is the increment ratio which the shearing strength increases along with the effective stress change when the water content is constant; *P* is the effective stress.

According to the real strength theory and the critical state's concept, when soilmass has withstood a simple loading–unloading cycle in stress history, it can be assumed that the failure point of overconsolidated soil is coincidence with the failure point of the normal consolidated soil at the same water content when stress path reaches at critical state line, as shown in Figure 1 and Figure 2. Then the effective stress of the overconsolidated soil is as follows:

$$p_{b}' = p + \Delta p_{b}' = p_{e} - \Delta p_{e}'$$
<sup>(2)</sup>

Based on the confirmation of a large number of tests, it can be considered approximately that the shapes of undrained stress path of the normal compacting soil sample is geometrical similarity under the different consolidation pressure, and the variation value of effective stress and consolidation pressure are in proportion (Wei 1987). Therefore their relation can be proposed from the following equation:

$$\Delta p_e' = \Delta p_a' \cdot \frac{p_e}{p_a} \tag{3}$$

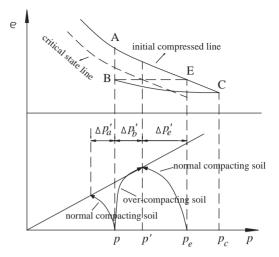


Figure 1. The constant consolidation pressure and the undrained stress path for the normal consolidated soil and the overconsolidated soil (Wei 1987).

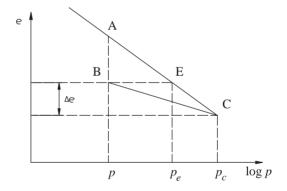


Figure 2. The relation curves for the normal consolidated soil and the overconsolidated soil.

The undrained shear strength of overconsolidated soil sample B is as follows:

$$S_{ub} = (p_t - u_{bt} + \Delta p_b) tg \varphi_e + \xi p_e$$
(4)

Let 
$$p = p_i - u_{bi}$$
 (5)

Taking the equation (2),(3),(5) into the equation (4), then:

$$S_{ub} = (p_{et} - \frac{p_{et}}{p_a} \cdot \Delta p_a') tg \varphi_e + \xi \cdot p_{et}$$
(6)

The undrained shear strength of normal consolidated soilmass A in initial consolidation pressure  $P_c$  is as follows:

$$S_{ua} = (p_a - \Delta p_a) \cdot tg\varphi_e + \xi \cdot p_a \tag{7}$$

$$S_{ub} - S_{ua} = \left[ p_{et} (1 - \frac{\Delta p_a'}{p_a}) - p_a + \Delta p_a' \right] \cdot tg\varphi_e + \xi \cdot (p_{et} - p_a)$$
$$= (1 - \frac{\Delta p_a'}{P_a})(p_{et} - p_a) \cdot tg\varphi_e + \xi \cdot (p_{et} - p_a)$$
$$= (\frac{p_{et}}{p_a} - 1) \left[ (p_a - \Delta p_a') \cdot tg\varphi_e + \xi \cdot p_a \right]$$
(8)
$$= (\frac{p_{et}}{P_a} - 1)S$$

Then 
$$\frac{S_{ub}}{S_{ua}} - 1 = \frac{p_{et}}{p_a} - 1$$

Therefore 
$$\frac{S_{ub}}{S_{ua}} = \frac{p_{el}}{p_a}$$
 (9)

In Figure 2: On the initial compression line AEC,

$$\frac{1}{C_c} = \frac{\log p_{et} - \log p_c}{\Delta e}$$

On the unloading and swelling line BC,  $1 \log p - \log p_{a}$ 

$$\frac{1}{C_s} = \frac{\log p - \log}{\Delta e}$$

Then 
$$\frac{C_s - C_c}{C_c} = \frac{\log p_{et} - \log p}{\log p - \log p_c}$$
(10)

Therefore 
$$\frac{p_{er}}{p} = \left(\frac{p}{p_e}\right)^{\frac{C_s - C_c}{C_e}}$$
 (11)

$$\frac{p_{et}}{p_t} = \frac{p_{et}}{p} \times \frac{p_t - u_t}{p_t}$$

$$\frac{S_{ub}}{S_{ua}} = \left(\frac{p_t - u_t}{p_c}\right)^{\frac{C_s - C_c}{C_c}} \times \frac{p_t - u_t}{p_t}$$

$$= \left(\frac{p_t - u_t}{p_c}\right)^i \times \frac{p_t - u_t}{p_t}$$
(12)

$$i = \frac{C_s - C_c}{C_c}$$

The equation (12) is the undrained strength ratio of the excavation unloading soil and the normal consolidation soil.

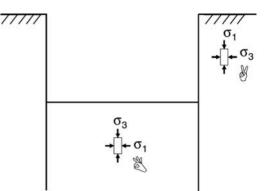


Figure 3. The diagram of the stress variation for excavation.

In the above deduction process:

 $S_{ub}$  – The undrained strength of the excavation unloading soils;

 $S_{ua}$  – The undrained strength of the normal consolidated soils;

 $u_t$  – The negative pore-water pressure of the unloading soilmass, with time dissipation;

 $P_t$  – The soilmass consolidation pressure of current state;

 $p_c$  – The soilmass consolidation pressure under natural state;

 $C_c$ ,  $C_s$  – The compression index and swelling index of soilmass.

The law is reflected in the equation (12) that soilmass strength reduces gradually with the negative pore-water pressure dissipation after unloading. when  $u_t = 0$ , soil is at the complete over-compacting state. The key that estimates the soilmass strength after unloading is to determine the parameter i. Because of the errors of the sampling disturbance and instrumentation equipment, the computed result of the parameter i which is determined by consolidation test's result  $C_c$ ,  $C_s$ , is bigger than the real value, needing to calculate the parameter i by the strength test.

After the synthetical comparison, Mayne (1980) propose that  $1 - C_s/C_c$  takes the statistical average value 0.64 to be quite reasonable according to the statistics of a large number of experimental data and in situ measurement; Furthermore, Zhang & Wei (1987) have confirmed this viewpoint. Therefore the parameter i is taken -0.64 in this paper.

# 3 EXPERIMENT ANALYSIS OF SOILMASS STRENGTH AFTER UNLOADING

# 3.1 Test plans

In the excavation process, as shown in Figure 3, the soilmass unit A of around foundation pit wall is lateral

 Table 1.
 The triaxial test plans of constant pressure consolidation.

	Consolidation pressure $\sigma_v = \sigma_H (kPa)$			
Variation mode of the stress	100	200	300	400
Unloading $-\sigma_v$ unloading failure	I01	I02	I03	I04
$\begin{array}{l} Unloading - \sigma_v \ loading \\ failure \end{array}$	J01	J02	J03	J04
$\sigma_v$ unloading failure $\sigma_v$ loading failure			K03 L03	

Table 2. The triaxial test plans of K<sub>0</sub> consolidation.

	Consolidation pressure $\sigma_v = \sigma_H/K_0$ (kPa)			
Variation mode of the stress	180	240	400	
$\sigma_H$ unloading failure	M01	M02	M03	

unloading, and vertical pressure is nearly invariable; The soilmass unit B of the lateral and vertical in the bottom of foundation pit simultaneously unloads, and the vertical stress drops more quickly than lateral stress, but still retains quite a part of incomplete unloading stress, therefore the influence of the unloading only possibly exists in a certain scope below foundation pit bottom surface. The stage excavation method is generally selected for the foundation pit. The first supporting structure immediately be taken when the first layer soil is excavated to reach at the design elevation; Then the second layer soil is excavated. During the excavation, the soilmass units are in the process that the soil is unloading and expanding and the negative pore-water pressure is dissipating slowly.

According to the unloading characteristic of above soilmass unit A, B, test plans are designed as shown in Table 1 and Table 2.

I, J group tests simulate the stress path of the unit B, and the soil samples are consolidated for 24 hours under constant pressure; then according to the stress path  $\Delta \sigma_v = \sigma_v/3$ ,  $\Delta \sigma_v/\Delta \sigma_H = 2$ , the test specimen are unloaded simultaneously on the vertical and the lateral, and the value was recorded when the negative pore-water pressure are stable after unloading; This process needs for 2–3 hour from starting unloading to stability of reading value, then turns on the drain valve, and makes the negative pore-water pressure dissipation, the soil sample completes consolidation under the new low stress condition, the consolidation time is 8 hours. Then the I group is loaded to the test specimen compression failure on the vertical, and the J group is unloaded to the test specimen extrusion failure on

Table 3. The basic index of mechanical property of Silt clay.

Parameters	Values
w	52.6%
γ	16.9 KN/m <sup>3</sup>
e	1.487
Sr	97.3%
Ip	23.2
I <sub>p</sub> I <sub>L</sub>	1.231
α <sub>1-2</sub>	$1.20  {\rm MPa}^{-1}$
c	11 kPa
$\varphi$	9.3°

Table 4. The negative pore-water pressure data of soil sample after unloading (kPa).

Serial number	I (J) 01	I (J) 02	I (J) 03	I (J) 04
Negative pore-water pressure (kPa)	_	_	-30.9	-34.1

Note: Because of instrument failure, the data of 01 and 02 can not be detected, the latter two data are average value of I, J group.

the vertical. As a reference test, K group and L group specimen are consolidated in the confining pressure of setting, without unloading disturbance, and respectively are loaded and unloaded to the test specimen failure on the vertical.

The unloading stress variation process of unit A is simulated by M group test.  $K_0 = 0.6$ , the consolidation pressure is exerted by staged loading; For avoiding the accidental failure of the test specimen, the staged loading is divided 10 levels to exert; The axial stress  $\Delta\sigma$ is exerted in each level loading, at the same time, the confining pressure  $\Delta\sigma_H = K_0 \Delta\sigma_v$  is exerting, consolidation time 24 hours. After consolidation completes,  $\Delta\sigma_v$  is maintained invariable, and the test specimen is compressed to failure by the lateral unloading.

# 3.2 Test results

The soil samples of tests are the typical soft soil of the Shanghai area, its basic index of mechanical property as shown in Table 3.

After unloading, the negative pore-water pressure is shown in Table 4.

# 4 THE COMPARISON ANALYSIS BETWEEN THEORY RESULT AND TEST RESULT OF THE SOILMASS STRENGTH AFTER EXCAVATING AND UNLOADING

The theory deduction and the test simulation about soil strength of the excavation has been discussed.

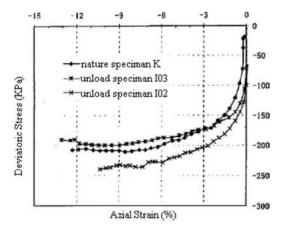


Figure 4. The normalization stress of the unloading soil of I group.

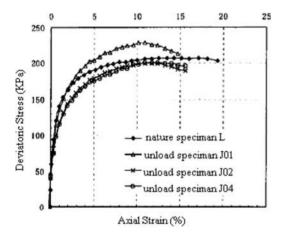


Figure 5. The normalization stress of the unloading soil of J group.

Now we will discuss the comparison reuslt of theory and test.

The stress–strain relation curve of different confining pressure of I, J two groups tests are normalized, and  $\sigma_m = (\sigma_1 + 2\sigma_2)/3 = 233$  kPa, then the curves can be drawn as shown in Figure 4 and Figure 5, in which K Line, L line are the soil stress-strain curves of natural compaction after normalization, and I02, I03, J01, J02, J04 are the normalizing stress-strain curve of over-compacting soil after the unloading.

After excavation unloading, the stress value of the soil sample stress-strain relations curve approaches or surpasses the stress value of the natural normal compacting soil as shown in Figure 4 and Figure 5; For eliminating the test error, after unloading, soil sample peak value ( $\sigma_1 - \sigma_3$ )<sub>max</sub> can be taken the average value of each normalized curve peak value, as shown

Table 5. The soil strength contrast between theory result and test result after unloading.

	Project				
	Peak val $(\sigma_1 - \sigma_3)$	ue ) <sub>max</sub> (kPa)			
Test number	soil	Soil sample of unloading disturbance	Test result S <sub>ub</sub> /S <sub>ua</sub>	Theory result S <sub>ub</sub> /S <sub>ua</sub>	Difference value (%)
I J	208.2 205.1	219.0 212.5	1.052 1.040	1.175 1.175	12.3 13.5

in Table 5; For comparing with test result, theoretical calculation is taken by the formula (12), the parameter  $P_c = 300$  kPa,  $P_t = 233$  kPa, the test results and the theoretical formula results, are shown in Table 5.

In the Table 5,  $S_{ub}$  is the undrained strength of excavation unloading soil in I, J series tests by normalization;  $S_{ua}$  is the undrained strength of the normal compressed soils in K, L series tests by normalization.

In above tests, the influence of negative porewater pressure (Table 4) is considered in soil sample. According to the computation of the formula (12), the soilmass strength ratio  $S_{ub}/S_{ua}$  is 1.229 and 1.235, which is higher about 5% ~6% than the ratio of the pore-water pressure dissipating completely.

The comparison from Table 5 can be found that theoretical calculation result is bigger about 10% than test result. Not only The soilmass is caused to be at the overconsolidated state, but also the soil microstructure is damaged, and the soilmass strength is reduced in the unloading process. In this triaxial test, soil samples is unloaded according to the stress path of the test, soil stress is redistributed, and consolidated, the overconsolidated soil is formated, soil structure of the original system is also damaged.

# 5 THE VARIATION OF SOILMASS STRENGTH PARAMETERS AFTER EXCAVATING AND UNLOADING

Based on the stress-strain relation curves of tests, according to the Mohr-Coulomb criterion, the soilmass strength parameters  $c\$\varphi$  of the simulating excavation unloading and the parameters  $c\$\varphi$  of routine-test are listed in Table 6.

The cohesion, the angle of internal friction that unloading failure of I and M group obtained are quite close, their *c* value is bigger than J group, but the  $\varphi$  value is smaller, furthermore, the values of c\$\$ $\varphi$ are obviously different from the result of conventional consolidated quick shear test. Because of lack

	Test number					
Parameter	A unit of M group (unloading failure)	B unit of I group (unloading failure)	B unit of J group (loading failure)	The result of conventional consolidated quick shear		
Cohesion c (kPa)	26.5	24.3	7.36	11		
Angle of internal friction $\varphi$ (°)	15	13	16	9.3		

Table 6. The strength parameter value c\$\$ $\varphi$  of the soilmass failure.

of the sufficient test data, the relationship between the unloading stress and the strength parameters c\$\$ $\varphi$  can not be obtained, which needs further test to determine whether these parameters have the inevitable relation.

# 6 CONCLUSION

This paper chooses the typical excavation as the test study object, designs and carries out different stress paths indoor triaxial tests in I, J, K, L, M group tests. Some useful conclusions are drawn by analyzing the influence of excavation on the result of theory and test, which are very significant for avoiding project accidents:

1. By the assumption that the soil as overconsolidated soil with dissipation of the negative pore-water pressure after unloading, the undrained strength ratio between the soils of excavation unloading and the normal consolidated soils is deduced, namely the formula (12). With the dissipation of soil negative pore-water pressure, the soilmass strength is reduced. According to the analysis result, the scope of reduction is not large. Under the above test stress condition, the strength ratio range of variation is about 5%–6%.

- 2. The undrained strength ratio  $S_{ub}/S_{ua}$  from the tests is smaller about 10% than undrained strength ratio from the above theoretical formula computation. The difference value in the test can be identified as the result that of unloading disturbance.
- 3. In the unloading process, the soilmass is caused to be at the overconsolidated state, the soil microstructure is damaged, and the soilmass strength is reduced.
- 4. The total stress strength parameters c\$\$ $\varphi$  obtained from the different stress path tests are much different from the parameters from the routine-test. Due to the insufficiency of data, the relationship between the unloading stress and the strength parameters c\$\$ $\varphi$  can not be obtained, which needs further test to determine whether these parameters have the inevitable relation.

# REFERENCES

- Broms, B.B. 1980. Soil Sampling in Europe: State-of-the-Art. Journal of the Geotechnical Engineering Div. 106: 65–98.
- Drnevich, V.P. & Massarsch, K.R. 1979. Sample Disturbance and Stress Strain Behaviour. ASCE Journal of the Geotechnical Engineering Division 105(GT 9): 1001–1016.
- Hvorslev, M.J. 1960. Physical component of the shear strength of saturated clays. *Research Conference on Shear Strength of Cohesive Soils*, ASCE: 169–274.
- Mayne, P.W. 1980. Cam-clay prediction of undrained strength. *Geotech Engrg Div* ASCE 106(GT11): 1219–1242.
- Rutledge, P.C. 1944. Relation of undisturbed sampling to laboratory test. *Transactions* ASCE, (109): 1155–1183.
- Wei, R.L. 1987. *Soft clay strength and deformation*. Beijing: China communications press.
- Zeng, X.Q. 1995. Subway project double thread tunnel parallel advancement interaction and construction mechanics research. Shanghai: Tongji University doctoral dissertation.
- Zeng, G.X., Pan, Q.Y. & Hu, Y.F. 1988. The Behavior of Excavation in Soft Clay Ground. *Chinese Journal of Geotechnical Engineering* 10: 13–22.

*Theme 2: Construction method, ground treatment, and conditioning for tunnelling* 

# Ten years of bored tunnels in The Netherlands: Part I, geotechnical issues

# K.J. Bakker

COB, Delft University of Technology, Delft, The Netherlands

# A. Bezuijen

Deltares, Delft University of Technology, Delft, The Netherlands

ABSTRACT: Ten years have passed since in 1997 for the first time construction of bored tunnels in the Netherlands soft soil was undertaken. Before that date essentially only immersed tunnels and cut-and-cover tunnels were constructed in the Netherlands. The first two bored tunnels were Pilot Projects, the 2nd Heinenoord tunnel and the Botlek Rail tunnel. Since then a series of other bored tunnels has been constructed and some are still under construction today. At the beginning of this period, amongst others Bakker et al (1997), gave an overview of the risks related to bored tunnels in soft ground and a plan for research related to the pilot projects was developed. After that in 1999 the 2nd Heinenoord tunnel opened for the public, the "Jointed platform for Bored tunnelling", in short GPB, was organized, to coordinate further research and monitoring of bored tunnels. This platform is supervised by the Center for Underground Construction. In this paper a summary is given of some of the most characteristic observations on these 10 years of underground construction in the Netherlands. In the first part of this paper the focus is on geotechnical interactions, and stability, whereas part two will focus more on structural related issues.

# 1 INTRODUCTION

In 1992 a fact-finding mission was sent to Japan by the Dutch government, which reported that it should be possible to construct bored tunnels in the Dutch soft soil conditions. Up to that time essentially only immersed and cut-and-cover tunnels were constructed in the Netherlands, as boring of tunnels in soft soil conditions, at that time, was considered to be too risk full.

After this conclusion things went quite fast; in 1993 the Dutch minister of Transport and Public works ordered the undertaking of two pilot projects, the 2nd Heinenoord Tunnel and the Botlek Rail Tunnel. The projects were primarily aimed at constructing new infrastructure and besides that for monitoring and research in order to advance the development of this new construction method for the Netherlands. The projects started in 1997 and 10 years have passed since then.

At the start of the pilot projects, the difficulties with respect to constructing bored tunnels in soft soil conditions were evaluated and a plan for monitoring and research was put forward, see Bakker et al (1997). Since then, the 2nd Heinenoord tunnel, see Fig. 1, and a series of other bored tunnels were constructed.

After the completion of the pilot projects a Joint Platform for Bored tunnels was established (GPB) that coordinates the monitoring and research at the various

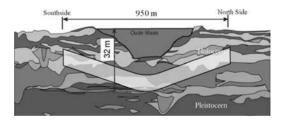


Figure 1. Geological profile at the 2nd Heinenoord tunnel.

other Dutch tunnelling projects. The GPB, an initiative of the larger clients for underground infrastructure on the government side, was organised under supervision of the Netherlands Centre for Underground Construction; COB. The research was organised in such a way that results of a project would be beneficial for a next project starting a little later.

Unquestionably a lot has been learned from the performed monitoring and research. The results of this process have been noticed abroad. In 2005 the Netherlands hosted the fifth International symposium of TC28 on "Underground Construction in Soft Ground". Researchers and experts from all over the world came to Amsterdam, to learn about the Dutch observations on tunnelling and to visit the construction works for the new North-South city metro system in Amsterdam. Table 1. Bored tunnels completed after 1997 in the Netherlands.

		Completion (Year)	Bored length (m)	Depth (m)	Outward Diameter (m)
2nd Heinenoord tunnel	Road	1999	945 (dual)	30	8.3
Western Scheldt tunnel	Road	2003	6700 (dual)	60	11.30
Botlek Rail tunnel	Rail	2004	1835 (dual)	26	9.60
Sophia Rail tunnel	Rail	2005	4000 (dual)	27	9.60
Pannerdensch Canal Rail tunnel	Rail	2005	1615 (dual)	25	9.60
Green Hart tunnel	Rail	2006	8.620 (single)	30	14.90

The above event was also the occasion for the presentation of a book; "A decade of progress in tunnelling in the Netherlands" by Bezuijen and van Lottum (2006), where this research is described in more detail.

This paper(s) gives some highlights of the main research result of the past decade.

# 2 REVIEW OF THE 1997 SITUATION AND WHAT CAME AFTER

In the design phase for the 2nd Heinenoord tunnel a main concern were the soft soil conditions in combination with high water pressures. In general in the Netherlands the water table is just below the soil surface. Furthermore the 8.3 m outward diameter for this first large diameter tunnel was a major step forward, compared to past experience in the Netherlands; experience that was mainly based on constructing bored tunnels, pipes or conduits up to about 4.0 m diameter. This gave some concern with respect to the amount of extrapolation of empiric knowledge.

With respect to the soft-soil conditions, the low stiffness of the Holocene clay and peat layers and the high groundwater table; nearly up to the soil surface, were considered a potential hazard and a challenge for bored tunnels. The soil profile at the 2nd Heinenoord tunnel, see Fig. 1, is indicative for the heterogeneous character and on occasion sudden changes in underground soil layering that one might encounter. In addition to the heterogeneity and the ground water, deformations due to tunnelling might influence the bearing capacity of any existing piled foundations in the vicinity. And as the common saying is that the Amsterdam Forest is underground, one might realize the potential risks involved for the North/South Metro works in Amsterdam.

Characteristic for a high water table is buoyancy; the effect that the tunnel might be floating up into the soft upper layers above the tunnel due to the gradient in the groundwater pressure. Besides the risk of breaking up of these soil layers, the rather flexible bedding of the tunnel and the deformations that this may cause need to be analysed. Therefore research was aimed at clarifying the effects of the soft underground, groundwater effects, and the effect of tunnelling on piled foundations.

After the successful construction of the two Pilot projects, a number of other bored tunnelling projects followed, see Table 1. Mention worth is that the Green Hart Tunnel holds until recently the record as the largest diameter bored tunnel in the world.

Still under construction are the tunnels for RandstadRail in Rotterdam, the Hubertus Tunnel for a road in The Hague and the North/South metro works in Amsterdam.

With respect to the construction of the North/South metro works in Amsterdam, the station works have made quite some progress and the bored tunnel is in a preparation phase. The elements of the immersed tunnel; the extension to Amsterdam North under the river IJ, are waiting for the completion of the immersion trench under the Amsterdam Central Station. For the bored tunnelling part, the TBM is expected to start excavation at the end of 2008.

Ten years after the pilot projects, the question arises whether the observations and related research have confirmed the above issues to be the critical ones or that advancing insight may have removed these issues from the "stage" and swapped these for other topics giving more concern.

In this paper some of the characteristic events and results of this past decade will be described. The choice for the topics being discussed is influenced by the projects that both authors were involved with, without intent to minimize the importance of other research that is not discussed in this paper. Further issues related to groundwater effects and grouting are described in more detail in a separate paper in this symposium by Bezuijen & Talmon (2008).

# 3 EXPERIENCES WITH BORED TUNNELS IN THE NETHERLANDS IN THE PAST DECADE

## 3.1 An instability of the bore front

During the construction of the 2nd Heinenoord Tunnel, approximately in the middle underneath the river Oude Maas an instability at the excavation front developed,

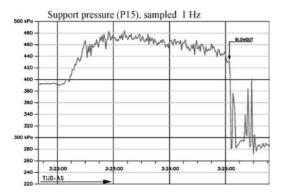


Figure 2. Support pressures before, during and after the "Blow out" at the 2nd Heinenoord tunnel.

see Fig. 2; afterward commonly referred to as "The Blow-out" (see also Bezuijen & Brassinga, 2001).

Backtracking the situation learned that after that a pressure drop was observed, in his efforts to restore frontal support, the machine driver first pumped bentonite to the excavation chamber; considering a deficiency in the bentonite system. When this did not help, air was pumped to the bore front; not realizing that the front itself already had collapsed. This collapse created a shortcut between the excavation chamber and the river. The action of pumping air was noticed by shipmasters on the river, which reported air bubbles rising to the water surface, which caused the failure to be known as the "blow-out". In this case the pumping of air had not been beneficial to the restoration of stability because pressure loss was not the cause but one of the results of the event.

This frontal stability at the 2nd Heinenoord tunnel has attracted some public attention. Presumably it is less known that loss of frontal stability has also occurred since then with some regularity at the other tunnels under construction in the years after, e.g. during construction of the Sophia Rail Tunnel and the Green Hart Tunnel, however without much delaying the construction process. At the 2nd Heinenoord Tunnel, construction work was delayed for several weeks before the crew succeeded in restoring frontal stability, filling up the crater in the river bottom with clay and bringing in swelling clay particles in the excavation room.

From the evaluation of the monitored pressures in the excavation room, it appeared that before the development of the instability, the frontal pressure was raised above the advised pressure for frontal support; i.e. at about 470 kPa instead of about 310 kPa. see Fig. 2 (pressure gauge P15 is in the excavation chamber at tunnel axis level).

In retrospect it was understood that during standstill, the pressures were raised to get a larger gradient in the pipes in order to improve the transport of excavated

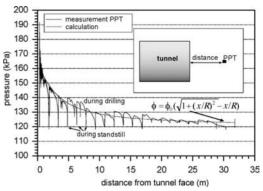


Figure 3. Pore water pressure distribution in front of the TBM.

material; i.e. Kedichem clay that was found in the lower part of the excavation front and appeared to be difficult to pump through the hydraulic muck transport system.

The measurements indicate that excavation had started without releasing pressure to the standard support level during excavation. In that condition instability developed within 15 seconds after that the wheel started cutting. At stand still, when sufficient time has passed for a proper vertical cake sealing of bentonite to build up at the front, a high support pressure is not much of a problem, as the pressures used are way below those that might override the passive resistance at the front. However, as the pressure itself is fluid pressure, when the cake-sealing is taken away during excavation, and water can penetrate the front, according to Pascal's law for a fluid without shear stresses, the pressure also works in the vertical direction, and if this pressure exceeds the vertical soil pressure this will trigger an uplift and possibly a breaking out of soil layers, and apparently that is what has happened here. In their paper on face support Jancsecz and Steiner (1994), for this reason gave a warning about the limits to the face support pressure, for situations with little overburden.

Research learns that for the fine sand that we have in the Pleistocene sands layers in the Netherlands, penetration of bentonite in the pores is negligible. Excavation therefore means removal of the cake sealing; Research by Bezuijen and Brassinga (2001), indicates that it normally takes about 4 to 5 minutes to build up a new cake sealing after the excavation wheel has removed the sealing during excavation. The time between passings of chisels, in the order of 20 seconds is too short for that. It must be emphasized that this effect is not only important for the upper limit to face support pressures, but may also give a limitation to the lower limit of the support pressure. A method to discount for this effect was given by Broere (2001), see also Fig. 4.

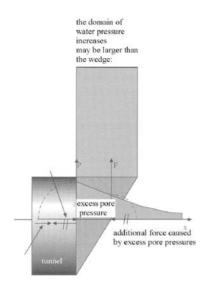


Figure 4. The effect of removal of the cake sealing during excavation on pore-pressures in the front. The influence zone for excess pore-pressures may be larger that the zone normally considered in stability analysis.

The situation of a low soil cover underneath the river bottom is not the only situation that might be critical to the above effect, also if the soil cover itself is relatively light, such as in the case of the thicker layers of peat overlaying the sand where the Green Hart Tunnel was excavated, this might lead to a critical situation. A local failure might be triggered where the generated excess pore pressure in front of the tunnel face can lift the soft soil layers.

The knowledge gained with the monitoring of the 2nd Heinenoord tunnel was applied for the Green Hart tunnel, and may have prevented instabilities at the bore front at larger scales; see Bezuijen et al. 2001 & Autuori & Minec (2005).

# 3.2 Tail void grouting and grouting pressures

To measure the soil pressures on a tunnel lining is difficult. In the start-up phase for the monitoring of the 2nd Heinenoord Tunnel, a number of international experts on tunnel engineering advised not to put too much effort on this topic, as "the results would probably be disappointing". Due to the hardening of the grout, the period for meaningful pressure measurements would be short and to prevent bridging effects the size of the pressure cells would have to be large and therefore costly.

Still, against this advice, the measurement of grouting pressures was undertaken, and repeated for a number of tunnel projects. It appeared that the interpretation was difficult when the grout has hardened, but for the fresh grout until 17 hour after injection

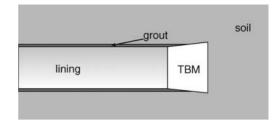


Figure 5. Under circumstances the Grout material from the tail void might flow into the gap behind the tail of the TBM, giving cause to increased loads.

it was possible to give an accepted interpretation of the measurement results (Bezuijen & Brassinga, 2004), and a lot of experience has been gained that has contributed to a better understanding of the grouting process and the pressures acting on the tunnel lining. With these results it was possible to predict grouting pressures and tunnel loading, see Talmon & Bezuijen (2005).

Based on various evaluations of the force distribution in the tunnel lining, see amongst others, Bakker (2000), it came forward that the initial in-situ soil stresses around the tunnel do not have a dominant influence on the compressive loading of the tunnel. Due to the tapering of the TBM and in spite of the tail void grouting there is a significant release of the radial stresses around the tunnel, see Fig. 5.

The final loading on the lining relates more to the effectiveness of the grouting process, the distribution of the grout openings and the consolidation of the grout than to the initial in-site soil stresses, see Bezuijen et al. (2004). Whether this reduction of the in-situ radial stresses is a lasting effect that will remain for the full lifespan of the tunnel may depend on the creep sensitivity of the soil, see also Brinkgreve and Bakker (2001), and Hashimoto et al. (2008).

# 3.3 Surface settlements

Hoefsloot et al. (2005), have shown that the application of a stress boundary condition between tunnel and soil in 3D tunnel analysis has a better corroboration between measurement and calculation of soil deformations around the tunnel and subsequently of the loading on the tunnel, than the application of the so called "contraction method".

Although this effect was known in the literature, see for example Mair and Taylor (1997), for the research team that worked at the 2nd Heinenoord tunnel the observation that the numerical predictions of surface settlements lacked accuracy was disappointing. At the start the expectations on numerical analysis had been quite high. Shortly after the first observations were evaluated it was realized within the team, that it were only the empirical predictions by Peck (1969) for a

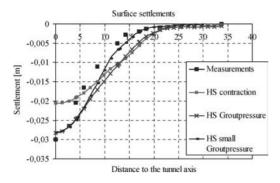


Figure 6. Surface settlements; measured and back-calculated with different material models.

volume loss of about 1% that gave a reasonable corroboration with the measurements. The finite element calculations, at that time mainly based on an application of the elastic-plastic Mohr-Coulomb model in combination with a contraction type of modelling for the tail void loss, predicted a too wide and too shallow surface settlement.

This disappointing result created a problem for the intentions to apply 3D numerical analysis in deformation predictions for tunnel projects in urban areas, such as for the Amsterdam North-South line metro works.

Since then, however, a lot of effort has been put in the improvement of the numerical prediction of soil deformations. To begin with it was the project team for the Amsterdam Metro works, see Van Dijk & Kaalberg (1998), that gave a first indication for an improvement, with the proposal to model the stresses at the tunnel soil interface instead of the deformations. With the introduction of this grout pressure model the results improved. Later on, when the physics in the process became better understood, i.e. the importance to account for the high stiffness of the soil in unloading, double hardening was introduced with the development of Hardening Soil, as a material model; with this development, the calculation results largely improved compared to the measurements, see Fig. 6. The latest development is the introduction of small strain stiffness in the Hardening Soil Model, see Benz (2006), which up to now gives the best results, see Möller (2005).

Theoretically the result might further be improved introducing anisotropy in the model; such models are being developed nowadays, e.g. in the framework of European Research; AMGISS, e.g. see www.ce.strath.ac.uk/amgiss.

#### 4 EVALUATION OF THE LEARNING ISSUES

Nowadays it's not the soil deformation during "normal" excavation process that makes us worry about surface settlements. With an average tail void loss of

about 0.5% of the diameter or less, the deformation might only be a problem for situations of underexcavation of buildings or if the structures are located very close to the excavation track. For tunnels in urban area, there is more concern with respect to bore-front stability; especially when the upper stratum of the soil above the Pleistocene layers, where the tunnels are usually positioned in, consists of soil with a relative low density, as in the Netherlands. For these situations with relatively light upper layers of peat or clays with organic parts, one has to be very careful controlling the support pressures during excavation, as on the one hand there is a lower bound value of the support to prevent cave in, but on the other hand, the upper limit triggered by an uplift of light upper layers may also be not far. This will limit the pressure window to work in.

Front instability has occurred at various tunnelling projects in the Netherlands. If the tunnel is outside any urban area this might not give too much problems; however if the tunnel is underneath a city road system, or close to pile foundations this may cause severe problems, as instability might cause a sinkhole in the pavement and foundation settlements.

With respect to the accuracy in the prediction of soil deformations: Apart from the well known empiric model of Peck (1969) that predicts the shape of the settlement trough but not the volume loss, the numerical models have become quite reliable in predicting surface and subsurface deformations, both vertical and horizontal. The improvement, mainly achieved in 2D analysis has opened up the possibility for a reliable deformation analysis in 3D of tunnelling in urban areas. For an adequate prediction of deformations it is important to model the grouting pressures as a boundary condition to the excavation, in combination with the application of a higher order material model, that takes into account the small strain deformation behaviour of sand, see Benz (2006).

Further it is recommended, and planned for, to integrate the Delft Cluster Grout pressure model in the Plaxis 3D Tunnel software. The latter would contribute to the applicability of the numerical models as a more general tool for underground construction. This would enable a better analysis for the loading on the tail of the TBM and of the tunnel lining.

Within certain limits some cost saving structural improvements are expected to be possible and, even more important, insight is obtained in the mechanism involved.

# 5 CONCLUDING REMARKS

Ten Years have passed since the first large diameter bored tunnelling project in the Netherlands in Soft soil was undertaken. Since then some world records with respect to tunnelling have been broken in the Netherlands; i.e. the largest diameter (for the Green Hart Tunnel), the highest outside pressure on a segmental tunnel (for the Westernscheldt Tunnel), the application of an Earth Pressure Balance shield in coarse sand, and the largest length of constructed tube in one day, (Pannerdensch Canal Tunnel).

Before the underground construction works were started, and the tunnelling projects were in a pre-design stage, the softness of the Netherlands underground attracted a large part of the attention, see Bakker (1997). In retrospect the influence of a low stiffness as a source of risk and influence on underground construction was confirmed, but sometimes in a different perspective, or related to other physical processes than foreseen.

With respect these new insights the following conclusions were drawn:

- 1 The low stiffness of the soil may also lead to increased flexibility of the tunnel tube. The deformation of the tube during hardening of the grout, and the additional Eigen stresses that this may cause is still a research topic.
- 2 For a proper prediction of surface settlements and soil deformations, it is important to model the grouting pressures at the interface between soil and tunnel (or grouting zone). Further to improve the prediction of the width of the settlement trough, the use of small strain analysis is advised.
- 3 During excavation in fine sand, such as the Pleistocene sand layers in the Netherlands, the supporting cake fluid will be removed by the chisels on the excavation wheel. Therefore, in cases of limited overburden the upperbound to the support pressure must be carefully determined to prevent instability of the overlaying soil.
- 4 In addition; for the determination of the lower limit to the support pressure, the increased pore pressures in the front also needs to be taken into account.

With acknowledgement to the Netherlands Centre for Underground Construction for their consent to publish about the research they commissioned and coordinated.

# REFERENCES

- Autuori, P. & Minec, S. 2005. Large diameter tunnelling under polders, Proceedings 5th Int. Symposium on Underground Construction in soft Ground, IS-Amsterdam 2005.
- Bakker, K.J., v.d. Berg, P. & Rots, J. 1997. Monitoring soft soil tunnelling in the Netherlands; an inventory of design aspects, Proc. ISSMFE, Hamburg.
- Bakker, K.J. 2000. Soil Retaining structures, development of models for structural analysis, Balkema, 2000, Rotterdam.
- Benz, T. 2006. Small strain stiffness of soils and its consequences, Doctor Thesis, IGS Universität Stuttgart.

- Bezuijen, A. & Brassinga, H.E. 2001. Blow-out pressure measured in a centrifuge model and in the field. Proc. Int. Symp. on Modern Tunnelling Science and Techn. Kyoto.
- Bezuijen, A., Talmon, A.M., Kaalberg, F.J. & Plugge, R. 2004. Field measurements of grout pressures during tunnelling of the Sophia Rail Tunnel. Soils and Foundations vol, 44, No1, 41–50, Feb.
- Bezuijen, A. & Talmon, A.M. 2005. Grout properties and their influence on back fill grouting. *Proceeding 5th Int. Symposium on Underground Construction in soft Ground*, IS-Amsterdam 2005.
- Bezuijen, A., Pruiksma, J.P. & van Meerten, H.H. 2001. Pore pressures in front of tunnel, measurements, calculations and consequences for stability of tunnel face. *Proc. Int. Symp. on Modern Tunnelling Science and Techn. Kyoto.*
- Bezuijen, A. & van Lottum, H. (eds). 2006. Tunnelling A Decade of Progress. GeoDelft 1995–2005.
- Bezuijen, A. & Bakker, K.J. 2008. The influence of flow around a TBM machine. Proceeding 6th Int. Symposium on Underground Construction in soft Ground, Shanghai.
- Bezuijen, A. & Talmon, A.M. 2008. Processes around a TBM. Proceeding 6th Int. Symposium on Underground Construction in soft Ground, Shanghai.
- Brinkgreve, R.B.J. & Bakker, K.J. 2001. Time-dependent behaviour of bore tunnels in soft soil conditions; a numerical study, *Proceedings*, *ICSMGE* Istanbul, Turkey.
- Broere, W. 2001. Tunnel face stability & new CPT applications, Delft University Press.
- Hashimoto, T., Ye, G.L., Nagaya, J., Konda, T. & Ma, X.F. 2008. Study on earth pressure acting upon shield tunnel lining in clayey and sandy grounds based on field monitoring. *Proceeding 6th Int. Symposium on Underground Construction in soft Ground*, Shanghai.
- Hoefsloot, F.J.M. & Verweij, A. 2005. 4D grouting pressure model PLAXIS. Proceeding 5th Int. Symposium on Underground Construction in soft Ground, IS-Amsterdam 2005.
- Jancsecz, S. & Steiner, W. 1994. Face Support for a large Mix-Shield in heterogeneous ground conditions. *Tunnelling 94*, British Tunnelling Association, 5–7 July 1994.
- Mair, R.J. & Taylor, R.N. 1997. Theme Lecture: Bored tunnelling in the urban environment. *Proc. 14th ICSMFE*, Balkema, Rotterdam.
- Möller, S.C. & Vermeer, P.A. 2005. Prediction of settlements and structural forces in linings due to tunnelling. *Proceed*ing 5th Int. Symposium on Underground Construction in soft Ground, IS-Amsterdam 2005.
- Pachen, H.M.A., Brassinga, H. & Bezuijen, A. 2005. Geotechnical centrifuge tests to verify the long-term behaviour of a bored tunnel, *Proc. 5th Int. Symposium on Underground Construction in soft Ground*, IS-Amsterdam 2005.
- Peck, R.B. 1969. Deep excavations and Tunnelling in soft Ground. *Proceedings 7th ICSMFE* Mexico.
- Talmon, A.M. & Bezuijen, A. 2005. Grouting the tail void of bored tunnels: the role of hardening and consolidation of grouts. *Proceeding 5th Int. Symposium on Underground Construction in soft Ground*, IS-Amsterdam.
- van Dijk, B. & Kaalberg, F.J. 1998. 3-D geotechnical model for the North/Southline in Amsterdam. In A. Cividini (Ed.), Application of numerical methods to geotechnical problems, Wien, 739{750. Springer-Verlag.

# Ten years of bored tunnels in The Netherlands: Part II structural issues

# K.J. Bakker

COB, Delft University of Technology, Delft, The Netherlands

# A. Bezuijen

Deltares, Delft University of Technology, Delft, The Netherlands

ABSTRACT: In 1997 for the first time construction of bored tunnels in the Netherlands soft soil was undertaken. Before that date essentially only immersed tunnels and cut-and-cover tunnels were constructed in the Netherlands. The first two bored tunnels were Pilot Projects, the 2nd Heinenoord tunnel and the Botlek Rail tunnel. Since then a series of other bored tunnels has been constructed and some are still under construction today. At the beginning of this period, amongst others Bakker (1997), gave an overview of the risks related to bored tunnels in soft ground and explained about a plan for research related to the Pilot projects. Ten years have passed, a lot of monitoring and research has been done. In this paper that is split in two parts a summary is given of some of the most characteristic observations of these past 10 years of underground construction in the Netherlands. In this second part, the emphasis will be on structural related issues discussed whereas in part one, frontal stability, grouting and soil deformations are discussed.

# 1 INTRODUCTION

In 1992 the Dutch government sent a fact-finding mission to Japan, to report on the possibility to construct bored tunnels in the Dutch soft soil conditions. Up to that time essentially only immersed and cut-and-cover tunnels were constructed in the Netherlands, as boring of tunnels in soft soil conditions, at that time, was considered to be too risk full.

After the report, that advised positive, things went quite fast; in 1993 the Dutch minister of Transport and Public works ordered the undertaking of two pilot projects, the 2nd Heinenoord Tunnel and the Botlek Rail Tunnel. The projects were primarily aimed at constructing new infrastructure and besides that for monitoring and research in order to advance the development of this new construction method for the Netherlands. The projects started in 1997 and 10 years have passed since then.

At the start of the pilot projects, the difficulties with respect to the construction of bored tunnels in soft soil conditions were evaluated and a plan for monitoring and research was put forward, see Bakker (1997). Since then, the 2nd Heinenoord tunnel, and a series of other bored tunnels were constructed. Unquestionably a lot has been learned from all the monitoring and research that was performed.

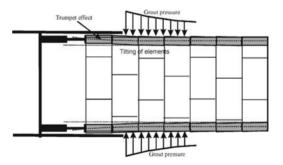


Figure 1. Trumpet effect in tunnel ring construction.

The results of this process have been noticed abroad. In 2005 the Netherlands hosted the fifth International symposium of TC28 on "Underground Construction in Soft Ground". The above event was also the occasion for the presentation of a book; "A decade of progress in tunnelling in the Netherlands" by Bezuijen and van Lottum (2006), where this research is described in more detail.

In the present paper some highlights of the main research result of the past decade will be given. The paper is split in two parts, where part one includes some general observations and discusses face support, grouting and surface settlements, whereas part two is more about structural issues.

# 2 REVIEW OF THE 1997 SITUATION AND WHAT CAME AFTER

A main concern with respect to boring tunnels in the Netherlands were the soft soil conditions; the low stiffness of the Holocene clay and peat layers and the high groundwater table; nearly up to the soil surface were considered a potential hazard and a challenge for bored tunnels.

Furthermore the 8.3 m outward diameter for the first large diameter tunnel was a major step forward, compared to past experience; experience that up to that time was mainly based on constructing bored tunnels, pipes or conduits up to about 4.0 m diameter.

In addition to that, in general deformations due to tunnelling might influence the bearing capacity of any existing piled foundations in the vicinity. And as the common saying is that the Amsterdam Forest is underground, one might realize the potential risks involved for the North/South Metro works in Amsterdam.

Characteristic for a high water table are buoyancy effects. Besides the risk of breaking up of the soft upper soil layers, the rather flexible bedding of the tunnel and the deformations that this may cause need to be analysed. Therefore research was aimed at clarifying the effects of the soft underground, groundwater effects, and the effect of tunnelling on piled foundations.

Ten years later, the question arises whether the observations have confirmed the above issues to be the critical ones. In this paper some of the characteristic events and results of this past decade will be described. The choice for the topics being discussed is influenced by the projects that both authors were involved with, without intent to minimize the importance of other research that is not discussed in this paper.

# 3 EXPERIENCES WITH BORED TUNNELS IN THE NETHERLANDS IN THE PAST DECADE

# 3.1 Structural damage

An early experience with the difficulties for bored tunnels in soft ground was the damage to the lining that occurred during the first 150 metres of construction of the 2nd Heinenoord Tunnel. On average the damage was too high compared to experiences from abroad and was considered to be unacceptable. Although, the integrity of the tunnel was not at stake, there was worry about the durability of the tunnel and the level of future maintenance.

Characteristic to the damage was cracking and spalling of concrete near the dowel and notches see Fig. 2. Quite often the damage was combined with



Figure 2. Damage to the Dowel and notch sockets.

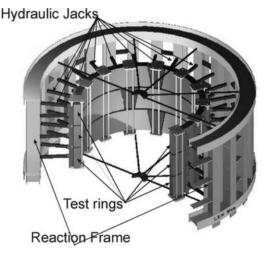


Figure 3. Large-scale tunnel ring testing in the Stevin Laboratories at Delft University (the diameter of the (gray) inner ring is 8.3 m).

differential displacements between subsequent rings and with leakage. The evaluation report, see Bakker (2000), attributed the damage to irregularities in the construction of the lining at the rear of the TBM and subsequent loading during TBM progress. Further a correlation of the damage with high jack forces was observed; these appeared to be necessary to overcome the friction in this part of the track, which prevented smooth progress.

With respect to the tunnel ring construction, it is difficult to erect a stress free perfect circular ring. The



Figure 4. Test site for the Pile-tunnel interaction test.

ring needs to be built onto the end of a former ring that already has undergone some loading and deformation from the tail void grouting while it partially has left the tail of the TBM, see Fig. 3. The further deformation is characterised by the trumpet shape of the tubing that develops, see Fig. 1, with the inevitable related stress development in the lining. The trumpet shape and the high jacking forces lead to local stress concentrations and irregular deformations in the lining and occasional to slipping between the different tunnel elements. The slipping of elements was blamed to the use of a bituminous material called Kaubit in the ring joint.

Originally Kaubit strips had been used in the ring joint. These Kaubit strips, of flexible bituminous like material, were used to prevent the occurrence of stress concentrations; so some slipping was meant to occur, but the "dynamic" character of the slipping that actually occurred that influenced the final geometry of the lining and had triggered cracking was unexpected. Especially the cracking and overloading of the dowel and notch system was unforeseen.

Failure of the dowel and notch system, see Fig. 2. led to spalling and in some cases to leakage. In the cases that leakage was observed this must have been correlated to damage to the notch at the outer side of the lining, creating a shortcut to water penetrating behind the rubber sealing there.

After the main conclusions were drawn, it was decided to exchange the Kaubit strips for thin plywood plates. Due to the larger stiffness and shearing resistance, shearing of the concrete elements at large was further prevented and the damage limited.

Besides this technical measure, the evaluation was the trigger for the undertaking of fundamental research into lining design that included large scale physical testing of tunnel tubing at Delft University see Fig. 4. In this project that was a combined effort of physical and numerical testing, the details of assembling tunnel segments into subsequent tunnel rings and these into a tube were investigated. Amongst others the main results of the project were reported by Blom (2002),

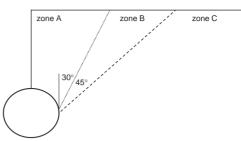


Figure 5. Zones that indicate different effects on piles foundations.

and Uijl et al (2003). Based on this research it was decided to omit the dowel and notches for the Green Hart tunnel; which led to a nearly damage free tunnel lining.

A different issue, not settled yet, is the durability of plywood and the consequences of wood rot on the long-term tunnel behavior. An unwanted loss of the longitudinal pre-stress of a tunnel might influence the tunnel flexibility and deformations, possibly leading to leakages. On the other hand, experience learns that compression largely increases the durability of wood. The ply wood material is compressed to a strain of more than 50% during tunnel construction. At such a high level of straining the wood cells might have collapsed.

# 3.2 Deformations of the TBM machine during construction of the Westernscheldt tunnel

During construction of the first tube for the Westernscheldt tunnel, unexpected deformations of the tail of the TBM were observed; i.e. the air space between tubing and tail of the TBM narrowed at a certain stage in an unexpected way. The shape of the observed deformations did not coincide with the assumed soil loading and gave the impression that it was a large deformations effect; i.e. buckling.

At first buckling was not accepted as a cause because the tapering of the TBM was assumed to give sufficient stress release to guarantee a sufficient decrease in isotropic stress. Further a certain bedding effect was assumed to be always present and the combination would make buckling unlikely. Buckling would only be plausible for a much higher loading of the tail of the TBM in combination with the absence of any bedding reaction.

However, the insights have changed since then. In general there may be no overall contact between the soil and the tail of the TBM; when grout is injected in the tail void, the increased pressure on the soil, compared to the original stress will push the soil from the TBM and grout will flow between the TBM tail and the soil, see Fig. 5 in part I of this paper. This means that the pressures on the TBM tail are higher than anticipated in the past and there might be no bedding reaction. This could well explain the occurrence of buckling and the deformations of the TBM tail.

A 1-D calculation model has been developed and is verified with FEM simulations (Bezuijen & Bakker, 2008). This model shows that also the high stiffness of soil during unloading, which led to the HS and the HS<sub>small</sub> material models, made it likely that the common tapering, approximately equal to an equivalent volume loss of 0.4%, is sufficient to lose the larger part of the effective radial stresses, which helps to develop a gap between the tail of TBM and the soil.

The grout pressures exerted on the tail of TBM might be much higher than the soil stresses, and in absence of bedding, buckling could well explain for the deformations.

# 3.3 *The influence of tunnel boring to piled foundations*

Large scale testing of pile foundations was performed during construction of the 2nd Heinenoord Tunnel. This was done in order of the Project Bureau of the Amsterdam North/South metro works to get a better understanding of the processes.

A trial field with loaded piles and pile configurations was installed in the area near and above the track of the TBM, see Fig. 4. One of the main concerns was that due to an increase in pore pressure the effective stresses around the pile tip might be affected and that a release in isotropic stresses might trigger a drop in pile bearing capacity.

However, against this reasoning there is also numerical and analytic evidence, (assuming cylinder symmetric analysis), that indicates that the release in stresses due to tunnelling is limited to a rather small plastic zone in the close vicinity of the tunnel lining, see also Verruijt (1993). The analytical model reveals that strain as a function of the distance drops as a function of  $1/r^2$ , which would indicate that the influence zone would be limited in size. This reasoning in combination with the fact that the strains due to tunnelling in general are quite small; the largest strains often being less than 0.5 or 1.0%, makes plastic zones further away than D/2, measured from the tubing, unlikely. Only above the tunnel this zone can be larger.

However, reasoning and analysis is one thing; measuring and validation is another; based on the field measurements and physical model research in Delft and Cambridge Kaalberg et al. (2005), proposed a zoning as shown in Fig. 5, with the following indicators; a zone "A" above the tunnel where the settlement of a pile is expected to be larger than the soil displacements. A zone "B" adjacent to the tunnel, with an inclined influence line, where the pile will follow the soil deformation at the tip of the pile, and further a zone "C", outside Zone B, where at soil surface level the settlement of the pile will be less than that of the soil surface. This zoning proposal more or less coincides with the main results as published by Selemetas (2005) that were mainly based of physical testing in a geotechnical centrifuge.

The results published by Kaalberg et al. and others are valid for the average volume loss that can be expected during tunnelling (0.5 to 1%) Earlier centrifuge testing by GeoDelft indicated that larger deformation effects are possible for higher volume losses (up to 7% was tested). Such volume losses are well above nowadays practice, but it means that during a calamity, piles over a larger area may be affected.

# 3.4 Longitudinal deformations of the tunnel tube

In the paper by Bakker et al. (1997), the development of longitudinal stresses in a tunnel lining due to irregular bedding in soft soil was mentioned as an item for research. Irregular bedding that could be the result of zones with different elasticity or else due to the stiff foundation of a shaft or bedding in the deeper Pleistocene layers; especially near the transition between Holocene and Pleistocene layers. The measurement of longitudinal stresses in itself has turned out to be cumbersome. Within the monitoring scheme for the 2nd Heinenoord a trial measurement was undertaken. In addition to that measurements from the Sophia Rail Tunnel were back-analysed with 4D finite element analysis, i.e. (time dependant 3D analysis), and after that the longitudinal stresses were also measured during the construction of the Green Hart Tunnel.

To begin with the latter situation: measurements were taken with a tubular liquid level devise of the longitudinal deformations of the tunnel during the grouting process. From these measurements the observation came forward that the tubing exhibited large vertical movements, up and down, between 20 to 30 mm during excavation and tail void grouting was measured, and a total vertical shift of the tubing vertical of about 60 mm at one location (See also Talmon & Bezuijen, 2008).

This amplitude was surely unexpected and is not fully accepted yet. Nevertheless it is clear that vertical deformations do occur in the zone where the grout material is still fluid, and during excavation and may lead to an alternating deformation; upwards when the TBM is excavating and grouting and downwards if the TBM is at stand still.

With respect to the 3D staged construction analysis of tunnel construction for the Sophia Rail Tunnel, that was undertaken for the COB F220 committee, a combined DIANA and PLAXIS 3D analysis was performed, see Hoefsloot et al. (2005). The outcome of these various analyses more or less coincided; which might have been expected as the mathematical base of both models is quite similar, and in general deformations remain small, so the soil reactions will most probably mainly have been elastic.

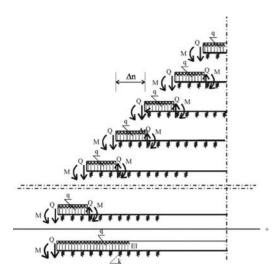


Figure 6. Conceptual model for the analysis of beam effects in the tube of a bored tunnel by Boogaards (1999).

The main conclusion with respect to this effect was that this issue can be properly analysed with a relatively simple model based on the concept of a beam with an elastic bedding and a series summation, such as developed by Boogaards & Bakker (1999), see Fig. 6 and later on applied by Hoefsloot (2002). See Fig. 7 for a comparison between model outcome and measurements from the 2nd Heinenoord tunnel.

However, using generally accepted parameters, the measured deformations are much higher than according to these models. Recently, Talmon et al. (2008) have presented results that may explain the lower stiffness that are found in the measurements (the lining stiffness can be lower due to only local contact between the elements and the soil stiffness reduces due to unloading of the soil around the tunnel), but these are not yet generally accepted.

# 4 CROSS PASSAGES

The design for the Westernscheldt tunnel in the Netherlands did trigger a debate on tunnel safety. Some major accidents with tunnel fires, such as occurred in the Channel tunnel and at the Mont Blanc tunnel in the Alps did reveal the vulnerability and relative unsafe situations in tunnels with oncoming traffic or in a single tube in general.

For the Westernscheldt tunnel, a twin tunnel with one way traffic per tube, the discussion focussed on what distance between cross passages would be acceptable to guarantee that escaping people would be able to find a safe haven by entering the other tube; assuming that the traffic is stopped, by an automatic control system. The outcome of these safety studies was a cross

Longitudinal bending moment in the tube

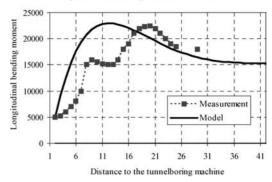


Figure 7. Longitudinal bending moment due to the construction forces measured and calculated acc. to Bogaards model.

connection at least every 250 m, which is nowadays more or less the reference situation in the Netherlands.

The task to construct these cross passages is a further technical effort. During the construction of the Botlek Rail Tunnel a vertical shaft and freezing were the main construction techniques as the cross passages could be positioned outside the area under the Oude Maas River. The positive experience with freezing for the Botlek Rail Tunnel was helpful in the decision making for the Westernscheldt Tunnel, but there the freezing was done from the tunnel tube as the track underneath the estuary is too long and too deep with respect of the water table to enable the shaft type method.

Although the method in itself is costly, its reliability is an important advantage and therefore it is also used for the cross passages of the Hubertus Tunnel and is expected to be used in future projects. For the single tube Green Hart Tunnel tunnel safety is achieved by construction of a separation wall with doors.

# 5 EVALUATION OF THE LEARNING ISSUES

The research on grout pressures, in combination with the structural research on lining design has gained us the insight that the lining thickness and the necessary reinforcement are mainly determined by the loading in the construction phase and to a lesser degree to the soil pressures. In engineering practice the thickness and reinforcement of the tubing is mainly determined by the most unfavourable jackforces during TBM excavation in combination with an unfavourable tail void grouting scenario. Difficulty with these is that it's the contractor's prerogative to decide on the necessary jack-forces that will enable him to construct the tunnel and also what scenario he will use for the tail void grouting. This might lead to conservative assumptions in the design office in order to avoid liabilities if a problem would occur during construction.

With respect to the generality of this conclusion it has to be considered that the main observations that were discussed relate to tunnels that are safely located in stiff Pleistocene sand layers. We must however consider the possibility of tunnels in softer soil layers that are more susceptible to consolidation and creep. Consolidation and creep might counteract the general tendency of stress release and arching in the soil and lead to a much higher radial loading. One might think of a soil pressure on the lining that may be on the level of the initial soil stresses before tunnel construction; the K<sub>0</sub> stress situation or even higher than these initial stresses. Such a situation was accounted for in the design for RandstadRail in Rotterdam, where a full steel lining was chosen for a part of the track where the tubing mainly rests in the upper much softer Holocene clay and peat layers, that foreseeable would have an extra loading on the lining due to consolidation and creep (Pachen et al. 2005).

However, with respect to lining design, within certain limits some cost saving structural improvements are expected to be possible and, even more important, insight is obtained in the mechanisms involved.

# 6 CONCLUDING REMARKS

Ten Years have passed since the first large diameter bored tunnelling project in the Netherlands in Soft soil was undertaken. Before the underground construction works were started, and the tunnelling projects were in a pre-design stage, the softness of the Netherlands underground attracted a large part of the attention, see Bakker (1997). In retrospect the influence of a low stiffness as a source of risk and influence on underground construction was confirmed, but sometimes in a different perspective, or related to other physical processes than foreseen.

With respect to the new insights gained the following conclusions were drawn:

- 1 The low stiffness of the ground support may give rise to increased vulnerability of the lining for jacking forces by the TBM during excavation. Care must be taken to precise shape of the elements and joints to prevent too high stresses during assembly.
- 2 The low stiffness of the soil may also lead to increased flexibility of the tunnel tube. The deformation of the tube during hardening of the grout, and the additional Eigen stresses that this may cause is still a research topic.
- 3 The stiffer Pleistocene sand layers might not always be able to follow the tapering of the TBM. It is expected that this may give rise to gapping behind the tail of the TBM. If grout penetrates this gap,

this may cause higher loads on the TBM than is normally assumed.

4 No proof was found that tunnel driving, in normal operation, might give cause to loss of bearing capacity of piles. Settlements in general are related to the settlement of the ground and the position of the pile toe with respect to the zones indicated in fig. 5.

With acknowledgement to the Netherlands Centre for Underground Construction for their consent to publish about the research they commissioned and coordinated, and with thanks to Cees Blom for the use of some of the figures.

# REFERENCES

- Bakker, K.J., v.d. Berg, P. & Rots, J. 1997. Monitoring soft soil tunnelling in the Netherlands; an inventory of design aspects, Proc. ISSMFE, Hamburg.
- Bakker, K.J. 2000. Soil Retaining structures, development of models for structural analysis, Balkema, 2000, Rotterdam
- Bezuijen, A. & Bakker, K.J. 2008. The influence of flow around a TBM machine. Proceeding 6th Int. Symposium on Underground Construction in soft Ground, Shanghai.
- Bezuijen, A. & van Lottum, H. (eds). 2006. *Tunnelling A Decade of Progress*. GeoDelft 1995–2005.
- Blom, C.B.M. 2002. Design philosophy of concrete linings for tunnels in soft Soil, Delft Univ. Press, The Netherlands.
- Bogaards, P.J. & Bakker K.J. 1999. Longitudinal bending moments in the tube of a bored tunnel. Numerical Models in Geomechanics Proc. NUMOG VII: p. 317–321.
- Hoefsloot, F.J.M. & Bakker, K.J. 2002. Longitudinal effects bored Hubertus tunnel in The Hague. Proceeding 4th Int. Symposium on Underground Construction in soft Ground, IS-Toulouse.
- Hoefsloot, FJ.M. & Verweij, A. 2005. 4D grouting pressure model PLAXIS. Proceeding 5th Int. Symposium on Underground Construction in soft Ground, IS-Amsterdam 2005.
- Kaalberg, F.J., Teunissen, E.A.H., van Tol, A.F. & Bosch, J.W. 2005. Dutch research on the impact of shield tunnelling on pile foundations. *Proceedings of 16th ICSMGE, Osaka.*
- Pachen, H.M.A., Brassinga, H. & Bezuijen, A. 2005. Geotechnical centrifuge testst to verify the long-term behavior of a bored tunnel. Proc. 5th Int. Symposium on Underground Construction in soft Ground, IS-Amsterdam 2005.
- Selemetas, D., Standing, J.R. & Mair, R.J. 2005. The response of full-scale piles to tunnelling. *Proceeding 5th Int. Symposium on Underground Construction in soft Ground*, IS-Amsterdam 2005.
- Talmon, A.M. & Bezuijen, A. 2008. Backfill grouting research at Groene Hart Tunnel. Proceeding 6th Int. Symposium on Underground Construction in soft ground, IS-Shanghai 2008.
- Uijl, J.A., den, A.H., Vervuurt, J.M., Gijsbers, F.B.J. & van der Veen, C. 2003. Full scale tests on a segmented tunnel lining. In *Proc. ITA World Tunnelling Congress 2003*, Amsterdam, The Netherlands, 12–17 April 2003.
- Verruijt, A. 1993. *Soil Dynamics*, Delft University of Technology.

# The influence of flow around a TBM machine

# A. Bezuijen

GeoDelft, Delft University of Technology, Delft, The Netherlands

# K.J. Bakker

Delft University of Technology, Delft, The Netherlands

ABSTRACT: The flow of grout and bentonite around a slurry shield TBM is investigated, using a 1-dimensional calculation model. The results show that the shield of the TBM is only partly in contact with the surrounding soil and that for a large extent it is in contact with the liquid grout and, depending on the amount of over cutting, also in contact with the bentonite that is injected at the tunnel face. These liquids also determine the forces on the TBM shield. The calculation model presents the pressure distribution on the shield and shows the influence of the soil properties, the overcutting and the properties of the grout and the bentonite. Assumptions in the model are checked with 2-D finite element calculations, plane strain and axi-symmetric, that show qualitative agreement, and a reasonable quantitative agreement. The model is useful to explain the phenomena and as a first estimation of the influence of flow around the TBM.

# 1 INTRODUCTION

Tunnelling through urban areas asks for a minimal ground loss. Modern TBM's can operate at an average ground loss of 0.5% or less (Bowers & Mos, 2005). To be able to calculate the ground loss that can be expected in this range it is necessary to improve the calculation methods. One decade ago it was sufficient to calculate whether the front pressure was sufficient to prevent a collapse of the bore front and not high enough to get a blow-out. Nowadays 4-D finite element modelling is used to estimate the ground loss and the settlement trough that is the result of it.

In most simulations on TBM tunnelling it is assumed that the non-excavated soil around the TBM slides over the shield skin of the TBM (Hoefsloot & Verweij, 2005; Kasper & Meschke, 2006). As a consequence the tapering of the TBM results directly into a volume loss and a settlement trough.

However, in reality the calculated settlement trough is too high, there may be a space between the shield skin of the TBM and the soil. This is because the cutting wheel of a TBM is in most cases a bit larger than the TBM itself. There is only a small difference (a few centimetres on a TBM with a diameter of 10 metres or more), but this is significant. It means that for a slurry shield bentonite can flow from the tunnel face back to the grout used to fill up the tail void. However, it is also possible that grout can flow from the tail void to the tunnel face. A 1-dimensional model is developed to get information on the order of magnitudes of these flows, the pressure distribution and the soil deformation. The model assumes a given pressure distribution at the tunnel face (the face pressure) and at the tail of the TBM (the grout pressure) and incorporates the yield strength of both the grout and the bentonite. It takes into account the overcutting of the cutting wheel and the conical construction of the TBM (with a bit smaller diameter at the tail compared to the front). Linear elastic soil behaviour is assumed.

The paper will briefly describe the model, more detailed information is presented by Bezuijen (2007), some example calculations will be presented and the assumptions will be checked by using 2-D finite element calculations.

# 2 GEOMETRY OF A TBM

A TBM shield seems to be a tube with a constant diameter, sketched in a way as in Figure 1. However, looking more in detail the shield is part of a cone. The diameter at the front is larger than at the tail. The difference is only small, normally around 0.4% of the diameter of the TBM. For a TBM shield with a diameter of 10 m, this means that the diameter of the shield is 4 cm smaller at the tail compared to the diameter just after the cutter head. This small difference in diameter allows the TBM to manoeuvre in the soil.

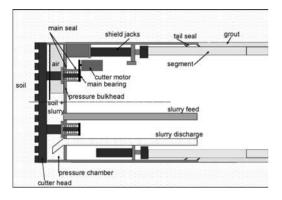


Figure 1. Sketch of a slurry shield TBM.

# 3 GROUT FLOW AROUND A TBM

#### 3.1 Theory

Assume a TBM with a small change in diameter (a diameter decrease from front to back). The TBM is boring in soil that is assumed linear elastic with a shear modulus *G*.

Assume that the soil in contact with the shield at every location of the shield. The tapering of the shield will lead to a decrease in stresses in the soil around the TBM going from the tunnel face to the tail. A simple approach is to neglect the influence of gravity and assume a tunnel that is positioned perfectly symmetric in the bore hole. In such a situation the relation between deformation and stress reduction can be written as (Verruijt, 1993):

$$\Delta \sigma = 2 \frac{\Delta r}{r} G \tag{1}$$

Where  $\Delta \sigma$  is the change in pressure,  $\Delta r$  the change in radius, *r* the radius of the tunnel and the grout and *G* the shear modulus of the soil around the tunnel.

Calculating the pressure from front to tail of the TBM, without the influence of grout or bentonite, will lead to an ongoing pressure reduction.

However, bentonite is injected at the front of the TBM and grout in the tail. Normally the bentonite pressure is lower than the initial soil pressure and the grout pressure is higher. Using eq. (1) this means that there will be a diameter decrease at the front and a diameter increase at the tail. However, when overcutters are used it is still possible that there is an opening at the front of the TBM between the tunnel face and the soil where bentonite can flow over the shield to the tail. Due to the high grout pressure it is also likely that grout flows from the tail over the shield to the TBM.

Somewhere in the middle around the TBM these flows will meet. Calculating the pressure distribution is now complicated by the fact that the direction of the

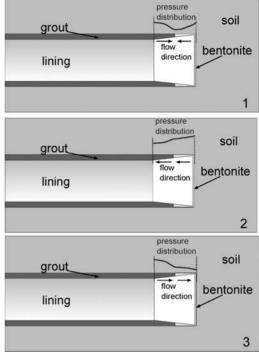


Figure 2. Possible flow directions and sketched pressure distributions along the TBM. The 3rd situation is worked out quantitatively, see Figure 3.

flow is of importance. Both grout and bentonite are usually described as Bingham liquids. This means that a certain pressure gradient is needed to start a flow. Therefore there are 3 possible flow situations, see also Figure 2.

- 1 Grout flows from the tail to the tunnel face and bentonite flows from the tunnel face to the tail (this situation can only occur when there is some volume loss in the gap between the TBM and the soil for example due to bleeding of the grout or penetration of the bentonite into the soil. For this situation the lowest pressure will be present there where the bentonite and the grout meet.
- 2 Bentonite is flowing backwards to the tail and pushing the grout out of the gap between the TBM and the soil. The pressure will be the highest at the tunnel face and will decrease when going to the tail. This cannot be a continuous situation, but can occur temporally.
- 3 Grout is flowing to the tunnel face and pushes the bentonite to the tunnel face. The pressure is highest at the tail close to the injection points of the grout and will decrease going to tunnel face.

The model developed can in principle cope with all these 3 options. However, for simplicity in this paper

we will only deal with the 3rd option because this is the most realistic situation for most of the drilling process. As the machine moves forward, a forward flow of material is needed to fill the annulus that is created by the TBM. It is therefore assumed that the grout flows along the TBM shield and pushes the bentonite away. Considering the advancing TBM during drilling, this can be a stable situation. Both the bentonite and the grout can be described as Bingham liquids. For the low flow velocities during tunnelling the yield stress of the liquid will be determining the pressure drop and the viscous forces can be neglected. Assuming that the flow induced friction will develop between the soil and the grout (in the 3rd option the grout and bentonite will not move with respect to the TBM in a stable situation), the pressure drop can be written as:

$$\Delta P = \frac{\Delta x}{s} \tau_{\gamma} \tag{2}$$

Where  $\Delta P$  is the change in pressure due to the flow,  $\Delta x$  a length increment along the TBM, *s* the gap width between the tunnel and the soil and  $\tau_{\gamma}$  the shear stress of the grout around the TBM.

The following calculation procedure is used. The soil around the tunnel is assumed to behave as independent slices with a thickness  $\Delta x$ . Knowing the geometry of the tunnel, the grout pressure at the tail and the bentonite pressure at the tunnel face, the soil pressure and the elastic properties of the soil, the gap width at the face and tail of the TBM can be calculated using Equation (1). That gap width can be used to calculate the pressure drop due to the flow of the grout to the front of the TBM over the distance  $\Delta x$  with Equation (2) and the pressure increase that will occur in the bentonite when it is pushed to the front. The resulting grout pressure and bentonite pressure is calculated independently. As long as at a certain location the calculated grout pressure is higher than the calculated bentonite pressure, it is assumed that the grout pushes the bentonite in the direction of the tunnel face and there will be grout at that location. In the case that the bentonite pressure is higher, there will be bentonite.

The result of such a calculation is shown in Figure 3. The upper plot of this figure shows the pressures and gap widths that would occur if there were only grout (G) or only bentonite (B) in dots and in lines the combined gap width and pressure. In this calculation the grout penetrates between the shield and the soil up to 1.8 m from the tail. The remaining part of the gap between the TBM and the soil is filled with bentonite. The amount of penetration varies with the pressures and shear strength that are chosen. The parameters used in this calculation are shown in Table 1. Although there is an open connection between the face and tail of the TBM (See the lower plot of Figure 3, the gap width of the combined calculation is never zero), there

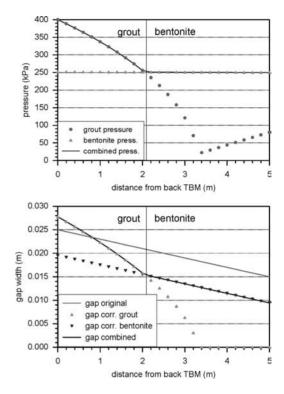


Figure 3. Pressures and gap width along a TBM. Grout pressures and bentonite pressures. Parameters see Table 1. Plots show pressures and gap width for the bentonite and grout pressure separately and the combined result.

Table 1. Input parameters used in calculation with bentonite and over cutting.

length TBM shield	5	m
Diameter	10	m
diameter reduction	0.2	%
over cutting	0.015	m
asymmetric (1) or	2	
symmetric (2)		
grain stress	150	kPa
grout pressure	400	kPa
pore pressure	200	kPa
pressure on tunnel face	250	kPa
shear modulus (G)	90	MPa
shear strength grout, 1 sided friction assumed	1.6	kPa
shear strength bentonite 2 sided friction assumed	0.08	kPa

is still a stable boundary between the grout en the bentonite. Since both liquids behave as Bingham liquids, there can be some decrease in the grout pressure and still the position of the grout-bentonite front will be the same.

According to the results, the tail of the TBM is never in contact with the soil. In reality there can be some contact due to forces not taken into account in this calculation (vertical loading from the lining, moments induced by the hydraulic jacks). However, it is clear that tapering of the shield does not necessarily lead to a certain volume loss, as was assumed by Kaspar & Meske (2006). The calculation results presented here are also in line with the TBM data, which normally show that more grout have to be pumped into the tail void than corresponding to the original tail void. The grout volume that flows into the gap between the soil and the TBM is not an extra volume, because it is a constant volume during tunnelling, but the grout volume increases because the grout pressure makes a wider gap.

In the Western Scheldt Tunnel project a soil sample was taken trough the tail of the shield. In that case no grout was found between the shield and the subsoil, only some bentonite in the sand (Thewes, 2007). This indicates that in reality the soil deformation is not as uniform as suggested here. At some locations the tunnel will remain in contact with the TBM. This will also be the result of the numerical calculations presented in the next session.

# 4 NUMERICAL SIMULATIONS

Some of the assumptions in the calculation model have been tested by means of numerical calculations, using the Plaxis programme.

The assumptions tested are the axial-symmetric deformation of the soil that is assumed and the influence of the non linear behaviour of the sand. Therefore a 2-dimensional plane strain simulation is performed using the mesh shown in Figure 4. The simulation is performed for soil conditions that are common in the western part of the Netherlands: A Holocene top layer or 10 m and a tunnel with a diameter of 10 m embedded in Pleistocene sand. The simulation (Brinkgeve et al., 2006). The parameters used are presented in Table 2.

In the simulation it is assumed that the pressure on top of the tunnel is 350 kPa at the beginning and the pressure is increased up to 400 kPa to simulate the increase in pressure due to grouting behind the TBM. In a second calculation the stress is first reduced to 320 kPa and increased to 400 kPa after that. This simulates the unloading in the soil that can occur due to the overcutting and/or lower bentonite pressures. The results will show to what extent a linear elastic soil behaviour can be assumed.

The pressure increase with depth in the grout is assumed to be 18 kPa/m. The calculated displacements around the tunnel opening due to the pressure increase are shown in Figure 5 for both loading situations. With

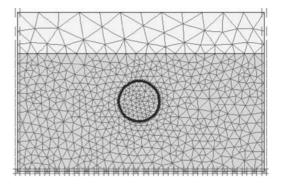


Figure 4. Mesh used in 2-D simulation. The diameter of the tunnel is 10 m.

Table 2. Input parameters for the Finite Element analysis. Parameters give (from top to bottom) the wet volumetric weight, elasticity parameters (Brinkgeve et al., 2006), cohesion, friction and dilatancy angle.

		Holocene	Pleistocene Sand	Grout
γ <sub>sat</sub>	kN/m <sup>3</sup>	18	20	pm
vur	_	0.35	0.2	0.499
Eref	MPa	10	180	pm
	MPa	10	180	0.45
Eref	Mpa	_	720	_
Gref	Mpa	_	675	_
Ŷ0.7		_	1.00E-04	_
c'	kPa	5	1	15
φ	0	20	32	0
ψ	0	_	2	_

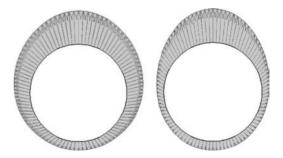


Figure 5. Deformation pattern around tunnel due to a pressure increase of 50 kPa starting at a pressure of 350 kPa (left), resulting in a maximum displacement of 22 mm. Right the same calculations but first a stress reduction to 320 kPa was applied, resulting in a max deformation of 25 mm.

the parameters used in the simulation, the maximum displacements are in the same order as the displacements calculated with the analytic model (27 mm), but this depends on the choice of the parameters.

It is clear that the displacement is not symmetrical around the tunnel axis. The soil will predominantly move upwards. For the situation where there was first unloading the deformation pattern is a bit more 'egg shaped' (see Figure 5).

The deformation pattern is therefore not completely axial-symmetric as is assumed in the analytical calculations. Furthermore, in reality the TBM can rotate with a horizontal rotation axis perpendicular to the axis of the TBM in the just created opening. This can also lead to an uneven distribution of the gap around the TBM. Therefore from this calculation it cannot be concluded what will be the gap width distribution between the TBM and the soil around the tunnel at different locations, what can be concluded is that the gap width shall not be evenly distributed along the TBM circumference. Simulations of the deformation of the soil movements for a tunnel at a larger depth resulted in a more symmetric deformation pattern.

# 5 DISCUSSION

The results of the numerical calculations show qualitative agreement with the results of the analytical model. However, the symmetric deformation that is assumed in the numerical model is not found in the numerical simulations, but looking at the results it is reasonable to assume such a symmetric deformation as a first assumption. The order of magnitude of the deformation is the same, but this is to a large extend a matter of 'tuning' the parameters. The parameters used in the small strain model lead to a stiffer behaviour of the sand than is normally assumed. When in a case study reliable soil data are available, both the elastic parameters and the parameters from the small strain model can be determined.

The influence of unloading before loading seems to be limited and therefore an elastic model can be used. Based on analytical calculations of Wang & Dussault (1994) it can be expected that the influence of unloading becomes more important when the pressures are decreased further.

The model, as it is now, is suitable to get a qualitative indication on what is the influence of bentonite and grout flow. In this way it is a step forward compared to models that simply assume that the soil flows over the TBM and that predict a settlement trough that is too large. For quantitative results and to see what parts of the TBM are in contact with soil and what are in contact with the grout or bentonite, more sophisticated models are needed then the one described here.

## 6 CONCLUSIONS

A one dimensional analytical model has been developed to describe the influence of bentonite and grout flow around a TBM on the pressure distribution in the soil around the TBM and the influence on the settlement trough. Calculations with this model show that the influence of the flow aloong the TBM can be significant, leading to calculated volume losses that are more in agreement with the measurements compared to results of calculations that do not take into account the flow around the TBM. A consequence is that the TBM is (partly) not in contact with the soil. At high soil pressures and therefore also high grout pressures this may increase the risk of deformation of the tail shield.

Comparing the results of the analytical model with the results of numerical calculations showed that some of the assumptions in the model (symmetric deformation, linear elastic soil behaviour) are a somewhat crude representation of reality. This means that it is worthwhile to include the principles described in the paper in more sophisticated numerical models. The model as described here can be used for a first estimate on what is the influence of bentonite and grout flow around a TBM on the settlement trough.

# REFERENCES

- Bezuijen A. 2007. Bentonite and grout flow around a TBM. *Proc. ITA 2007, Prague*.
- Bezuijen, A. & Talmon, A.M. 2003. Grout the foundation of a bored tunnel. Proc ICOF 2003 Dundee.
- Bezuijen, A., Talmon, A.M., Kaalberg, F.J. & Plugge, R. 2004. Field measurements of grout pressures during tunneling of the Sophia Rail tunnel. *Soils and Foundations* 44(1): 41–50.
- Bowers, K.H. & Moss, N.A. 2005. Setllement due to tunnelling on hte CTRL London tunnels. Proc. 5th. Int. symposium on Geotch. Aspects of Underground Construction in Soft Ground, Amsterdam.
- Brinkgreve, R.B.J., Bakker, K.J. & Bonnier, P.G. 2006. The relevance of small-strain soil stiffness in numerical simulation of excavation and tunnelling projects. *Proceedings Numog XII*, Graz, Austria.
- Hoefsloot, F.J.M. & Verweij, A. 2005. 4D Grouting pressure model PLAXIS. Proc. 5th. Int. symposium on Geotch. Aspects of Underground Construction in Soft Ground, Amsterdam, pp 529–534.
- Kasper, T. & Meschke, G. 2006. On the influence of face pressure, grouting pressure and TBM design in soft ground tunnelling. *Tunnelling and Underground Space Technology* 21 160–171.
- Thewes, M. 2007. Private communication.
- Verruijt, A. 1993. Soil Dynamics. Delft University of Technology, b28.
- Wang, Y. & Dusseault, M.B. 1994, Stresses around a circular opening in an elastoplastic porous Medium Subjected to repeated hydraulic loading. *Int. J. Rock Mech. Min. Sci. & Geomech abstr.* 31(6): 597–616.

# Mechanisms that determine between fracture and compaction grouting in sand

# A. Bezuijen & A.F. van Tol

Delft University of Technology/Deltares, Delft, The Netherlands

# M.P.M. Sanders

Delft University of Technology, Delft, The Netherlands Present affiliation: Royal Haskoning, The Netherlands

ABSTRACT: Laboratory tests have shown that injection of grout in sand can lead to different shapes of the grout in the sand. At low water cement ratios (1-2) there is usually compaction grouting, which leads to a spherical shape of the grout. At higher water cement ratios (5-20) fracturing of the sand will occur, leading to rather thin grout structures. In field observations this difference is less obvious. It is assumed that in most cases fractures occur, although it is hardly ever possible to examine what is made in the soil. This paper describes conceptual models and calculation models that explain, at least qualitatively the differences in the behavior of the grout and discusses a possible reason for the difference between the model tests and the field tests.

# 1 INTRODUCTION

Compensation grouting has been successfully applied in several projects to prevent or to compensate for surface settlements induced by for example tunnelling (Mair & Hight, 1994; Chiriotti et al. 2005; Christiaens et al. 2005). Compensation grouting uses hydraulic fracturing to get a heave that can compensate the settlement or compaction to densify the soil with limited heave. In the latter situation the compaction lead to improvement of the soil characteristics.

Laboratory tests have been performed to investigate the mechanisms that are of importance for compensation grouting in clay (Jaworski et al. 1981, Mori & Tamura 1987, Andersen et al. 1994, Chin & Bolton 1999, Au, 2001). Comparable tests have been performed for compensation grouting on sand (Chang, 2004, Kleinlugtenbelt et al. 2006, Bezuijen et al. 2007 and Gafar & Soga, 2006). These tests showed that compensation grouting by fracturing is not so straight forward in sand. After quite a number of tests with a water cement ratio of 1-2 it appeared that there were hardly any fractures in the sand, but a more or less spherical shape of grout is formed in the sand, see Figure 1. Fractures could be obtained when grout with a higher water cement ratio (5-20) is used (Bezuijen et al. 2007).

Similar results were found in at least 3 different laboratories. Different results were, however reported from the field (Grotenhuis, 2004) based on measured



Figure 1. Results from Cambridge University (left) and GeoDelft that show more a spherical shape than fractures.

surface displacements. Normally it is not possible to investigate the fractures created, but this was possible during the construction of the station for the high speed train in Antwerp and during the construction of the cross-passages at the Hubertus Tunnel in The Hague, the Netherlands.

In Antwerp compensation grouting was applied to prevent settlements on the existing station (Christiaens et al. 2005). The resulting fractures were found in a soil layer with a lot of shells. A clear fracture was found between the shells. Here it is possible that the shells have influenced the fracture. The Hubertus Tunnel case was an unintended fracture that occurred when the lances for freezing were brought in and the grout pressure used to install these lances was large enough to create fractures.

The paper will describe the Hubertus Tunnel case more in detail, since this is one of the very few cases where fractures in relatively homogeneous sand at 11 m depth and 5 m below the waterline could be studied and describes the principle and results of calculation models that are developed to describe various parts of the grouting process. Possible causes of the differences between the field test and the model tests are: 1) the preconditioning has an influence. 2) the in-homogeneities in 'real' soil serve as a trigger for the start of the process. 3) The installation of the TAMs (TAM stand for Tube-a-Manchette, a tube with injection openings covered by a rubber Manchette through which the grout can be injected) which causes unloading of the soil has an influence. In this paper we will elaborate the 3rd possibility and mention some aspects of the 2nd. The first possibility is not dealt with in this paper.

Results of the model tests itself will be described and analyzed in another paper on this conference (Gafar at al., 2008).

# 2 THE HUBERTUS TUNNEL

The Hubertus Tunnel is a double track road tunnel that is constructed in dune sand. The tunnels have a diameter of 10 m. Five connections were made between the two tunnels. Soil freezing was used to make these connections. Lances were constructed from the tunnel lining into the sand to freeze the soil. A connection between the two tunnels was made through the linings and the concrete connection structure was made. For a cross-section see Figure 2.

The tunnel crown is placed 11 m below the soil surface and the phreatic level is 5 m below the surface. The vertical pressure at the top of the tunnel is estimated to be around 140 kPa.

Grout was used to install the lances for freezing. During the installation of these lances for the first connection, the pressure was increased so far that the grout created hydraulic fractures. These fractures became visible when the frozen soil was removed.

Figure 3 shows an example of fractures in the sand around the tunnel and between the tail void grout and the sand.

The picture is taken standing in the newly created opening, from left to right you see the opening, the tunnel lining, the tail void grout, grout from a lance that penetrated between the tail void grout and the sand and the frozen sand with a fracture in the upper part of the sand. There were also fractures that were only visible in the sand.

The normal pressure to inject the grout is 3 bar. However, according to the employees of the contractor on the site the pressure could have been higher during this injection because there were doubts on the

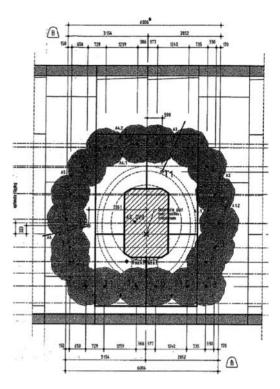


Figure 2. Sketch of a connection between the tunnels made by freezing.

pressure gauge. The grout had a water-cement ratio (W/C) of approximately 1 and 3% of bentonite was used in the mixture.

# 3 COMPARISON WITH LABORATORY TESTS

The laboratory tests performed at GeoDelft, as described by Bezuijen et al. (2007) and Gafar et al. (2008) have been performed in a cylindrical tank with a diameter of 0.9 m and a height of the sand sample of 0.85 m. The sand was pressurized to get a vertical effective stress of 100 kPa. Different grout mixtures were injected and different injection procedures were used. Most mixtures injected had 5% or 7% bentonite. A fracture like behaviour only occurred when the W/C ratio in the mixture was 2 or higher. Lower W/C ratios would not create fractures, but lead to the shapes shown in Figure 1 (right picture). The maximum injection pressures varied in most cases between 20 and 30 bar.

Comparing the main results of the model tests with that of the field observations, there seems to be a discrepancy. The grout that does not lead to fractures in the model, does result in fractures in the field. The injection pressure in the field situation described is not known, but it is unlikely that where the normal

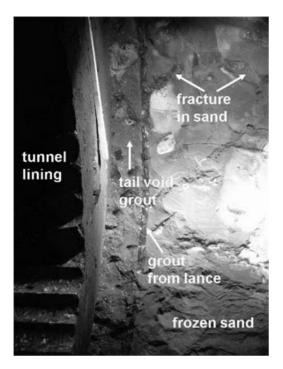


Figure 3. Fractures found during the construction of the first connection between the tunnels of the Hubertus tunnel.

injection pressure is 3 bar, it was 28 bar during the construction of the first cross-section (this would be the injection pressure based on the laboratory tests and extrapolating the results of these tests to 140 kPa vertical pressure).

A possible reason for the discrepancy between the model tests and the field behaviour can be the soil boundary condition, as will be explained in the next sections.

### 4 RELEVANT GROUT PROPERTIES

# 4.1 Plastering

Bezuijen & Van Tol (2007) have described how plastering of the grout influences the fracturing behaviour. Fracturing occurs because the radial stress in the sand around a fracture is higher than the angular stress. Using the Mohr-Coulomb criterion, the relation between radial stress and angular stress perpendicular on the radial stress can be written as:

$$\sigma_{\theta} = \frac{1 - \sin\phi}{1 + \sin\phi} \sigma_{r} \tag{1}$$

Where  $\sigma_r$  is the radial stress,  $\sigma_{\theta}$  the tangential stress and  $\phi$  the friction angle of the sand. Taking for example

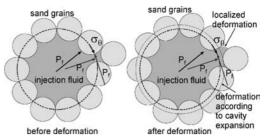


Figure 4. Sketch with possible deformation modes of the injection hole. In reality there will be more grains around the injection hole. The figure just shows the principle.

a friction angle of 35 degrees, the tangential stress is only 0.27 times the radial stress.

Looking at the scale of the grains, sand is not homogeneous and isotropic, as assumed in continuum mechanics. At the boundary of the opening there will be some grains that are in closer contact and some between which there is some space, see Figure 4.

Such space can be sufficient to have the fluid pressure not only in radial but also in tangential direction. The fluid pressure is with perfect plastering equal to the radial stress and much higher than the tangential stress and therefore this will lead to an opening of the space between the grains and a fracture can occur. Some in-homogeneity in the soil as is often present in a field situation (more than in the model test) will facilitate the fracture formation, because weak sections in the soil will deform. If there is a beginning of a fracture then the fluid pressure will penetrate further in the sand and the fracture will easily grow further. A fracture will stop to propagate when the pressure in the fracture tip drops due to friction losses, leak-off or increase of the volume with injection fluid.

Plastering and the formation of a filter cake in the injection hole will hamper fracturing of the sand, because now possible irregularities at the boundaries of the injection hole will be filled with plaster see Figure 5. This plaster also has certain strength and therefore prevents the fluid pressure from penetrating into the space between the grains.

As is also mentioned in Bezuijen en Van Tol (2007), plastering is caused both by consolidation of the grout and by leak-off of grout in the sand while larger particles in the grout are filtered by the sand, remain at in the injection hole and cause a plastering layer at the boundary between the injection liquid and the sand.

With a constant injection pressure, the consolidation of the grout mixture, without leak-off, can be approximated by formula (Bezuijen et al. 2007):

$$s_1 = \sqrt{2k \frac{n_i - n_e}{1 - n_i} \Delta \phi t}$$
<sup>(2)</sup>

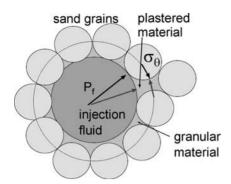


Figure 5. Influence of plastering.

Where  $s_1$  is the thickness of the consolidated layer (the plastering layer),  $n_i$  the initial porosity,  $n_e$  the porosity after consolidation, k the permeability of the consolidated layer,  $\Delta \phi$  the difference in piezometric head over the column and t the time.

This formula implies that the thickness of the plastering layer increases with the square root of the applied pressure, a lower pressure will lead to a thinner plastering layer.

## 4.2 Leak-off

Assuming that the flow during leak-off can be described as a Bingham liquid flow, the relation between the difference in piezometric head and the thickness of the plastering layer of courser materials can be written as (for v > 0, flow into the sand):

$$\Delta\phi = \frac{s_1 v}{k_1} + \frac{s_2 v}{k_2} + \alpha_1 s_1 + \alpha_2 s_2 + Rv$$
(3)

Where  $\Delta \phi$  is the difference in piezometric head,  $s_1$  the thickness of the grout cake (the plastering layer),  $s_2$  the distance the liquid has penetrated into the sand,  $k_1$  and  $k_2$  the permeability of the cake and the sand respectively for the penetrating liquid, v the velocity,  $\alpha_1$  and  $\alpha_2$  factors for the cake and the sand respectively that determine the drop in piezometric head caused by the yield stress in a Bingham liquid and *R* the flow resistance from the soil around the fracture. When this equation is dominant, the thickness of the plastering will be proportional with the applied difference in piezometric head.

Experimental research by Sanders (2007) has shown that consolidation determines the thickness of the plastering layer for grout with a low W/C ratio, leak off is the dominant mechanism for the thickness of this plastering layer at W/C ratios of 5 and more. Normally a filter cake caused by consolidation of leakoff is made of finer particles than the sand of the subsoil. Due to this leak off will stop when there is some thickness of the filter cake.

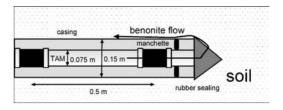


Figure 6. Sketch of casing and TAM during installation.

# 5 INFLUENCE OF SOIL DEFORMATION

In the experiments preformed in Waterloo, Cambridge and Delft, the sand around the injection point is reasonable uniform and the vertical confinement pressure is constant over the soil sample. Injection of grout will lead to cavity expansion until a fracture occurs.

A different pressure distribution is present around the injection points at the Hubertus Tunnel and in general when compensation grouting is applied in the field. At the Hubertus tunnel the freezing lances were applied between the two tunnels. Consolidation of the tail void grout leads to a stress reduction in the vicinity of the tunnel (Bezuijen & Talmon 2003). Arching between the 2 tunnels can lead to a further reduction of the stresses. Also the placement of lances, or in case of compensation grouting of TAMs, leads to a reduction of stresses. The TAMs are placed using a casing; see Figure 6 for a sketch. Figure 6 shows the situation where the TAM is placed by displacement of the soil. Another option is that the bentonite is used to remove the soil. The latter option leads to more unloading of the soil. Whatever option is chosen, when the casing is removed there will be some unloading of the soil although grout is injected to stabilize the bore hole, as appears from (minimal) settlements measured during the installation of the TAMs. (Kleinlugtenbelt, 2006).

The situation that occurred during compensation grouting can be compared with the situation of a cavity under repeated loading. Such a situation is analyzed by Wang & Dusseault (1994). They used cavity expansion theory to calculate the stress distribution in the soil around a bore hole during unloading and re-loading. In the case of compensation grouting, the removal of the casing will result in unloading and the reloading will occur during cavity expansion. Cavity contraction and expansion theory cannot describe when a fracture will start, but it can be used to calculate what pressures will lead to a plastic zone in the soil. Some plastic deformation in the sand is necessary before a fracture can occur. Plastic deformation will lead to dilatancy in the sand, which results in more space between the grains. This makes it easier for the grout to separate the grains further according to the mechanism described in Section 4.1 leading to a fracture, because there are locally zones with a higher permeability.

Parts of Wang & Dusseault's analyses will be presented here in a shortened and slightly adapted version to allow calculation of the influence of previous unloading on the loading pressure at which plastic deformation occurs. It is assumed that after installation of a TAM there is a pressure release due to the removal of the casing that leads to an active failure of the soil around the hole. This means that the radial stress,  $\sigma_r$ , is smaller than the tangential stress,  $\sigma_{\theta}$ . After installation, the radial pressure is increased due to the fracturing process and passive yield will occur. In such a situation  $\sigma_r$ , is larger than the tangential stress,  $\sigma_{\theta}$  and a fracture can occur. Here we assume that the fracture initiation pressure is related with the pressure that results to passive failure of the cavity.

Assuming a linear Mohr-Coloumb criterion, the plastic stresses must fulfill the criterion:

$$\sigma'_{\theta} - N\sigma'_{r} + S = 0 \tag{4}$$

Where the prime indicates the effective stress.

The material parameters N and S are different for active and passive yield and can be written as:

$$N_a = [1 + \sin\phi]/[1 - \sin\phi]$$

$$N_p = [1 - \sin\phi]/[1 + \sin\phi]$$
(5)

and

$$S_a = -2c_0 \cos\phi / [1 - \sin\phi]$$
  

$$S_p = 2c_0 \cos\phi / [1 + \sin\phi]$$
(6)

(The publication of Wang and Dusseault discriminates between peak values and residual values, here we use only one value.)

Here  $\phi$  is the friction angle and  $c_0$  the cohesion of the soil material. Active yield will occur when:

$$p_a < \frac{2\sigma'_0 + S_a}{1 + N_a} \tag{7}$$

Where  $p_a$  is the pressure at the boundary of the opening in the active state (assuming a perfect plastering on this boundary) and  $\sigma'_0$  the initial effective stress around the opening.

When the effective pressure around the TAM remains higher than the criterion mentioned in Eq. (7), there will be no plastic deformation and the limit pressure for passive plastic deformation will remain the same as if there was no unloading on the soil.

In case the criterion of Eq. (7) is fulfilled, there will be an active plastic zone. Increasing the pressure in the cavity afterwards will lead to, the situation sketched in Figure 7. There will be an active plastic zone and within that a passive plastic zone. When the pressure is increased further the active zone will disappear, but

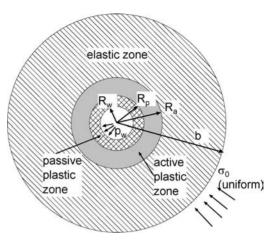


Figure 7. Definition sketch, for the situation with active and passive soil around the cavity, see also text.

Table 1. Input parameters used in calculation.

$\sigma_0$ original stress in sand $\phi$ friction angle sand	100 35	kPa degr.
$c_0$ cohesion	0	kPa
$R_{\rm w}$ (radius of tube, see Figure 7)	0.035	m
b (radius with constant press)	5	m

our interest is the pressure at which the passive zone started.

During unloading, when the active zone is formed  $P_r$ , Based on the equations presented by Wang and Dusseault, the pressure in the cavity, can be expressed as a function of the thickness of the plastic zone  $R_a$  and other parameters (see also Figure 7):

$$P_{r} = \left[ p_{0} - H \frac{b^{2} - R_{a}^{2}}{b^{2} R_{a}^{2}} - S_{a} \right] \left( \frac{r_{w}}{R_{a}} \right)^{N_{a}}$$
(8)

Where:

$$H = \frac{\sigma'_{h} (N_{a} - 1) - S_{a}}{b^{2} (N_{a} + 1) + (1 - N_{a}) R_{a}^{2}} R_{a}^{2} b^{2}$$
(9)

When after active yield the pressure is increased the first soil will come in the passive plastic state when:

$$p_{p} > \frac{2p_{a} + S_{p} - S_{a}}{1 + N_{p}}$$
(10)

Without the active failure this relation would read:

$$p_p > \frac{2\sigma'_0 + S_p}{1 + N_p} \tag{11}$$

Table 1 presents the input parameters used for calculations using the formula's presented above. Results

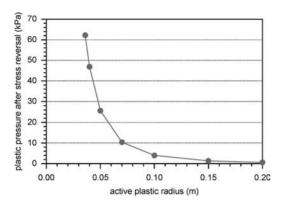


Figure 8. Pressure that is necessary to create passive plastic deformation after that first active yield is imposed as a function of the original active plastic zone.

are shown in Figure 8. Without active plasticity the pressure where passive plasticity starts is 157 kPa. From these results it is clear that plastic deformation in the active state can considerably reduce the borehole pressure at which passive plastic deformation occurs. The pressure needed to achieve passive plastic deformation decreases as the active plastic radius created by the low pressure before the pressure increase is larger. This presents quantitative information for the statement already mentioned by Wang & Dusseault: 'Our study suggests that initial active formation damage reduces the pressure required to initiate such a fracture'.

## 6 DISCUSSION

In Section 4 it was shown qualitatively that only with a limited filter cake between the grout and the subsoil a fracture can occur in homogeneous soil. In in-homogeneous soil the formation of a fracture is easier, because also the initial deformation will not be symmetric. For this reason there was a fracture between the tail void grout and the sand at the Hubertus Tunnel.

The calculation model by Wang and Dusseault, as described in Section 5, shows that the pressure to create a fracture is considerably lower in case of a cavity that has been subjected to plastic unloading, compared to a cavity that is still in an undisturbed state. This last phenomenon is the most reasonable explanation that the injection pressure in the field is lower than the pressure measured in the laboratory. A lower injection pressure also means that the filter cake due to plastering and leak-off is thinner and thus that the grout mixture is less critical. This may be an explanation that fractures are created in the field for conditions that does not lead to fractures in the laboratory. Further experimental work has to prove and quantify this idea. Consequence may be that the compensation grouting is sand is influenced not only by the grout, but also to a large extend by the stress distribution around the bore hole where the TAM is installed. The installation procedure will influence the shape of the fracture and the pressure needed to create a fracture.

# 7 CONCLUSIONS

From our study we came to the following conclusions:

- 1 Fractures can occur in a field situation where for comparable soil and using the same grout only cavity expansion is measured in a model test. This can be caused by the heterogeneity in the field and by the way the TAMs are installed in the field.
- 2 Unloading of the soil before it is loaded by injection of grout will lead to a reduction of the injection pressure.
- 3 A reduced injection pressure will lead to a reduced cake thickness because both bleeding and leak-off will be reduced.
- 4 In further experimental research it will be necessary to investigate the influence of the installation procedure and in-homogeneity of the soil.

# ACKNOWLEDGEMENT

This work was supported by Delft Cluster. The authors wish to thank The Hubertus Tunnel Combination for the information presented on the first cross-connection of this tunnel.

# REFERENCES

- Andersen, K.H., Rawlings, C.G., Lunne, T.A. & Trond, H. 1994. Estimation of hydraulic fracture pressure in clay. *Journal of Canadian Geotechnical* 31: 817–828.
- Au, S.K.A. 2001. Fundamental study of compensation grouting in clay. PhD thesis. University of Cambridge.
- Bezuijen, A., Sanders, M.P.M., Hamer, D. & Tol, A.F. van 2007. Laboratory tests on compensation grouting, the influence of grout bleeding. *Proc. World Tunnel Congress*, Prague.
- Bezuijen, A. & Talmon, A.M. 2003. Grout the foundation of a bored tunnel. *Proc ICOF*. Dundee: Thomas Telford.
- Bezuijen, A. & Tol, A.F. van 2007. Compensation grouting in sand, fractures and compaction. Proc. XIV European Conference on Soil Mechanics & Geotechnical Engineering, Madrid.
- Chang, H. 2004. *Hydraulic fractures in particulate materials*, Phd. Thesis, Georgia Institute of Technology, November.
- Chin, C.Y. & Bolton, M.D. 1999. Factors influencing hydrofracture in clay. Proc. 13th ASCE Engineering Mechanics Conference, Baltimore.
- Chiriotti, E., Avgnina N. & Grasso P. 2005. Compensation grouting for TBM tunneling beneath shallow cover. Proc. 5th Int. Symposium Geotechnical aspects of underground construction in soft ground, Amsterdam 2005.

- Christiaens M., Hemerijckx E. & Vereerstraeten J.C. 2005. Tunnelling under the city centre of Antwerp. A new underground Railway link for the HSL Paris-Brussels-Amsteram. Proc. 5th Int. Symposium Geotechnical aspects of underground construction in soft ground, Amsterdam 2005.
- Gafar, K. & Soga, K. 2006. Fundamental investigation of soil-grout interaction in sandy soils. Report. University of Cambridge.
- Gafar, K., Soga, K., Bezuijen A., Sanders M.P.M. & Tol A.F. van. 2008. Fracturing of sand in compensation grouting. Proc. 6st Int. Symposium on Geotch. Aspects of Underground Construction in Soft Ground, Shanghai.
- Grotenhuis, R. 2004. Fracture Grouting in Theory, Modelling of fracture grouting in sand. MSc thesis, Delft University of Technology.
- Jaworski, G.W., Seed, H.B. & Duncan, J.M., 1981, Laboratory study of hydraulic fracturing. Journal of the Geotechnical Engineering Division 107(6): 713-732.

- Kleinlugtenbelt, R. 2005. Compensation grouting, laboratory tests in sand. MSc thesis. Delft University of Technology. Kleinlugtenbelt, R. 2006. Private Communications.
- Kleinlugtenbelt, R., Bezuijen, A. & Tol A.F. van. 2006. Model tests on compensation grouting. Proc. World Tunnel Congress, Seoul.
- Mair, R.J. & Hight, D.W. 1994. Compensation grouting. World Tunnelling November: 361-367.
- Mori, A. & Tamura, M. 1987. Hydro-fracturing pressure of cohesive soil. Journal of the soil and foundations. Japanese Society of Soil Mechanics & Foundation Engineering 27(1): 14-22.
- Sanders, M.P.M. 2007. Hydraulic fracture grouting, laboratory tests in sand. MSc thesis. Delft University of Technology.
- Wang, Y. & Dusseault, M.B. 1994. Stresses around a circular opening in an elastoplastic porous Medium Subjected to repeated hydraulic loading. Int. J. Rock Mech. Min. Sci. & Geomech abstr 31(6): 597-616.

# Research of non-motor vehicle-rail transit-tube interchanging transport system pattern

A.Z.G. Deng & Q.H. Zhang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: In the light of the relationship between land use and rail transit, it considers that the development of the public transport is one of the effective ways to solve the problems in traffic. Urban rail transit that can lead the development of the cities should be the backbone in the public traffic. This paper presents the research about non-motor vehicle-rail transit interchanging transport system patterns. The interchange pattern is proved to be feasible by investigation and calculation research. The purpose is to make public transportation more effective, and elevate the leading role of rail transit. The paper also indicates the effective ways to solve the problems in traffic in order to exalt the proportion of the urban rail transit.

# 1 GENERAL INTRODUCTION

Nowadays, there are many researches about the interchange of rail transit and public transportation (such as airport, railway, subway) along with the increasing urban rail transit lines in Beijing and Shanghai etc. However, these researches hardly mention the interchange of non-motor vehicle traffic to rail transit. Through the investigation on general distribution station (refers to the station around the inhabited area, public transportation, non-motor vehicle and walking system primarily), this paper presents the research about non-motor vehicle-rail transit interchanging transport system patterns. The purpose is to facilitate the public transportation, and elevate the leading role of rail transit. With the development of rail transit network, it is very important to investigate the model of the affected factor in station and the land use around to provide a comfortable, safe and convenient environment in transit interchanging. The paper proposes the effective ways to solve the problems in traffic, in order to exalt the proportion of the urban rail transit.

# 2 THEORY OF THE LAND USE AROUND THE RAIL TRANSPORT STATIONS

The fundamental characteristic of the railway station can be summarized as the following: forming the buildings around the railway station, which generate the core-axis pattern, developing the traffic organization and function layout with the center of railway station. Land is important to a city, and reasonable use of land will do benefit to the economy, society as well as the ecosystem. The aim of land use shows complex, multidimensional and integrative in space form, which request close cooperation between infrastructure development and civil development to optimize land use. In the range of railway station, it should be emphasized of the relation and merge between different city functions, which can make people enjoy the use of railway and embody the theory of "people oriented".

In developed countries, some new urban planners break through the limit of the traditional urban planning theory under the city's sustainable development background, and proposed Transit-Oriented Development Model, i.e. TOD Model. It is expected the use of public transport to lead the development of a city, and back to the model of using bicycle and walking. The main content of TOD is: Using public transportation station as a center, walking distance as a radius. In this area, high density of the land use is emphasized and public transportation facilities are arranged around station. By walking, bicycle and public bus etc, high effective interchange to replace the leading role of car in city.

Because of the diversity between China and west developed countries on population density and city development stage, there are some differences of using TOD theory, explained as follows:

1. Different background: After the World War II, the cars become popular in cities of US, the energy

waste becomes serious, suburbanization and urban blight of the city downtown. The TOD concept was right proposed under this background. But cities in China are still in highly development. The main problem is how to develop public transportation, to solve the traffic jam carried up by high density of city's population and deterioration of the environment.

- 2. Population density: In US, low population density, and the main transportation home-based-trips are cars used. But in China, population density is higher. Even the residential area in suburb, the population density is still higher than foreign country.
- 3. Willingness to use public transportation: Because of the distributed suburb, high percentage of people owing cars and more highways, the public transportation has less attraction to publication in western countries. But in China, it is because of the large population, short of effective facilities, and undeveloped construction technique and management technique that there always exits the traffic jam, lower speed of transportation and lack of comfortable conditions. There will be a vicious circle along with the more cars using.
- 4. The difference of influence radius of station: In foreign TOD model, there are a 600 meter influence radius with the center of the station. And the interchange with rail transit is most based on walk system. But in China, the density of rail transit network is lower, instead of higher population density, and the influence radius of railway station is larger, people use interchange mostly by public bus and bicycle etc with rail transit, which makes the interchange pattern of the railway station and surrounding area different.

It can be seen from the operation of foreign big cities' transportation that the percentage of railway passenger is 45% to 60% to the total one. But in China, the percentage of bus makes over 75%, and that of railway is much lower. It is because that the development of rail transit transportation is on the beginning, and hasn't formed a transport net. The density of station and network is over 1500 km in London, New York, Paris and Tokyo, and there is a high railway density in down town also. The density of railway and stations in town is shown in Table 1. It will generally take 5 to 10 minutes for people walking from home to the railway station. Take Beijing for example, the density of subway in down town is only 0.32 km/km<sup>2</sup>, which is one tenth of that compared to developed countries. It is pointed out in literature that the downtown district, middle district and outskirt district of a big city can be taken as 1.6, 0.8 and 0.4 km/km<sup>2</sup> based on China's situation and it is found acceptable. It is still a gap of the density of the railway system with developed countries.

Table 1. The density of station and network in London, New York, Paris and Tokyo.

	London	New York	Paris	Tokyo
Density of network km/km <sup>2</sup>	3.49	3.47	3.76	2.6
Density of station unit/km <sup>2</sup>	2.52	3.92	4.58	1.59

In conclusion, due to low network density in china, the application of TOD model in China's cities will be modified as: it should be based on the station as a center, rail transit, bus and non-motor vehicles as an interchange for development. The main task is not to increase the residential density but how to combine land use and public transport together, which can provide a comfort, convenient and attractive public transport system. By doing so can we make use of the network effectively as the backbone in city's development.

# 3 RAIL TRANSPORT SYSTEM PATTERNS IN DIFFERENT CHARACTERISTIC STATION

To different railway transportation types of stations (It is classified into large scale interchange station, general interchange station and general distribution station) area, accordingly there are different major functions (seen in Table 2). Generally, for station in downtown area (large-scale interchange station or general interchange station), the commercial business development area consist up the majority of this district, and the residential development area takes up the minority of this district. For station in outskirt or newly developed area (general interchange station) or general station), the most suitable development in this area should be residential development.

In planning the station and surrounding area, the main concern point should be the land use around the station. For example, in city CBD area, the station should be built as the commercial center of, especially in interchange station of multi-trip railway lines. The underground space exploitation should also be considered to develop a district, which is incorporating transportation and business or other functions to form an underground urban complex.

For interchange station in outskirt of the city or in development zone, it should be considered together with other transport method e.g. private cars, buses, non-engine cars.

For stations at suburbs of the city, the majority method of interchange is using Cars Peripheral Park and Ride (P&R). The P&R refers to the interchanging facilities for people in outskirt parking cars around the railway station or public transportation's end stop. By using public transportation system to down town, the Table 2. Classification in different rail transit station.

	Classification in different rail transit station	Land use around station	Different interchange forms
Rail transit station	Large-scale interchange station	Town center commercial circle and CBD area	Within rail transits (3 or over 3 lines)
		City suburb	Rail transit with air plane
		City distribution center	Rail transit with railway Rail transit with Road passenger transportation Rail transit with public transport
		residential district	Ran transit with public transport
	General interchange station	City sub-center	Within rail transits (2 lines) Rail transit with public transport
		Development zone	Peripheral Park and Ride (P&R)
	General distribution station	Residential district	Rail transit with public transport Rail transit with non-motor vehicle Rail transit with walk system

use of private cars in down town area is decreased. The outskirt interchange facilities are the link of cars and public transportation system. It major function is to provide the effective, safe, convenient, comfortable linkage between outskirt cars system and downtown public transportation system, so as to increase the attraction of the public transportation system and decrease the use of cars in down town area. By doing so, the transportation pressure in down town area could be diminished and realize the sustainable development of the city's transportation system. This method is suitable for outskirt of the city. The railway station must provide the sufficient parking facilities, parking lots to meet the interchange need. The parking facilities must be near to the station, and have connection path to the station. The enough roads should be planned and constructed.

Regarding in urban general distribution station, preceded by text analysis, the main consideration is the walk system, the non-motor vehicle transportation system and the bus, the taxi convenient interchanges.

The walking system is the most primary connection with the rail transit, the content mainly includes sidewalks system in station, the facility and the passengers separates from vehicle facility plan design, the guidance informational sign design, walking routes organization design and so on. Therefore the walking system should be given priority in rail transit design. Considered transportation station around the land use intensity is high, passengers activity is also frequent, and in order to increase commercial stores suitable sidewalks should be provided. Traffic island and crosswalk must be considerable and designed in this area. The walk system in the station which connect platform meets the convenient needs in the station, but also achieve the evacuated request, simultaneously also must have distinct guidance symbol.

The bicycle is one of effective ways connected with rail transit also. Enough bicycle parks should be provided in joints of station design, appropriative parking zone be setting which is connected with underground tunnel, accommodation road should be constructed in order to lower the effects from non-motor vehicle in traffic.

The interchange between bus transportation and rail transit mainly includes the roadside parking pattern, parallel pattern, vertical pattern and centralized pattern. Bus accommodation road can be presented and the bus stop should be built near subway station' entrance-exit which can provide comfortable, safe and convenient environment in transit interchanging.

# 4 RESEARCH OF NON-MOTOR VEHICLE-RAIL TRANSIT-TUBE INTERCHANGING TRANSPORT SYSTEM PATTERN

Bicycle is one of the effective manners in public traffic. Especially in china at present, inferior service in public traffic, undeveloped mechanization. Bicycle shows cheap, convenient and unpolluted characteristics, it takes important status in our country public traffic. The interchanging content mainly includes bicycle park system in station, the facility of the passengers separates from vehicle facility planned and suitable routes organization designed near the station.

Bicycle-rail transit-tube transferring transport system patterns design should follow principles:

1. The interchange pattern is suitable in the city suburb or in residential district around rail transit station (General in distribution station), and it can be not applied in city center.

- In order to avoiding occupying of the space, decrease the effects to the transport, it should provide enough quantity if the bicycle special parking spots.
- 3. Appropriative bicycle parking lots on the ground should be provided near the station which passenger flow is large, and passengers may take interchange through tunnel; dispersed bicycle parking zone may be designed around the station which has few passenger flow, yet is not so closed to the station' entrance-exit in order to avoid effecting passengers.
- In park essential facilities must be supplied and should arrange the specialist to manage with inexpensive charge.
- 5. To display bicycle superiority in short-distance home-based-trips, and limits its proportion in long-distance home-based-trips. Reasonable bicycle routes and accommodation road system can reduce the bicycle' effects in traffic and provides comfortable, safe and convenient environment in transit interchanging.

In general distribution station, which interchange with rail transit almost are public traffic (bus) and non-motor vehicle (such as automobile, bicycle and walk system). At present in many stations usual show: bicycles are laid on the street or some temporary openair non-motor vehicle parks spot all around, it has occupied the limited station' space. Simultaneously the bicycle around the station may on the passenger 'way, hinder the pedestrian traffic, and potential security risk exists also. Although many stations peripheral non-motor vehicle parks have take specialists, accommodation tunnel which may connect rail transit and bicycles are not been designed. It enlarges interchanging distance, takes inconvenient and lowers the interchanging efficiency.

Based on the interchange principle between nonmotor vehicle and rail transit introduced above, it is proposed in this paper the Bicycle-rail transit-tube transferring transport system patterns concept by using a general underground subway station as an example.

For a general station, the length is about 200 m, the width is about 20 m (30-35 m if including exit). Four exits are designed on both sides of the station, and are constructed in conjunction with the ventilation shaft etc affiliates as shown in Figure 1. The station is situated under the cross, and four exits are situated at four corners of the road. In construction of the station, the main structure will be constructed first, and then the exit. It can be seen from the plane layout that the two exits in the same direction would be constructed separately, and no connection between the two exits. The shadow area in the middle has no use. If it can be constructed at the same time with the exits of the station, the two exits and the middle part could form the underground parking bicycle etc. By doing so, the land around the station can be used intensively, and

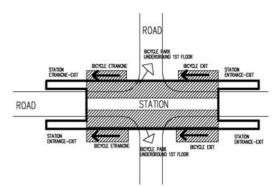


Figure 1. Bicycle-rail transit-tube interchanging transport system patterns in General distribution station on the 1st flood underground.

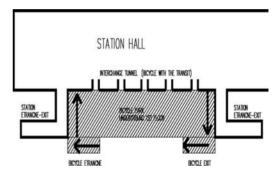


Figure 2. Sketch map of Bicycle parks underground.

decrease the station ground area occupation area and passenger flow on the ground. The phenomenon of scramble between bicycle and car will be decreased, and the interchange efficiency of non-motor vehicle and railway will be highly increased.

The underground non-motor vehicle is shown as shadow area in fig. 1, the four corners of the road could be set as bicycles parking exit (shown in Fig. 2) which makes the passengers from both directions can park their bicycles effectively and interchange with rail transit (interchanging may use the mutual wall between station hall and bicycle parking area or use tunnel). For consideration of parking area of bicycle, this shadow area is about one third of the station main area (about 60 m) with the width of 20 m, area of  $1200 \text{ m}^2$  (total area  $2400 \text{ m}^2$ ). The major design reference can be planed by Bicycle Parking Design Standard.

The principle factors considered for the scale of a normal park for bicycles include the number of bicycles reaching to station, the area occupied by one bicycle, and the piece of the bicycle park. According to the piece, the scale of the bicycle park can be calculated as following:

$$S_{bic} = S_{bic} V_{bic} / \lambda_{bic} \tag{1}$$

Table 3. The main design index in bicycle park.

		Parking width		Spac betw		Aisle width	
Parking type		single (m)	double (m)	bicyc (m)			
Diagonal	30°	1.00	1.60	0.50	1.2	0 2.0	
	45°	1.40	2.26	0.50	1.2	0 2.0	
	$60^{\circ}$	1.70	2.77	0.50	1.5	0 2.0	
Vertical		2.00	3.20	0.60	1.5	0 2.0	
		Unit ar	ea				
		Single one-sid	Sing e two-	le sides	Double one-side	Double two-sides	
Parking type		m <sup>2</sup>	m <sup>2</sup>		m <sup>2</sup>	m <sup>2</sup>	
Diagonal	30°	2.20	2.00		2.00	1.80	
	45°	1.84	1.70		1.65	1.51	
	$60^{\circ}$	1.85	1.73		1.67	1.55	
Vertical		2.10	1.98		1.86	1.74	

where:  $S_{bic}$  denotes the needed scale for bicycles in the station (m<sup>2</sup>);  $V_{bic}$  is the number of bicycles during rush hour piece/hour);  $\lambda_{bic}$  is the velocity of the bicycle park;  $s_{bic}$  is the averaged area occupied by a bicycle (m<sup>2</sup>).

The velocity can be set as 1 due to little change of the number of bicycles because the users of the bicycles in the station are almost only to work or back from work by bicycles. The area occupied by a bicycle, obtained from Table 3, is  $1.7 \text{ m}^2$ . Through calculating, the capacity of the underground bicycle park, excluding the public area, is 1200 units.

Through the sampling survey of numbers of bicycles near the available stations in Shanghai rail transit carried out by the author, the following point are given: the number of persons using bicycles for interchanging transportation, including the persons who arbitrarily lay their bicycles at the edges of road or near some shops, is about 800 to 1000. According to the above results, it can be seen that the capacity of the underground park area in a normal station can meet the requirement of paring bicycles for transportation interchanging.

For the station of main body lay on the side of road, the underground park area for bicycles can be set up as shown in Figure 3. For this type of parking area, the ceiling can be mounted by transparent materials for accessing of sunlight. The vegetable processing on the top of the park will improve the inner environment in an extent through the hole between the parking area and station hall.

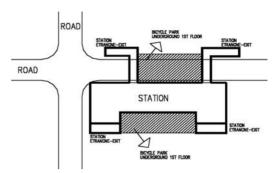


Figure 3. Sketch map of Bicycle-rail transit-tube interchanging transport system patterns (station on one side of road).

Taking the variety of the plane layout of the railway transportation stations into accounting, the underground parking area for bicycles combined railway station, proposed in this paper, is a conceptual pattern. The exact arrangement will be adjusted according to the actual plane layout of stations. The principle idea of the model is to efficiently utilize the area near the entrances and exits, realize the interchanging between bicycles and railway transportation without extra cost, and increase the efficiency of interchanging.

For the underground parking area of bicycles in a station, the construction is feasible in techniques. In general, the stations which need this type of parking area are in the sub-center, outskirt or near the satellite city. The main buildings near the stations are residential districts also, the construction of the station will almost not affected by the environment around. The parking area can be constructed with the main structure of the station by open excavation method in foundation. Also, it's feasible to firstly construct the main structure of the station and then construct the parking area combined the entrances and exits by open excavation method.

#### 5 CONCLUSIONS

Through the analysis above, it can be concluded that the underground bicycle parking area for transit interchanging is feasible in reality. The advantages as below:

 The users of bicycles can interchange rail transit through the entrance between the under ground parking area and station conveniently, which realize the interchange without extra cost, increasing the efficiency in a great extent, providing the comfortable environment in transit interchanging at same time and expressing the conception of people oriented.

- 2. The underground interchange between bicycle and railway transit decreases the occupied area of the station, facilitate the travelers using other types of transportation which transfigure the environment around the station.
- 3. This pattern of transit interchanging can decrease the flow of bicycles on the ground, increases the efficiency of motor passing, eliminates the possibility of traffic accident and realizes the Pedestrian System Separated from Vehicle System by passenger interchanging through tunnels.

The cost of construction for the underground bicycles parking area is higher than normal station. Nevertheless, synthetically considering the Traffic Diversion on the ground, the effect on the passenger flow by other patterns of transportation, and the environment near the station, the society and economic benefit resulting from what it brings will exceed far from what it costs.

#### REFERENCES

- Ge, L. 2005. A Study on Planning & Design Methods for Urban Passenger Transit Hubs. *Doctoral Dissertations In Southeast University*. Nanjing.
- Gu, B.N. & Rao, X.P. 2001. On the Reduction of Subway Station Scale. *Urban mass transit.* 2001(3):14–17.
- Huang, J.Z. 2006. Models for Suburbanization Land use and Corresponding Transportation Development in Metropolis. Beijing: China Architecture & Building Press.
- Yang, D.Y. & Han, H. 2000. A Study on Metropolitan Rail Transit and Transportation Structure. Urban mass transit. 2000(4):10–15.
- Zheng, M.Y. 2006. Urban Develop in Light-RailAge. Beijing: China Railway Publishing House.

# Shotcrete excavations for the Munich subway – Comparison of different methods of face support in settlement sensitive areas

J. Fillibeck & N. Vogt

Zentrum Geotechnik, Technische Universität München, München, Germany

ABSTRACT: For the construction of shallow tunnels in settlement-sensitive urban areas it is very important to reduce the settlements and to increase the stability of the tunnel face during the excavation. In the case of shotcrete excavation, the use of different methods of face support has become more and more common. These methods are: ground freezing, pipe roofs, jet grouting and injection support. The paper shows the experience made to due the installation of the above mentioned face supports, especially since the specific focus is related to the arising settlements. If only small deformations are allowed to occur, as the examples show, deformations that have to be considered during the construction process as well as those to establish the bearing load; they could be significant depending on the process. Suggestions have been made as to ways in which the deformations can be reduced by making additional measurements.

#### 1 INTRODUCTION

For the construction of safe shallow tunnels in settlement-sensitive urban areas, it is very important to reduce the settlements and to increase the stability of the tunnel face during the excavation. The use of different methods of face support in the case of shotcrete excavations is becoming increasingly common. These methods are ground freezing, pipe roofs, jet grouting and injection support.

The report presents the experience gained in the installation of the abovementioned working face supports, with particular focus on the induced settlements. Four different projects of Munich's subway are described. After a short project description the results of the measurements are illustrated (geodetic and borehole measurements) and evaluated. With this background, the different methods of face support are compared and the different advantages and disadvantages are discussed. Finally, special attention is focused on the installation process. Proposals are made for the reduction of settlements in future tunnel projects.

#### 2 GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS

In the Munich subsoil the quaternary gravels follow under fillings of small thickness. The quaternary gravels can reach a thickness of more than 20 m. They predominantly consist of medium density layers, are laminated and have, depending on the deposit conditions and their age, a differing amount of sand and fine grain. The average permeability amounts to approx.  $k = 5 \cdot 10^{-3}$  m/s. Tertiary layers lie below the quaternary gravels. They consist of changing layers of fine-to medium-grained sands with high density and clays or silts in stiff to firm consistency. The thickness of the layers can change excessively within a small distance. The average permeability of the sand amounts approximately from  $k = 1 \cdot 10^{-4}$  to  $1 \cdot 10^{-5}$  m/s, the tertiary clay and silt can for all practical purposes be assumed impermeable.

The quaternary gravels possess a mostly free phreatic water level, which can reach ground level. There are still confined aquifers within the sand layers with fine-grained cover. The pressure of the groundwater approximately corresponds to that of the free phreatic surface in the quaternary gravels.

#### 3 HEADING WITH GROUNDFREEZING UNDER THE CITY HALL OF MUNICH

#### 3.1 Construction process

The extention of the station Marienplatz of the subway lines U3/ U6 under the Munich City Hall was built by the company Fa. Max Bögl GmbH & Co KG. The project was finished in 2006.

Parallel to the two existing platforms, two directly joining tunnels were built in shotcrete method under atmospheric conditions with a vertical distance of

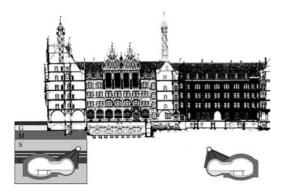


Figure 1. Cross section with City Hall, tunnels and geologic situation.

about 10 m to the city hall. In order to avoid damage to the landmark city hall, the deformations had to be strictly limited. The construction company Fa. Bögl, planned freezing arches in the context of an alternate bid, in order to support the crown and to keep the retaining water away from the tunnel face. The freezing arches were provided for through pilot galleries above the crown (Figure 1).

The tunnels are embedded in the tertiary layers (figure 1). The water bearing sand layers had to be dewatered with the help of filter wells.

#### 3.2 Measurements to reduce frost heave

For the successful realization of the specific proposal it was crucial to reduce frost heave in such a way that no damage occur to the city hall. Frost heave can be essentially attributed to two reasons:

- homogenous frost heave  $(\Delta h_{vol})$  because of a 9% increase in volume caused by the changeover from water to ice.
- growing of ice lenses with corresponding frost heave  $(\Delta h_{icel.})$  because of the tendency of the soil to draw water near the interface of the frozen to the unfrozen soil (zero-degree-front). This frost heave increases with time.

The frost heave tests, which were performed in the laboratory of the Centre for Geotechnics at the Technical University of Munich (Zentrum Geotechnik, TU München), showed, that in the tertiary fine-grained soils frost heave  $\Delta h_{icel.}$  still occurs at load-levels of more than 400 kN/m<sup>2</sup> if water can be drawn at the interface to the permeable sand layers. Therefore the alternating layers of permeable sands and frost-sensitive clays present a critical risk source when frost heave is considered.

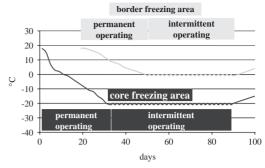


Figure 2. Operating – Control of the artificial ground freezing.

In order to reduce frost heave to a minimum the following measurements had to be taken:

- measuring and controlling the temperature in the soil with the help of 5 measuring cross sections per tunnel. Every cross section includes 18 thermo couples.
- reducing the operation time of the frozen arches by dividing the tunnels into 3 different sections: north, middle and south.
- further partitioning within the freezing sections by the installation of groups of freezing tubes with separate control.

In figure 2 the temperature development of a section is shown schematically. Twenty days after starting the freezing process in the core of the freezing body the freezing of the border area started. After the frozen body reached approx.  $-22^{\circ}$ C in the core area and  $0^{\circ}$ C in the border area, the freezing tubes were operated intermittently, an average of 8 to 24 hours in the core area and 12 to 24 hours in the border area. Due to intermittent handling, the zero-degree-front does not move outside (enlargement of the frozen body), but stays in a narrow zone, which again and again gets frozen and defrosted. Thus frost heave reduces significantly. The operation of one section could be stopped after approx. 90 days. The adjacent defrosting process took about three months.

#### 3.3 Measuring of the settlements

The deformations which occurred during the construction process were measured by a geodetic precise levelling system on the surface and a closed water levelling system in the 2nd basement of the city hall.

The closed water levelling system consisted of 10 measuring points with a resolution of 1/10 mm. The measuring results could be checked online at all times. Figure 3 shows the location of the measuring points S03, S06 and S09 of the closed water levelling system as well as the development of the settlement and heave

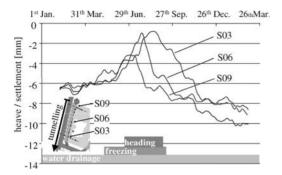


Figure 3. Vertical displacements of the measuring points S03, S06 and S09.

depending on the time. The three measurement points were situated in the sections north, middle and south.

The settlements at the beginning of the freezing process result from groundwater drawdown. At the onset of freezing the expected frost heave started. It reached a maximum value of 3 to 5 mm. The settlements due to the tunnelling process occurred after the heading had passed the measuring points and they still continued after the freezing process was stopped. The settlements slowed down continuously and stopped 3 months later with a maximum settlement of about 10 to 12 mm.

Figure 3 clearly shows the temporary displacement of the settlements according to the heading. The drive reached the measuring points in descending order, resulting in them reaching the maximum heave successively.

The measured deformations were approximately the same as the calculated ones. Thereby half of the settlements could be attributed to dewatering measures, which lead to large area settlements and correspondingly low differential settlements. Furthermore, no settlement damages were determined at the city hall, so it can be assumed that the heading was very successful. It was essential that for the success of the project, larger frost heave by ice lenses could be avoided by applying the above mentioned measures. Frost lenses would otherwise have led to a softening of larger soil areas and therefore to larger settlements and settlement differences.

#### 4 JET GROUTING COVER FOR A LARGE CROSS SECTION FOR U3 NORTH LOT 1

#### 4.1 Construction process

The consortium Ed. Züblin AG/Max Bögl GmbH & Co KG carried out the construction of the subway lot U3 North -1 in the north of Munich. The works were completed in 2006. The shotcrete headings with a total length of around 1950 m were driven with and without

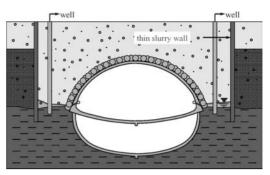


Figure 4. Heading W3 and W4 with jet grouting cover.

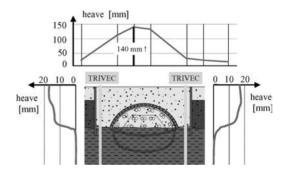


Figure 5. Deformations in cross section MQ 8 after construction of the jet grouting cover.

compressed air support and with several methods of crown support which will be introduced hereafter.

The headings W3 and W4 with a cross section area of up to  $200 \text{ m}^2$  began from a starting shaft with a top heading. First watertight pits with thin slurry walls were produced (figure 4) in order to lower the groundwater table. The safety of the excavation face was increased by 13 jet grouting covers (total length of about 15.5 m each, overlap 4.3 m) as well as further jet grouting piles in the face of the crown.

The quaternary gravels were cut with suspension (simplex – method) at a pressure of up to 400 bar at the cone. At anytime during the making of a jet grouting pipe a controlled outflow of the suspension is required, assuring that the pressure does not lift the soil. The top heading followed after the installation of the jet grouting took place. The heading of the bench and invert began on finishing all top headings.

## 4.2 *Heave during the installation of the jet grouting cover*

Figure 5 shows the deformation in the cross section at a distance of 50 m away from the starting shaft directly after the installation of the jet grouting cover.



Figure 6. Required suspension backflow during jet grouting.

The heave above the crown reached in MQ 8 about 140 mm and in total a maximum of 250 mm. At first, the heave was deemed uncritical because there were no buildings close to the tunnel, however they result particularly in soil strains in a narrow band directly above the interface to the tertiary soils. Because of the proximity of the thin slurry wall to the jet grouting cover, the heave led to a crack in the thin slurry wall, making the wall permeable.

The heave results from the fact, that the outflow of the suspension in the annular space of the jet grouting piles, which are faced upwards, can not be controlled in a suitable way (figure 6). Owing to the lack of backflow in the layered soil with strongly differing conductivity the overpressure spread over a greater area, resulting in a rising of the soil above the jet grouting cover.

The large heave could only be limited by reducing the overpressure through the installation of further boreholes from the ground level, resulting in high costs.

With increasing soil cover the heave reduced because of the increasing load. However, even under more than 12 m of soil cover and the installation of the jet grouting cover in tertiary clays, the heave still amounted to approximately 20 mm.

For further projects, where only very few deformations are allowed during the installation of the jet grouting cover, sufficient attention should be paid to the control of the suspension backflow during grouting. This problem could for example be resolved by improving the technique of the grouting machine or with the help of a double tube, which is pulled a little ahead during jet grouting or likewise with the help of special valves which control the pressure of the backflow.

If the pressure gets to high during grouting, heave can be avoided or at least reduced by additional horizontal or vertical arranged boreholes.

	U5/9 Ostbahn- hof	U5/9 Theresien- wiese	U3N1 W4 / W3
tunnelling cross section	$\oplus$	CD	θ
cross section area [m <sup>2</sup> ]	200	175	170 - 200
covering [m]	9,3	9,6	6,5/11
max. settlement [mm]	36	38	26 / 40

Figure 7. Comparison of settlements of different shotcrete tunnels, driven under atmospheric conditions.

#### 4.3 Settlements during the heading

After the installation of the jet grouting cover, the top heading with temporary shotcrete invert followed step by step, over the whole heading distance. After this the heading of the bench and invert followed. The settlements which occurred during the headings amounted to a maximum of 26 mm in cross section MQ 8 and 30 to 40 mm in the area with larger soil cover.

In order to be able to judge the results, in figure 7 the above mentioned measurements are compared with those of atmospheric shotcrete headings having nearly the same soil cover but were driven in partial face advance without jet grouting cover.

Overall maximum settlements were measured as almost the same size, which means that the jet grouting cover does not reduce the settlements, in comparison to tunnels driven in partial face advance. As the sliding micrometer measurements show, the forces which were taken from the jet grouting cover, lead to concentrated high stresses in the small bedding area of the jet grouting cover. This stress concentration leads to comparatively high compressions and settlements. On the other hand partial face advance leads to less stress concentration, however the different delayed headings lead to multiple load rearrangements and therefore the surface experiences approximately the same settlements.

Finally it can be concluded that with the jet grouting cover the settlements are not reduced in comparison to those caused by partial face advances. However, the face stability clearly increases by using a jet grouting cover.

#### 5 PIPE SCREEN COVER FOR THE UNDERPINNING OF A BUILDING IN U3 NORTH LOT 1

The two shotcrete headings of section W1 in the above described subway Lot U3 North-1 in Munich had a cross sectional area of  $A = 41 \text{ m}^2$  and were driven in the tertiary soils under atmospheric conditions with the help of wells, dewatering the tertiary sand layers.

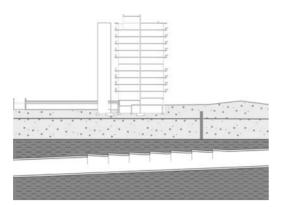


Figure 8. Crossing the building Werner-Friedmann-Bogen, longitudinal section.

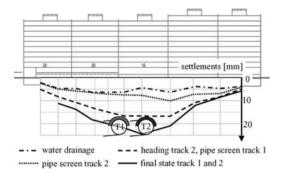


Figure 9. Settlements of Werner-Friedmann-Bogen during tunnelling (cross section).

In this section the underpinning of the Werner-Friedmann-Bogen, a building complex with 12 floors, is of special interest. The foundation pressure of the 3 m wide strip foundation, which lies in the centre of the building and carries the main loads, amounting to nearly  $300 \text{ kN/m}^2$ .

At a vertical distance of approx. 12 m between the foundation and the crown, a pipe screen cover was planned as an additional measure of protection (figure 8), because the tertiary soil cover amounted only 4 m and full water pressure was acting from the quaternary to the surface of the tertiary soils. At the southwest side of the Werner-Friedmann-Bogen an underground garage follows.

For every pipe screen 38 pipes were installed. The length of the pipes amounted to 12 m with an overlap of 4 m. The bore diameter amounted to 146 mm with a 6 mm annular space.

In figure 9 the settlement trough along the Werner-Friedmann-Bogen is shown as dependent on the development of the heading.

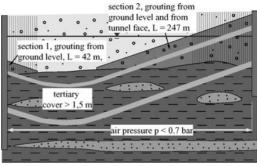


Figure 10. Longitudinal section of the grouting section O2.

Due to the dewatering of the tertiary sand layers settlements of 5 mm to 7 mm were recorded. The installation of the pipe screen and the forward directed settlements of the heading of track 2 increased the maximum settlements to approx. 10 mm. The largest settlements resulted from the 2 headings. Finally the maximum settlements amounted to 25 mm.

As a comparison with measuring of further cross sections without pipe screen shows, the maximum settlements were measured under the Werner-Friedmann-Bogen. It is clear that the foundation loads lead to higher settlements and because of the smaller soil cover only limited arching develops. Furthermore, installation also cause settlements. However, it is crucial to settlements, that the pipe screens as well as the surrounding soil layers experience some deformation, before the system can carry the expected load in both the longitudinal and lateral directions. That is why the predominant settlements respectively occur shortly before and directly during the heading.

It can therefore be concluded, that the pipe screen primarily increases the safety of the tunnel face. For the installation and formation of the bearing effects, deformations are however necessary, which lead in this case to settlements of 25 mm. Pipe screens are only applicable for the reduction of settlements, if substantially higher settlements are expected without them.

#### 6 HEADING WITH COMPRESSED AIR SUPPORT AND GROUTING IN U3 NORTH LOT 1

The geological conditions in the section O2 of the subway lot U3 north-1 are shown in figure 10. In this section a shotcrete heading with compressed air support was provided for. If the thickness of the tertiary soils above the crown reached less than 1.5 m, the overlaying quaternary gravels were grouted. In the grouting section 1 with a length of approx. 40 m the gravels were grouted from the surface. In section 2 the surface was not accessible. The gravel was therefore grouted from

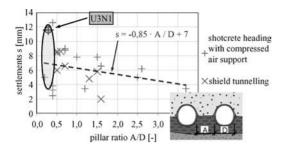


Figure 11. Surface settlements due to shotcrete tunnelling using compressed air in Munich with dependence on the pillar ratio A/D.

the tunnel. At the end of the heading the grouting was done again from the surface.

The aim of the grouting was to reduce the permeability of the gravel to  $k \le 5 \cdot 10^{-5}$  m/s. This was controlled by permeability tests in the bore hole.

After the grouting activity the heading followed in shotcrete method with compressed air support with a maximum overpressure of 0.7 bar. Settlements were measured between 3 mm and 11 mm (without considerating of settlements due to water drainage). In order to assess this result, the results of settlement measurements of headings in Munich, with compressed air support and without grouting (shield tunnelling and shotcrete method) depending on the pillar ratio A/D are compared with the abovementioned result of the U3N1 measurement in figure 11.

It can be seen that the settlements measured if grouting was applied do not differ from those without grouting. It appears that the grouting did not reduce the settlements.

Overall, the results confirm, that shotcrete headings with compressed air support lead only to very small settlements with small tangential inclinations, which cause no damage to conventional buildings. Considering that the measured compressed air consumption was almost just as small as the calculated value, the very extensive grouting measure (21500 m grouting boreholes with more than 43000 grouting sleeves have been installed) can be determined as very successful.

#### 7 CONCLUSIONS AND FINAL REMARKS

In order to construct shallow tunnels in settlementsensitive urban areas with the shotcrete method, measures have to be taken to increase the face stability and to reduce the settlements.

Besides the common measures (for example reducing the length of the advance step, etc.) special crown support measures are often used for this purpose. In this paper, the experiences of the authors demonstrated the purpose of using a frozen cover, a pipe screen cover, a jet grouting cover and a grouting cover. It has been shown, that the installation of crown supporting measures can have an extensive influence on the appearing deformations

As the settlements caused by shotcrete headings with conventional cross sections (approx.  $40 \text{ m}^2$ ) in the Munich underground are comparatively small (smaller than 20 mm to 25 mm for atmospheric headings and smaller than 10 to 20 mm for headings with compressed air support), the crown supporting measures do not have, as the examples show, decisive advantages regarding the deformations. However, if the crown supporting measures are used in a adequate way, a considerably higher safety potential occurs. This has to be considered if a decision has to be made, as to whether or not crown supporting measures are necessary in difficult sections.

### Fracturing of sand in compensation grouting

K. Gafar & K. Soga University of Cambridge, Cambridge, UK

A. Bezuijen, M.P.M. Sanders & A.F. van Tol *TU-Delft/GeoDelft, Delft, The Netherlands* 

ABSTRACT: The phenomenon of fracturing in sand as a result of compensation grouting was studied. Processes of fracture initiation and propagation were explained and a parametric study was conducted in order to investigate the factors that cause sand fracturing to occur. Experimental results indicate that fracture initiation requires the existence of a local inhomogeneity around the injection position. Grout mixture in terms of water-cement ratio and fines content had major roles in sand fracturing, whereas injection rate had a minor influence under the tested conditions.

#### 1 INTRODUCTION

Compensation grouting has been widely in use to control ground settlements during tunneling processes. Nevertheless, its use is still hindered in many cases by the uncertainties in the grout mechanical behavior. As the current grouting practice is highly dependant on field experience rather than scientific knowledge of soil-grout interaction behavior, issues such as suitable injection pressure, soil fracturing and bleeding (amount of water forced out of the grout mixture) still need further investigation.

In particular, soil fracturing stands out as one of the major challenges that could affect the results of a grouting project. Accidentally-created fractures could cause considerable damage to near-by structures, whereas failing to create fractures when they are required could result in tunneling-induced settlements (for example) not being fully compensated for.

While fracturing of cohesive soils was extensively studied by many researchers (e.g. Jaworski et al. 1981, Mori & Tamura 1987, Andersen et al. 1994, Chin & Bolton 1999, Soga et al. 2005 & 2006), there has been limited work for compensation grouting in cohesionless materials (e.g. Chang 2004). Fracturing of sand was studied in relation to the oil industry (Khodaverdian & McElfresh 2000, Bohloli & de Pater 2006) and horizontal directional drilling (Bezuijen et al. 2002). Recently, the prospected use of fracture grouting in Amsterdam to tackle settlements encountered during the construction of the North-South Metro line triggered a thorough research into the phenomenon of sand fracturing. This paper follows on from the work reported by Sanders (2007) and Bezuijen & van Tol (2007). Utilizing the reported optimum grout mixture for soil fracturing, a series of laboratory scale grout injection tests was performed in which various factors affecting fracturing of sand were studied.

#### 2 THEORY & BACKGROUND

Hydraulic fracturing is defined as the condition leading to the creation and propagation of a thin physical separation in a soil or rock mass due to high fluid pressures. The fracturing process is mainly characterized by fracture initiation, propagation and orientation. Understanding the factors controlling these parameters is the first step in understanding and predicting the grout fracturing behaviour. The factors affecting fracturing phenomenon could be divided into two groups: factors related to soil, such as soil properties (particle size, shape and distribution, relative density, cohesion, friction angle etc), stress state and the magnitude of confining pressure and factors related to grout itself and the grouting process, such as grout rheology (components and viscosity), injection rate and injection pressure.

#### 2.1 Fracture initiation

The two main theories explaining fracture initiation are tensile failure and shear failure. Jaworski et al. (1981) suggested that, for a hydraulic fracture to occur, the effective stress has to become tensile and equal in

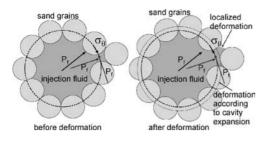


Figure 1. Realistic particle arrangement around an injection hole and possible deformation modes.  $P_f$  is the injection pressure to cause plastic deformations in soil and  $\sigma_{\theta}$  in the effective stress around the injection hole. (Bezuijen & van Tol 2007).

magnitude to the tensile strength of the soil. This situation is clearly unconceivable in case of cohesionless soils like sand. The other theory on fracture initiation proposes a shear failure as the main reason for fracturing in clays. Mori & Tamura (1987) suggested that shear failure occurs within a short duration under a high injection rate, with the duration being too short for the grout to penetrate into micro fissures to create wedge action and/or enter the soil pores to weaken the soil strength.

For fracturing in sand, Bezuijen et al. (2007) and Bezuijen & van Tol (2007) suggested that the local contact forces between sand grains have to be eliminated. As in case of cohesionless soils there is no tensile strength between the grains, this means that the fracturing pressure has to overcome the effective stress in the direction perpendicular to the fracture. In reality, sand is never perfectly homogeneous and therefore, the arrangement of particles around a certain boundary (injection hole, for example) will be in such a way that some particles are in closer contact than others, as shown in Figure 1.

When injection is conducted, grout will fill the space between particles and start to push them apart. Whether this initiated fracture will propagate further or the result will be a roughly symmetric cavity expansion depends mainly on the properties of the injected grout. This explanation of fracture initiation in sand agrees with the findings of Thallak (1991), who, based on the micromechanics of granular media, suggested that hydraulic fracture initiation requires the local contact forces between particles to become zero or tensile. As this requirement is complicated by local force distribution at microscopic scale (which can lead to a wide variation in the fracturing pressure), it was concluded that hydraulic fracture initiation depends mainly on the local microscopic inhomogeneities in the soil.

#### 2.2 Fracture propagation

Starting from the condition shown in Figure 1, a roughly symmetric cavity expansion will always be the

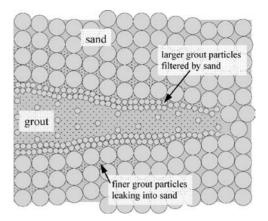


Figure 2. Leak-off and filter cake formation associated with fracturing.

result if the injection pressure is high enough and close to perfect cavity expansion pressure. Factors that dictate how high the injection pressure will be are mainly the grout materials and rheology, confining pressure and stress state and soil density. Results reported by Kleinlugtenbelt (2005), Gafar & Soga (2006), Sanders (2007) and Bezuijen & van Tol (2007) confirm that high injection pressures are associated with high grout viscosities (low w/c ratio or more cement content for cement-based grouts). Gafar & Soga (2006) reported that injection pressure in the case of no confinement was increased by 15 times when a 100 kPa confinement was introduced. Bezuijen & van Tol (2008) highlighted the influence of the stress state. Unloading of the soil around a cavity as a result of equipment installation leads to plastic deformations in the soil, which may result in lower injection pressures.

Bezuijen & van Tol (2007) explained that, for fractures to propagate, grout mix has to contain enough content of fine bentonite particles and has enough water to ensure good flowability at the same time. In this case, pressure application will cause the water in adjacent sand pores to be replaced with a mixture of water and finer particles leaking from the grout mix, as shown in Figure 2. This permeation action is termed as leak-off. With introduction of fines in the soil matrix, the leak-off will reduce the permeability of sand around the injection hole, causing grout bleeding to be slowed down.

On the sides of the propagating fracture, a filter cake is formed as a result of the accumulation of larger cement particles filtered at the sand-grout boundary. Filter cake formation is crucial for the propagating fracture to be able to keep itself open and to sustain forces induced by grout penetration. At the same time, bleeding is restricted, keeping a good workability of the grout. A suitably high initial water-cement ratio (w/c ratio) will ensure that the grout mix will have

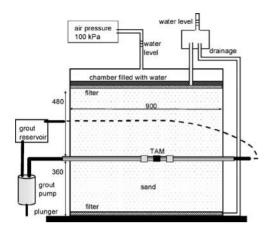


Figure 3. Schematic diagram of the experimental setup, showing the dimensions in mm (Bezuijen & van Tol 2007).

sufficient water content to keep a fracture propagating without the need for very high pressures. Eventually, the pressure at fracture tip will not be enough to overcome the effective pressure in the direction perpendicular to fracture. Nonetheless, the pressure at the tip will still be high enough for leak-off and bleeding to continue. This will cause a filter cake to be formed at the fracture tip, completely blocking further grout propagation.

#### **3 EXPERIMENTAL SETUP**

Grout injection tests were conducted in a cylindrical steel container of a 900 mm diameter and changeable height. Two sample heights were used: 840 mm (for Tests 1 and 2) and 600 mm (for the rest of the tests). Figure 3 shows a schematic diagram of the experimental setup. The injection tube position was fixed at 360 mm above the bottom of the container. A PVC plate rests on the top of the saturated sand sample, tightly sealing it off from an upper water chamber. Confinement is applied by means of pressurizing this water chamber using compressed air. Air pressure is applied through a glass cylinder that also shows the change in water level resulting from soil heave. This change is continuously measured during the test by means of a differential pressure gauge. Two tubes connect the top (through the water chamber) and the bottom of the soil sample to another graduated glass cylinder which rests on the top of the setup, providing a double drainage system.

A simplified model of the tube à Manchette (TAM), as shown in Figure 4, runs across the diameter of the cylindrical container. The tube has an internal diameter of 22 mm, with 4 equally spaced 7 mm holes at the centre. A rubber sleeve covers the holes and 2 rings, one on either side, prevent the injected grout from flowing along the tube.

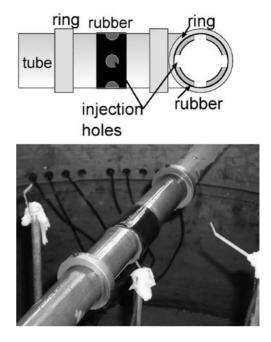


Figure 4. Simplified model of Tube à Manchette (TAM) used for injection. Actual space between the two rings is 40 mm during injection (Bezuijen et al. 2007).

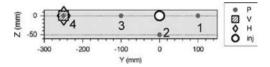


Figure 5. Location of instruments with respect to the injection tube (inj). P are the pore pressure transducers, V measures the vertical pressure and H the horizontal (Bezuijen et al. 2007).

Changes in pore water pressure during injection were monitored by four pore pressure transducers, distributed around the injection point as shown in Figure 5. The readings from these transducers were not used in this paper. Two total stress cells were used to record the change in horizontal and vertical pressures.

Two types of sand were used: Baskarp sand  $(d_{50} = 130 \,\mu\text{m})$  and Leighton Buzzard type D sand  $(d_{50} = 234 \,\mu\text{m})$ . In both cases, sand was wet-pluviated into water in the model container. Loose sand was then densified to required relative density (70%) by dropping the whole container over 25 mm as many times as required.

In order to raise  $K_0$  to a closer value to 1, sand was "pre-stressed" by applying a confining pressure of 300 kPa at the beginning. Confinement was reduced to 100 kPa prior to grout injection. This pre-stressing

Table 1. Summary of conducted experiments.

No	W/C ratio	Rate (l/m)	Bentonite (%)	Materials/ sand	Remarks
1	5.0	10.0	7.0	GD/Ba	Repeatability test
2	5.0	2.0	7.0	GD/Ba	Slower inj.
4	5.0	10.0	7.0	GD/Ba	Wall friction
5	5.0	10.0	7.0	Ca/Ba	Effect of materials
6	1.0	2.0	4.0	Ca/LB	Lower w/c ratio
7	1.0	10.0	4.0	Ca/LB	Faster inj. rate

Notes: Rate = injection rate, GD = GeoDelft cement and bentonite, Ca = Cambridge cement and bentonite, Ba = Baskarp sand, LB = Leighton Buzzard sand. Bentonite percentage is by weight of mixing water.

was only partially successful and values of starting  $K_0$  were still less than, but close to, 1.

Grout injection was conducted using a plunger pump. A bladder (not shown in Fig. 3) was used as an interface between pumped water and injected grout in order to avoid damaging the pump by the grout. The injection pump was capable of reaching a maximum pressure of 4 MPa. Injected grout was allowed to set for 24 hours before the sand was dug out and the shape of hardened grout was photographed.

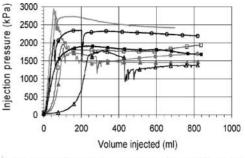
#### 4 RESULTS & DISCUSSION

Using the same injection setup, Sanders (2007) and Bezuijen & van Tol (2007) reported that the best grouting efficiency was attained by using a cementbentonite grout of a w/c ratio of 5.0 to fracture Baskarp sand. Ordinary Portland cement and sodium-activated bentonite (7% by weight of mixing water) were utilized. Injection was made under an injection rate of 10 liters per minute (1/m).

The current series of tests adopted the above mentioned test as a reference and Table 1 summarizes some of the experiments carried out. Two sets of grout materials were used: (a) rapid hardening, ordinary Portland cement and sodium-activated bentonite (GeoDelft, the Netherlands), and (b) normal hardening ordinary Portland cement and sodium bentonite (University of Cambridge, UK). Injection pressures resulting from all the tests are shown in Figure 6.

#### 4.1 Repeatability check

Due to a problem with sample preparation that yielded a sample which was not perfectly homogeneous, the repeatability test (Test 1, using grout with w/c ratio of 5.0) resulted in a single fracture which propagated to



- Ref. test 

Test 1

Test 2

Test 4

Test 5

Test 6

Test 7

Figure 6. Change of injection pressure with injected volume for different tests.

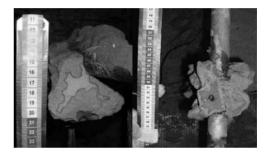


Figure 7. Fractures from (a) reference test (left) and (b) Test 2 with slower injection rate (right).

near the container wall. Nevertheless, the initial pressure was similar in magnitude to the starting pressure of the reference test (2.7 and 2.8 MPa respectively). The pressure during injection was about 40% less than the injection pressure during the reference test, which highlights the effect of soil inhomogeneity.

#### 4.2 Effect of injection rate

Reducing the injection rate in Test 2 by a factor of 5 still yielded fracturing of the sand model. The grout mix contained enough water and fine particles. Recorded injection pressure was about 10% lower than the value for faster injection rate. Sectioning of hardened grout revealed that thicker fractures were formed, with narrower leak-off zone and thicker filter cake, as shown in Figure 7b.

Under the slower injection rate, there is more time for the filter cake to develop. According to Bezuijen et al. (2007), the thickness of filter cake increases with the square root of the time that the grout is pressurized. For a given injection pressure and injected volume, reducing the injection rate by a factor of 5 will lead to approximately 2 times thicker filter cake. Formation of a thicker filter cake will hamper further leak-off, as the finer particles are blocked by the filter cake that is already formed.

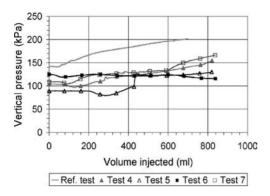


Figure 8. Comparison of the change in vertical pressure with injected volume between the reference test and the tests with reduced sample height.

#### 4.3 Side wall friction

The sample height for the rest of the tests was 600 mm. Test 4 was a repetition of the reference test under the new sample height. Results showed that the injection pressure under the new testing condition was about 1 MPa lower, even though the injected grout managed to fracture the sand in a similar way. The change in overburden pressure that corresponds to changing the height of sand above the injection point is only a few kilo Pascals. Therefore, it could not have been the reason for this reduction in injection pressure. However, measurements of total vertical stress showed that the friction between the sand and the walls of the container have reduced the vertical stress at the injection level from that applied at the top of the sample (Fig. 8). This explains the reduction in recorded injection pressures for all the tests conducted under the new sample height.

#### 4.4 Grout materials

The ordinary Portland cement used in the experiments was CEM I cement and either sodium bentonite or sodium-activated bentonite was mixed. Bruce et al. (1997) reported that sodium bentonite is the best type of bentonites to be added to cement grouts. This is mainly attributed to its swelling potential, as it swells up to 18 times its original volume. Sodium-activated bentonite, on the other hand, could swell up to 10 to 15 times.

Results showed that there was no significant difference resulting from using sodium bentonite instead of sodium-activated bentonite in terms of fracturing sand. Comparing the injection pressures of tests 4 and 5 (Fig. 6), the only difference is the slightly slower build-up of pressure in case of sodium bentonite. This should have resulted from a problem with the injection system at the beginning of injection, as almost no heave or drainage was recorded over the delay period. The hardening speed of the used Portland cement (rapid



Figure 9. Dehydrated layer around the boundary of hardened grout for (a) Test 6 (left) and (b) Test 7 (right).

for GeoDelft and normal for Cambridge) did not affect the results, as most of the processes that influence sand fracturing happen well before hardening. In terms of grouting efficiency (defined as heaved volume divided by injected volume), both types of bentonite gave more or the less the same efficiency.

#### 4.5 No-fracture tests

Two injection tests were conducted using a low w/c ratio grout (w/c ratio of 1.0, 4% sodium bentonite added; see Tests 6 and 7 in Table 1). The tests were carried out in sand models of type D Leighton Buzzard sand ( $d_{50} = 234 \mu m$ ).

Injection under both slow (Test 6) and fast injection rates (Test 7) yielded no fracturing. The injection pressures were very close to each other, but slightly higher than the values for fracturing experiments. The slower injection rate gave more uniform shape of hardened grout (Fig. 9a), whereas some fingering was observed for the faster injection rate (Fig. 9b). In both cases, sectioning the hardened grout revealed a layer of dehydrated material around the grout-soil boundary.

With higher cement content and less water in the grout mix, no leak-off occurred and there was limited amount of free bentonite to develop a filter cake. Bleeding did occur and this in turn reduced the grout mobility. It is possible that the calcium in the cement changes the coagulation structure of the bentonite by cation exchange, increasing the permeability of the consolidated grout and accelerating bleeding (Sanders 2007). Most of the grout stayed around the injection point. The existence of a dehydrated layer at the groutsoil boundaries suggests that more bleeding happened around the boundaries, which is in agreement with the theory suggested by McKinley & Bolton (1999). The faster injection rate allowed less time for bleeding and hence, the grout managed to create some fingering before it became too viscous to flow.

Consolidation of the grout leads to the possible local irregularities at the boundaries of an injection hole to be filled up, or plastered, in the way shown in Figure 10.

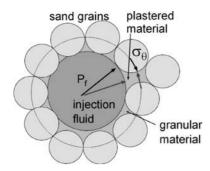


Figure 10. Influence of plastering on fracture initiation (Bezuijen & van Tol 2007).

Such plastering will hamper fracture initiation, as it prevents the fluid pressure from penetrating into the space between sand particles. As the created plaster will have a certain strength, part of the injection pressure will be acting on the plaster rather than being used to push particles away from each other to initiate a fracture.

#### 5 CONCLUSIONS

The experimental work conducted confirmed that fracture initiation in sand requires some local inhomogeneity around the injection point, rapid development of a filter cake with a limited thickness and a grout with low viscosity and a limited yield stress. Whether the initiated fractures will propagate or not depends mainly on the grout mixture. Water-cement (w/c) ratio and fine particles content play a major role in fracturing of sand. Grouts with high w/c ratios and enough fines will exhibit a leak-off of fine particles, accompanied with the formation of a filter cake, which results in fracturing. For grouts with low w/c ratios and large grout permeability, bleeding is the dominant process, leading to non-fracturing of sand.

For a suitable grout mixture, faster injection rates will result in thinner fractures, whereas slower rates give thicker fractures with less leak-off and thicker filter cake. If the w/c ratio is too low, no fractures will be formed, regardless of the injection rate.

The longer fractures experienced under relatively low injection pressures in the field are mainly due to the natural inhomogeneity of sand layers. Injection in almost perfectly homogeneous sand in the laboratory gives shorter fractures and requires higher injection pressures.

#### REFERENCES

Andersen, K.H., Rawlings, C.G., Lunne, T.A. & Trond, H. 1994. Estimation of hydraulic fracture pressure in clay. *Journal of Canadian Geotechnical* 31: 817–828.

- Bezuijen, A., Pruiksma J.P. & Pater C.J.de 2002. Maximum pressures in tunnelling limited by hydraulic fractures. *Proc. International Tunnelling Association Conference*, Amsterdam.
- Bezuijen, A., Sanders, M.P.M., Hamer, D. & Tol, A.F.van 2007. Laboratory tests on compensation grouting, the influence of grout bleeding. *Proc. World Tunnel Congress*, Prague.
- Bezuijen, A. & Tol, A.F.van 2007. Compensation grouting in sand, fractures and compaction. Proc. XIV European Conference on Soil Mechanics & Geotechnical Engineering, Madrid.
- Bezuijen, A. & Tol, A.F.van 2008. Mechanisms that determine between fracture grouting and compaction grouting in sand. Proc. 6th International Symposium on Geotechnical. Aspects of Underground Construction in Soft Ground, Shanghai.
- Bohloli B. & Pater C.J.de 2006, Experimental study on hydraulic fracturing of soft rocks: Influence of fluid rheology and confining stress. *Journal of Petroleum Science* and Engineering 53 (1–2): 1–12.
- Bruce, D.A., Littlejohn, G.S. & Naudts, A.M.C. 1997. Materials for ground treatment- A practitioner's guide. *Proc. Grouting: compaction, remediation & testing*, Logan.
- Chang, H. 2004. *Hydraulic fractures in particulate materials*. PhD. thesis. Georgia Institute of Technology.
- Chin, C.Y. & Bolton, M.D. 1999. Factors influencing hydrofracture in clay. Proc. 13th ASCE Engineering Mechanics Conference, Baltimore.
- Gafar, K. & Soga, K. 2006. Fundamental investigation of soil-grout interaction in sandy soils. Report. University of Cambridge.
- Jaworski, G.W., Seed, H.B & Duncan, J.M. 1981. Laboratory study of hydraulic fracturing. *Journal of the Geotechnical Engineering Division* 107(6): 713–732.
- Khodaverdian, M.M. & McElfresh, P.M. 2000. Hydraulic fracture stimulation in poorly consolidated sand: mechanism and consequences. SPE Annual Technical Conference, Dallas.
- Kleinlugtenbelt, R. 2005. Compensation grouting, laboratory tests in sand. MSc thesis. Delft University of Technology.
- McKinley J.D. and Bolton M.D. 1999. A geotechnical description of fresh cement grout – Filtration and consolidation behaviour. Magazine of Concrete Research 51(5): 295–307.
- Mori, A. & Tamura, M. 1987. Hydro-fracturing pressure of cohesive soil. Journal of the soil and foundations, Japanese Society of Soil Mechanics & Foundation Engineering 27(1): 14–22.
- Sanders, M.P.M 2007. Hydraulic fracture grouting, laboratory tests in sand. MSc thesis. Delft University of Technology.
- Soga, K., Gafar, K.O., Ng, M.Y.A. & Au, S.K.A. 2006. Macro and micro behaviour of soil fracturing. *Proc. International Symposium on Geomechanics and Geotechnics of Particulate Media*, Yamaguchi.
- Soga, K., Ng, M.Y.A. & Gafar, K. 2005. Soil fracturing in grouting. Proc. 11th International Conference of the International Association of Computer Methods and Advances in Geomechanics, Tornio.
- Thallak, S. 1991. Numerical simulation of hydraulic fracturing in granular media. Phd thesis. University of Waterloo.

# Historical cases and use of horizontal jet grouting solutions with 360° distribution and frontal septum to consolidate very weak and saturated soils

#### G. Guatteri

Novatecna Consolidações e Construções S.A - São Paulo, Brazil Terrajato - Lisbon, Portugal

A. Koshima, R. Lopes & A. Ravaglia Novatecna Consolidações e Construções S.A – São Paulo, Brazil

#### M.R. Pieroni

Novatecna Consolidações e Construções S.A – São Paulo, Brazil Terrajato – Lisbon, Portugal

ABSTRACT: Horizontal jet grouting technology has proved to be quite a versatile tool for dealing with the particular geological conditions encountered when excavating tunnels in soil mass, especially in the presence of water flow and high hydraulic gradient. We describe the state-of-the-art application known as 360° distribution, in which horizontal jet grouting columns are executed around the excavated section, including the invert, and at the far end of the conical treatment, to create a watertight chamber. This results . in a heading that is constantly protected by pre-consolidated soil and minimizes the effects of excavation on nearby structures. We also describe a special tool (called the 'preventer') designed to control drilling and injection fluid outflow, which proved to be absolutely essential to successful outcomes. We also present the experience gained by the authors in a number of projects in Brazil and Venezuela and the results of a full scale test, executed for the first time in Europe.

#### 1 INTRODUCTION

Horizontal jet grouting has proved to be an efficient and versatile technical solution and is increasingly being used for tunnelling in difficult soils, usually with high granulometry, presence of intense water flow and high hydraulic gradient.

Further developments have facilitated more non-TBM excavations in locations with low overburden and superstructure presence, such as urban areas where stable and safe conditions during excavation are required at all times, with minimum impact on nearby structures.

One type of application that is not yet in common use is the one in which horizontal jet grouting columns are executed all around the tunnel cavity in what is called a 360° distribution (roof, sidewalls and invert) and in which, depending on the circumstances, consolidation is extended to form a horizontal soil-cement frontal wall (septum) at the far end of the conical treatment.

Therefore, for the heading of the tunnel, a sequence of watertight chambers are created so that the excavation activity is always protected by pre-consolidated soil. We provide a brief description of this innovative technical solution, its main geometrical characteristics and use conditions.

From the executive point of view, we describe the use of a special tool named the Preventer, which was developed to control the outflow of drilling and injection fluids, and is absolutely essential in order to obtain good performance.

Finally practical cases, executed by the authors, are described, along with the results of the first full scale test of a 360° jet grouting treatment executed in Europe.

#### 2 360° DESIGN

In recent years, horizontal jet grouting technology has frequently been used for tunnels where safety was a priority and more traditional techniques were not feasible or might not guarantee good performance. As readers will know, this applies mainly to consolidating soil mass with poor geo-mechanical characteristics which do not allow excavation in safe conditions, mainly in sandy, silty or clayly soils or a combination of these, with or without presence of water.

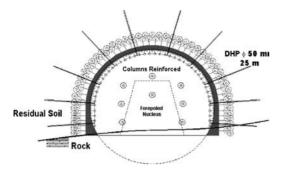


Figure 1. Typical distribution for roof and lateral protection and forepoled nucleus.

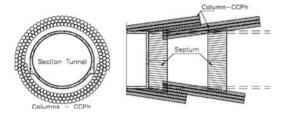


Figure 2. Typical distribution of a full face  $360^\circ$  distribution.

This technology is normally associated with the NATM. In a typical configuration, columns are executed side by side to form a pre-consolidated arch which follows the external profile of the excavated section.

Depending on the actual soil conditions, the consolidation effect may be improved by jet grouted columns executed inside the section to reinforce the tunnel's nucleus, or by means of horizontal drains drilled at the bottom or around the section. In more severe situations, the jet grouted columns may be reinforced by means of steel pipes or fiberglass, and drains may be equipped with vacuum systems. Fig. 1 shows a typical traditional distribution.

The encouraging results obtained from the horizontal jet grouting technique and the growing needs for executing tunnels in increasingly difficult conditions, at shallower depths, generally in alluvial or even residual deposits but with high granulometry, intense water flow, hydraulic gradient and the presence of important superstructures, as in many urban projects, prompted the development of the '360° distribution'.

The main characteristics of the 360° solution, compared to the traditional, are: a) soil consolidation, previously limited to roof and sidewalls, is now extended to the invert, and b) the execution of a jet grouted plug at the bottom of the conical treatment.

Thus a sealed chamber can be created, and in terms of stability, soil-mass loads around the cavity are redistributed to prevent possible inflows of material from

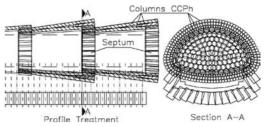


Figure 3. Example of 360° distribution for half-section and bench heading.

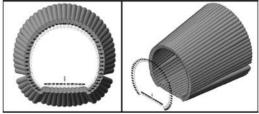


Figure 4. 3D Analysis of a 360° distribution.

the excavation face and the invert, which are quite common in sandy soils.

From the hydraulic point of view, the drastic reduction of water inflow during the excavation phase reduces soil-mass dewatering and therefore keeps settlement under control and minimizes effects on nearby structures while the heading is executed. Fig. 2 shows a typical example of a 360 degrees distribution with full-face excavation.

Depending on the tunnel dimensions, the solution may also be adjusted for use with a half-section heading. In this case, the bench or final invert may be executed, if required by the geological conditions, with the help of vertical or inclined jet grouting to ensure safer conditions and optimize working schedule (fig.3).

As far as the operational aspect is concerned, the 360° solution became feasible due to advances in drilling equipment, involving redesign and improvement of its maneuverability, set up and mast alignment. Special homothetic templates with the projection of the consolidation elements for the perfect alignment of the drilling string had to be developed and implemented.

Also, 3D graphic tools were introduced in order to verify the integrity and continuity of the treatment designed (fig.4).

Special consideration must be given to the device used to control the outflow of drilling and injection fluids, which evolved from a similar one used in the oil industry, hence the name 'preventer'. Proper use of this device proved to be essential to achieve appropriate

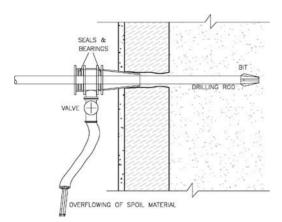


Figure 5. Preventer typical configuration.

monitoring and control of movement of excavated soil mass.

#### 3 THE PREVENTER CONTROL VALVE

The "preventer valve" (or "blow-up preventer") is used to avoid gas leaks and explosions when drilling oil wells.

Preventer valves and retainers at the mouth of the hole control outflows of solid and liquid spoils from drilling when ground is sandy and susceptible to "piping." They also facilitate control of slurry (excess soil-cement mixture) during jet grouting jobs, thus avoiding "piping" along the hole or the column.

Note that a drilling operation in non-cohesive soils below the water table produces withdrawal of material due to the intense flow established by the water table hydraulic gradient and water needed for the drilling operation itself.

In such case, it quite common to lose control of the volume of material flowing out of the hole if there is not proper perception of the phenomenon, thus triggering a piping effect.

This piping occurs when pressure inside the hole during the drilling operation is less than the effective hydrostatic pressure acting into the ground. And this can occur also during the jet grouting phase.

In this situation, with the *preventer* installed at the hole mouth, we may control pressure inside the hole, thus avoiding decompression of the surrounding material leading to an increase of permeability.

This device (fig.5) has a system of valves and seals to maintain strict control of volumes of materials withdrawn during drilling or injection. At the end of the jetting operation, it can also be sealed for the time needed for the soil-cement mixture to set.



Figure 6. Tunnel face with starting drilling points.



Figure 7. Septum and installation of yielding arch.

#### 4 HISTORICAL CASES

#### 4.1 Copacabana subway tunnel (Rio de Janeiro – Brazil)

Rio de Janeiro's subway company planned a 750 m extension to line 1, from Cardinal Arcoverde Station to the center of Copacabana, also building Siqueira Campos Station and turn-offs or maneuvering areas. The underground structure comprised two independent but juxtaposed "eyeglass type" tunnels crossing gneissic rock, saprolite and micacious residual silty-sandy soil and sandy clayish soil of marine origin, of high hydrogeological complexity; the route also crossed a densely populated built-up area.

One of the greatest challenges was the NATM section of the tunnels under the direct foundations of a 50-year-old 7-storey building, standing on sandy sediment (beach sand), with around 6m overburden.

These tunnels were previously treated with horizontal jet grouting forming a closed 360° conical chamber with treatment throughout the excavation cross section and front septum protecting each advance module. (figs. 6, 7).

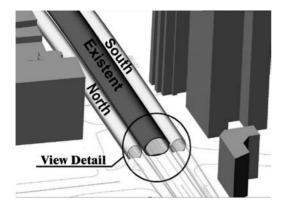


Figure 8. Plaza Italia/Capuchinos Twin Tunnels.

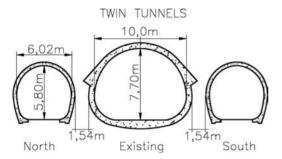


Figure 9. Section details.

The work was successfully concluded on time and the preventer valve was a crucial factor in ground treatment. The maximum settlement recorded was 30 mm.

#### 4.2 Plaza Italia Tunnel – Line 4 subway (Caracas – Venezuela)

Part of the new Line 4, the North and South Tunnels Initial Sections, heading from Plaza Itália toward Capuchinos Station, involves the expansion of an existing station where two tunnels of approximately 6.5 m diameter were to be excavated on each side of an existing tunnel, which had to be maintained operational (fig. 8 and 9). In this case the existing structure, acted as a barrier to the ground water flow, raising its level and pressure.

On starting the excavation, instead of the residual soil predicted by the original geological investigation, tunnelers found highly permeable alluvial sandy soil with gravel at the bottom of the excavation section, topped by a clayly layer of residual origin (micaceous schist), and a sedimentary fine sandy layer, at the upper part of the tunnel. The overburden was around 10 meters.

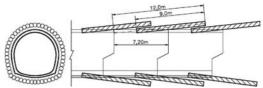


Figure 10. 360° horizontal jet grouting scheme.

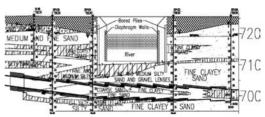


Figure 11. Geological profile and tunnel path details.

In order to cope with these difficulties, various preventive measures were taken in the design and execution phases. First of all, the design was adjusted to the in situ conditions, extending the vault treatment to the invert or locally, whenever it was necessary, so as to pre-consolidate and reduce water flow (360° Horizontal Jet Grouting, fig. 10).

Due to the presence of water under pressure, the drilling and jet grouting operations at the invert had to be executed using the *preventer* valve.

Since the central portion of the section proved to be in clay material, there was no need to execute a jet-grouted frontal septum.

However, specific measures to control pressure and water flow using deep horizontal drains, equipped with check valves when necessary, had to be taken.

#### 4.3 Tamanduatei river – service tunnel (São Paulo– Brazil)

To our knowledge, this is the first case worldwide in which the 360° distribution has been adopted as a pre consolidation shape in a NATM tunnel.

In this case the challenge was the tunnel running at about 10 m under street level, but deepening to 25 m when crossing the river (Fig. 11) due to the project requirement for a minimum of 5 m under the bottom of the deepest diaphragm-walls containing the canalized river; this structure would pose a serious risk of becoming a preferential water-path communicating with the river bed.

The geological profiles (Fig. 11) showed that the tunnel is embedded in an alluvial mass deposited during the more recent Tertiary period. The matrix comprises coarse, medium and fine light-gray sands, with N(SPT) 10 to 40 and is very pervious at the tunnel's levels.

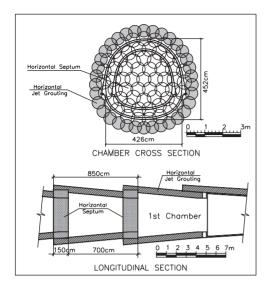


Figure 12. Treatment scheme with full section septum.

Inside this matrix, two highly plastic silty clay layers, 2 to 5 m thick, are intercalated but only randomly intersecting the tunnel excavation.

Water table observations showed a quick response during rainy periods. The same effect was produced whenever the level of the river rose due to rainfall anywhere along its upper course.

Before the excavation, as additional precaution, watertightness was checked by installing three horizontal drains inside the chamber so as to verify the presence of water under pressure and/or continuous flow.

#### 4.4 Full scale test for a railroad tunnel (Barcelona – Spain)

The Madrid-Zaragoza-Barcelona-French Frontier high speed rail link's section connecting to Sants Station in Barcelona, required a tunnel with a  $120 \text{ m}^2$  cross section located underneath the path of the existing ground level railway.

The geological investigation detected the presence of alluvial sandy material with high water table and high permeability, which was potentially dangerous for the tunnel's stability and the existing nearby structures, without special soil consolidation treatment.

In order to verify the efficiency of the horizontal or inclined jet grouting consolidation techniques in submerged conditions, it was decided to carry out a full scale test to confirm the feasibility of some alternative technical solutions proposed, verify the consolidation design parameters and check the effect of the activity at ground level, recording settlements and water table variations.

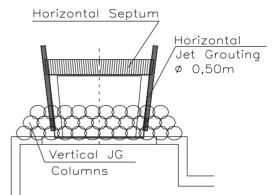


Figure 13. Full Scale Test - General lay-out.



Sandy to Silty Soil Silty Silty Clay

Figure 14. Geological Profile related to the chamber position.

On a slightly smaller scale (80% of the original dimensions), the test reproduced the executive conditions of a consolidated horizontal half-section heading with all the structural and waterproof elements executed in accordance with the procedures and specifications stipulated for the actual structure.

Fig. 13 shows the general lay-out and geometry of the proposed treatment. The heading has been dimensioned to simulate the excavation of a stretch of tunnel 5 meters long between two jet grouted septums. The soil cover above the tunnel's crown is around 6 meters. Also the logistic and operational conditions facing the excavation crew during the actual work were accurately reproduced.

The geological profile at the test area is shown in fig.14. Basically it consists of a sequence of sandy to silty soils with SPT varying from 5 to 15, intercalated with very thin layers of silty clay. The presence of stiff gray clay was detected only well below the invert.

Between the tunnel's crown and invert levels, the soil is a very fine silt that is under hydraulic gradient and flowing out of the excavation almost without control. Since above the crown level there is no more

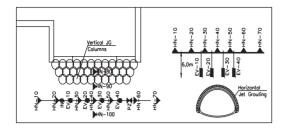


Figure 15. Instrumentation lay-out.

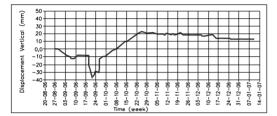


Figure 16. Bench Mark HN 40 - Readings.

consistent strata, even a small outflow is immediately reflected at the surface.

To verify the effects of the jet grouting injection onto nearby structures and the efficiency of the chamber's watertightness, piezometers and bench marks were installed, properly distributed above and around the test area (see fig. 15).

A number of horizontal drains with vacuum system were installed inside the excavation section in order to facilitate the chamber dewatering. A connection between the chamber and the soil mass outside would be detected by the external piezometers. The readings of settlements and water table levels were taken daily.

Fig. 16 and 17 show a summary of the data recorded throughout the test period. Despite very poor soil characteristics, it was possible to keep soil movements around the chamber and at ground level under control, with an average variation of about  $\pm -20$  mm.

The preventer (control valve system) was permanently installed except in a particular situation when, due to a technical problem with this device, a massive loss of material was experienced. Once the preventer was repaired, the original ground conditions were reinstated by injecting and recompressing the ground area affected. A pressure gauge was installed at the mouth of the device to verify the build-up of pressure inside the soil mass. A maximum value of 0.4 Mpa was recorded.

Piezometers readings were stable during the jet grouting consolidation phase. During the chamber excavation, one piezometer showed an anomaly indicating a possible gap in the consolidated area.

The problem was solved by reinforcing the consolidation with a small number of additional columns on

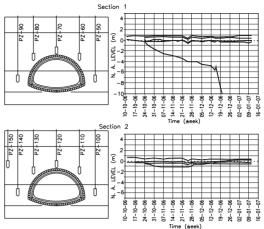


Figure 17. Piezometers Readings.

one side of the external alignment limited to the area affected by the phenomenon. The chamber excavation was successfully concluded confirming the stability and watertightness of the proposed treatment.

The test results were fully adopted in the design and subsequent execution of the main tunnel under the existing railway.

#### 5 CONCLUSIONS

The 360° consolidation treatment is the state-of-theart in terms of pre-consolidation technique associated with NATM mainly in alluvial or residual formations, with high granulometry and relevant presence of water pressure, and in general in all types of soil were jet grouting technique can form the column element.

To ensure the level of efficiency required, the instruments to pursue treatment's imperviousness and geometry are the *preventer* valve that is essential in order to avoid uncontrolled loss of material due to piping together with the special homothetic templates for perfect alignment of the drilling string.

The 360° consolidation scheme proved to be extremely flexible, as it can be adjusted to actual in situ geological conditions, and when executed in the appropriate manner and with proper equipment, achieves extremely good results in terms of ground movements, with a recorded average of 20 to 30 mm thus allowing a proper control of side effects on existing structures.

So far this technique has be implemented in urban tunnels with a minimum overburden of around 6 meters. The encouraging experience gained since 1998 when was executed for the first time, is at present leading the authors to study its application in even shallower tunnels with a soil cover of 3 to 4 meters.

#### REFERENCES

- Guatteri, G. Mosiici, P. Doro Altan, V. Koshima, A. 1998. Brazilian Experience in Jet Grouting Treatments in Difficult Tunnels. World Tunnel Congress 98, ITA CBT, Tunnels and Metropolises, São Paulo.
- Guatteri, G. Koshima, A. Lopes, J. Doro Altan, V. 2000. 360° Jet Grouted Conical Chamber Allow Safe Tunneling Under River Within a Highly Pervious Environment. Geoenge 2000 – An International Conference on Geotechnical & Geological Engineering; Melbourne, Australia.
- Guatteri, G. Koshima, A. Lopes, J. Pieroni, M. 2004. Resume of Brazilian Experience about Use of Jet Grouting in Tunnels and Underground Excavations. 1° Brazilian Congress of Tunnels – São Paulo.
- Guatteri, G. Koshima, A. Ravaglia, A. Duarte, M. 2007. Jet Grouting Solutions for the Caracas Subway Line 4. Challenges in Urban Projects – XIII Panamerican Conference on Soil Mechanics and Geotechnical Engineering – Venezuela
- Sózio, L. 2002. Jet Setting in Rio. Tunnels & Tunneling International, I-1,15.

# The effects of sample dimension and gradation on shear strength parameters of conditioned soils in EPBM

M. Hajialilue-Bonab, M. Ahmadi-adli, H. Sabetamal & H. Katebi *University of Tabriz, Tabriz, Iran* 

ABSTRACT: Mechanical properties of conditioned soils in EPBM tunneling consist of lots of unknowns. In this research, the tests has been arranged to fulfill of four goals. Firstly the effects of conditioning on the shear strength variation have been investigated. Secondly an investigation on effects of conditioning on shear strength parameters  $(C, \varphi)$  has been performed. In third step the results exerted from two previous stages have been compared for two shearing apparatuses of conventional and large shear boxes. The last goal of the research is exploration of effects of changes in conditioning parameters on shear strength. It is found that the C& $\varphi$  for tested soils obtained from large shear box are usually greater than the results of the same soil in conventional shear test. This result is less significant for conditioned soil with compared to unconditioned soil and it is found to be a function of injected foam content.

#### 1 INTRODUCTION

Determining the shear strength parameters of the conditioned soils usually lead to the use of conventional shear box tests because of no availability to large shear boxes. In order to use this apparatus, soil gradation must be modified based on the codes which demands to elimination of great mass of coarse particles. This elimination affects the shear strength parameters especially in conditioned soil. In this research the effects of sample dimension and soil gradation on properties of foam conditioned soils have been investigated. A modification on soil gradation has been done in samples of large and conventional shear boxes. The variables in this research are normal stress and the injected foam quantity in to each of soils.

#### 2 MATERIALS AND PROCEDURES IN SOIL CONDITIONING

- 2.1 Conditioning parameters
- Concentration (C<sub>f</sub>) This parameter is the content of foaming agent in water unit weight.
- Foam Expansion Ratio (FER) This is the volumetric expansion of a unit volume of foaming solution.
- Foam Injection Ratio (FIR) It consists of the ratio of injected foam volume to volume of the conditioned soil.

The last parameter is of great importance in successful soil conditioning.

#### 2.2 Laboratory foam generation system

To produce a conditioner (foam) with specific and controlled parameters, a foam generating system in laboratory scale was designed and constructed. This apparatus has the ability of controlling foam properties along the production and can produce uniform foam. The schematic plan of this system which manufactured in soil mechanic laboratory of Tabriz University is shown in figure 1.

Foam generation is performed by mixing process of air and liquid (consist of foam agent) under pressure. In the first step the reservoir must be filled by foam Solution. Foam generation is performed by mixing process of air and liquid (consist of foam agent) under pressure. In the first step the reservoir must be filled by foam solution. A regulator controls the air pressure supplied to the reservoir. Liquid valve is opened and flow control valve is used to adjust the flow through the liquid flow meter to the foam generator unit inlet. The pressure in the air flow line is controlled with a control valve of liquid flow meter and measured by a pressure gauge. Air pipe valve is opened to allow the air to flow through the control valve and the air flow meter to the generator unit inlet. The pressurized air and foam solution then flow through the generator unit to produce the foam. The design of the foam generator allows the liquid and

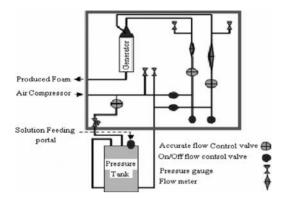


Figure 1. Schematic of laboratory foam generation system.

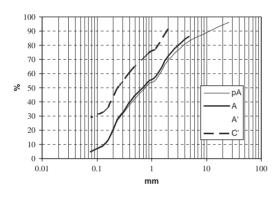


Figure 2. Particle distribution of soils used in this research.

air flow rates and pressures to be adjusted and monitored separately to control the properties of the foam produced.

#### **3 SHEAR TESTS PATTERNS**

In order to perform large shear box  $(300 \times 300 \times 150 \text{ mm})$  tests and conventional shear box  $(60 \times 60 \times 20 \text{ mm})$  tests the soil excavated from Tabriz metro line has been used. This soil labeled pA. Considering the maximum dimension of the soil particles with respect to the shear box dimension (large, conventional), soil gradation has been modified to A, A' (by elimination of the oversized particles) and C' (same as A' with 30% more in fine content). Particle size distribution for these soils has been illustrated in figure 2. Direct shear tests on soils A, A' and C' were performed using conventional and large shear boxes. The normal stresses on samples also were 37.52, 64.75, 119.29 and 228.28 (kPa).

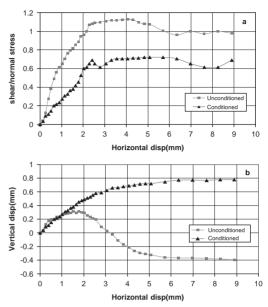


Figure 3. (a) Variation in ratios of shear to normal stress and (b) vertical displacement of sample cap versus horizontal displacement in Conventional shear boxes on soil A' under normal stress 37.52 kPa.

#### 4 RESULTS AND DISCUSSIONS

### 4.1 Conditioning effects on shear strength variation trends

The behavior of soil A' in conditioned and unconditioned state under different normal stress (37.52, 64.75 and 119.29 kPa) in direct shear tests have been shown in Figure 3 to 5. The variation of shear/normal stress ratio versus horizontal displacement has been investigated. The variation of vertical displacement with respect to horizontal displacement during shear test has been also considered. Figure 3 shows the results for soil A' in both conditioned and unconditioned state. A reduction of almost 0.4 in maximum values of stress ratios can be observed. The curvatures indicate some peak point in lower stress ratios in unconditioned soil test. This peak is going to disappear by increasing the stress level. But in conditioned samples there is no peak point in any stress level. It can be observed that increasing in stress level can convert the dilative behavior to contractive behavior. However the conditioned samples give a completely contractive behavior regardless of initial specific volume.

### 4.2 Conditioning Effects on shear strength parameters

Table 1 shows calculated C& $\phi$  for performed tests. The friction angle for peak and residual states are given.

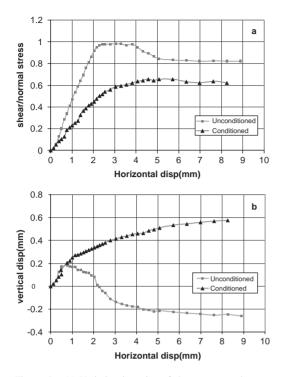


Figure 4. (a) Variation in ratios of shear to normal stress and (b) vertical displacement of sample cap versus horizontal displacement in Conventional shear boxes on soil A' under normal stress 64.75 kPa.

The residual friction angle represents the state of the soils in excavation process.

Comparison of corresponding shear tests performed on both conventional and large shear boxes indicate that the large boxes give internal friction angle 6 to 7 degrees greater than conventional shear tests. However in the tests on conditioned samples this reduces to 3 to 4. Large apparatuses also result in lower cohesion in comparison to the standard samples. Also in conditioned samples this trend is traced but it is less intensive.

#### 4.3 Effects of variation in conditioning parameters on shear strength parameters

The needed conditioner for soils used in tests has been predicted using the Kusakabe 1999 formula.

$$\mathcal{Q} = \frac{a}{2} [(60 - 4.0X^{0.8}) + (80 - 3.3Y^{0.8}) + (90 - 2.7Z^{0.8})] \tag{1}$$

Where X = the percentage of soil passing 0.074 mm; Y = the percentage of soil passing 0.25 mm; Z = the percentage of soil passing 2.0 mm; a = 1.0 for  $C_u$  > 15, a = 1.2 for 15 >  $C_u$  > 4, a = 1.6 for 4 >  $C_u$ 

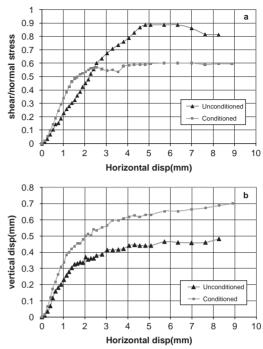


Figure 5. (a) Variation in ratios of shear to normal stress and (b) vertical displacement of sample cap versus horizontal displacement in Conventional shear boxes on soil A' under normal stress 119.29 kPa.

Using this formula for soil A, the FIR is 50%. Soil A has been tested with FIR equal 50%, 30% and 70% and results are shown in figure 6. For example in treatment with FIR = 30% an increasing of 0.11 in normal/shear stress ratios can be observed. Contrarily, with FIR = 70% no remarkable effect was observed. It is clear that increasing FIR do not change the normal/shear stress ratios while a little reduction on FIR has considerable effect on this ratio.

Figure 7 gives the results for another group of similar tests on soil A' with various FIR. If one apply the soil gradation A' to the Kusakabe formula, FIR will result in 40%. By increasing foam consumption.

In soil samples, a negligible reduction in stress ratios can be observed, but a little reduction in foam consumption will result a significant increase in these ratios. This relation in conventional shear tests is about 0.17. This means that consuming more foam indicates slight effect on shear strength of samples. During shearing, pore pressure will generate but in granular soil this will dissipate very quickly. In the presence of foam the dissipation of pore pressure will postpone and the shear strength will decrease in foamed granular soil. Comparison of changes in stress ratios in large and conventional tests shows that conventional direct shear tests results are more sensitive to the changes in

	Fine content	Amount of conditioner	Normal stress	τ <sub>μ</sub> kPa	ذ peak	C kPa	ذ residual
Large shear box	5	0	37.52	45.322	50.38	9.221	44.2
ar			64.75	72.888	48.39		
she			119.29	125.08	46.36		
ê	5	50	37.52	26.095	34.82	4.42	31.1
,arg			64.75	44.93	34.76		
П			119.29	75.929	32.48		
al	5	0	37.52	42.281	48.41	14.62	37.4
ů ,			64.75	63.667	44.51		
inti Doy			119.29	105.95	41.61		
Conventional shear box			228.28	186.68	39.28		
Con the con	5	40	37.52	27.076	35.81	7.259	28.4
<u> </u>			64.75	43.066	33.63		

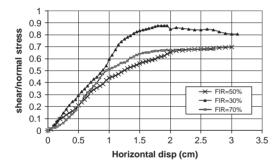


Figure 6. Variation in ratios of shear to normal stress versus horizontal displacement in large shear boxes on soil A under normal stress 64.75 kPa.

FIR than the results in the same samples in large shear test. The results for other tests give the same conclusion. This can be observed in Figure 8. This figure give some more information about soil C' which is similar to soil A' with some more fine content. It's obvious that the increasing FIR in samples that have excessive fine causes more significant loss in internal friction angle.

But in samples which have more fines this phenomena firstly occurs steeply but in FIRs higher than optimum this trend is not observed at all.

It can be concluded that the soil which have more fine content need less FIR to get a specific reduction of shear strength.

In order to study the possible effects of variation in foam expansion ratio (FER), a foam in three FERs of 5, 10 and 22 were prepared for similar shear tests on

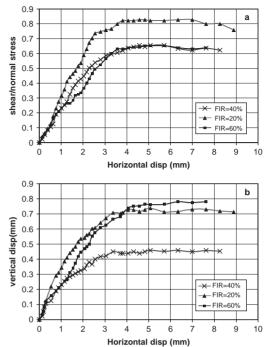


Figure 7. (a) Variation in ratios of shear to normal stress and (b) vertical displacement of sample cap versus horizontal displacement in Conventional shear boxes on soil A' under normal stress 64.75 kPa.

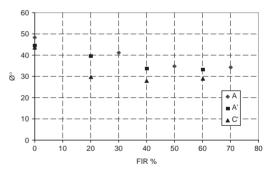


Figure 8. Internal friction angle versus variations in FIR.

soil C'. Figure 9 shows that a reduction on FER causes no effect on shear parameters and stress ratios.

The reason which can be stated is that the low FER indicates very wet foam so that it is unable to reduce the relative density and consequently increase the specific volume. On the other hand when the FER increases an unstable state occurs and consequently no changes in stress ratios happen. It is resulted that a FER closer to 15 gives a stable state for foam.

Figure 10 gives a view of obtained variations in stress ratios of conditioned samples during shearing

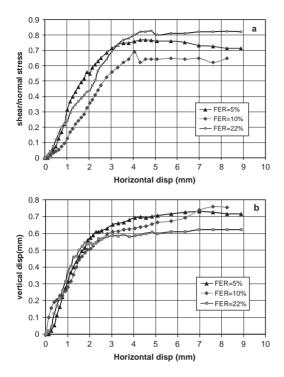


Figure 9. (a) Variation in ratios of shear to normal stress and (b) vertical displacement of sample cap versus horizontal displacement in Conventional shear boxes on soil C' under normal stress 64.75 kPa.

because of changes in Cf. Three tests performed on C' with three different initial Cf of 3%, 4% and 5%. If we consider the possible changes in stress ratios there is no significant difference between these samples. But the volume of samples tends to reduce during the test. The authors considered this logical as well as declaration of producers of conditioners.

#### 5 CONCLUSIONS

- The soil conditioning cause a decrease of 7 to 11 degrees in internal friction angle of soils. A loss of 49.1 kPa for the cohesion of conditioned soil was observed.
- 2. Decreasing FIR from its optimum given by Kusakabe formula, will greatly affect the trend of reduction in shear strength. But increasing in that quantity does not show remarkable effect. FER lower than 10 and more than 18 causes no conditioning and consequently the changes in strength are negligible.
- 3. Large shear test gives greater internal friction angle about 6 to 7 degrees in unconditioned samples and

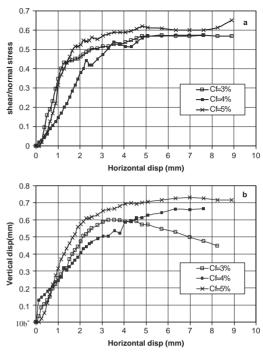


Figure 10. (a) Variation in ratios of shear to normal stress and (b) vertical displacement of sample cap versus horizontal displacement in Conventional shear boxes on soil C' under normal stress 64.75 kPa.

3 to 4 angles in conditioned samples comparing conventional shear test.

#### REFERENCES

- Ahmadi adli, M, "Investigation on mechanical properties of conditioned soils for EPB mechanized tunneling in Tabriz", 2006, M.Sc. thesis, Faculty of Civil engineering, University of Tabriz. Tabriz: Iran.
- Defence Standard 42–40. 2002. Foam Liquids, Fire Extinguishing (Concentrates, Foam, Fire Extinguishing). Ministry of Defence. Issue 2. USA.
- EFNARC. 2005. Specifications and Guidelines for the Use of Specialist Products for Mechanized Tunnelling (TBM). In Soft Ground and Hard Rock. EFNARC, UK.
- Merritt, A. 2004. Conditioning of clay soils for tunneling machine screw conveyors. PH. D. thesis, St. John's college, Cambridge university. London: England.
- Milligan, G.W.E. 2000. Soil conditioning and lubrication in tunneling, pipe jacking and micro tunnelling. A state of art review. http/www.civils.eng.ox.ac.uk/research/ pipejacking.htm
- Psomas, S. 2001. Properties of foam/sand Mixtures for tunnelling applications. M.Sc. thesis, St. Hugh's college, Oxford university. London: England.

### Experimental study on compressibility behavior of foamed sandy soil

M. Hajialilue-Bonab, H. Sabetamal, H. Katebi & M. Ahmadi-adli

University of Tabriz, Tabriz, Iran

ABSTRACT: In order to assess the influence of different foam types on compressibility behavior of conditioned sand, a set of tests were performed on three gradation of sandy soil. Some index tests were also undertaken for verifying foam agents characteristics and foam generator quality. Details and discussion about different aspects of mentioned cases have been presented in this paper. Compressibility tests were performed by a 151 mm diameter Rowe Cell and foam generation was carried out by foam generator which was constructed by the authors.

#### 1 INTRODUCTION

Ideal ground conditions for EPB machines consist of soils with relatively high fines contents such as clayey silts or silty sands, with a consistency to form a low permeability, soft plastic paste when excavated. These properties allow the support pressure to be transferred uniformly to the tunnel face and controlled flow of the soil through the machine. The compressibility of soil has an important function on machine performance. However, natural soils rarely have these ideal properties, and conditioning of the soil is usually necessary to change its properties to suit the machine. Soil conditioning for EPB machines involves injecting conditioning agents into the excavated soil. The objective is to modify the soil properties to form a soft plastic paste, leading to improvements in the machine performance and control of the excavation process in a wide range of soils. The specific treatments required to effectively condition different types of soil. Many factors influence the specification and performance of soil conditioning treatments.

Soil conditioning is clearly performed by means of some chemical & physical materials such as foam and polymer which lead to decrease permeability, increase plasticity, etc. The SCA (Soil Conditioning Agent) is also responsible for controlling the rheological properties of the extracted soil, and minimizing wear and abrasion of the cutter head during tunneling. In order to achieve all of these objectives, the SCA composition must be tailored to the properties of the soil. Foams, mainly consisting of surfactants, are used in fine-grained materials to reduce material adhesion to exposed EPBM surfaces, and to fluidize the muck (reduce balling and enhance flow ability). Foams are also used in granular deposits to improve material rheology and reduce soil permeability. While the use of SCAs in EPBM tunneling is on the rise, the use of SCAs in general is very much a "black box" technology. According to Psomas (2001) foam/sand mixtures exhibit low shear strength and they have very compressible, so in order to assess the influence of foam and polymer on shear strength and Compressibility behavior and influence of foam parameter in those, supplementary tests performed with variety of foam agents and foam parameters such as FER, FIR and Cf on three gradation of sandy soil (see Sabetamal 2006). This paper focused on the results of compressibility tests which have been undertaken by two types of foam agents.

#### 2 EQUIPMENT AND MATERIALS

#### 2.1 Foam generator

Lab scale TBM foam generator with full controlling possibility on various part of system was designed and constructed. The operation of the laboratory foam generator is described here with reference to the schematic diagram shown in figure 1. Foam generation is performed by mixing process of air and liquid (consist of foam agent) under pressure. In the first step the reservoir must be filled by foam solution. Then it flows on route 2 via air pressure. A regulator (b) controls the air pressure supplied to the reservoir and the compressed air is flowed in route (1). The entrance air pressure is measured by gauge "a". Valve (2) is opened and flow control valve is used to adjust the flow through the liquid flow meter to the foam generator unit inlet. Valve (1) is opened to allow the air to flow through the control valve and the air flow meter to the generator unit inlet. The air pressure is measured by a pressure gauge (e). The pressurized air and foam solution then flow

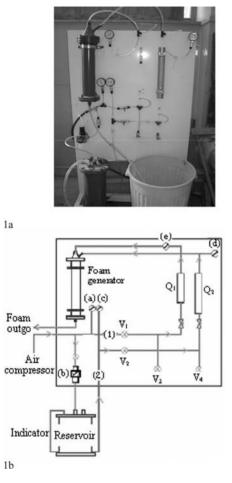


Figure 1. a: Photo of laboratory foam generator. b: Schematic of laboratory foam generator.

through the generator unit to produce the foam. The design of the foam generator allows the liquid and air flow rates and pressures to be adjusted and monitored separately to control the properties of the foam.

#### 2.2 Rowe cell

Compressibility tests were carried out in a standard 151 mm diameter Rowe Cell. Stiff porous discs were placed on the top and bottom of the sample so that the boundary conditions are fixed strain rather than free strain. All tests were undertaken at seven stress level and each level composed of two stages: undrained stage (immediate settlement is obtained) and drained stage (settlement due consolidation is obtained). At each stress level, the measurement of drained liquid volumes in second stage leads to calculate the air and liquid void ratio. Vertical settlement and vertical

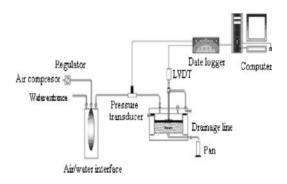


Figure 2. Compressibility tests equipment set up.

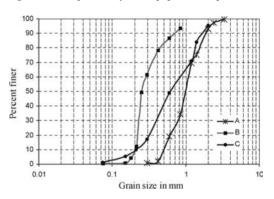


Figure 3. Sand's gradations.

Table 1. Sand's properties.

Soil	GS	emin	emax	D50 mm
A B	2.65 2.65	0.452 0.660	0.695 0.940	1.0 0.25
C	2.65	0.580	0.766	0.60

pressure are measured using LVDT and pressure transducer respectively according to figure 2.

#### **3 STUDIED MATERIALS**

#### 3.1 Sand

The gradation curves of different sandy soils which were used in this research are shown in figure 3. The soil type A and B are quite uniform. The soil type C is a mix of soil A and B. The properties of these sandy soils such as  $G_s$ ,  $e_{min}$ ,  $e_{max}$  are shown in table 1

#### 3.2 Foam agents

Four different types of foam agent and one type of polymer were used in foam index tests. Only two of them were used in compressibility tests. Agent type D is produced in Iran and vastly used in Shiraz metro project. Other agents (named A, B, C) and polymers (named P) are produced by Degussa. The foam generator was able to operate with all of them that produce acceptable quality micro-foam.

#### 4 FOAM INDEX TESTS

The foam properties depend on its different compounds like air, water, surfactant and sometimes polymer. The parameters which characterize foam are:

- Surfactant Dosage =  $C_f$  [%]
- Polymer Dosage =  $C_P$  [%]
- Air Ratio (Foam Expansion Ratio) = FER
- Foam Injection Ratio = FIR
- 25% or 50% drainage time according to "Minis try of defense Standard 42–40"

The amount of air introduced to the soil can be changed with the air ratio FER which characterizes the ratio between air and liquid volume. The foam injection ratio FIR indicates the volume of foam used per  $1 \text{ m}^3$  excavated soil. In order to verifying the properties of produced foam and to investigate the capability of foam generator to produce qualified foam with different agents, 50% drainage time were tested for produced foams.

According to the index tests, it is found that to produce of foam with equal FER, the amount of air and liquid flows must be adapted with each foam agent type. The amount of FER with respect to Cf for foam D at the same condition is shown in figure 4a. It can be observed that, with increasing of Cf, FER will increase.

The stability times for different foam are shown in figure 4b. The stability time varies from 6 to 15 minutes for different foams. By adding 2.5% polymer to foam agent B the stability time will be doubled. This can be observed for all foam agents. This is an important result as the stability time is very important for some kind of excavations.

#### 5 COMPRESSIBILITY TESTS

The compressibility tests were normally performed at four stress level in which the stress be doubled with respect to previous level. Because of sample sensitivity to stress level, the tests were carried out at seven stress levels. For this reason the incremental ratio decreased from 2 to 1.414 from tertiary step (i.e. doubling in two load steps). During each load stage the drainage valve was closed at the beginning, in order to measure an undrained compression. Such a compression would be negligible for a saturated specimen, but for

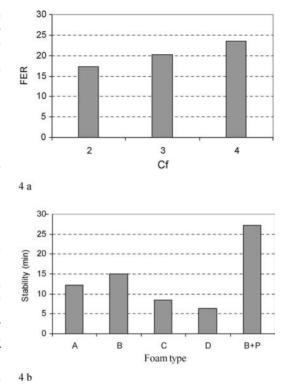


Figure 4. a: Effect of Cf on FER for foam D b: Comparing of foam stability.

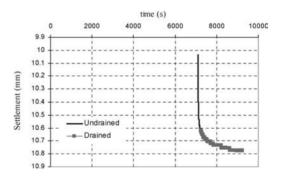


Figure 5. Drained and undrained settlement at end stage (@450 kPa).

the sand/foam mixtures there is a significant compression of the air bubbles in the foam. Then the drainage valve was opened and fluid allowed to drain from sample. A typical plot of displacement against time which shows both the undrained and drained phases is given in figure 5.

Figures 6, 7 illustrate the variation of void ratio against vertical stress in semi logarithmic scale for

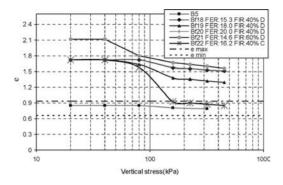


Figure 6. Void ratio against load logarithm for sand B.

sands B, C. Only the last points for each load increment are plotted, i.e. the void ratio achieved at the end of the drained stage. The soil type A was coarser than B and C and in order to achieve a homogeneous medium for this soil, other conditioning agent such as polymer and bentonite was needed. Concentration (Cf) of foam agent for all samples is 3%. Index f at curves indicates foamed sands. The lowermost curve is for unconditioned sand, initially prepared in a loose state.

At low stress level the void ratio lies just below the maximum void ratio for the sand. As the stress increases, the void ratio reduces by a small increment, but the sand remains in a loose to medium density. (Demonstrating the static stress is not sufficient for sand compaction). The upper curves are for different tests on foam/sand mixtures. The differences between the initial void ratios at low stress levels are entirely due to the amounts of added foam. At last stage the void ratio of the sand/foam mixture is still well above the void ratio at 0% relative density. It was unsurprising that sand/foam mixtures could be made at high void ratios, but it was quite unexpected that such high void ratios could be sustained at a remarkably high stress level. Note that the sand/foam mixture has a truly composite action rather than the sum of the component parts. Sand would have been compacted to a much lower density, and the foam by itself would have been crushed at such a stress but the sand/foam mixture is stable in a remarkably loose state. This may have fundamental implications for tunneling operations.

According to performed tests, the foam/sand mixtures void ratio are greater than maximum void ratio of dry sand and this characteristics of foam/sand mixtures is independent with respect to the foam types. With increasing of FIR the void ratio increases intensively but increasing of FER would have negligible effects on void ratio. In figures 6, 7 two curves show different behavior with respect to other curves. These two curves are Bf22 and Cf10 tests that are prepared by foam C. The other samples were prepared with foam D.

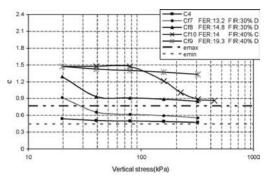


Figure 7. Void ratio against load logarithm for sand C.

Void ratio in foamed sands has been composed by two components: air and liquid void ratio, majority of void ratio in foam/sand mixture consists air void ratio. If the foam bubbles are stable, the compressibility of bubbles could lead to high compressibility of mixture. This amount of compressibility via water drainage will be not possible and also it need more time to drain. The role of consolidation settlement is much lesser than immediate settlement (Fig. 5).

According to figures 6, 7, the samples that are prepared by foam D show lower compressibility and lesser sensitivity with respect to samples that are prepared by foam C.

This phenomenon presumably is related to low stability of foam D against loading. But different drainage times for two types of agents are not considerable. To investigate this phenomenon, the volume change of this mixture was verified by each volume change component (air and water). In figure 8, the void ratio changes for soil and its component, air and liquid, have been drawn with respect to pressure for two sand (B, C) which mixed by foam type C and D. The initial and final values are used for this plot. General tendency of void ratio changes is distinct. It is clear that the water void ratio variations for samples which prepared with foam D (Figs 8a, 8c) are grater than air void ratio variations. The increasing of air void ratio has being seen in some cases (Bf20) because of excess sample drying in drainage process. Although in Bf22 and Cf10 tests (Figs 8b, 8d) air void ratio decrease more than water void ratio. It means that the amount of drained water is lesser than previous cases, and the majority of volume changes were taken place via compressibility of foam bubbles.

The air void ratio changes are compared in figures 9, 10. The air void ratio for samples with foam D gives either a negligible change or a small increase (except Cf7). It can be concluded that the foam D increase the water content and consequently the void ratio of mixture preserving the homogeneity. But the compressibility's is lesser because of low stability against loading and during the test process foam bubbles wash

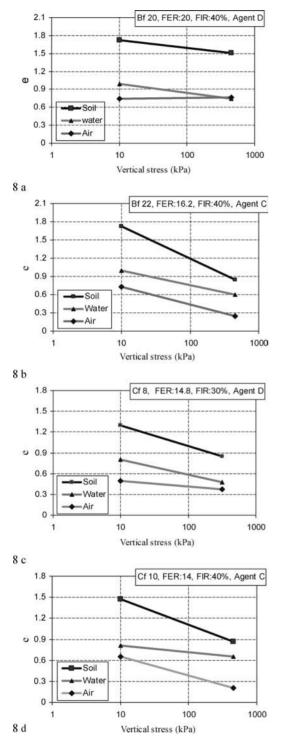


Figure 8. Void ratios variation versus vertical stress for sand B,C with two different foam agents.

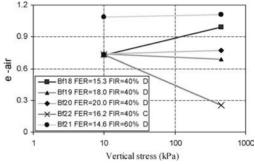


Figure 9. Air void ratio versus vertical stress for sand B.

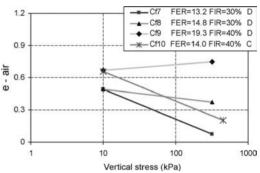


Figure 10. Air void ratio versus vertical stress for sand C.

out via water drainage in second stages of tests. Foam C also cause the increasing of water content in the mixture, moreover the sample is high compressible consequently shows a well sensitivity with respect to loading (Figs 6, 7). This difference between two types of foam agents are related to their chemical compositions. Note that the similar shear strength was obtained for samples prepared by two different agents (Sabetamal 2006). But depends on their compressibility different settlements were obtained. Thus, the foam D must be improved for that's low stability time and low compressibility in order to use in TBM machine in sandy soil.

#### 6 CONCLUSION

- Index tests on various foam agents show that the main differences between these agents are on stability or drainage time. The polymer increase the stability time significantly.
- 2. One of the foam/sand mixtures characteristics is high compressibility. Existence of foam bubbles in the soil skeleton increase void ratios, this feature for all foam agents is common but the behavior of sample during loading is affected by foam type. Thus, use of same foam agent for different condition is not suitable.

- 3. Large part of void ratio in foam/sand mixture consists of air void ratio. If the foam bubbles are stable against vertical stress and during the test process foam bubbles do not wash out via water drainage, the compressibility of bubbles could lead to high compressibility of mixture.
- 4. Some foam agents such as foam D increase the void ratio at first but because of their unfitted chemical compositions they can not sustained against loading.
- 5. With increasing of soil particle size at uniform gradation, use of foam alone as a conditioning agent is not sufficient as the foam bubbles easily escaped from the soil voids and assembled on soil mass. Consequently the composite action of soil/foam fails. So in coarser soils other conditioning agent such as bentonite as a filler of voids must also be used.

#### REFERENCES

- Defence Standard 42–40. 2002. Foam Liquids, Fire Extinguishing (Concentrates, Foam, Fire Extinguishing). Ministry of Defence. Issue 2. USA.
- EFNARC, 2005. Specification and guidelines for the use of specialist products for soft ground tunneling.
- Head, K.H. 1994. Manual of soil laboratory testing: shear strength, Compressibility and Permeability. 2nd edition, Vol.2, Pentech press: London.
- Milligan, G.W.E. 2000. Soil conditioning and lubrication in tunneling, pipe jacking and micro tunneling. A state of art review. web site: http/www.civils.eng.ox.ac.uk/research/ pipejacking.htm
- Psomas, S. 2001. Properties of foam/sand mixtures for tunneling applications: M.Sc. thesis, St. Hugh's college, Oxford University. London: England.
- Sabetamal, H. 2006. Study of soil conditioning on granular soils for EPB Tunneling: M.Sc. thesis, Faculty of Civl engineering, University of Tabriz. Tabriz: Iran.

## Study on earth pressure acting upon shield tunnel lining in clayey and sandy grounds based on field monitoring

T. Hashimoto, G.L. Ye, J. Nagaya & T. Konda

Geo-Research Institute, Osaka, Japan

#### X.F. Ma

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: In this study, firstly, a series of field monitoring data on earth pressure in soft clay, hard clay and sand ground are analyzed respectively. In the soft clay ground, the earth pressure fluctuated with backfill grouting pressure at first, it settled down toward a steady value between (static vertical pressure  $Pv0 \pm$  cohesion C) and distributed uniformly over the ring finally. In the hard clay ground, the earth pressure was more greatly influenced by the backfill grouting, especially for the lateral earth pressure in hard clay with  $2C/p_{v0} \approx 0.5$ . In the sand ground, although the earth pressure was also influenced by the backfill grouting, the distribution was relatively uniform than in hard clay ground since the hydraulic pressure accounts for a large portion in earth pressure. The insights obtained from this study can contribute to an improvement of load considerations in shield lining design.

#### 1 INTRODUCTION

In the design code of shield tunnel in Japan (JSCE, 1996), the earth pressure acting upon the segment lining is calculated by the overburden pressure or Terzaghi's loosening earth pressure according to the stratum condition and the overburden height only. However, it is known that the earth pressure is also influenced by the construction conditions (e.g. back-fill grouting, position adjusting of shield machine), and the interaction between the ground and man-made structures (e.g. tunnel lining, pile foundation). In most cases, these factors work together and undistinguishable. Therefore, nowadays the mechanical behavior of the earth pressure upon shield tunnel lining has not been clearly clarified yet.

Some researches have been done on this problem in the last decades. After the Terzaghi's theory on loosening earth pressure, Murayama (1968) studied the vertical earth pressure in sandy layers by trapdoor tests. According to the test finding that the sliding surface is similar to logarithm spirals he proposed a formula to calculate the vertical earth pressure. However, the up-to-data shield technology equipped with precise pressure control system at cutter face and simultaneous backfill grouting system makes it possible to build a tunnel without loosening the surrounding ground. Therefore, the actual earth pressure cannot be correctly predicted by conventional methods (Ohta et al., 1997, Hashimoto et al., 1997). Moreover, Suzuki et al. (1996) reported that the maximum loads occurred during backfill grouting in a shield tunnel with large overburden.

In this study, firstly, three cases of field monitoring on earth pressure in soft clay, hard clay and sand grounds are analyzed carefully. And focus is set on the long-term behavior. The results – distributions of earth pressure, axial force and bending moment of lining – are compared with the calculated value by conventional design method respectively. And more than twenty measurement data are summarized and organized according to the strength of the ground to find out some empirical rules of earth pressure. The insights obtained from this study can contribute to an improvement of load consideration in shield lining design.

#### 2 BRIEF DESCRIPTION OF MONITORING SITES

In order to clarify the characteristics of earth pressures acting upon the linings in clayey and sandy grounds, three typical monitoring jobs are chosen from three

Table 1. Descriptions of shields and geology conditions.

Site name	Kadoma (Soft clay)	Osakajo-A (Hard clay)	Osakajo-B (Sand)
Shield type	Mud-soil pressure balanced	Earth pressure balanced	Earth pressure balanced
Segment type Shield diameter	Ductile \$\$300 mm	RC \$5300 mm	RC \$5300 mm
Backfill grouting type	Simultaneous	Simultaneous	Simultaneous
Overburden height	14.09 m	28.2 m	16.8 m
Around soil type	Alluvial clay	Diluvium clay	Sand
SPT-N value Unconfined compressive strength	3 ~ 5 170 ~ 200 kPa	8 ~ 9 540 kPa	>50 -

types of ground respectively, namely, soft clay, hard clay and sand grounds. Some basic information on the shields and geological conditions are listed in Table 1, and the soil profiles of monitoring sites are shown in Figure 1. Moreover, two comparing monitoring sections were setup at Osakajo-A and Osakjo-B sites to check the influence of backfill grouting. In order to obtain a reliable earth pressure data, a pad type earth pressure cell (Hashimoto et al., 1993), as shown in Figure 2, was adopted in all monitoring jobs. The water pressures were recorded by piezometers from the grouting holes.

### 3 FIELD MONITORINGS OF EARTH PRESSURES ACTING UPON LINING

The time histories of observed earth pressures around the lining in 3 types of ground are shown in Figure  $3 \sim 5$ . The left part of the figures represents the shortterm data, and the right part is the long-term one. The earth pressure and water pressure are noted as EP and WP hereinafter. Without mentions, the earth pressure means total earth pressure.

### 3.1 In soft clay ground

Figure 3 shows the changes of observed EPs and WPs at crown, left spring-line, right spring-line and invert of the lining in soft clay ground. After the tail passing, the EPs fluctuated greatly by backfill grouting pressure, and the fluctuation almost disappeared at 7th rings after tail passing, which indicated the extent of influence from grouting holes is about 7 rings in such a ground condition. After that, the EPs and WPs decreased gradually in the first  $1 \sim 2$  months, and then

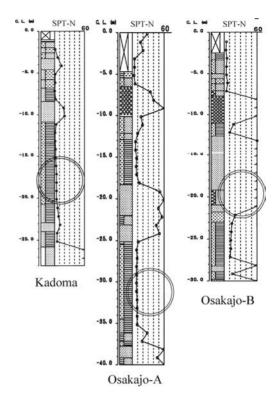


Figure 1. Soil profiles at monitoring sites.

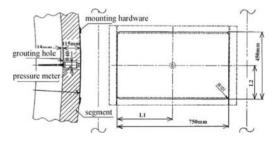


Figure 2. Pad type earth pressure cell.

the WPs remained unchanged while the EPs turned to increased little by little, finally reached a constant status (except R-spring). These final constant values lie between (static pressure  $P_0 \pm$  cohesion C). Above phenomena can be explained as following. The excess porewater pressure adjacent to lining, which was generated during shield advancing and backfill grouting, dissipated in the first 1~2 months resulting dropdowns of EPs and WPs, and then the excess porewater pressure in the farer surrounding soil dissipated along with the decoration of soil skeleton, resulted in a very slow buildup of effective EPs upon lining.

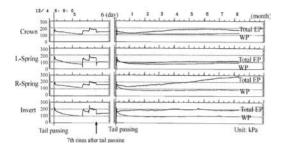


Figure 3. Observed earth pressure acting upon lining in soft clay ground (Kadoma Shield).

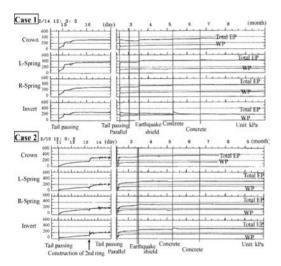


Figure 4. Observed earth pressure acting upon lining in hard clay ground (Osakajo-A Shield).

### 3.2 In hard clay ground

Figure 4 shows the changes of observed EPs and WPs at two compared cases in hard clay ground. In Case 1, a regular backfill grouting was implemented. In Case 2, in order to eliminate the influences (the squeezing force to the ground) of backfill grouting, a special grouting with smaller injection pressure and injection ratio was used, as shown in Table 2. Since the longterm changes in both cases were influenced by the parallel shield passing, earthquake and concrete casting of invert and arcade, and showed increasing trends. the attention was paid to the changes before the 2nd shield passing. In Case 1, after all the EPs primarily climbing up to  $300 \sim 400$  kPa under the influence of backfill grouting, the EPs at crown and invert turned to decrease. On the other hand, the EPs in Case 2 was smaller (about 200 kPa) at first and then turned to be increased by backfill grouting of next several

Table 2. Backfill grouting of Osaka-A shield

	Case 1	Case 2
Injection pressure	150 kPa	50 kPa
Injection ratio	139%	100%
Grouting material	Standard strength*	Low strength

\* Two components grouting material with a gel time less than 10 seconds.

Table 3. Backfill grouting of Osaka-B shield

	Case 1	Case 2
Injection pressure	300 kPa	170 kPa
Injection ratio	135%	100%
Grouting material	Standard strength	Low strength

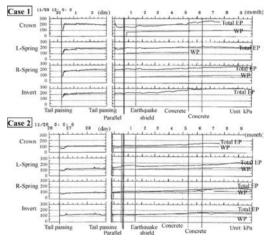


Figure 5. Observed earth pressure acting upon lining in sandy ground (Osakajo-B Shield).

rings. Comparisons between two cases show that the final differences were very small. The final EPs at the crown in both cases were  $370 \sim 380$  kPa. The figure shows that the main part of each EP at different location was determined at the first 10 rings, which indicates that the EP largely depends on the backfill grouting in the hard clay ground. And it also can be seen that the settling down of EP only needed several days, much quicker than in soft clay.

### 3.3 In sand ground

The same as that in Osaka-A shield, here also two cases of comparing measurements were carried out, as shown in Table 3. Figure 5 shows the changes

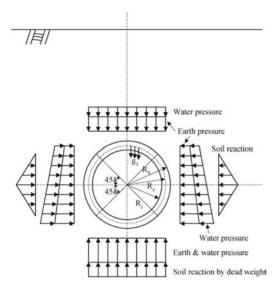


Figure 6. Conventional design method in Japan (effective stress method).

of observed EPs and WPs in both cases. In Case 1, right after the backfill grouting of current ring the EPs was  $100 \sim 150$  kPa, then rose up to 200 kPa after the grouting of next  $2 \sim 3$  rings, and further increased to  $200 \sim 250 \,\mathrm{kPa}$  4 months later. On the other hand, in Case 2, right after the backfill grouting of current ring the EPs were only  $80 \sim 110$  kPa, and did not increased significantly after next several rings. The final values were  $120 \sim 160$  kPa even 4 months later. The figure also shows that although the long-term EP in both cases influenced by the parallel shield passing and earthquake, and therefore showed an increasing trend, however, the magnitudes did not change a lot from those after the first several rings' backfill grouting. And comparing to the EPs in clayey grounds, the settling down of EPs in the sandy ground was much faster.

### 4 COMPARISON BETWEEN DESIGN AND OBSERVATION (HASHIMOTO ET AL. 2002)

In Japan, the conventional model and the bedded frame model are main design method for shield lining. In most common situation, such as the original designs of the three tunnels in previous session, the engineer prefers the conventional model for the sake of convenience. We also used conventional model for comparison in this study. Figure 6 is a conceptual figure of conventional model, which considers the EP and WP as well as soil reaction as lining loads.

The comparisons between the observed EP and member forces as well as the calculated values are

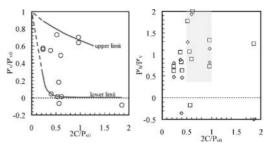


Figure 7.  $p'_{\nu}/p'_{\nu 0}$  vs.  $2C/p_{\nu 0}$  and  $p'_{h}/p'_{\nu}$  vs.  $2C/p_{\nu 0}$  (clayey ground).

shown in APPENDIX. The EPs over the ring in all ground conditions were relative uniform over the ring by comparing with the design values. Consequently the axial force and the bending moment were smaller than the design ones. As mentioned in previous session, the EP was greatly influenced by the backfill ground in relative hard ground, and it is not difficult to image that the backfill grouting can flow into the tail void quickly and cover the whole ring with a uniform pressure. Therefore, the uniform distribution of EPs is directly related with the backfill grouting.

### 5 RELATION BETWEEN EARTH PRESSURE AND GROUND CONDITION AND ITS INTERPRETATION

Besides aforementioned three measurement data, we also collected more than twenty data (long-term data) from other monitoring jobs in Japan, including the shield tunnels for subway, sewer, power-supply and communication, and summarized them according to the strength of the ground. The authors would like to emphasize that based on these limited data it is still difficult to execute a quantitative analysis. Therefore in this session a qualitative analysis on the behavior of EPs in various ground conditions is given out.

### 5.1 In clayey ground

The relation between  $p'_{\nu}/p'_{\nu 0}$  and the normalized strength  $2C/p_{\nu 0}$  is plotted in Figure 7(a), where  $p'_{\nu}$  is the effective vertical EP at the crown,  $p'_{\nu 0}$  is the effective vertical EP at the crown,  $p'_{\nu 0}$  is the effective overburden pressure,  $p_{\nu 0}$  is the total overburden pressure, and *C* is the cohesion. It is well known that in natural clayey ground, there has such an empirical relation as  $q_u (=2C) = 0.3 \sim 0.4 p_{\nu 0}$ . When the ground is unconsolidated with a very small  $2C/p_{\nu 0}$  (<0.3), the tail void is easy to collapse immediately after tail passing, and cannot be easily filled by the backfill grouting in time. Therefore, the surrounding soils will yield and most of the overburden weight will act upon the tunnel  $(p'_{\nu}/p'_{\nu 0} \rightarrow 1)$ . And when the ground is consolidated or

a little over-consolidated  $(2C/p_{v0} \approx 0.5)$ , the grouting material can fill the tail void in time before collapse. On the other hand, when the stiffness of the grouting material and surrounding ground are close, their interaction becomes active. Consequently,  $p'_{\nu}/p'_{\nu0}$  depends on the grouting pressure and varies largely according to the construction conditions. Furthermore, when the ground is stiff enough  $(2C/p_{\nu0})$  becomes large), the shrinkage of grouting material during hardening will be larger than the deformation of surrounding ground. The backfill grouting has minor effect on the EP, resulting in a small  $p'_{\nu}/p'_{\nu0}$ . The data in Figure 7(a) indicate those kinds of phenomenon. For a better understanding, two boundary lines are drawn in the figure, and for the areas lacking of data broken lines are drawn.

The relation between  $p'_h/p'_v$  and the normalized strength  $2C/p_{\nu0}$  is shown in Figure 7(b), where  $p'_h$  is the effective horizontal EP at the spring line, and  $p'_v$  is the effective vertical EP at the crown. It is found that when  $2C/p_{\nu0}$  is small (<0.3),  $p'_h/p'_v = 0.45 \sim 0.8$ . Considering the coefficient of lateral earth pressure at rest ( $K_0$ ) of clayey ground is about 0.5 and the location of spring line is deeper than the crown, such values of  $p'_h/p'_v$  are rational. When  $2C/p_{\nu0} \approx 0.5$ , the value of  $p'_h/p'_v$  scatters in a wide range, implying that the circumferential distribution of earth pressure in such ground also depends on the backfill grouting and other construction conditions.

#### 5.2 In sandy ground

The relation between  $p'_{\nu}/p'_{\nu 0}$  and the equivalent SPT (Standard Penetration Test) N value - which is regarded as a strength index - is plotted in Figure 8(a). By applying the Terzaghi's loosening earth pressure to an assumptive shield tunnel, we can get a curve of loosening earth pressure against SPT-N value, as the dot line in Figure 8(a). However, due to the influences in construction process, the EP acting on the lining will vary above or below the theoretical line. When the ground condition is relatively good and the loosening earth pressure is small, a proper backfill grouting can reduce the EP dramatically, otherwise a large grouting pressure may remain on the lining. The former phenomenon often occurs in the sand ground mingled with cohesive silt/clay. The data with SPT-N between  $40 \sim 90$  in Figure 8(a) are just some vivid instances. Furthermore, when the ground becomes very dense sand or gravel, similar to the clayey ground with large  $2C/p_{\nu 0}$ , the shrinkage of grouting material may larger than the deformation of surrounding ground, and the EP consequently becomes very small, such as the date with SPT-N > 100 in the figure.

Although there is no enough observed data in the SPT-N < 40 ground, we can make an estimation from the knowledge of soil mechanics. When the soil is very loose, the dependency on construction shall be weakened (the same as the soft clay ground), and then the

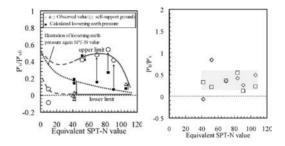


Figure 8.  $p'_{\nu}/p'_{\nu 0}$  vs. SPT-N and  $p'_{h}/p'_{\nu}$  vs. SPT-N (sandy ground).

distribution of EP will get close to the theoretical loosening earth pressure, as the broken line in Figure 8(a). For a better understanding, the authors also draw two boundary lines in the figure.

The relation between  $p'_h/p'_v$  and the equivalent SPT-N value is shown in Figure 8(b). It is found that  $p'_h/p'_v$  scatters between  $0 \sim 0.6$ , indicating that the circumferential distribution of earth pressure in sandy ground also depends on the backfill grouting and other construction conditions.

The shapes of the boundary lines in Figure 7(a) and 8(a) have the same characteristics: two ends are narrow while the middle is wide. This kind of shape clearly tells us that when calculating the EP in the ground with medium stiffness, more attention should be paid to the backfill grouting and other construction conditions than those in soft or very hard ground. It should be pointed out that, however, the backfill grouting must be considered when dealing with the settlement in soft clay ground (Hashimoto et al. 1999).

### 6 CONCLUSIONS

In order to clarify the mechanical behavior of earth pressure acting upon shield tunnel lining in various ground conditions, saying soft clay, hard clay and sand grounds, a series of field monitoring has been carried out. By analyzing the observations carefully, following conclusions were obtained.

- The earth pressure is influenced by the injection of backfill grouting at 7 ~ 8 rings away in the case of simultaneous backfill grouting.
- 2. In the soft clay ground, the earth pressure fluctuates with the backfill grouting in the early phase. However, it settles down to a steady value between (static pressure  $P_{\nu 0} \pm$  cohesion *C*) finally, regardless of the backfill grouting pressure.
- 3. In the hard clay and sand ground, the initial earth pressure that builds up gradually by backfill grouting remains in the long-term earth pressure. In other words, the earth pressure depends on the backfill grouting to such an extent that sometimes the

earth pressure will be larger than the prediction by Terzaghi's loosening earth pressure.

- 4. The settling down of earth pressure in soft clay ground needs several months, while those in hard clay and sand grounds only need several days or even several hours.
- 5. The distributions of earth pressure are more uniform than predictions by conventional design method in all ground conditions. In other word, the bending moment is apt to be overestimated, especially in the sandy ground where hydrostatic pressure plays a dominant role in earth pressure. Therefore, it is suggested that when designing the lining, the influence of backfill grouting should be taken into consideration.
- 6. By analyzing more than twenty monitoring data, it is found that in the clayey ground, if  $2C/q_u < 0.3$ , a large portion of the overburden will act upon the lining. If  $2C/p_{\nu0} \approx 0.5$ , the magnitude and distribution of earth pressure depend largely on the backfill grouting. And in the sandy ground, if the equivalent SPT-N value lies between  $40 \sim 80$ , the magnitude and distribution of earth pressure also depend largely on the backfill grouting.

### REFERENCES

- Hashimoto, T., Nagaya, J. & Konda, T. 1999. Prediction of ground deformation due to shield excavation in clayey soils. *Soils and Foundations*, Vol.39, No.3, pp.53–61.
- Hashimoto, T., Nagaya, J., Konda, T. & Tamura, T. 2002. Observation of lining pressure due to shield tunneling. *Geotechnical aspects of underground construction in soft* ground, IS-Toulouse, Kanster et al. (eds), Specifique, pp.119–124.
- Hashimoto, T., Yabe, K., Yamane, S. & Ito, H. 1993. Development of Pad type earth pressure cell for shield segment. *Proc. of 28th annul report of JGS*, pp.2055–2058. (in Japanese)
- Japanese Society of Civil Engineers (JSCE). 1996. Japanese Standard for Shield Tunneling. 3rd edition.
- Murayama, S. 1968. Earth pressure on vertically yielding section in sand layer. DPRI Annuals, No.11(B), Kyoto University, pp.549–565.

- Ohta, H., Shiotani, T., Sugihara, K., Hashimoto, T. & Nagaya, J. 1997. Consideration of Design Earth Pressure for Shield Tunnel Based on Measurement. *Proc. of Tunnel Engineering, JSCE*. Vol.9. pp.37–42. (in Japanese)
- Suzuki, M., Kamada, T., Nakagawa, H., Hashimoto, T. & Satsukawa, Y. 1996. Measurement of earth and water pressure acting on the great depth shield tunnel segments. *Geotechnical aspects of underground construction in soft ground*, *IS-London*, Mair & Taylor (eds), Balkema, pp. 613–619.

### APPENDIX

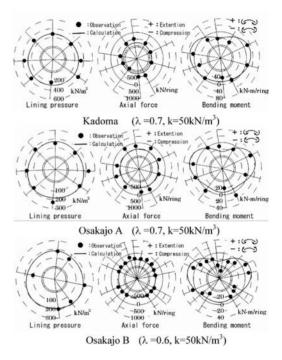


Figure A1. Comparison of observed and designed values of earth pressure and member forces (Hashimoto et al, 2002).

### The double-o-tube shield tunnel in Shanghai soil

### C. He

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, School of Civil Engineering, Tongji University, Shanghai, P.R. China

### L. Teng

Shanghai Tunnel Engineering CO. LTD, Shanghai, P.R. China

### J.Y. Yan

Shanghai Metro Operation CO. LTD, Shanghai, P.R. China

ABSTRACT: The high building density of shanghai and the wish to construct bigger city transport system lead to an obvious conflict. The double-o-tube shield tunneling was introduced to save underground space. In 2002, Shanghai has built its first double-o-tube shield tunnel in No.8 line. The stress and displacement distribution around the double-o-tube shield tunneling was investigated from an in-situ test.

### 1 GENERAL INSTRUCTIONS

Shanghai, as one of the biggest city in China, the density of building is quite high. But with the big development of urban city, more and more building and transportation constructions are needed. So the questions that use limited space of shanghai to satisfy all kinds of needs come out these years. In subway area, the double-o-tube shield tunneling was introduced to save underground space. In 2002, Shanghai has built its first double-o-tube shield tunnel in No.8 line. Figure 1 shows the double circular shield which



Figure 1. Break through of a double circular shield.

just finished one part of tunnel and broke through the working shaft.

The origin of the double circular shield tunnel may be traced to 1981, when a basic patent was applied for in Japan. The patent was registered in 1987, and a horizontal double circular shield tunnel field trial was performed in the same year. In 1988, a vertical double circular shield tunnel trial was conducted in the field. In the construction of the national road No. 54 tunnel in Hiromiosa, a double circular shield, which was designed and manufactured by Ishikawajima-Harima Heavy Industries Co. Ltd, was used to build the first double circular shield tunnel in the world with length of 853.8 m (Moriya, Y., 2000).

The shield used in Shanghai subway was made in Japan by Ishikawajima-Harima Heavy Industries Co. Ltd and assembled in Shanghai by Shanghai TBM Company. The width of shield is 11.12 m; the diameter of a circular is 6.52 m. The diameter of mostly single circular shields used in Shanghai is 6.4 m. The length of shield is 12.76 m. The distance between the lining and shield is 11 cm. Figure 2 shows the main dimension of double-o-tube shield tunnel shield machine.

The double-o-tube shield tunneling Method is applied for an earth pressure balanced shield machine with interlocking spoke-equipped multiple cutters that are positioned in the same plane. Adjacent cutters rotate in the opposite directions to avoid touching or smashing one another and are thus controlled synchronously. The double circular shield machine is

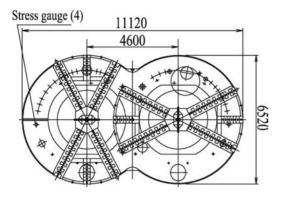


Figure 2. The dimension of the double circular shield.

equipped with cantilever-arm-type erector to erect joint and panel segments, so it provides wide working space. The double-o-tube shield tunnel is composed of 11 prefabricate concrete segments (including the columns in the middle of the tunnel) per section.

### 2 COMPARISON BETWEEN SINGLE AND DOUBLE-O-TUBE SHIELD TUNNEL

Compared with the single circular shield, the most important advantage of the double-o-tube shield tunnel is saving space. Most of Shanghai metro was built under the road, and the space is limited when the road is relatively narrow. Apparently, the double circular shield tunnel may pass narrower underground corridors, and the impact on nearby structures is minimized (Moriya, Y., 2000 and R.C. Sterling, 1992). Meanwhile, the double circular shield tunnel has an optimized cross-section with a minimized section area, enabling the most efficient use of underground space. Moreover, the cross-passage between two circular shield tunnels is unnecessary for a double circular shield, and construction risk is therefore eliminated. Figure 3 shows you the concept that double circular shield tunnel can save space comparing with two single tunnels.

On the other hand, the double-o-tube shield tunnel has its own disadvantages. The first one is cost. China can produce the single circular shield now, but the double circular shield is made in Japan and assembled in China. Moreover, the construction cost of the double-o-tube shield tunneling is more expensive than two single circular tunnels. Secondly, the double circular shield is not easy to be controlled in small curvature route. Sometimes it needs extra load to balance the shield. Thirdly, the speed of double-o-tube shield tunneling is slower than two single circular tunneling which shared the same shield. The main reason is that the linings fix spends more time.

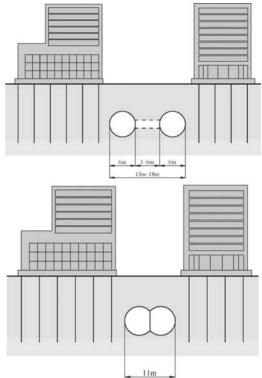


Figure 3. Comparison between single shield tunnel and double circular shield tunnel.

During the tunneling process, there are three important construction parameters: face pressure, speed and grout volume. The face pressure which the double-o-tube shield tunneling used is little higher than which used in the single circular tunneling. Higher face pressure can reduce the settlement while tunnel is finished, however it would induce the higher heave before the opening face.

The speed of the double-o-tube shield tunneling is much lower than the single circular tunneling. The main reason is the lining fix cost more time in the double-o-tube shield tunneling. And maintaining a certain low speed is efficient way for the settlement control. According to the construction experience of Shanghai, lower speed correspond smaller ground settlement especially for shallow cover depth case.

Although grout pressure is more frequently mentioned in theoretical and numerical calculation of the tunneling as the mechanical parameter, in the practical tunneling work of Shanghai, grout volume used to present the effect of grout instead of grout pressure. The main reason is that the grout pressure is unstable and hard to control for real work. The grout volume which the double-o-tube shield tunneling need is much more than the volume which single circular tunneling

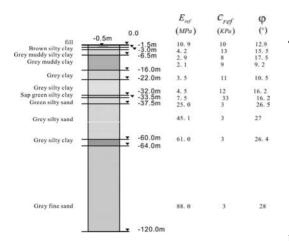


Figure 4. Typical profile of ground condition in Shanghai.

need. But the grout volume is not a fixed value. It is depend on different soil types, cover depth, grouting point, and grout material. Even while all the factors are same, the grout volume would still be different for different company which in charge of tunneling. It is more based on the experience but not theoretical result.

### **3 GEOLOGY CONDITION**

Since Shanghai is located near the confluent of Huangpu River and Chang jiang (Yangzi River), the soil has been progressively constituted by sedimentation and ended in a superposition of layers. The Shanghai soil can be assumed as homogeneous in most Shanghai land except partial place where some soil layers are missed. In Shanghai area, the mucky soil stratum is down to about 30 m deep from ground surface. It is basically saturated fluidplastic or soft-plastic clay with low shear strength  $(0.005 \sim 0.01 \text{ Mpa})$ , high water content (above 40%), high compressibility  $(0.5 \sim 1.0 \text{ MPa}-1)$ , sensitivity varying in  $4 \sim 5$ , and evident rheological behavior (Wang Zhen Xin & Bai Yun, 2004). The water table is ranged from  $0.3 \text{ m} \sim 1 \text{ m}$ . In this very soft ground, deep excavations for constructing underground metro station and high building basement encounter many difficulties in environment protection. The general ground condition of Shanghai Metro Line is shown in Figure 4.

There are mainly two soil types which double-o-tube shield tunnel cross through. One is the Grey muddy clay and Grey clay which are the typical soft clay layers. Another situation is in the place where the Grey muddy clay is missed and Grey clay is very thin. It means that layer Grey muddy silty clay almost connects with layer Grey silty clay. So the tunnel mainly crosses the silty clay.

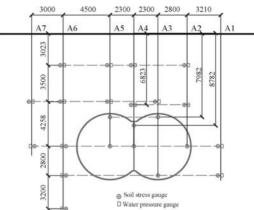


Figure 5. Gauges layout in cross section A.

Along the tunneling route, most of the cover depth varies from 6 m to 16 m. There are two typical cases according to the different cover depth. One is 8 m cover depth representing shallow cover. Another one is 16 m for deep cover. Surface settlement due to tunneling is significant different between these two cases.

### 4 IN-SITU TEST

## 4.1 Stress distribution in the soil due to the double-o-tube shield tunneling

Geotechnical engineers discussed on a theoretical base about the soil stress distribution due to double-o-tube shield tunneling at the beginning of double circular method applied in Shanghai. They try to compare the soil stress distribution due to double-o-tube shield tunneling with the results from single circular tunneling. Two theories came out. One opinion is that the soil stress distribution from the double-o-tube shield tunneling could be assumed as the combination of soil stress from two separate single circularities which their location are coincident with double-o-tube shield tunnel. Other engineers prefer to believe that it should be modeled from a bigger equivalent single circular which share same center point with the double-o-tube shield tunnel.

An in-situ test has been done by Shanghai Tunnel Engineering Co. Ltd to investigate the stress and displacement distribution around the double-o-tube shield tunneling. There are 4 sections in which the displacement inspector and stress gauge were placed along the line of tunneling.

Cross section A was designed for observing the soil stress increment and water stress increment. Figure 5 shows the layout of gauges on cross section A.

Figure 6 shows the vertical soil stress increment in the plane which in 1.5 m ahead of the opening face.

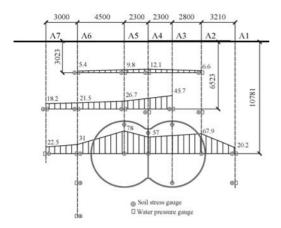


Figure 6. The vertical stress increment in 1.5 m ahead of the opening face.

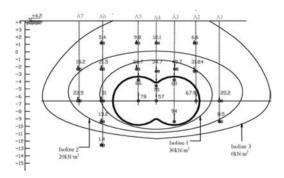


Figure 7. The vertical stress increment distribution in 1.5 m ahead of the opening face.

There are three soil stress increment distribution line in figure 7 with different depth. It is drawn from 4 inspectors at 3.02 m deep, 4 at 6.52 m and 6 at 10.78 m. It is different tendency for these distribution lines. A triangle shape which the largest soil stress increment occurred on the middle vertical line of double-o-tube shield tunnel is got for the top distribution line which near to ground surface. For the bottom distribution line which lies on the same depth level with tunnel horizontal central line, the stress increment in the middle point of tunnel is smaller than the one in the center point of two circles.

Figure 7 shows the soil stress distribution due to the double-o-tube shield tunneling of one section. The black line in figure represents the isoline soil stress around the tunnel at the time while the opening face is 1.5 m behind.

From the result, the soil stress distribution induced by tunneling is more closed to an ellipse than the combination of two single circularities or a big circularity.

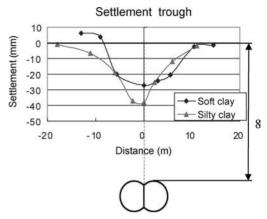


Figure 8. The settlement troughs with cover depth of 8 m.

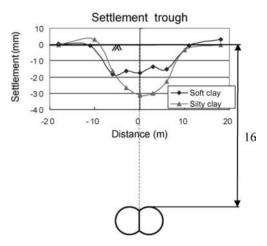


Figure 9. The settlement troughs with cover depth of 16 m.

And the effective influence width from double-o-tube shield tunneling is mostly 2.5 times to width of tunnel.

## 4.2 Surface settlement due to the double-o-tube shield tunneling

The requirement of surface displacement due to tunneling is  $+1 \sim -3$  cm in Shanghai. In the beginning of the double-o-tube shield tunneling, the settlement was surprisingly high. Many people suspected this tunneling method at the beginning of doubleo-tube shield tunnel applying, however after some projects; the engineers acquire the experience of the double-o-tube shield tunneling, the settlement has been under control in most conditions.

Figure 8 and Figure 9 give the surface settlement trough due to tunneling in two cases. And each figure shows two soil types which tunnel cross through.

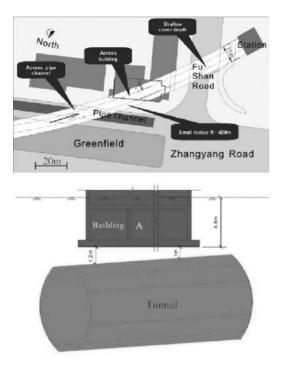


Figure 10. The plan view and transverse section of the double-o-tube shield tunnel crossing building A.

It is hard to obtain equal volume loss for each circle in the advance of tunneling, especially in the curve. So from these two figures, it is quite normal phenomena that the settlement trough is asymmetry to the center of shield.

Concerning the influence of soil type, the maximum settlement of silty clay is higher than the value of clay. Meanwhile the settlement trough of clay is wider. The main reason is that the influence area in the soft clay is lager than the one in silty clay with the same construction conditions, not only in the cross section but also in the longitudinal direction.

Comparing the data in different cover depth, the maximum settlement of 16 m cover depth is less than the one of 8 m cover depth. In the same time, the settlement trough of 16 m is wider than the one of 8 m cover depth. This phenomenon is coinciding with the Peck formula.

### 5 DOUBLE-O-TUBE SHIELD TUNNEL CROSSING BUILDING

Here is an interesting project during the double-o-tube shield tunneling. There are three key construction phase of this double-o-tube shield tunnel line. The shield need across three key section- pipe channel, five-floor buildings and shallow cover depth

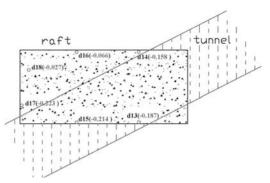


Figure 11. The measurement results of building settlement after tunnel crossing.

area. The current paper will represent the second key section which tunnel crossing a five stories building.

Figure 10 shows the plane view of three key construction phases and the cross section of second one. The distance of the building and the tunnel is very close; it ranged from 1.2 m to 1.0 m. It is great risk that the tunnel crosses a building with such close distance. Especially this building with raft foundation has already had 0.3% inclination. According to Chinese foundation code, the maximum inclination of building should be controlled under 0.4%. Meanwhile it has been mentioned before that there are not enough experiences of the double-o-tube shield tunneling comparing with the single circular tunneling.

Shanghai Tunnel Engineering Co. Ltd take series actions to control the ground settlement and reduce the influence of tunneling to building raft foundation. In the process of tunneling, the face pressure remains high level. The grout volume happened in the tail is larger than the value which used normally. The tunneling speed was maintained at very low in the whole process. Moreover, extra grout was used while the tail passed. With all these construction method, it results in that the settlement and the inclination of the building was controlled well.

There were 6 displacement inspectors installed in the foundation of the building. The final measurement data were given in Figure 11.

The blue number shows the location of displacement inspectors. The red number is the settlement of raft foundation respectively corresponding to each inspectors and the unit is centimeter. The measurement of settlement ranged from 0.3-2.2 mm. It means that the tunneling is successful. The building is almost intact and no change can be detected by eyes.

### 6 CONCLUSIONS

The double-o-tube shield tunneling has been applied in Shanghai successfully. The settlement can be controlled in a very low value. The construction is more based on the experience but not theoretical result. Furthering research need to be done to get a theoretical guarantee for future work.

### ACKNOWLEDGEMENTS

The work described in this paper was part of the project funded by Shanghai Tunnel Engineering Co. Ltd. The authors would like to acknowledge the collaboration throughout the project of Shanghai TBM Company. Particular thanks are due to Tang xuan and Wu huiming.

### REFERENCES

- Moriya, Y., 2000. Special shield tunneling methods in Japan. In: Proceedings of the International Conference on Tunnels and Underground Structures, Singapore, pp. 249–25.
- Shanghai Tunnel Engineering Co. Ltd., 2006. The report of double circular tunneling (1), Sept.
- Sterling, R.C. 1992. Developments in excavation technology, a comparison of Japan, the US and Europe, *Tunneling and* Underground Space Technology. 7 (3), pp. 221–235.
- Wang, Z.X. & Bai, Y. 2004. Urban soft ground tunneling in China— experiences from Shanghai and other cities, *Proceedings of World Tunnel Congress and 13th ITA*, Singapore, pp. 22–27.

### Frozen soil properties for cross passage construction in Shanghai Yangtze River Tunnel

### X.D. Hu

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

### A.R. Pi

Department of Civil Engineering, Anhui Institute of Architecture & Industry, Hefei, P.R. China

ABSTRACT: Eight cross passages between the two tubes of Shanghai Yangtze River Tunnel are constructed by Artificial Ground Freezing Method. The formations around the tunnel are characterized with saline soil, which raises such disadvantages as freezing-point depression, freezing phase lengthening, the growth rate decrease of frozen bodies and the strength loss of the frozen ground. In order for successful construction, the properties of the artificial frozen soils are made out by test, such as salinity, freezing point, uniaxial compressive strength, thermal conductivity, frost heave ratio, thaw consolidation ratio and other related parameters. Based on the test results some suggestions are given for freezing scheme design, the cross passages construction and freezing process monitoring.

### 1 INTRODUCTION

As in many other tunnel projects (Crippa et al. 2006, Hu et al. 2006 & Zhao et al. 2005), eight cross passages between the two tubes of Shanghai Yangtze River Tunnel (see Figure 1) are constructed by artificial ground freezing method. Shanghai Yangtze River Tunnel is situated at the estuary of the Yangtze River, where saltwater intrusion occurs frequently (Shen et al. 2003). It is supposed that the saline concentration of soil at the riverbed is higher than usual, thus the higher risk in constructing the cross passages by artificial ground freezing method. The properties of frozen saline soil have a remarkable change (Roman et al. 2004) which is disadvantageous to the construction. As a result, it is of great necessity to proceed a research of the properties of the saline soil such as the salinity, the freezing point of the saline soil and the frozen soil strength, etc. The research concentrated on the basic geotechnical parameters, the physical mechanic parameters and the thermodynamic parameters of the designed soil layers, including gray mucky clay ([4]), gray clay ([5]1), gray clayey silt ([5]2), gray silty clay ([5]3) and gray clayey silt ([7]1-1).

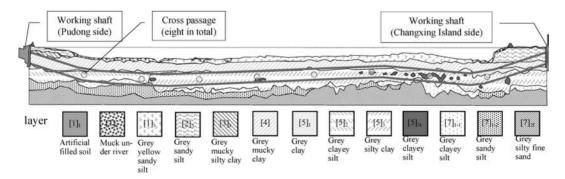


Figure 1. Layout of the cross passages of Shanghai Yangtze River Tunnel.

Soil layer (undisturbed)	Water Content w (%)	Density $\rho$ (g/cm <sup>3</sup> )	Permeability coefficient K (cm/s)	Void ratio $e_0$	Plastic limit w <sub>P</sub> (%)	Liquid limit w <sub>I</sub> (%)	Salinity (‰)	Chlorinity (‰)
[4]	44.02	1.69	$3.05 \times 10^{-7}$	1.40	40.78	23.72	18.370	10.169
[5] <sub>1</sub>	38.21	1.77	$2.15 \times 10^{-7}$	1.07	33.97	21.68	17.392	9.627
[5] <sub>2</sub>	29.96	1.80	$8.55 \times 10^{-5}$	0.94	30.42	19.92	13.228	7.322
[5] <sub>3</sub>	31.44	1.78	$1.99 \times 10^{-6}$	1.01	36.73	21.17	8.039	4.450
$[7]_{1-1}$	26.83	1.86	$7.90 \times 10^{-4}$	0.85	36.02	20.35	14.744	8.161

Table 1. Results of geotechnical tests.

### 2 TESTS

### 2.1 Requirement and standard

The tests are proceeded according to the standards of People's Republic of China as follows:

- Standard for soil test method (GB/T50123-1999)
- Coal Industrial Standard (MT/T593-1996)

### 2.2 Content

2.2.1 Basic geotechnical tests

The basic geotechnical tests include test of density, water content, permeability coefficient, void ratio, plastic limit, liquid limit, salinity and chlorinity.

### 2.2.2 Frozen soil tests

Frozen soil tests include:

1. Uniaxial compressive strength test

Uniaxial compressive strength tests are conducted at the temperature of  $-8^{\circ}$ C,  $-15^{\circ}$ C,  $-20^{\circ}$ C and  $-25^{\circ}$ C respectively to achieve the uniaxial compressive strength, modulus of elasticity and Poisson's ratio.

- Frost heave test Frost heave tests are conducted to achieve frost heave ratio and frost heave force.
- 3. Thaw consolidation test
- 4. Thermal conductivity test
- 5. Freezing point test
- 6. Specific heat capacity test

### 3 RESULTS

### 3.1 Basic geotechnical tests

The results of basic geotechnical tests are presented in Table 1.

### 3.2 Frozen soil tests

### 3.2.1 Uniaxial compressive strength test

(1) Uniaxial compressive strength

Table 2 shows the result of uniaxial compression test, which indicates the fact that the uniaxial compressive strength of the frozen soil rises when

Table 2. Results of uniaxial compressive strength.

Soil layer	$-8^{\circ}C$	-15°C	-20°C	-25°C
[4]	3.14	4.07	5.29	5.94
[5]1	3.19	4.82	5.47	6.58
[5]2	4.18	5.26	6.17	7.07
[5]3	3.07	3.95	5.16	6.60
$[7]_{1-1}$	4.29	5.44	6.55	7.61

Table 3. The relationship between uniaxial compressive strength  $\sigma$  (MPa) and temperature *T* (°C).

Soil layer	Linear fitted relationship	
$   \begin{bmatrix}     4\\     5_1\\     5_2\\     5_3\\     [7]_{1-1}   \end{bmatrix} $	$\begin{split} \sigma &= -0.1708 \ T + 1.706 \\ \sigma &= -0.1943 \ T + 1.7118 \\ \sigma &= -0.1704 \ T + 2.7725 \\ \sigma &= -0.2073 \ T + 1.1713 \\ \sigma &= -0.1965 \ T + 2.6328 \end{split}$	$R^{2} = 0.9844$ $R^{2} = 0.9901$ $R^{2} = 0.9982$ $R^{2} = 0.9641$ $R^{2} = 0.9949$

temperature falls. Furthermore, these two variables obey a favorable linear relationship. The linear fitted correlation between uniaxial compressive strength  $\sigma$  and temperature *T* according to the test data is presented in Table 3 and Figure 2.

(2) Modulus of elasticity

The result of the test indicates that, generally, modulus of elasticity of the frozen soil rises when temperature falls. Data acquired from the test is shown in Table 4. The exponential correlation between modulus E of elasticity and temperature T could be fitted in form of

$$E = ae^{-bT} \tag{1}$$

as presented in Figure 3 and the coefficients a and b are shown in Table 5.

(3) Poisson's ratio

The result of Poisson's ratio is shown in Table 6, which demonstrates that when temperature falls, the Poisson's ratio of the frozen soil tends to decline, though this trend is not remarkable.

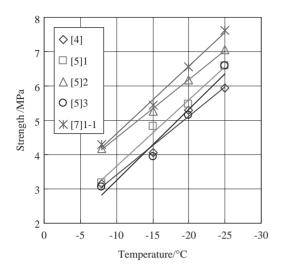


Figure 2. The relationship between uniaxial compressive strength and temperature.

	Modulus of elasticity/MPa				
Soil layer	-8°C	−15°C	-20°C	-25°C	
[4]	53.1	78.7	146.5	246.8	
[5]1	58.2	97.9	165.2	269.0	
[5]2	50.7	82.4	155.7	273.5	
[5]3	57.1	93.2	161.1	306.1	
$[7]_{1-1}$	79.2	155.7	238.1	401.0	

Table 4. Result of modulus of elasticity.

Table 5. The coefficients *a* and *b* in the relationship between modulus *E* (MPa) of elasticity and temperature *T* ( $^{\circ}$ C).

	Exponential fitted relationship			
Soil layer	a	b	$R^2$	
[4]	23.173	0.0921	0.9726	
[5]	26.975	0.0907	0.9939	
[5]2	20.923	0.1005	0.9824	
[5]3	23.848	0.0985	0.9796	
$[7]_{1-1}$	37.176	0.0945	0.9989	

### 3.2.2 Frost heave test

Frost heave ratio test: frost heave test without axial confinement so that the specimen can expand freely. The relationship between axial displacement and time is measured at the time according to code requirement. The maximum of frost heave  $\delta_{max}$  should also be recorded. Frost heave ratio of the specimen means the ratio between the maximum of frost heave  $\delta_{max}$  and initial length of the specimen.

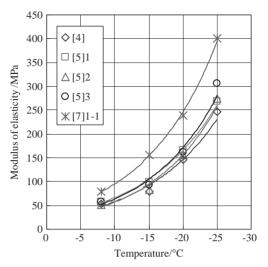


Figure 3. The relationship between modulus of elasticity and temperature.

Table 6. Result of Poisson's ratio.

	Poisson's ratio					
Soil layer	-8°C	−15°C	-20°C	-25°C		
[4]	0.281	0.263	0.230	0.214		
[5]1	0.277	0.251	0.229	0.204		
[5]2	0.298	0.269	0.248	0.221		
[5]3	0.259	0.242	0.217	0.206		
$[7]_{1-1}$	0.266	0.244	0.218	0.180		

Table 7. The results of frost heave force and frost heave ratio tests.

Soil layer	Frost heave force (MPa)	Frost heave ratio (%)
[4]	0.66	7.93
[5]1	0.78	6.84
[5]2	0.84	7.26
[5]3	0.71	6.35
$[7]_{1-1}$	0.69	6.48

Frost heave force test: frost heave test proceeded with displacement confinement. Lengthwise confinement is applied to the upper end of the specimen and then frost heave force is measured by load sensor. The relationship between frost heave force and time is recorded during the whole process to get the maximum of frost heave force  $\sigma_{max}$ .

The maximum of frost heave ratio and frost heave force of each soil layer is presented in Table 7.

Table 8. The result of the thaw consolidation ratio.

Thaw consolidation ratio (%)
6.83 7.09 7.76 7.25 6.05

Table 9.The result of the thermal conductivity.

	The thermal conductivity (W/( $m \cdot H$		
Soil layer	positive	negative	
[4]	1.39	1.62	
[5]1	1.43	1.75	
[5]2	1.47	1.68	
[5]3	1.42	1.74	
$[7]_{1-1}$	1.54	1.81	

### 3.2.3 Thaw consolidation test

The temperature of the specimen is  $-8^{\circ}$ C, the hot end temperature is  $40 \pm 2^{\circ}$ C. The dimension of the specimen is  $\Phi$ 79.8 × 40 mm. The thaw consolidation ratio is determined as follows:

$$a_0 = 100\Delta h_0 / h_0 \tag{2}$$

where a0 = thaw consolidation ratio (%);  $\Delta h0 =$  quantity of thaw consolidation (mm); and h0 = initial height of the specimen (mm).

The result of thaw consolidation test of each soil layer is shown in Table 8.

### 3.2.4 Thermal conductivity test

The result of the thermal conductivity is presented in Table 9. In this table, positive is for the unfrozen soil and negative for the frozen soil.

### 3.2.5 Freezing point test

Generally speaking, the crystallization from liquid to solid of the water in the soil undergoes three stages: First a small group of molecule is formed which is called the crystallization center or growth point, then it grow up to a bigger crumb called crystal nucleus, which eventually develops to ice crystal. The temperature of the formation of the ice crystal is called freezing point or ice point. The crystallization center is formed at a certain temperature below the freezing point. As a result, the formation of frozen soil is consisted of four stages, i.e. supercooling, jump, invariableness and degradation. At the stage of jump, the electric potential will suddenly reduce, and then become stable at a certain number at the temperature when freezing begins.

Soil layer	Freezing point (°C)			
[4]	-2.5			
[5]1	-2.3			
[5]2	-1.7			
[5]3	-2.1			
$[7]_{1-1}$	-1.7			

Table 10. The result of freezing point.

Table 11	Result of	undisturbed	soil s	necific	heat cana	city
Table 11.	Result Of	unuistarbeu	sons	peenie	neat capa	city.

Soil layer	Specific heat capacity/(g·K)
[4]	1.56
[5]1	1.62
[5]2	1.69
[5]3	1.65
$[7]_{1-1}$	1.51

According to this principle, the freezing point can be calculated as follows:

$$T = V / K \tag{3}$$

where T = freezing point (°C); V = the stabilized value after the jump of the hot electric potential  $(\mu\nu)$ ; and K = the demarcation coefficient of thermoelectric couple (°C/ $\mu\nu$ ).

The result of the freezing point of each soil layer is presented in table 10.

### 3.2.6 Specific heat capacity test

The specific heat capacity test is performed with the specific heat capacity testing device DTBR-01. Introduce coolant of the same temperature and volume into both sides of the equipment. Temperature sensors are previously put into coolant. The specimen with temperature sensor embedded is put into one side. According to the law of conservation of energy, the energy for one degree changing can be determined by the relationship between the three values of temperature. The result is presented in table 11.

### 4 DISCUSSION AND SUGGESTIONS

### 4.1 Salinity and its influence

The salinity of pore water in the undisturbed soil ranges from  $8.039 \sim 18.370\%$  (see Table 1). To find out the salinity difference between the soils near the sea and far from the sea, an additional test was made for a soil layer – the gray clayey silt ([7]<sub>1–1</sub>). In the result, the salinity of the specimen of the layer in the urban area (at the site of the restoration project of the collapsed tunnels of Shanghai Metro Line 4) is 6.320%, while that of the same layer at the estuary is 14.744% (see Table 1). Therefore, a conclusion could be drawn that the salinity of pore water in the strata near the sea is higher than that distant form the sea.

Influence of salinity on frozen soil properties could be freezing-point depression, freezing phase lengthening, the growth rate decrease of frozen bodies and the strength loss of the frozen ground, on which a great attention must be paid.

Given lack of enough test data, no clear relationships have been found between the salinity and frozen soil strength, frost heave force, frost heave ratio, thaw consolidation ratio and other parameters, as well as freezing point, but further tests are being performed by the authors.

### 4.2 Freezing-point depression

The freezing point ranges from -1.7 to  $2.5^{\circ}$ C (see Table 10). It is found that the freezing-point depression of the soil at the estuary is greater than that distant from the sea. Fro example, the freezing-point temperatures of the gray clayey silt  $[7]_{1-1}$  near the sea and distant from the sea are  $-1.7^{\circ}$ C (see Table 10) and  $-1.4^{\circ}$ C (Xiao, et al. 2003), respectively. On the other hand, soil layers are sorted according to the descending order of the magnitude of freezing-point depression, i.e. mucky clay, clay, silty clay and clayey silt.

Freezing-point depression causes a smaller thickness of the soil frozen wall than the thickness when the freezing point is considered as 0°C. Attention must be paid upon the thickness loss of the frozen soil wall due to freezing-point depression.

### 4.3 Frozen soil strength

The uniaxial compressive strength of the frozen soils and the temperature of the specimen bear a favorable linear relationship, with regression coefficient all higher than 0.96. Meanwhile, the moduli of elasticity of the frozen soils have an exponential correlation with the temperature, with an average regression coefficient of 0.985. The regression formulas can be used to calculate the strength values at any temperature within the range of the test temperature.

A suggestion here is that the regression formulas should be used to calculate the bearing capacity and deformation of the frozen wall.

### 4.4 Frost heave

The frost heave force ranges from 0.66 to 0.84 MPa. The frost heave ratio ranges from 6.35 to 7.93%. According to Code for Design of Soil Foundation of Building in Frozen Soil Region (JGJ118-98), when the frost heave ratio of the clay is higher than 6%, the soil is categorized as highly frost-heaving soil. Therefore

great emphasis should be laid on the design and application of the construction. It is suggested to install a steel reinforcement frame around the opening as a protection device to prevent the tunnel lining from excessive deformation due to the frost heave.

### 4.5 Thaw consolidation

The ratio of the thaw consolidation ranges from 6.05% to 7.76%. The thaw settlement of the ground around the cross passages could be remarkable. Therefore, it is advised to perform enforced thawing as an effective measure to reduce thaw settlement.

### 4.6 Thermal properties

The thermal conductivity for unfrozen soil ranges from 1.39 to  $1.54 \, W/(m \cdot K)$ , and  $1.62 to 1.81 \, W/(m \cdot K)$  for frozen soil. The soils are hard to be frozen, i.e. the growth rate of the frozen body could decrease or the freezing phase could be lengthened, because the coefficient of thermal conductivity is unfavorably low. It is necessary to enhance freezing process monitoring and to use the monitored data to judge the size of the frozen wall instead of rude estimate of the freezing phase.

### 5 CONCLUSIONS

Although the strength of the frozen soils at the estuary of the Yangtze River is fairly high, attention must be paid upon the disadvantageous influence of freezing-point depression due to the high salinity on the calculation of the thickness of the frozen soil walls for constructing the cross passages.

Further study should be carried out to find out the influence of salinity of pore water in the saline soils upon the geotechnical and physical-mechanical properties discussed before.

### ACKNOWLEDGEMENTS

The work were supported by the National Natural Science Foundation of China (No. 50578120), the National High Technology Research and Development Program of China (863 Program) (No. 2006AA11Z118) and Shanghai Leading Academic Discipline Project(Project Number: B308).

### REFERENCES

Crippa C. & Manassero V. 2006. Artificial Ground Freezing at Sophiaspoortunnel (The Netherlands) — Freezing parameters: Data acquisition and processing. *GeoCongress* 2006: Geotechnical Engineering in the Information Technology Age, Atlanta, Feb 26 – Mar 1, 2006. Reston, VA: ASCE.

- Heilongjiang Province Academy of Cold Area Building Research. 1999. Code for Design of Soil Foundation of Building in Frozen Soil Region (JGJ118-98). Beijing: China Architecture & Building Press.
- Hu X.D. & Chen R. 2006. Construction technology of freezing method applied to cross-passage of double-deck cross-river road tunnel. *Low Temperature architecture technology*. (5). 64–66.
- Roman L.T., Aksenov V.I. & Veretehina E.G. 2004. Characteristics of influence of the content salts on the frozen soils strength. *Journal of glaciology and geocryology* 26(Suppl): 35–38.
- Shen H.T., Mao Z.C. & Zhu J.R. 2003. Saltwater Intrusion in the Changjiang Estuary. Beijing: China Ocean Press.
- The Ministry of Coal Industrial P.R.C. 1996. Coal industrial standard: Physical-mechanical properties test of artificial frozen soil (MT/T593-1996). Beijing.

- The Ministry of Water Resources P.R.C. 1999. *Standard* for soil test method (GB/T50123-1999). Beijing: China Planning Press.
- Xiao Z.H., Hu X.D., Pi A.R. & Liu R.F. 2003. Study on uniaxial compressive strength of Shanghai soils under secondary freeze-thaw action. *Rock and Soil Mechanics* 27 (Suppl): 497–500.
- Zhao Y.H., Hu X.D., Zhao G.Q. 2005. Experimental study on artificial frozen soil boundary detecting during crosspassage construction in tunnel. *Progress in Electromagnetics Research Symposium, Hangzhou, China, 22–26 August 2005.* Cambridge: The Electromagnetics Academy.

# The influence of engineering-geological conditions on construction of the radioactive waste dump

J. Kuzma & L. Hrustinec

Slovak University of Technology, Faculty of Civil Engineering, Bratislava, Slovakia

ABSTRACT: Secure stability and reliable serviceability of radioactive dump is a difficult engineering problem. Due to difficult geological formations determined mainly by great compressibility, low shear strength of soils, and high ground water level, or great upward hydrostatic pressure will these demands increase. Influence of required reliability and lifespan on the structure of these specific objects is considerable. We are trying to contribute to a problem solving of these difficult and complicated problems in submitted contribution.

### 1 INSTRUCTION

The Republic Radioactive Waste Dump (RRWD) located about 2 km from the area of the Mochovce Nuclear Plant in south-western Slovakia, Central Europe, was built to store waste with low and medium activity arising from operation and disposal of nuclear power facilities as well as from research institutes, laboratories, hospitals and other institutions engaged in activities generating nuclear waste. Ensuring stability and reliable operation of RRWD in Mochovce is a challenging engineering task. The dump is built in demanding engineering-geological conditions determined in particular by high compressibility, low shearing strength of soil and high underground water level and/or high lifting forces exerted in the basement. Also required reliability and lifetime have a significant influence on layout of these specific structures. This paper discusses a solution for this complex engineering task.

The topic of how to secure a reliable storage of final processed low and medium radioactive waste in the long term is a complex interdisciplinary issue. The time aspect is a significant factor affecting a complex approach to this problem and specific periods of active operation and planned lifetime of the repository must be taken into account. The repository is built in fairly difficult geological and hydro-geological conditions. It consists of a complex of buildings and technological facilities. In the first construction stage, two double lines of storage boxes with the total surface area of approx. 112.000 m<sup>2</sup> were built. The general situation around the dump and its adjacent territory is shown on Figure 1. Section of the RRWD is shown on Figure 2.



Figure 1. View of placement of the Republic Radioactive Waste Dump (RRWD) in Mochovce.

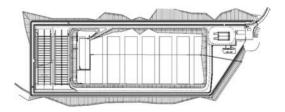


Figure 2. Section of the RRWD in Mochovce.

Such large structural work challenging in terms of operational safety will have a significant impact on the surrounding natural environment during its lifetime. Extensive construction activities, active operation of the repository and the final coverage will have a significant influence on a change in the balanced state of the rock environment. This change in the balanced state will cause deformation symptoms at the affected territory with a number of structural buildings and functional (technological) units. Knowing the extent of the modified basement and construction at the territory concerned will be decisive to determine stability of the territory and functionality of the already built or planned drainage systems, engineering and technological distribution lines. This paper will discuss in detail the issue of repository reliability in terms of assessing the basement according to requirements of group I and II limit states, i.e. according to safety and usability of the structural work. A set of geotechnical issues is related to the draft construction project of the final repository coverage with its primary function to prevent water infiltration to premises housing the storage boxes after the active operation of the structural work until its lifetime.

### 2 MORPHOLOGICAL, GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS AT THE REPUBLIC DUMP

Republic Radioactive Waste Dump is located in southwestern Slovakia, Central Europe. The terrain is slightly folded with an unstable superficial water river. The basement is made of sandy limestone clay with spots of dusty fine-grain sand and clay silt. These spots form lenses separated from each other. Sediments of Ivanian strata (lower Panonium) with thickness of 11.5-23.0 m were identified at the northern part of the repository. They are represented by sandy clay with thin spots of dusty sands and isolated lenses of gravel sands. Quaternary deposits are represented primarily by deluvial rain-wash sand clay with thickness up to 3.0 to 4.0 m. The basement directly under the first double-line of boxes is made of residual quaternary deposits (clays), especially in the middle of the box structure and under the bottom doubleline. Panonian deposits (clays, sands) are found underneath.

Hydrogeological conditions in the RRWD basement and in its direct vicinity are one of the crucial factors with regard to operational safety of the dump. It can be said that the hydrogeological conditions at the site are fairly complicated due to the complex nature of the geological environment with flows of underground water. Surveys had done show existence of several continuous water-saturated aquifers of underground water. The highest main aquifer (H) is an aquifer with a free water level, fairly sensitive to precipitation. Other two aquifers are more permeable, hydraulically fairly isolated sandy spots in deeper zones of the Sarmatian basement. The underground water has lifting characteristics and its piezometric level reaches 1.0 to 2.0 m above the free level of the H aquifer. The original hydrogeological conditions were significantly

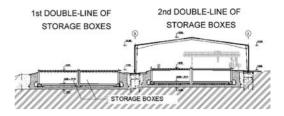


Figure 3. Typical cross section of the 1st and 2nd double-line of storage boxes in RRWD in Mochovce.

modified by constructional activities. In terms of assessing the foundation conditions, the engineeringgeological and hydrogeological conditions at the area of interest can be assessed as difficult.

### 3 CONSTRUCTIONAL AND TECHNICAL CHARACTERISTICS OF THE REPOSITORY

At present, the Republic Radioactive Waste Dump consists of two double-lines of storage boxes made from reinforced concrete. Four boxes are grouped into one dilatation block with layout dimensions of  $18.6 \times 6.0$  m and height of 5.5 m, placed on a common foundation slab 0.6 m thick. There are five dilatation blocks in one line. An expansion gap between the blocks is planned to be 50.0 mm thick. The storage boxes are covered by 0.5 m thick panels from reinforced concrete. A system of two drainages (control drainage and monitored drainage) is built at the bottom of the storage boxes and under the foundation. The main goal of the dump drainage system is to ensure inspection and safe removal of potential leaks of underground and surface water contaminated by radioactive substances from the inner space of the storage boxes into the basement and/or from the basement into the storage boxes. Side walls and the bottom of the monolithic storage boxes from reinforced concrete are protected by a layer of clayish soil. A 1.0 m thick layer of compacted clayish soil is located under the tanks to ensure waterproof basement for the storage boxes. Placed on this layer is a 0.6 m thick layer of sandy gravel. Side walls are protected by a 3.5 m thick vertical clay sealing linked with a horizontal sealing soil layer, forming a compact unit - the so-called clay sealing tub. The double-lines are protected against climatic effects by a mobile steel hall when the storage boxes are filled with waste. Considering the general reliability of RRWD, the structures of the storage boxes are the most important and decisive building structures. At the same time, these structures will face the highest demands due to effects of forces, deformation and radiation. A typical cross and longitudinal section of the 1st and 2nd double-line is shown on Figure 3 and Figure 4.

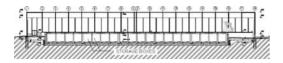


Figure 4. Typical longitudinal section of the 1st double-line of storage boxes in RRWD in Mochovce.

### 4 ASSESSING STABILITY IN RRWD ACCORDING TO LIMIT STATES

In order to make a general assessment of RRWD safety and usability, it is necessary to examine a set of geotechnical issues predetermined by the challenging nature of building structures at the dump and by complex engineering-geological conditions. In terms of compliance with requirements according to group I and II of limit states, solutions for the problems can be divided into two main sets as follows:

- Stability calculations to assess overall stability of RRWD and the adjacent rock environment affected by the construction in specific stages of construction, operation and expected lifetime of the repository, including basement for the storage boxes.
- Deformation calculations to assess final and uneven settlement of decisive building structures of the dump, taking into account specific stages of construction, operation and expected lifetime of the dump.

To solve the first set of problems related to security and operative capacity of RRWD in Mochovce in every construction stage and after the construction, it was necessary to make a series of stability calculations to determine:

- Stability of the 1st and 2nd double-line,
- Stability of adjacent slopes,
- Bearing capacity of reinforced concrete boxes.

The second set of problems deals with how to ensure usability and smooth operation of the building structures and technological units at the dump in terms of size of the final and uneven settlement of the storage boxes. A requirement to assess horizontal shifts of storage box walls at places of expansion gaps, planned to be 50.0 mm wide, stems out from the structuraltechnical solution of decisive bearing structures, including bearing walls of the storage boxes. Considering the height of storage box walls, even a small uneven settlement or inclination of tough reinforced concrete structures of the storage boxes can cause their damage, resulting in a loss of their planned function. The extent of settlement and uneven settlement of the repository basement must be known to assess also function of the drainage and sealing systems located in revision and inspection shafts. In case of drainage systems, this

involves especially the required gradient needed for a gravitational drainage of leaked water. The structural design of the dump shows a concentrated intensity of load in the geometric centre of the double-lines with storage boxes, so the method and system of placing fiber-concrete containers into the storage boxes of the dump is very important.

The effects of uneven settlement of the basement result in formation of local depressions in the drainage system, so their thickness must follow the extent of the uneven settlement of the basement to ensure the required gradient for gravitational water drainage in the long term. Occurrence of local depressions would result in higher and permanent hydraulic load of specific sealing layers in the dump, affecting physical properties of the soil in the sealing layer, in particular the state of consistency and then strength and deformation properties. Such changes would result in higher permeability of the sealing tub, higher compressibility and lower bearing capacity of the RRWD subsoil. Localization of maximum tensions and deformations given by effects of external and deforming load can indicate potential defect areas that must be taken into account in the design of structures of specific layers for the final coverage.

The most demanding and the most important task in the process of addressing these problems is to transform actual material properties of the natural rock to an idealized calculation model that must realistically depict the actual behavior of the assessed structures. It is therefore necessary to pay proper attention to this task. This issue is complicated even more by significant non-homogeneity of the natural rock environment, but primarily by the fact that material properties of fine-grain soil in the subsoil are very variable in time and tension. It is very difficult to define complex constitutional relationships in calculation models that would truly describe the actual processes in the soil massif resulting from effects of the load. This task can be simplified to a certain degree by a reasonable idealization (simplification) of actual soil properties, respecting the inevitable requirement to preserve the physical essence. It is therefore necessary to determine physical, strength and deforming properties of the basement soil and of the soil incorporated in the sealing tub based on standard laboratory test on integral and undamaged samples in accordance with applicable standards.

To define calculation models and limit conditions of specific tasks, it is necessary to define the following input data:

- Spatial arrangement and geometrical shape of building structures and construction at the area concerned;
- Extent and distribution of permanent, incidental, extraordinary loads and their combinations;

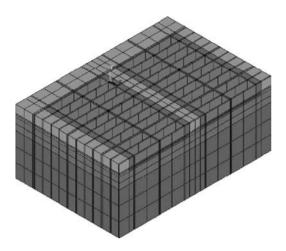


Figure 5. Calculation model divided into finite elements.

- Physical, strength and deforming properties of artificial building materials (concrete and reinforcedconcrete structures);
- Physical, strength and deforming properties of the natural rock environment at the area of concern and of the soil incorporated in RRWD.

In order to assess general reliability of the dump, it is not permissible to exceed any limit state and to allow deformation of any structural unit during the whole period of active operation of the repository (organization of filling the storage boxes by fiber-concrete containers) and during the period of the planned and/or actual lifetime of the dump once it is finally covered. Specific calculations must therefore depict all the crucial load states that occur or might occur in the future. The extent of load on specific building structures and subsoil in RRWD and effects of the load on the environment is a very variable quantity in terms of time. It will depend mostly on the construction process applied for the main dump building structures, schedule of organized filling of the storage boxes by fiber-concrete containers during active operation of the dump and constructional stage of the final dump coverage. The intensiveness of load will therefore depend on a progressive increase in permanent, long-term and short-term, incidental and extraordinary load and their combinations.

Mathematical modeling using the method of finite elements was used to calculate tensions and deformations in the RRWD subsoil. The spatial model for the model calculations was developed according to assumptions of the linearly elastic half-space theory (Figure 5).

Basic parameters of the geotechnical model arise from an engineering-geological survey and structural design of the dump. Geotechnical calculations define

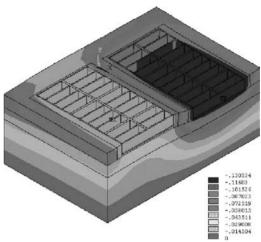


Figure 6. Isoareas of subsoil settlement for storage boxes calculated for the Load State 2.

four load states (LS) describing the following crucial stages of dump construction and operation:

- LS 1 Load on the subsoil due to own weight of the reinforced-concrete storage boxes,
- LS 2 Load after filling the 1st double line with waste containers,
- LS 3 Load after filling the 2nd double line with waste containers,
- LS 4 Load after closing the 1st and 2nd double lines and the final coverage of the dump.

Resultant settlement (vertical displacement) of the subsoil under the storage boxes calculated for the Load State 2 is shown on Figure 6.

The factor of time is very important to address these problems, because the expected lifetime for RRWD was determined by the Slovak Nuclear Supervision Office to be 300 years based on the period of the institutional inspection of the dump. Consolidation processes in the dump basement and progressive redistribution of forces occurring in dependence on the rate of loading the foundation gap (filling the storage boxes by containers) are closely related to the time factor. Calculations on forecasts make it possible to design a suitable method of container placement, with favorable effects on the overall average settlement and uneven deformations that will have been developed depending on time in couples of decades. Stability of the Republic Radioactive Waste dump must be ensured with required reliability during the whole term of its lifetime.

### 5 CONCLUSION

Construction of specific buildings intended to store low and medium radioactive waste is an extremely challenging task for engineers. Given the demands and required reliability of storing radioactive waste in RRWD Mochovce, the issue must be discussed in its complexity. The time aspect is a significant factor affecting a complex approach to this problem and specific periods of active operation and planned lifetime of the dump must be taken into account. To formulate specific geotechnical problems, it is necessary to focus primarily on assessment of structural works in terms of safety and usability of building structures and technological units. Much emphasis should be given on definitions of limit conditions for specific tasks with focus on their variability in time. A correct definition of interactions between the subsoil and the building structure to ensure the required level of environmental protection is an essential requirement to address such a complex interdisciplinary issue.

### ACKNOWLEDGEMENT

This paper was developed in terms of a grant task of the Slovak Republic Ministry of Education No 1/2135/05.

### REFERENCES

Kuzma, J. et al. 1998. Experimental Assessment of the Republic Radioactive Waste Dump in Mochovce. Bratislava.

### Critical ventilation velocity in large cross-section road tunnel fire

### Z.X. Li, X. Han & K.S. Wang

Shanghai Institute of Disaster Prevention and Relief, Tongji University, Shanghai, P.R. China

ABSTRACT: Based on CFD method, this paper conducted simulation analysis of lorry and car fires for large cross-section road tunnel. The smoke distribution feature was discussed and the corresponding results of critical ventilation velocity were also compared with the values calculated by related empirical formula. It would be helpful to work out a ventilation system strategy for the road tunnel.

### 1 INTRODUCTION

Recent incidents however have drawn widespread attention to the risks of fires in road, rail and masstransit tunnels (e.g. King's Cross, Channel Tunnel, Mont Blanc, Tauern, Gotthard). The incidents have resulted in casualties as well as large direct and indirect economic damage. The control of smoke flow during a road tunnel fire is often an important part of fire safety measure. On the other hand, CFD (Computational Fluid Dynamics) simulations are increasingly used to estimate the effects of fires in road tunnels. However, little work has been done to analyze comprehensive fire characteristics of the large cross-section road tunnel. On the basis of CFD method and with the help of FDS simulation software, this paper conducted simulation analysis of lorry and car fires for large cross-section road tunnel. Two fire scenarios were constructed and five different kinds of longitudinal ventilation conditions were considered. The smoke distribution feature was discussed and the corresponding results of critical ventilation velocity were also compared with the values calculated by related empirical formula. It would be helpful to work out a ventilation system strategy for the road tunnel.

### 2 CFD SIMULATION SCENARIOS

### 2.1 Brief introduction of FDS

In these simulations, FDS (Fire Dynamics Simulator) 4.06 which was released by NIST (National Institute of Standards and Technology, USA) was used. FDS is a Computational Fluid Dynamics (CFD) model with LES (Large Eddy Simulation) of fire-driven fluid flow. The model solves numerically a form of the Navier-Stokes equations appropriate for low-speed,

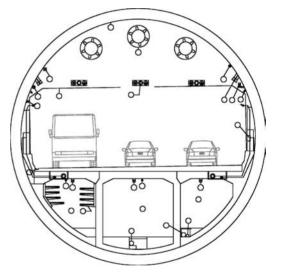


Figure 1. Cross-section of road tunnel.

thermally-driven flow with an emphasis on smoke and heat transport from fires. The partial derivatives of the conservation equations of mass, momentum and energy are approximated as finite differences, and the solution is updated in time on a three-dimensional, rectilinear grid. Thermal radiation is computed using a finite volume technique on the same grid as the flow solver.

### 2.2 The design of the fire scenario

As for large cross-section road tunnel, the fire smoke distribution feature under different ventilation situation was simulated. One road tunnel which has 100 m length and 15 m diameter was constructed as

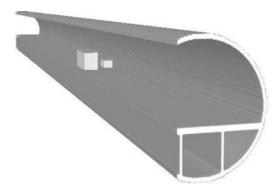


Figure 2. CFD model in scenario 1.



Figure 3. CFD model in scenario 2.

simulation model, as shown in Figure 1, Figure 2 and Figure 3. One lorry which designed as fire source was located at 37 m from the upstream ventilation cross-section and 63 m from the downstream ventilation cross-section, as shown in Figure 2. The lorry was supposed to be on fire first, and the heat amount released by the lorry would emblaze a car near the lorry. Two fire scenarios were set up, including: (1) scenario 1: the car at the right of the lorry; (2) scenario 2: the car in front of the lorry. Both scenarios involved five different kinds of longitudinal ventilation conditions. The ventilation velocities were set as 2 m/s, 2.5 m/s, 3 m/s, 3.5 m/s, and 4 m/s respectively.

During the simulation, the lorry was burning first. Then at a certain time, when the heat amount the car received exceeding  $16 \text{ kW/m}^2$ , the car would begin to burn. The heat release rate of the lorry and the car were 20 MW and 5 MW respectively. The total heat release rate was shown in Figure 4 and Figure 5 separately.

### 3 ANALYSIS OF CFD SIMULATION RESULTS

### 3.1 The brief analysis of the simulation results

Since the heat release rates of two scenarios were almost of same value, the simulation results demonstrated that the relative position of the lorry and the car didn't distinctively affect the smoke distribution feature. As for the same environmental conditions, there was not much difference between these two

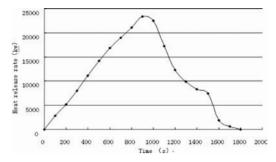


Figure 4. Heat release rate of fire source in scenario 1.

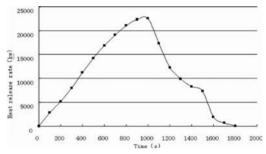


Figure 5. Heat release rate of fire source in scenario 2.



(a) ventilation velocity: 2m/s, simulation time: 1000s



(b) ventilation velocity: 2.5m/s, simulation time: 1020s



(c) ventilation velocity: 3m/s, simulation time: 1000s



(d) ventilation velocity: 3.5m/s, simulation time: 1030s

Figure 6. The smoke distribution feature under ventilation velocities of 2 m/s, 2.5 m/s, 3 m/s and 3.5 m/s respectively in scenario 2.

fire scenarios, as shown in Figure 6, Figure 7 and Figure 8 respectively. For example, under the same longitudinal ventilation condition, the smoke density at the location of 10 m upstream the fire source was of similar distribution type.

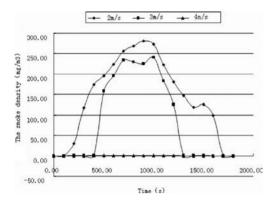


Figure 7. Smoke density at the location of 10 m upstream the fire source in scenario 1.

The simulation results illustrated that the road tunnel fire yielded high production of smoke. As a result of fire temperature, the smoke would have a lower relative density than the surrounding air in the tunnel and therefore rose to the ceiling of the tunnel. Consequently the smoke would remove itself away from the fire source in a layer along the ceiling. Along the bottom the cold air would be sucked towards the fire. This created two flowing layers, i.e., the hot smoke layer and the cold smoke-free layer of air. The phenomenon was called stratification.

The hot smoke layer gradually formed as the hot smoke curled upwards. With continuous yielding of smoke from the fire source, the hot smoke layer swelled rapidly and spread towards both sides of the tunnel. Meanwhile, the cold air from the lower part of the tunnel flew to the fire source. In this way, the symmetry re-circulated air was formed on both sides of the fire field. When the longitudinal ventilation was started, the smoke on both sides of the fire source emerged asymmetry. If the ventilation velocity is quite small, it couldn't prevent the smoke from diffusing in the direction opposite to the ventilation stream. This situation, namely smoke flows against the ventilation, is adverse to the prevention of the smoke spread (the hot smoke maybe emblaze the traffic in the upper side of the fire) and would be harmful to the safety of the firemen. In order to prevent the backflow of the smoke, the longitudinal ventilation velocity should be greater than the critical one.

As ventilation velocities were set as 2 m/s and 3 m/s, and at the location of 10 m upstream the fire source, the maximum smoke density would approached  $250 \text{ mg/m}^3$  and  $280 \text{ mg/m}^3$  separately. While the ventilation velocity was set as 4 m/s, the corresponding smoke density almost kept zero. It obviously indicated that the critical ventilation velocity went between 3 m/s and 4 m/s.

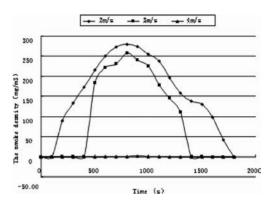


Figure 8. Smoke density at the location of 10 m upstream the fire source in scenario 2.

### 3.2 Obtain of the critical ventilation velocity

Oka and Atkinson had proposed a method to calculate the critical velocity, which was to get the smoke backflow distance in different ventilation through the detectors first. Then the value of the distance was placed in the reference frame for backflow distance and ventilation velocity. To joint these points in the reference frame and extend the line to the y-axis, the y-axis value of the point of intersection was just the critical velocity. This paper would also obtain the critical ventilation velocity in this way.

The spread of the smoke under the ventilation velocities of 2 m/s, 2.5 m/s, 3 m/s and 3.5 m/s were presented in above mentioned figures. These figures could give out the related distance values which the smoke spread upstream the fire source. Based on the detectors, we could get the backflow distance in four situations of 120 m, 80 m, 48 m and 10 m. These numerical values were put up in the reference frame for backflow distance-ventilation velocity. Jointing the points and extending the line in accordance with the method referred previously, then the critical ventilation velocity could be obtained as 3.65 m/s.

### 3.3 *Comparative analysis of the critical ventilation velocity*

Oka and Atkinson had set up a formula for calculation of tunnel critical ventilation velocity as follows:

$$Q^* = \frac{Q}{\rho_0 C_p T_0 g^{\frac{1}{2}} H^{\frac{5}{2}}}$$
(1)

$$V^* = \frac{V_{critical}}{\sqrt{gH}}$$
(2)

$$V^* = 0.35(0.124)^{1/3} (Q^*)^{1/3}, Q^* < 0.124$$
(3)

$$V^* = 0.35, \ Q^* < 0.124$$
 (4)

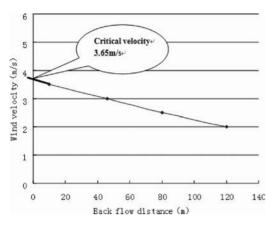


Figure 9. Critical ventilation velocity for large cross-section road tunnel.

where Q = the heat release rate, kw; V<sub>critical</sub> = the critical velocity, m/s;  $Q^* =$  dimensionless heat release rate, and V\* = the dimensionless critical velocity,  $\rho_0 =$  ambient air density, kg/m<sup>3</sup>; CP = specific heat capacity, kJ kg<sup>-1</sup> K<sup>-1</sup>; T<sub>0</sub> = ambient temperature, K; g = acceleration due to gravity, m/s<sup>2</sup>; H = height from the surface of fire source to tunnel ceiling, m. Using these values we could obtain the critical ventilation velocity of the tunnel as 3.208 m/s. Comparing the simulation value with the one given out by formula (1), the relative error was 12.12%.

Wu and Bakar further amended the formula (1). They considered the influence of the breadth of the tunnel and replaced the height of the tunnel with the hydraulic tunnel height H, which was defined as the ratio of 4 times the cross-sectional area to the tunnel wetted perimeter. Thus, the new formula was illustrated as follows:

$$V''=\frac{V}{\sqrt{g\overline{H}}}$$
(5)

$$Q'' = \frac{Q}{\rho_0 C_p T_0 \sqrt{g\overline{H}^5}} \tag{6}$$

$$V'' = 0.40[0.20]^{-1/3}[Q'']^{1/3}, Q'' \le 0.20$$
(7)

$$V''=0.40, \ Q'' \ge 0.20$$
 (8)

where Q'' = dimensionless heat release; V'' = dimensionless critical ventilation velocity, other

parameter were of the same meaning as formula (1). Through formula (5), we could obtain the critical ventilation velocity as 2.893 m/s. Comparing this value with the simulation one, the relative error was 20.7%.

### 4 CONCLUSION

The simulation results demonstrated that specifying a critical ventilation velocity was helpful to prevent back layering of smoke during a tunnel fire. Hence, it was very important for smoke management of large crosssection road tunnel. In order to find cost-effective methods to upgrade fire safety in existing tunnels or tunnels to be built, the practical tests as well as simulation analysis should be further carried out. Hence, the corresponding performance-based calculation method could be put up to improve the computation of critical ventilation velocity. Meanwhile, more influencing factors need to be considered, such as the impact of road tunnel physical characteristics and the traffic capacity, etc.

### ACKNOWLEDGMENTS

The support of the Natural Science Foundation of China (Grant No. 50678124) is gratefully appreciated.

### REFERENCES

- Bettis, RJ., Jagger, SF., Wu, Y. Interim validation of tunnel fire consequence models; summary of phase 2 tests.*The Health and Safety Laboratory Report IR/L/FR/93/11, The Health and Safety Executive.* UK, 1993.
- Danziger, NH., Kennedy, WD. Longitudinal ventilation analysis for the Glenwood canyon tunnels. Proceedings of the Fourth International Symposium Aerodynamics and Ventilation of Vehicle Tunnels. York, UK, 1982: 169–186.
- Heselden, AJM. Studies of fire and smoke behavior relevant to tunnels. Proceedings of the Second International Symposium of Aerodynamics and Ventilation of Vehicle Tunnels. Paper J1, 1976.
- Kennedy, WD., Parsons, B. Critical velocity: past, present and future. Paper presented in the One Day Seminar of Smoke and Critical Velocity in Tunnels. London, 2 April 1996.
- Oka, Y., Atkinson, GT. Control of smoke flow in tunnel fires. *Fire Safety Journal*. 1995; 25(4): 305–322.
- Wu, Y., Bakar, MZA. Control of smoke flow in tunnel fires using longitudinal ventilation systems-a study of critical. *Fire Safety Journal*. 2000; 35(4): 363–390.

# Metro tunnels in Buenos Aires: Design and construction procedures 1998–2007

### A.O. Sfriso

Department of Estabilidad, University of Buenos Aires, Argentina

ABSTRACT: The City of Buenos Aires, Argentina, is expanding it's metro network. Some 13 km of new tunnels have been excavated since 1998 and some 20 km are scheduled for construction in the near future. Many major improvements have been implemmented during these years in the fields of design and construction procedures. Some of the achievements and lessons learned are described in this paper, including: characterization of Buenos Aires soils for the numerical modeling of NATM tunneling, description of the design and construction procedures in use and some comments on the observed ground behavior during construction.

### 1 INTRODUCTION

The City of Buenos Aires is extending it's metro network as shown in Figure 1. On-going projects are: Line A, extended 4 km, Line B, extended 4 km; Line E, extended 2 km; and new Line H, 5 km long. Some 20 km of new Lines F, G, I are scheduled for construction in the near future (SBASE 2006).

Landmarks of new construction procedures are: i) introduction of shotcrete, Line B, 1998 (Fig. 2); ii)

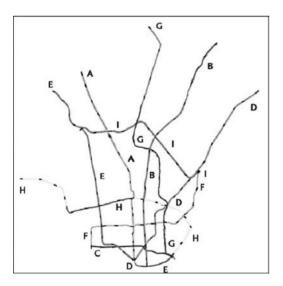


Figure 1. Metro network in Buenos Aires. Existing (A, B, C, D, E, H) and new projects (F, G, I).

belgian tunneling method, Line H, 2000 (Fig. 3); iii) full face excavation, Line B, 2004 (Fig. 4).

Geotechnical and structural analysis techniques evolved concurrently, from earth-load theory to state



Figure 2. First use of shotcrete, Line B.



Figure 3. Belgian tunneling method, Line H, 2000.



Figure 4. Full face excavation, Line B, 2004.

of the art computer simulation of construction procedures and calibration of constitutive models via back analysis of monitoring data (Núñez 1996, Sfriso 1996, 1999, 2006).

### 2 CHARACTERIZATION OF BUENOS AIRES SOILS FOR TUNNELING

Buenos Aires City soils have been described in other contributions (Bolognesi 1975, Fidalgo 1975, Núñez 1986a, 1986b). Briefly, the Pampeano formation underlying Buenos Aires is a modified Loess, overconsolidated by dessication and cemented with calcium carbonate in nodule and matrix impregnation forms. Except for the heaved upper three to six meters, penetration resistance is systematically  $N_{SPT} > 20$  with some heavily cemented zones that exhibit very weak rock behavior with  $N_{SPT} > 50$  (Nuñez 1986b).

The most used site investigation technique in Buenos Aires is SPT penetration using a 2 ½" sampler along with standard lab testing and CTUC testing on recovered samples. Some plate load testing and Menard pressuremeter testing have been recently included as part of the field investigation specifications for metro projects (Sfriso 2006).

### 2.1 Underground construction in the Pampeano Formation

The Pampeano formation is very favourable for underground construction due to its high stiffness, reliable compressive strength, rapid drainage and good frictional behavior when drained.

Two particular characteristics of the formation must be accounted for in the design of underground projects: i) the Pampeano formation is fissured and has lenses of quasi-granular behavior, forcing the installation of a primary support close to the face in order to avoid

Table 1. Assumed in-depth variation of  $K_0$ .

	Depth m	$K_0$ –	
0	to 8/12	0.55–0.70	
8/12	to 20/24	0.65–1.00	
20/24	to 30/32	0.55–0.80	

Table 2. PLT Modulus of subgrade reaction.

_	Depth m	Primary loading MN/m <sup>3</sup>	Un-Reloading MN/m <sup>3</sup>
0	to 8/12	200–300	500-800
8/12	to 12/14	400–600	800-1200
12/14	to 20/24	600–800	1200-1800
20/24	to 30/32	250–500	600-1400

crown overexcavation; and ii) materials drain at a speed comparable to that of the construction.

Due to these factors, the max allowable drift without support is about 2.5 meters. Up to this maximum, the unsupported drift has very little influence on the resulting settlements, as soil behavior remains quasi-elastic (Sfriso 2006, Núñez 2007).

### 2.2 In situ stresses

It is accepted (Bolognesi 1991, Núñez 1986a, 1986b, Sfriso 1999, 2006) that upper Pampeano soils are overconsolidated by dessication to an equivalent pressure 0.8-1.2 MPa. Table 1 lists the assumed in-depth variation of  $K_0$  used for the design of underground structures (Sfriso 2006). These figures have not been actually measured directly but estimated after backanalysis of monitoring data.

### 2.3 Modulus of subgrade reaction

The most reliable information of in-situ stiffness is retrieved via plate loading tests performed in vertical shafts or pilot tunnels. Typical in-depth variation of the modulus of subgrade reaction, as determined by PLT, is listed in Table 2.

### 2.4 Parameters for numerical modelling

Hyperbolic model (Duncan 1970, Vermeer 1998) has been extensively used for the numerical analysis of underground construction in Buenos Aires soils (Sfriso 1999, 2006). After eight years of continuous usage and calibration, a set of input parameters for the Plaxis implemmentation of the hyperbolic model (Vermeer 1998) has been found to best represent

Table 3. Material parameters used for numerical simulations.

	Fill		0-8/12		8/12-20/24		> 20/24	
	min	max	min	max	min	max	min	max
c <sub>u</sub> (KPa)	20	50	50	100	110	220	40	120
φ <sub>u</sub> (°)	8	15	10	20	5	20	0	5
c' (KPa)	0	5	10	25	25	50	15	30
φ' (°)	28	30	28	31	30	34	28	31
ψ (°)	0	0	0	3	0	6	0	3
$E_{50}^{r}$ (MPa)	10	20	60	100	75	150	60	100
E <sup>r</sup> <sub>ur</sub> (MPa)	25	50	150	250	180	300	140	220
m (–)	0	0	0	0	0	0	0	0
v (-)	0.20	0.20	0.20	0.30	0.20	0.30	0.25	0.35
R <sub>f</sub> (-)	0.85	0.90	0.80	0.90	0.80	0.90	0.80	0.90

the observed behavior of tunnels, caverns and open pit excavations. This set is listed in Table 3. Stressstrain relationship of the HSM model is reproduced in Equations 1a to 1d.

$$\sigma_{1} - \sigma_{3} = \begin{cases} \text{load: } 2E_{50} \left( 1 - R_{f} \frac{\sigma_{1} - \sigma_{3}}{\sigma_{3}N_{\phi} + 2c\sqrt{N_{\phi}}} \right) \varepsilon_{1} \\ \text{unload: } E_{w}\varepsilon_{1} \end{cases}$$
(1a)

$$N_{\phi} = \tan^2 \left[ \frac{\pi}{4} + \frac{\phi}{2} \right] \tag{1b}$$

$$E_{ur} = E_{ur}^{r} \left( \frac{\sigma_3 + c \cot[\phi]}{c \cot[\phi] + p_{aim}} \right)^m$$
(1c)

$$E_{50} = E_{50}^{r} \left( \frac{\sigma_3 + c \cot[\phi]}{c \cot[\phi] + p_{atm}} \right)^{m}$$
(1d)

In Equations 1a to 1d and Table 3,  $\sigma_I$  and  $\sigma_3$  are the major and minor principal stresses,  $\varepsilon_I$  is the major principal strain, *c* is either undrained cohesion  $c_u$  or drained cohesion c',  $\phi$  is either undrained friction angle  $\phi_u$  or drained friction angle  $\phi'$ ,  $\psi$  is dilatancy angle,  $E_{50}^r$  and  $E_{ur}^r$  are reference loading/unloading Young's modulus, *m* is stiffness exponent,  $\nu$  is Poisson's ratio and  $R_f$  is the failure ratio.

### **3 CONSTRUCTION PROCEDURES**

### 3.1 Tunnels

Construction procedures evolved from german method (Fig. 2) to belgian method (Fig. 3) and have probably reached an optimal stage with full face excavation (Fig. 4). Figure 5 shows the cross section of a typical two lane, full face tunnel, as used in Lines B and H. A 15 cm unreinforced shotcrete layer and 1.0 m spaced lightweight lattice girders account for the primary support of the tunnel, later supplemented with 30–40 cm of cast-in-place unreinforced concrete.

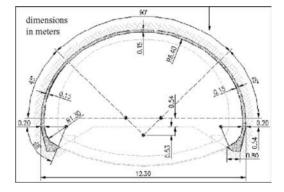


Figure 5. Cross section of a typical two lane, full face tunnel, lines B and H.

The Metro authority requires that tunnels remain dry during operation, thus rendering cast-in place secondary lining as the cheapest option, when compared to membrane barriers and secondary shotcrete lining. Some full face, all-shotcrete sections with impermeabilization barriers have been successfully built in lines H and B.

No closure of the structural ring is needed for stability, and therefore advance rates of 2.5 m-3.5 m per 12 hr shift are consistently achieved. After the tunnel is excavated, a cast in place invert is placed in 5 m-6 msegments, allowing for the placement of the secondary lining in single poured 5 m segments. Figure 6 shows a tunnel after placement of the invert, while Figure 7 shows the formwork being driven into the tunnel.

### 3.2 Underground stations

Underground caverns for metro stations have been built using many techniques including: i) cut & cover slab-on-piles; ii) underground excavated main cavern & open pit excavated upper hall; and, iii) underground excavated main cavern & upper hall.



Figure 6. Tunnel after placement of the invert, Line B.



Figure 7. Formwork used to cast the secondary lining, Line B.

As per 2007, three stations are in excavation stage. The first two are Echeverría Station (Fig. 8) and Villa Urquiza Station, Line B, where the german tunnelling method was used. The third station is Corrientes Station, Line H, where full face excavation is being used (Fig. 9).

Corrientes Station is the latest and more challenging improvement to construction procedures used in Buenos Aires metro tunnelling so far. It is an underground cavern 14.1 m high, 18.9 m wide and 135 m long (Fig. 10). On top of the main cavern, a 6 m high access hall shall be excavated full-face after completion of the secondary lining of the main cavern.

The primary lining of Corrientes Station is formed by 20–40 cm mesh reinforced shotcrete placed in two layers, and 1.0 m spaced lightweight lattice girders. The construction procedure is shown in Figure 11. Full-face excavation is accomplished via a series of four benches, each one 5 m long. Two excavators are permanently set at the two top and two bottom benches, respectively. The bottom bench excavator alternatively lies on soil or on top of the cast-in-place invert, included into the primary support lining to reduce



Figure 8. Echeverría Station, Line B. German tunnelling method.



Figure 9. Corrientes Station, Line H. Full face excavation.

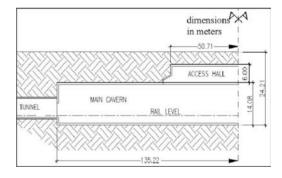


Figure 10. Longitudinal sketch of Corrientes Station, Line H.

costs and time schedule. While excavation of Echeverría Station took some 14 months, it shall took some 6 months to complete Corrientes Station, with advance rates 1.0 m per day.

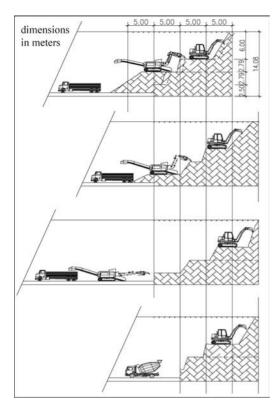


Figure 11. Construction procedure for Corrientes Station, Line H.

### 4 DESIGN PROCEDURES

### 4.1 Primary lining

The preliminary design of the primary lining is largely based on experience. By the time the tunnel shown in Fig. 2 was being analyzed, a simplified design method was developed to estimate forces acting in the crown of the primary support of circular sections (Núñez 1996). The expressions are

$$N = \frac{1}{2} \left( K_0 + \frac{2}{3} \frac{1 - K_0}{1 + a} \right) \frac{2D - A}{3D} p_v D$$
(2a)

$$M = \frac{1 - K_0}{16} \frac{a}{1 + a} \frac{2D - A}{3D} p_v D^2$$
(2b)

$$a = 16 \frac{E_r (1 - v_r^2)}{E(1 - v_r^2)} \left(\frac{e}{D}\right)^3$$
(2c)

where *N* is the normal force at crown, *M* is the flexure moment at crown,  $p_{\nu}$  is the vertical pressure on the crown, *D* is the tunnel diameter, *A* is the unsupported drift, *E*,  $\nu$  are the elastic parameters of the soil mass and  $E_r$ ,  $\nu_r$  are the elastic parameters of the support system. Structural forces obtained with equations 2a, 2b and 2c compare within 10%–15% with those computed using the more involved procedure by Einstein & Schwarz (Einstein 1979).

### 4.2 Simulation of construction procedures

Construction procedures are simulated using 3D elastoplastic models that allow for the estimation of surface settlements, the computation of face stability and the determination of structural forces acting on the primary lining. Structural forces computed with 3D FEM are some 20% lower than those obtained with Eqns. 2a, 2b, 2c. These equations, when applied to the tunnel shown in Fig. 5, resulted in M = 0.55 KNm/m and N = 416 KN/m. 3D numerical models yielded M = 0.47 KNm/m and N = 415 KN/m. (Sfriso 2006, Núñez 2007).

### 4.3 Secondary lining

Metro authority requires that the secondary lining be designed using earth-load procedures and beam on springs analyses. Both primary lining and the effect of construction procedures are disregarded in the design of the secondary lining.

### 5 GROUND BEHAVIOR

Ground behavior has been largely elastic for all construction procedures and underground structures built so far. Disturbance to surrounding structures and facilities has always been minimal, and surface settlements in the range 2 mm–8 mm for tunnels and 4 mm–15 mm for underground caverns have been observed for all construction procedures and soil covers. While this is a desirable behavior from the point of view of construction and safety, it also means that uncertainty of the predictions remain high, because it is unknown how safe the construction procedures really are.

A numerical excercise has been performed to compare the construction procedures for safety and impact to surroundings. A tunnel section 10 m wide, 8 m high with a soil cover of 5 m was used, and the low side parameters listed in Table 3 were adopted. The results are listed in Table 4 (Sfriso 2006). It can be noticed that the german method proved to be the least safe construction method, due to the low safety of the unsupported access tunnels excavated to build the side walls (Fig. 12).

Echeverría Station and Corrientes Station have proven enlightening experiences for the purpose of checking predictions. For safety considerations, it was decided that Echeverría Station be excavated using the german method, and a max. surface settlement of 15 mm was predicted. After a series of small access

Table 4. Numerical comparison between construction procedures for tunnels.

	German	Belgian	Full face
Max. surface settlement, undrained parameters (mm)	4.9	4.3	4.6
Max. surface settlement, drained parameters (mm)	7.4	5.3	6.7
Max. angular distorsion, undrained parameters $(10^{-3})$	0.26	0.22	0.18
Max. angular distorsion, drained parameters $(10^{-3})$	0.30	0.26	0.26
Factor of safety, undrained parameters (–)	2.6	>7	4.7



Figure 12. Unsupported pilot tunnel for side walls, german method of tunnelling.

tunnels were excavated to improve the construction schedule, and before the main cavern excavation was started, a surprisingly high 10 mm settlement was observed at surface.

When the numerical model was re-run with the access tunnels included but without any change in material parameters, the observed surface settlement could be reproduced. It turned out that the unconfinement produced by the too many intersecting small tunnels was responsible for the undesired behavior and yielded a temporarily unsafe condition. It was dediced that the abandoned access tunnels be supported by struts (Fig. 13).

At Corrientes Station, the observed surface settlement 5 mm–8 mm is much lower than the predicted value of 20 mm. After interpretation of the monitoring data, it has been concluded that the unload Young's modulus of Pampeano soils is lower than originally estimated, and that the deposit rebound is partly

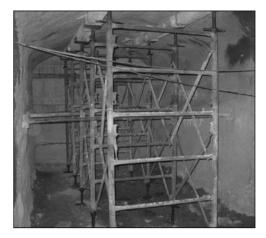


Figure 13. Strut supported access tunnels, Echeverría Station, Line B.

responsible for the small settlements observed. Being the first large closed ring structure ever built in Buenos Aires, Corrientes Station is the first opportunity to properly calibrate the unloading Young's modulus and the effect of soil rebound.

### 6 REMAINING CHALLENGES

The advancement in design and construction procedures is an endless activity. Despite the efficient methods actually in use, some remaining challenges need to be addressed in the near future. These are: i) the implemmentation of a reliable procedure to measure  $K_0$ ; ii) the abandonment of cast-in-place concrete and "dry" tunnels; iii) the use of robot-placed, fiber reinforced shotcrete; iv) the implemmentation of more advanced topographic guiding systems; v) optimizations in the usage of lattice girders; and vi) better control of ground water during construction.

### 7 CONCLUSIONS

13 km of metro tunnels have been excavated in Buenos Aires in the period 1998–2007. Construction procedures in 2007 include shotcrete and full face excavation both in tunnels and caverns, while design procedures include state of the art numerical simulation of construction processes. Best fit parameters for the constitutive models used were introduced and some observed features of soil behavior have been described.

The Pampeano formation underlying Buenos Aires City is very favourable for underground excavation due to its high stiffness, reliable compressive strength, rapid drainage and good frictional behavior when drained. Ground behavior has been largely elastic for all construction procedures and underground structures built so far. Disturbance to surrounding structures and facilities has always been minimal, and surface settlements in the range 2 mm–8 mm for tunnels and 4 mm–15 mm for underground caverns have been observed for all construction procedures and soil covers.

Corrientes Station is the latest improvement to construction procedures used in Buenos Aires metro tunnelling so far. An underground cavern 135 m long shall be completely excavated in six months with surface settlements less than 10 mm and minimal disturbance to surroundings.

### ACKNOWLEDGEMENTS

The working teams from the companies involved in these projects, Techint, Dycasa, Roggio and SBASE, shared their impressions, opinions and experience with the writer, allowing him to learn many things from them. Eduardo Núñez taught the writer the basics, the involved theories and the tricks of tunnelling, carefully supervised the writer's early works, and remains as the day-by-day writer's source for expert advice.

### REFERENCES

Bolognesi, A. 1975. Compresibilidad de los suelos de la Formación Pampeano. V PCSMFE, Argentina, V: 255–302.

- Bolognesi, A. and Vardé, O. 1991. Subterráneos en Buenos Aires. IX PCSMFE, Chile, III:1329–1350.
- Duncan, J. and Chang, C. 1970. Nonlinear analysis of stress and strain in soils. JSMFD, ASCE, 96, SM5, 1629–1653.
- Einstein, H. and Schwartz, C. 1979. Simplified analysis for tunnel supports. *JGED*, ASCE, 105, 4, 499–518.
- Fidalgo, F., De Francesco, F. and Pascual, R. 1975. Geología superficial de la llanura Bonaerense". VIArg. Geol. Conf, 110–147.
- Núñez, E. 1986a. Panel Report: Geotechnical conditions in Buenos Aires City. V Conf. IAEG, 2623–2630.
- Núñez, E. and Mucucci, C. 1986b. Cemented preconsolidated soils as very weak rocks. V Conf. IAEG, 403–410.
- Núñez, E. 1996. Túneles de sección circular en la formación pampeano. *bull. SAMS*, 29, 1–15.
- Núñez, E. 2007. Casagrande lecture: Uncertainties and Approximations in Geotechnics.XIII PCSMGE, Venezuela. In press.
- SBASE, 2006. Institutional web page. www.sbase.com.ar.
- Sfriso, A., 1996. Túneles de sección circular en la formación Pampeano – comentario. *bull. SAMS*, No. 29, 16–19.
- Sfriso, A. 1999. Tunnels in Buenos Aires: Application of numerical methods to the structural design of linings. XI PCSMGE, Brasil, 637–642.
- Sfriso, A. 2006, Algunos procedimientos constructivos para la ejecución de túneles urbanos. XIII CAMSIG, Argentina, 1–17.
- Vermeer, P. 1998. Plaxis Users Manual. Balkema, Rotterdam. Ne, 577 p.

# Study on the earth pressure distribution of excavation chamber in EPB tunneling

T.T. Song & S.H. Zhou

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. Chnia

ABSTRACT: In shield tunneling, the excavation stability is very important. The balance pressure which supports the work face is offered by the the mucks in the earth chamber. The ideal supporting pressure on the excavation face is trapezoidal. In fact, the pressure is irregular. So, the conceptions of earth pressure supporting ratio (*EPSR*), regular modulus of earth pressure and buffer ability of chamber are put forward. Earth pressures in two different situations of shield tunneling in clay ground and cobble sand ground are studied. The studies show that EPSR and regular modulus of earth pressure in soft ground are both better than those in cobble ground. EPSR and regular modulus of earth pressure are two appropriate assessing indexes for EPB tunneling.

### 1 INTRODUCTION

With the advantages of surrounding influence, excavation speed, structure quality, working environment etc., the shield tunneling method is popularly applied in the tunnels of metro, railway, road, municipal engineering and so on.

Earth pressure balance machine (EPB) and slurry shield tunneling are the two mostly common method in metro tunnels. Comparing with slurry shield method, the EPB has the merits of small construction yard, less cost, simple technology. So the EPB is more popular in metro tunnels.

EPB machine applies pressure to working face by the mucks which are cut from the face sometimes with injection of soil conditioning additives. With the pressure the working face can maintain stable. The stability of working face is a key factor in the EPB tunneling. The accidents brought by destabilization of working face are the main accidents according to statistics (Qin Jianshe, 2005).

Present researches about the shield tunneling pressure mainly focus on the theoretical pressure needed for the working face (Anagnostou G, Kov'ari K., 1996; Abdul-Hamid Soubral, 2000 & Qin Jianshe, 2005). As for how to apply the pressure and the pressure properties are seldom mentioned and researched. In this research, features of working face pressure, distribution of pressure in the excavation chamber and the pressure in the clay and cobble sand strata are researched. Some conceptions are put forward, and principles are summarized.

### 2 CLASSICAL EPB THEORY

The excavating system of EPB machine is composed by shield, cutter head embedded with cutters, working chamber, pressure wall, screw conveyor, and thrust cylinders (see Fig 1). When excavation, thrust cylinders apply force to pressure wall, and the pressure wall applies the pressure to the working face by the mucks in the earth chamber. The pressure is used to balance the water and soil pressure in ground.

EPB machine adjusts pressure to balance water and soil pressure in working face by the manners of changing the pressure of thrust cylinders and the rotation speed of screw conveyor (see Figure 1). When it is necessary to increase the pressure in earth chamber, the rotation speed should be decreased and the thrust force should be increase vice versa.

Classical earth pressure assumes these:

- 1. The pressure applied by shield machine to the working face is trapezoidal. And the trapezoidal pressure balances with the water and soil pressure in working face in order to maintain facing balance.
- 2. The pressure applied to the ground is equal to the pressure in the pressure wall (M. Herrenknecht & U. Rehm, Aug, 2003). So, the working face pressure is acquired through the pressure values shown in the operating panel.

Obviously, the conditions for classical earth pressure are satisfied with the followings:

1. The mucks in earth chamber should have the properties of ideal plastic fluidity.

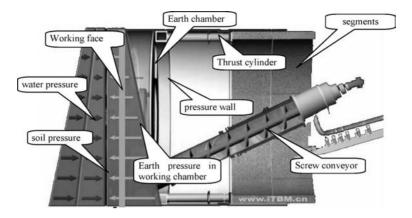


Figure 1. Soil pressure + water pressure = earth pressure in working chamber (Wassmer, Treceno & ANdreossi, 2001).

- 2. The opening rate of cutter head is enough.
- 3. There is no pressure loss in the screw conveyor.

In fact, EPB can't reach this ideal state in the practice of excavation.

#### 3 DEFINITIONS RELATED EARTH PRESSURE

The pressure applied by shield machine is related to the stability of working face directly. In practice, it's hard to observe and gauge the pressure of the working face. So, some indirect ways to estimate the pressure of working face, as below:

- 1 The pressure sensor in the pressure wall. This way is thought as the most efficient way. But it has difference between the two pressures.
- 2 The force of thrust cylinders and the torque of cutter head. They increases with the earth pressure increase;
- 3 The volume of mucks discharge. More discharge, less earth pressure.

For further research, the following definitions are given:

#### 3.1 Earth pressure supporting ratio (EPSR)

The classical earth pressure theory assumes that the supporting pressure on the working face is supplied by earth in the earth chamber (see Fig 1). In fact, the supporting pressure is composed by two parts: earth in working chamber and the plane of cutter head. As to the Figure 2, the earth pressure is mainly transferred though the open of cutter head. But the plane also contributes supporting pressure. So the earth pressure supporting ratio (*EPSR*) is defined as the ratio of earth pressure in the total supporting pressure.

$$EPSR = \frac{\text{earth supporting pressure}}{\text{total supporting pressure}}$$
(1)

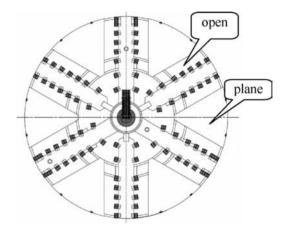


Figure 2. Schematic plan of open and plane.

where EPSR = Earth Pressure Supporting Ratio; earth supporting pressure: the total pressure of earth pressure on the working face;

total supporting pressure: the total supporting pressure on working face including earth pressure and cutterhead plane supporting pressure;

The total supporting pressure is assumed the balance pressure on the working face.

The ratio is related to the shape of cutter head, opening rate and the control of earth pressure. The value of the ratio is between 0 and 1 which indicates the state of earth control when excavating. For example, the ratio of cutter head of spoke shape is bigger than that of plane shape.

#### 3.2 *Regular modulus of earth pressure (RMEP)*

Five pressure sensors are located on the pressure wall (see Fig 1). The position of the sensors are shown in Figure 3, and the height is shown in Table 1.

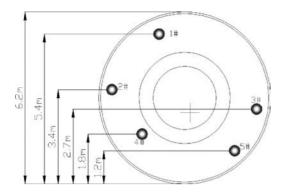


Figure 3. The Distribution of earth pressure sensors.

Table 1. Position of the sensors.

Sensor No.	1	2	3	4	5
Position(m)	1.2	1.8	2.7	3.4	5.4
r osition(iii)	1.2	1.0	2.7	5.1	5.1

The regular modulus of earth pressure *RMEP* is used for indicating the regularity of the earth pressure in working chamber. The value of *RMEP* is defined as related coefficients:  $R^2$  between values of earth pressure and the corresponding vertical position. The value of the modulus is between 0 and 1 which indicates the fitting extent of the fitted regression line. The ideal value is 1. The value is closer to 1, better regular of the earth pressure is.

$$R^2 = 1 - \frac{SSE}{SST}$$
(2)

where

$$SSE = \sum (Y_i - \hat{Y}_i)^2$$
(3)

$$SST = (\sum Y_i^2) - \frac{(\sum Y_i)^2}{n}$$
(4)

 $Y_i$ : actual data points of earth pressure

 $\hat{Y}_i$ : predicted value of the regression model

The detail definition of  $R^2$  can referred Richard A. Johnson (1992).

#### 3.3 Buffer ability of working chamber

It can't keep the balance pressure when the mucks in the working chamber are over discharged due to improper operation. The volume of the working chamber should have the ability of maintaining the pressure due to improper discharging operation. So the chamber should buffer the earth pressure when the mucks are sudden loss.

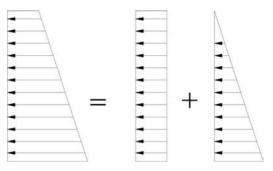


Figure 4. Total pressure makeup: additive pressure of facing + lateral pressure of mucks.

Here, the buffer ability of chamber is defined as the proportion of discharged mucks in the whole chamber at the condition of screw conveyor working for 5 minutes with maximum rotation speed.

$$buffer ability = \frac{discharge vol. in 5 min.}{vol. of working chamber}$$
(5)

It is deemed that the buffer ability is good if the proportion of discharge mucks is less than 30%.

#### 4 PROPERTIES OF EARTH PRESSURE OF EPB EXCAVATION

#### 4.1 Analysis of pressure makeup

The pressure wall is in the behind of working chamber. The wall undertakes the main face pressure. Here the series S type EPB shield machine of Herrenknecht are take an example to research the wall pressure. The earth pressure sensors in the wall are distributed as the Figure 3. There are five sensors in the wall. The serial number and the position are shown in Figure 3.

The measured pressure shape is trapezoidal in most common situation (see Fig 1 & Fig 4). The trapezoidal pressure is composed of two parts: one is rectangular and another is triangular. The triangular can be deemed as the lateral pressure of the mucks, and the rectangular can be deemed as the additive pressure applied by the water and soil pressure of the working face.

Analysis hereinbefore, there won't be additive pressure if the pressure shape is triangular. At this situation, if the thrust force is big and the face is stable, the cutter head will undertake large portion of face pressure. So, the earth pressure support ratio *EPSR* is low which lead to the large contact stress between cutter head and face and large torque of cutter wheel. Usually, in this situation, the wear of cutters and cutter head are serious.

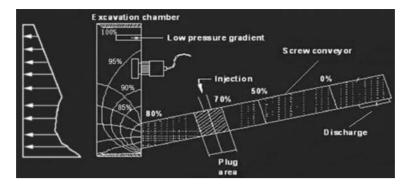


Figure 5. Earth pressure distribution diagram (Herrenknecht & Rehm, 2003).

Table 2. The ground parameters of Guangzhou.

$\rho$ (g/cm <sup>3</sup> )	w(%)	e	c(kPa)	$\Phi(^{\circ})$	Es1-2(MPa)
1.95	31	0.8	22.8	18.1	4.7

#### 4.2 Pressure properties in clay

Usually, the mucks of clay strata have a good plastic fluidity which can apply a regular pressure to the excavation face. So, the earth pressure supporting ratio is high; and the modulus of the earth pressure is high too. The stability of working face can be well controlled if the working chamber satisfies the mucks buffer.

Nevertheless, there is pressure loss in the screw conveyor in this kind of strata. The pressure gradient curve is shown in Fig 4. It is inevitable that pressure falls in the screw conveyor because the pressure is 0 in the discharging outlet. The pressure gradient depends on the fluidity and impermeability of the mucks. The pressure in the inlet of screw conveyor has fallen to 80% (see Fig 5). It indicates that the pressure isn't regular due the existence of screw conveyor though the mucks is of plastic fluidity. (see left in Fig 5)

The excavation data of ring No. 219 in one section of line 5 of Guangzhou metro are taken for an example. The cutter head of the EPB machine is plane type, and opening rate is about 28%. There are two types of cutters: disc cutter and scrape cutter. The length of working chamber is 1 m. The position of earth pressure sensors are shown in Figure 3. The tunnel is in the stratum of <4-2> whose main parameters are shown in Table 2.

The Figure 6 is the figure of earth pressure data which are acquired from the earth pressure sensors in pressure wall. The number order of the pressure sensors is from the top down. They are 1#, 2#, 3#, 4# and 5#. From the data it can obtain that the regular modulus of earth pressure *RMEP* is about 0.98. This value shows the earth pressure of pressure wall is more regular.

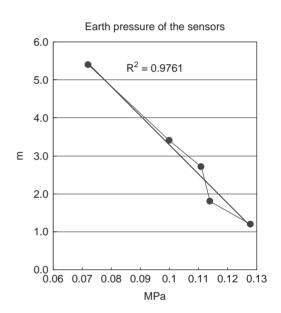


Figure 6. Mean earth pressure in the pressure wall.

Table 3. Assessment of earth pressure supporting ratio.

A /	Additive stress/ Max. stress				
Actual torque/ Max. torque	0~30%	30~50%	>50%		
<0.8	< 0.5	$0.5 \sim 0.8$	>0.8		
>0.8	< 0.3	$0.3 \sim 0.5$	>0.5		
<1.1					
>1.1	< 0.1	$0.1 \sim 0.3$	$0.3 \sim 05$		

The proportion of earth pressure and cutter head is hard to be divided. The earth pressure supporting ratio can't be calculated precisely. So, it can be judged by the earth pressure in the pressure wall and the torque of cutter head by the following way. The additive

Machiner type	Excavation diameter(m)	Rating torque(KN·m)	Max. thrust(kN)	Max. speed(mm/min)	Opening ratio	Cutter head
EPB TBM	6.28	5980	34210	80	28%	disk and scrape

Table 4. Main index of the EPB (S365).

Table 5. The indexes of the stratum.

$\rho(g/cm^3)$	w(%)	c(kPa)	$\Phi(^{\circ})$	E <sub>0</sub> (MPa)	K(m/d)
2.2	31%	0~1	38	45	27

pressure can be known from the pressure wall. Because the additive pressure is applied by face, higher additive pressure, higher earth pressure supporting ratio. If the cutter head plate undertakes more facing pressure the torque of the cutter head will become bigger. According to empirical summary and modification, the method using additive pressure and torque to estimate earth pressure supporting ratio is put forward. (shown in Table 3)

The actual torque of this ring is 3650 kNm while the rating torque is 4500 kNm. The value of actual torque/rating torque is 0.81; and the value of additive stress/max. stress is 0.32. So according to the earth pressure and the torque and assessing by the Table 3, the *EPSR* is about 0.65.

#### 4.3 Pressure properties in cobble sand strata

The line 1 of Chengdu metro is in strata of cobble and sand. The EPB machine is  $\Phi 6.28 \text{ m}$  of Herrenknt. The main parameters of the machine are shown in Table 4.

From the analysis hereinbefore, the *EPSR* is about  $0.6 \sim 0.7$  in the soft clay strata. And in most of the situations the EPB tunneling can't reach total earth balance in working face. The values of *EPSR*s are usually controlled bigger than 0.5. The main balance pressure is earth in this stratum. The regularity of the earth pressure is good. The regular modulus of earth pressure reaches 0.98.

The tunnel line is in the strata of cobble and sand. The cobble content is high and the permeability coefficient is high. The main indexes of strata <3 - 7> are listed in Table 5.

The pressure data in the Figure 7 are collected in the ring of 112 which is the mean value in the ring. And the torque of cutter head is 3020 kNm in this ring. The *EPSR* is about 0.4 from the Table 3 and Figure 7.

The regular modulus of earth pressure *RMEP* in the chamber is 0.93 which is less than the value in clay. The pressure gradient can reflect the muck unit weight. The gradient value is the product of unit weight and lateral

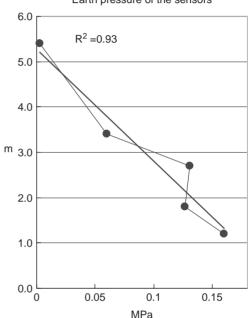


Figure 7. Mean earth pressure in the pressure wall.

pressure coefficient. Due to the irregularity of earth press in cobble and sand strata, it's hard to calculate the unit weight. However, it shows the unit weight is bigger according to Figure 6 and Figure 7.

The data show that the *EPSR* is low. So the cutter head plate will undertake bigger pressure. Low *EPSR* is unfavorable for wear of cutters and cutter head. So the *EPSR* should be increased by taking some measures. Such as soil conditioning should be taken to increase the mucks' plastic fluidity. Simultaneously, the regular modulus of earth pressure can be increased.

#### 4.4 Buffer ability of working chamber

The discharging ability of the screw conveyor is as below:

$$Q = \eta \frac{\pi}{4} (D_1^2 - D_2^2) Pn$$
 (6)

where: $\eta$ - discharging efficiency, if there is no loss,  $\eta = 1$ . In common  $\eta$  is about 0.9;  $D_1$ - inner diameter of screw conveyor;  $D_2$ - outer diameter of center shaft; P- Distance of paddle; n- Rotation speed.

According to formula (6) the maximal discharging ability of screw conveyor is  $1.2 \text{ m}^3$ /min. The volume of discharging is 6 m<sup>3</sup> in 5 min. The cubage of the chamber is about 30 m<sup>3</sup>. According to formula (5), the buffer ability is 20% (6/30 = 20%) of the cubage of chamber (shown in formula 7).

buffer ability = 
$$\frac{1.2*5}{30}$$
\*100% = 20% (7)

According to calculation, 30% loss of working chamber volume can cause big ground settlement. The bigger buffer proportion can cause more ground loss and instability of working face. If the proportion is over 30%, it can be deemed that the buffer ability of the chamber is not enough. If it is below 30% it can be deemed that the chamber has the ability of earth pressure buffer for improper operation.

#### 5 CONCLUSIONS

From the analysis and study of the theory and measured data, the main conclusions are as below:

- 1 The classical earth pressure theory has limited and premises.
- 2 The pressure in the pressure wall is divided into two parts: additive pressure of working face and lateral pressure of mucks.
- 3 The conceptions of earth pressure supporting ratio, regular modulus of working chamber and buffer ability of chamber are put forward to research the pressure properties.
- 4 Clay stratum and cobble sand stratum are taken for examples to study.

- 5 The research shows that the actual pressure can't reach the situation of classical pressure.
- 6 *EPSR* and regular modulus in clay are both higher than in cobble and sand. So the invalid wear is higher in cobble and sand strata.

#### REFERENCES

- Abdul-Hamid Soubral 2000. Three dimensional face stability analysis of shallow circular tunnels. *International Conference on Geotechnical and Geological Engineering*, 2000. Australia: Melbourne.
- Anagnostou G & Kov'ari K. 1996. Face stability conditions earth pressure balanced shields. *Tunneling and Underground Space Technology*11(2):165–173.
- Herrenknecht, M. & Rehm, U. 2003. Earth Pressure Balanced Shield Technology. *Internal Lecture in Colorado School* of Mine, USA.
- Hisatake M. & Eto T Murakami T. 1995. Stability and failure mechanisms of a tunnel face with a shallow depth. Proceedings of the 8th Congress of the International Society for Rock Mechanics. Japan: Fujii.
- Johnson, R.A. & Bhattacharyya, G.K. 1992. Statistics: principles and methods. New York : Wiley.
- Qin, J.S. 2005. study on mechanism of the face deformation and failure in shield tunneling. *PH.D thesis, University of Hohai*.China: Nanjing.
- Shikibu, N. 1995. Stability of face during shield tunneling— A survey of Japanese shield tunneling 1995. In: Fujita, Kusakabe. Underground Construction in Soft Ground. Rotterdam: Balkema.
- Stallebrass, S. E., Grant, R.J. & Taylor, R.N. 1996. A Finite element study of ground movements measured in centrifuge model tests of tunnels. *Geotechnical Aspects of Underground Construction in Soft Ground, 1996.* Rotterdam: Balkema.
- Wassmer, L., Treceno, O. & ANdreossi, E. 2001. Tunnel boring machine(TBM) Application in Soft Ground Condition. *I.M.I.A. Meeting 2001*, Australia: Sydney.

### Backfill grouting research at Groene Hart Tunnel

#### A.M. Talmon

Deltares (Delft Hydraulics) & Delft University of Technology, Delft, The Netherlands

#### A. Bezuijen

Deltares (GeoDelft) & Delft University of Technology, Delft, The Netherlands

ABSTRACT: Measurements on back-fill grouting at Groene Hart Tunnel (GHT) are described and analysed. The research utilises TBM data, monitored grout pressures and measured rheological properties of tunnelling fluids. Grout injection strategy and vertical grout pressure distribution behind the TBM were found to agree with established theory, like found in other tunnelling projects. However, on some occasions different smaller vertical grout pressure gradients were found, which might have been caused by an incidental upward movement of the tunnel. It is shown that the consolidation of grout determines the decay of grout pressures. The associated net hydraulic resistance of surrounding soil is substantially higher than according to radial flow theory. This higher resistance is attributed to bentonite originating from the face of the TBM, having invaded the grout-soil interface.

#### 1 INTRODUCTION

The Groene Hart Tunnel (GHT) in the Netherlands is part of a high speed railway line between Amsterdam and Brussels. The location of the tunnel is about 20 kilometers South-West of Amsterdam. The length of the tunnel is 7.16 km. The tunnel is situated at a depth of about 30 meters. Hydro-geotechnical conditions are described by Aime et al. 2004. This double track tunnel has a diameter of 14.5 meters. Aristaghes et al. 2002 reported on the design of the tunnel.

Grouting research at GHT was conducted as a cooperation between Delft Institutes, HSL project organisation and COB-research. The study focused on the interaction of soil, grout and tunnel construction at the injection of grout. Three specific aims were: 1) Analysis of grout pressures measured at GHT. 2) Quantification of the consolidation of grout and its influence on grout pressures. 3) Modelling of the grout pressure distribution; specifically from grout injection at the TBM to the buoyancy dominated region further behind the TBM. Novel conditions at the time were the impressive diameter of the tunnel: 14.5 m, in contrast to the 10 m diameter tunnels monitored up to then, the critical hydro-geotechnical soft soil conditions and a different type of grout. The geotechnical profile is shown in Fig.1.

Laboratory characterisations were conducted on the grout: vane testing for rheological properties, consolidation experiments, and tests to determine to

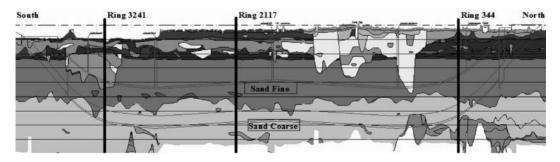


Figure 1. Geotechnical profile Groene Hart Tunnel and measurement locations: Instrumented plot no 1 at ring 344 (29/03/2002), COB-passage at ring 2117 (03/06/2003) and Noordplaspolder at ring 3241 (05/11/2003).

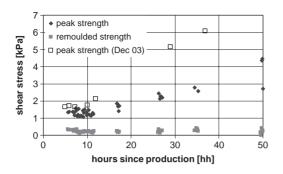


Figure 2. Peak shear strength and remoulded strength of GHT tail void grout, determined by vane testing.

what degree of grout consolidation the grout pressure transducers used in the lining will give reliable results. Also the rheological and consolidation properties of bentonite that could surround the grout were measured.

The construction of Groene Hart Tunnel was monitored on a number of locations. Three locations were available for Delft Institutes research: "Instrumented plot no 1", a so-called "COB-passage" and a passage in "Noordplaspolder", see Figure 1.

In each of these locations grout pressures on the tunnel lining were measured. The COB-passage had additional instrumentation: pore water pressure measurements, axial and tangential strain measurement in the tunnel lining, convergence measurements, vertical position measurements of the tunnel lining and tilt measurements of the tunnel lining. This paper focuses on grout pressures.

#### 2 PROPERTIES OF TUNNELING GROUT

#### 2.1 Rheology of grout

Yield stresses are governing grout pressures behind a TBM. Shirlaw et al. (2004) describe the ingredients of the cement-less grout employed at Groene Hart Tunnel (GHT): sand, flyash, lime and chemical additives. They also address some of its physical properties. Fresh grout samples were taken (27/03/2002) from a supply container located in the TBM. The grout was tested in Delft. The density of the grout is 1850 kg/m<sup>3</sup>, the water content is 0.201.

Peak shear strength and remoulded shear strength were measured with a vane apparatus as a function of time. Figure 2 shows the results. The peak shear strength of fresh grout is about 1 to 1.5 kPa. The tests also show that peak shear stresses increase slowly in time. This is caused by puzzolanic reaction of the flyash present in the grout. The remoulded shear strength, on the contrary, is nearly constant in time. The grout was fine grained, though no sieve tests were conducted for quantification. Similar tests were conducted on grout collected in December 2003. Also the shear modulus of grout was determined. This property is needed to calculate the transition to elastic behaviour (=input to DCLong: Talmon & Bezuijen 2005).

The peak shear strength, instead of the remoulded strength, is here the relevant parameter, since through continued consolidation the failure surfaces are closed continuously, and it is here that the peak strength has to be surmounted continuously.

#### 2.2 Consolidation of grout

In porous soil conditions elevated grout pressures, compared to the pore water pressure at the same location, lead to consolidation of grout, or grout bleeding. The consolidation of grout, in turn, influences the effective stress distribution some distance behind the TBM. Consolidation starts at the grout-soil interface. and proceeds slowly into the grout layer that fills the tail void (this layer is called a grout cake). Consolidation properties of GHT grout were determined in Delft by means of element tests such as described by Bezuijen & Talmon (2003). Permeability and porosity change of the grout are the governing parameters. Relevant results to the calculation of grout consolidation in the tail void are: permeability grout cake  $k = 2.4 \times 10^{-8}$  [m/s], porosity fresh grout  $n_i = 0.31$  and porosity of the consolidated grout cake  $n_e = 0.24$ .

#### 3 LABORATORY TESTING OF GROUT PRESSURE SENSORS

Pressure sensors (SenSym/ICT 19C 100 P) have been mounted in the tunnel lining segments. A cavity (D = 82 mm, h = 23 mm) filled with tail brush grease separated the sensor membrane from the grout (this construction is comparable to that at Sophia Rail Tunnel, which is described by Bezuijen et al. 2004).

Since at the GHT beam action of the tunnel lining was the primary subject of the COB-research, it had to be sure that the grout pressure data was reliable, specifically under consolidated/hardened conditions of the grout. The functioning of these grout pressure sensors was tested in the laboratory, Bezuijen (2004). A sketch of the test cell is shown in Figure 3. A pressure sensor was placed in a grease filled cavity position, as at GHT.

The grout in the test cell was loaded by increasing the air pressure in the top of the cell. The resulting grout pressure was the same as in the field.

Up to about one hour, this sensor adequately measured externally imposed variations of total pressure, see Figure 4. After that the pressure readings were in

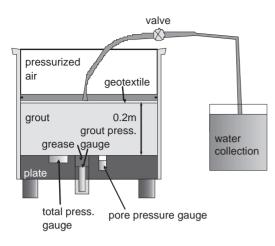


Figure 3. Laboratory testing of grout pressure sensor.

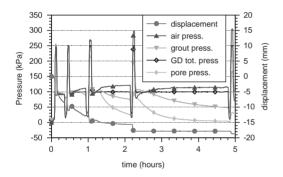


Figure 4. Example of pressure sensor response.

between the imposed total pressure and the pore-water pressure.

During this process another flush mounted pressure sensor with very stiff membrane, continued to measure the total stresses. It is concluded that after one hour consolidated grout reached the pressure sensor. Subsequently grout-arching might have occurred in front of the GHT pressure sensor when loaded dynamically. As a consequence, pressure readings at GHT have to be interpreted with care. It should however be mentioned that these tests might have been too stringent; the imposed pressure variations are not likely to occur in the field. Instead there is a slow decay of grout pressure with time, given arching less chance to occur. Furthermore the sensor is located at the impermeable lining. In Section 6 it will be shown that in a tunnelling process it can take more than 25 hours before the grout close to the lining is consolidated

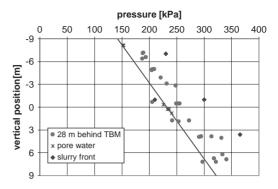


Figure 5. Pressures measured on tunnel lining at 28 m behind TBM.

#### 4 GROUT PRESSURES MEASURED AT GHT

### 4.1 Measured grout pressures at "Instrumented plot no.1"

Twelve grout pressure sensors were utilised in "Instrumented plot no 1". The time-series were relatively short: the data covered at max 3 rings behind the TBM. The wireless FM-data transfer of the sensors suffered interruptions and malfunctioning.

The grout was injected through six injection pipes. The pressure drop over the pipes was about 3 bar. This pressure drop is equivalent to a wall shear stress of 1 kPa. This is significantly higher than found for cementious grouts such as at Sophia Rail Tunnel and Botlek Rail Tunnel, where about 0.2 kPa was found (Talmon et al. 2001).

Pump-stroke counts showed that about 60% of the grout was injected through the three injection ports in the upper half of the TBM. Thus a net downstream of grout is created at the back of the TBM.

Despite the sparse set of tail void pressure data, a picture emerged that the vertical grout pressure gradient behind the TBM ( $\leq 10 \text{ kPa/m}$ ) could have been rather small in comparison to earlier tunnelling projects such as Sophia Rail Tunnel (Bezuijen et al. 2004).

#### 4.2 Measured grout pressures at COB passage

At COB passage, tail void grout pressures were measured by a total of 32 pressure sensors. Ring 2117 and 2118 were each circumferentially equipped with 10 pressure sensors. The remaining 16 sensors were equally distributed over ring 2119, 2120 and 2121.

The grout pressure profile, 28 m behind the TBM, is shown in Figure 5. At this point it approaches the hydrostatic pressure distribution. Immediately behind the TBM, measured pressures are about 100 kPa above pore water pressure. Only two sensors located in the

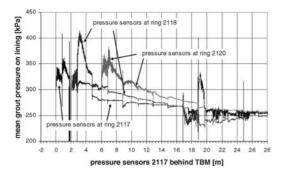


Figure 6. Mean grout pressure of ring 2117, 2118 and 2120.

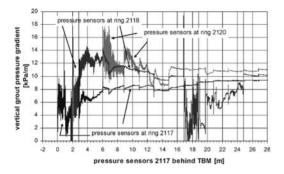


Figure 7. Vertical grout pressure gradients measured at ring 2117, 2118 and 2120. Between 16 and 25 metres electronics did not function correctly.

lower half of the tunnel, strangely, showed pressures equal to pore water pressure.

Average grout pressures measured at ring 2117, 2118 and 2120 are shown in Figure 6. It shows that grout pressures decay with distance from the TBM, in correspondence with the fundamental processes described in Talmon & Bezuijen (2005).

The pressure distribution over the circumference of the tunnel lining is determined by specific weight of the grout and flow resistance when the grout is distributed around the lining. The variation of the vertical grout pressure gradient with distance behind the TBM is shown in Figure 7. This figure shows that at upon exiting of ring 2117 this vertical pressure gradient is rather small: 6 kPa/m. This implies that there has been a down-flow of grout with a high flow resistance.

Measured vertical displacements of the tunnel lining (measured by a water levelling system measuring vertical displacements between ring 2117, 2118 and ring 2057) indicate that the tunnel lining had moved to a high position before the pressure sensors came into the grout. Before exiting of Ring 2117, the lining had already moved upward some 6 cm, see Figure 8. Ring 2057, located 120 m behind the TBM served as a reference.

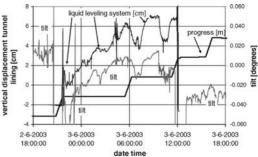


Figure 8. Vertical position of ring 2117 measured by a liquid leveling system and tilt of ring 2117 measured when the grout sensors first came into the grout (3-6-2003 06:00).

Ring 2117 and 2118 were also equiped with tilt sensors that were attached to tunnel ring segments situated at tunnel axis height. The time trace of the tilt meters varied in concert with vertical positions measured with the liquid levelling system. Near the end of drilling for ring 2120 (3-6-2003 12:00), the liquid levelling system had to be dismantled, the tilt measuring system continued measuring though. The tilt-measuring system showed that after the drilling of ring 2120, the tilt had dropped 0.04 degree, which according to Figure 8 corresponds to a vertical downward movement of about 6 centimetres since ring 2117 and 2118 came into the grout.

Overviewing the data, it is concluded that when ring 2117 and 2118 came into the grout the tunnel lining was at a temporally high position. The course of tilt meter data, vertical pressure gradient and vertical position correspond qualitatively. A representative vertical grout pressure gradient at the back of the TBM is 12 kPa/m.

#### 4.3 Measured grout pressures at Noordplaspolder

The data from Noordplaspolder produced a more familiar picture: vertical grout pressure gradients of about 15 kPa/m immediately behind the TBM and a vertical gradient of about 8 kPa/m at 17 meters behind the TBM, see Figure 9.

#### 5 ANALYSIS OF GROUT PRESSURES BEHIND THE TBM

A finite difference model for the calculation of grout pressures immediately behind a TBM was first presented by Talmon et al. (2001). The model calculates the distribution of grout issued from the injection ports, and consequently predicts the pressure distribution directly behind the TBM. A tail void of constant thickness is assumed and the computational domain stretches for about 5 meters behind the TBM.

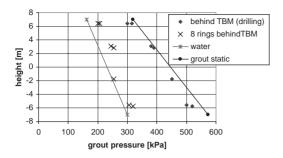


Figure 9. Vertical grout pressure profiles measured at ring 3241 of Noordplaspolder.

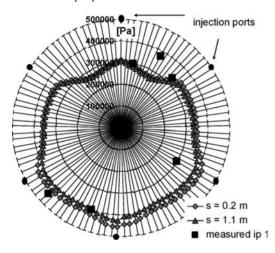


Figure 10. Grout pressures measured up to 1 m behind the TBM at "Instrumented plot no 1" and calculated pressures (model DCgrout). s = axial coordinate. Two-sided friction grout layer:  $\tau_y = 1.5$  kPa, 60% of grout is injected through injection ports in the upper half of the TBM.

When the results of the "Instrumented plot no 1" came available a calculation of the grout pressure distribution immediately behind the TBM was made, see Figure 10. Inputs were the vane test results of March 2002 and the measured distribution of flow rates over the six grout-injection pipes.

Only if two-sided friction of the grout layer is assumed, like in the original model, a fair correspondence with the measurements is achieved. A pressure difference of nearly 150 kPa is calculated between the bottom and the crest of the tunnel. This pressure difference is equivalent to a vertical grout pressure gradient of 10 kPa/m. For reference, self-weight of the grout would produce a vertical pressure gradient of 18.1 kPa/m. For 300 kPa at the crest, this would lead to a pressure of 562 kPa at the bottom of the tunnel.

When first data of the COB-passage came available it was found that the distribution of flow rates over the injection ports was quite comparable: 55% of the grout was injected through the three upper ports. The grout pressure data (ring 2117) showed a small vertical grout pressure gradient of about 4 a 6 kPa/m, see Figure 7. This could not be explained by means of the DCgrout model. The measured vertical pressure gradient more is compatible with a situation where all of the grout is forced downwards. In that case the vertical pressure gradient is given by:  $18.1-2*\tau_v/h = 3.1$  kPa/m.

At ring 2118 and 2120 a vertical grout pressure gradient of about 12 kPa/m is measured behind the TBM. Here the results correspond more with a situation where grout distributes in both the up- as downward direction from the injection ports.

The pace of the construction cycle at COB-Passage was slowed down by installation of instrumentation.

The measured pressure distribution at Noordplaspolder is more in line with earlier experience at other tunnels: f.i. Sophia Rail Tunnel (Bezuijen et al 2004).

Given the rheological properties of grout, the measured vertical grout pressure gradients can only be achieved when there is friction on both sides of the grout layer: friction at the soil-grout interface and friction at the tunnel-grout interface. Given the high frictional resistance measured over the supply lines, it seems justified to conclude that this grout is providing significant friction with the concrete tunnel lining (contrary to cementious grout employed in previous projects).

It has been found that the pressure distribution immediately behind the TBM varies strongly between measurement locations. A likely candidate that might have affected the pressure distribution is vertical movement of the tunnel lining. The higher yield stress of the grout used, compared to the grout in the earlier tunnels, has as a consequence that the movements of the lining have a larger influence on the pressure distribution around the lining.

#### 6 CALCULATION OF THE CONSOLIDATION OF GROUT

Pressurized grout situated against a permeable surface is subject to fluid loss. Two different situations can be discerned: conditions during stand still of the TBM and conditions during drilling of the TBM.

Under stand-still conditions no fresh grout is supplied and grout pressures drop slowly. The pressurised soil will unload. Filtration properties of grout were measured in the laboratory, and a model was made for grout consolidation during stand still (see Bezuijen and Talmon 2003). This situation is verifiable on measured pressure decay during standstill. Involved properties are permeability of grout, porosity changes of the grout, hydraulic resistance of surrounding soil and elasticity of surrounding soil.

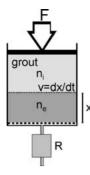


Figure 11. Skematisation of consolidation of grout. Elevated grout pressures are represented by the force F, the hydraulic resistance of pore water flow is represented by R.

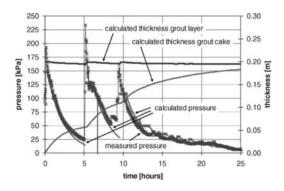


Figure 12. Numerical simulation of grout pressure decay behind the TBM at COB-passage compared with measurement: calculated pressure decay and calculated thickness of the grout cake, at  $R = 2 \ 10^7$  [s].

Under drilling conditions fresh grout is supplied and the grout pressure is constant. The surrounding soil will be under a constant pressure, and only permeability and hydraulic resistance are relevant.

The consolidation process is schematised in Figure 11. The hydraulic resistance R was back-calculated from observed grout pressure decay. Measured pressure decay and calculated grout pressures match well, see Figure 12. This back-calculated hydraulic resistance is however substantially higher than that of surrounding soil. A typical value for the hydraulic resistance of radial pore water flow into the surrounding soil is R = 65000 [s], Bezuijen (2005). It is hypothesised that bentonite slurry from the face of the TBM has invaded the grout-soil interface.

The consolidation theory allows the calculation of the development of the grout cake along the grout-soil interface, see Figure 12. The thickness of this filter cake is important to the modelling of the grout layer as an interface between tunnel lining and soil: it increases the integral stiffness of the grout layer. The foundation of the tunnel occurs by consolidation of the grout. This conclusion was also reached for the Sophia Rail Tunnel (Bezuijen et al. 2004), where cementious grout was used.

#### 7 CONCLUSIONS

The fine grained grout at GHT has a yield stress of about 1 a 1.5 kPa and, due to absence of cement, the yield stress of this grout increases only slowly with time. It was shown that consolidation of grout determines the decay of grout pressures over at least five tunnel rings behind the TBM. The developing grout cake increases the integral stiffness of the grout layer and hence the tunnel is founded.

The associated net hydraulic resistance of surrounding soil is substantially higher than according to radial flow theory. This higher resistance is attributed to bentonite originating from the face of the TBM, having invaded the grout-soil interface.

Given the high frictional resistance measured over the supply lines and the small vertical grout pressure gradients behind the TBM, it is concluded that this grout is providing significant friction with the concrete tunnel lining (contrary to cementious grout employed in previous projects).

For a number of reasons the pressure readings at GHT were less reliable than at other tunnels. The GHT data is nonetheless valuable because it shows the general applicability of the results from earlier research at Sophia Rail Tunnel, it is providing data on the performance of cement-less grout with high yield stress and it shows a marked influence of vertical movements of the tunnel lining on the grout pressure distribution.

#### ACKNOWLEDGEMENT

The work was conducted as a co-operation between Delft Cluster, HSL-South Organisation and Centre for Underground Construction. The authors gratefully acknowledge HSL South Organisation, and their treasurer the Dutch Public Works Department, and Centre for Underground Construction for permission to publish.

#### REFERENCES

- Aime, R., Aristaghes, P., Autuori, P. & Minec, S. 2004. 15 m Diameter Tunneling under Netherlands Polders, Proc. Underground Space for Sustainable Urban Development (ITA Singapore), Elsevier.
- Bezuijen, A. 2004. Calibration grout pressure sensor GHT/Results of measurements (in Dutch: Ijking groutdrukopnemers GHT/Resultaten metingen), COB report F512-O-04-127.

- Bezuijen, A. 2004. Vin tests grout GHT/Results of measurements (in Dutch: Vinproeven op grout GHT/Resultaten metingen), GeoDelft 403050/0008.
- Bezuijen, A. 2005. Analysis of pore pressures close to the tunnel; influence of grout flow, (in Dutch: Analyse waterspanningen naast tunnel; invloed groutstroming op waterspanning), COB report F512-05-04.
- Bezuijen, A. & Talmon, A.M. 2003. Grout the foundation of a bored tunnel, 2003, Proc. int. conf. on Foundations "Innovations, Observations, Design & Practice" (ICOF 2003 Dundee), Thomas Telford, London.
- Bezuijen, A., Talmon, A.M., Kaalberg, F.J. & Plugge, R. 2004. Field measurements on grout pressures during tunnelling of the Sophia Rail Tunnel, J. of Soils and Foundations of the Japanese Geotechnical Society, Vol.44, no.1, pp39–48.
- Shirlaw, J.N., Richards, D.P., Ramond, P. & Longchamp, P. 2004. Recent experience in automatic tail void grouting with soft ground tunnel boring machines, in *proc. Under*ground Space for Sustainable Urban Development (ITA Singapore), Elsevier.
- Talmon, A.M. & Bezuijen, A. 2005. Grouting the tail void of bored tunnels: the role of hardening and consolidation of grouts, 5th int. Symp. Geotechnical Aspects of Underground Construction in Soft Ground, 15–17 June, ISSMGE-TC28, Amsterdam, The Netherlands.
- Talmon, A.M., Bezuijen, A., Aanen, L. & van der Zon, W.H. 2001. Grout pressures around a tunnel lining, proc. IS-Kyoto conference on Modern tunneling Science and Technology, pp817–822.

### Longitudinal tube bending due to grout pressures

#### A.M. Talmon

Delft Hydraulics & Delft University of Technology, Delft, The Netherlands

A. Bezuijen

GeoDelft & Delft University of Technology, Delft, The Netherlands

#### F.J.M. Hoefsloot

Fugro Ingenieursbureaux B.V., Leidschendam, The Netherlands

ABSTRACT: A mathematical relation is described between the distribution of grout over injection ports, rheological properties and vertical grout pressure gradient immediately behind a TBM. This vertical grout pressure gradient is an important loading parameter in beam action calculation of tunnel linings. Longitudinal moments in the lining are of importance in tunneling especially for the design of the lining. These moments are determined by the uneven distribution of jack forces from the TBM but also by the buoyancy forces that are exerted by the grout that is still in the liquid phase close behind the TBM.

#### 1 INTRODUCTION

Earlier work (Bezuijen & Talmon 2005) has shown that the grout properties have a large influence on the longitudinal moments and that it is even possible that these moments prevent an adequate tunnelling process.

This earlier work did not take into account that the tunnel is stage constructed and that a specific beam calculation method has to be used to take that into account. An analytical staged beam calculation will be used in combination with measured and calculated grout pressure distribution to calculate longitudinal bending moments in the lining. The calculation method itself will be described in another paper to this conference (Hoefsloot, 2008).

The vertical grout pressure distribution immediately behind a TBM is governing to a large extent the stresses in the tunnel lining. This vertical grout pressure distribution is determined by the injection strategy and rheological properties of the grout. A finite difference model was published by Talmon et al. (2001) for the calculation of the grout pressure distribution up to a distance of a few tunnel rings behind a TBM. Using this model it was learned that there might be a simple relation between the distribution of grout injection rates over the injection ports and the vertical grout pressure gradient immediately behind a TBM.

#### 2 SIMPLE MODEL FOR VERTICAL GROUT PRESSURE DISTRIBUTION AT TBM

#### 2.1 Injection conditions

Many TBM's are equipped with six injection ports. Typically the flow rate through the ports located in the upper half of the TBM is larger than in the lower half. This way, the grout is forced to flow downwards, partially. As a consequence, the vertical grout pressure gradient will be lower than the static grout pressure gradient. Such a grout injection pushes the tunnel downwards. A definition sketch of grout distribution is given in Figure 1. Data on typical grout injection at Groene Hart Tunnel is given in Table 1.

The outer diameter of the Groene Hart Tunnel lining is 14.50 m. The theoretical thickness of the grout layer is 0.18 m.

## 2.2 *Mathematical modeling of tangential grout flow*

Each injection port has its own supply area. Grout flow velocities in the grouting annulus are small. Given the Bingham-like rheological character of grout, only its yield-stress is relevant. As a consequence, for both clock and anti-clock wise direction, grout pressures decay linearly from injection ports. Grout pressure

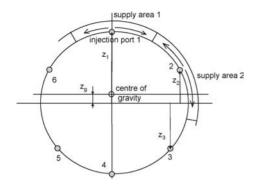


Figure 1. Definition sketch tangential grout distribution behind a TBM, grout supply area and the z-coordinate of the centre of gravity of grout injection.

 Table 1.
 Measured distribution of grout flow rates over the injection ports at Groene Hart Tunnel.

Injection point	position [degrees]	vertical position [m]	contribution to total grout injection
1	8.4	7.32	0.2
2	65.2	3.10	0.178
3	122	-3.92	0.156
4	180.9	-7.40	0.133
5	237.7	-3.95	0.156
6	294.5	3.07	0.177
total			1

decay halts upon convergence with grout from adjacent injection ports. The width of supply area is dictated by the relative injection rates of associated injection ports. Considering the width of supply area, and the fact that no discontinuities in grout pressure are allowed, the precise locations of the stagnation points between supply area are calculated. Next, with yield stresses on both sides of the grout layer, the grout pressure distribution is calculated mathematically. The location of injection ports and the definition of supply area is given in Figure 1.

An example of the calculation results for Groene Hart Tunnel is given in Figure 2. In this calculation both sides of the grout layer experience tangential shear stresses equal to the yield stress of fresh grout. Physical properties are listed in Table 2.

In reality there are no sharp discontinuities at the confluence of the supply area. These discontinuities are an artifact inherent to the employed schematization. The calculated discontinuities are acceptable because locally clock-wise and anti-clock-wise flow compensate.

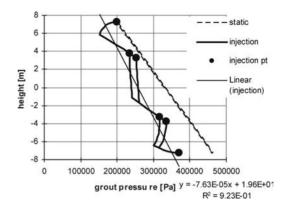


Figure 2. Calculated grout pressure distribution from grout injection. Injection ports are indicated. A linear approximation of calculated grout pressure distribution is indicated. The theoretical static grout pressure distribution is shown for comparison. Grouting parameters are according to Table 1, physical parameters are according to Table 2.

Table 2.	Input-parameters for the calculation of
grout pres	ssure distribution at Groene Hart Tunnel.

D <sub>tunnel</sub> [m]	14.5
h <sub>annulus</sub> [m]	0.18
$\rho_{\text{grout}} [\text{kg/m}^3]$	1850
$\tau_{\rm v}$ [Pa]	1500
P <sub>crest</sub> [Pa]	200000

### 2.3 Simplification of vertical pressure gradient calculation

The loading force (F) of grout on a tunnel lining, per unit length, is given by:

$$F = A \frac{dp}{dz} \tag{1}$$

The average vertical grout pressure gradient might be determined from linear approximations as shown in Figure 2. This is however not the most straightforward method. The mathematical model presented in Section 3 gives grout pressures at each circumferential position. By integration of calculated grout pressures the loading force (F) is calculated. By virtue of the above equation the corresponding vertical grout pressure gradient is determined next.

A number of calculations was conducted for different injection strategies, where the grout distribution between the upper and lower part of the TBM varied. Upon inspection of the results it was found that the centre of gravity of grout injection is a decisive

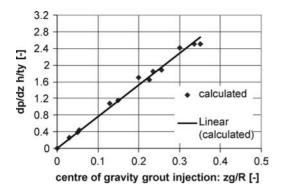


Figure 3. Calculated dimensionless contribution of grout-injection to vertical grout pressure gradient. Location of injection ports according to Table 1.

parameter. The z-coordinate of the centre of gravity is calculated by:

$$z_g = \frac{1}{Q_{total}} \sum (z_i Q_i)$$
<sup>(2)</sup>

with:  $Q_i$  = grout-flow-rate of injection port (*i*),

Q<sub>total</sub> = total grout-flow-rate of all injection ports,

 $z_i$  = vertical coordinate injection port (*i*),

 $z_g = z$ -coordinate centre of gravity

Definitions for the centre of gravity are shown in Figure 1.

Calculation results for vertical grout pressure gradient from grout injection (without static grout pressure) are shown, in Figure 3. Quasi independent of the precise distribution, a linear relation is found between the centre of gravity of grout injection and vertical grout pressure gradient. With six-injection ports, without grout overlapping adjacent injection ports, the maximum vertical position of the centre of gravity is  $z_g(max) = R/3$ , and the influence of grout injection is maximal:  $dp/dz = 8/\pi \tau_y/h$ . In that case the injection port at the crest of the tunnel supplies 1/3 of total grout injection.

Adding the static grout pressure to the results of Figure 3 leads to the following "design-formula" for vertical pressure gradient immediately behind a TBM:

$$\frac{dp}{dz} = \frac{24}{\pi} \frac{\tau_y}{h} \sum_{i=1}^{6} \left(\frac{z_i}{R} \frac{Q_i}{Q_{total}}\right) - \rho_{grout} g \tag{3}$$

For the grouting conditions at Groene Hart Tunnel, given in Table 1, the vertical position of the centre of gravity is at  $z_g = 0.047$  R. According to the simplified theory presented above, a vertical grout pressure gradient of 15.6 kPa/m would be expected at Groene Hart Tunnel. Measured vertical pressure gradients measured at Groene Hart Tunnel were: (~5 kPa/m at ring 2117, 12 kPa/m at ring 2118 and 17 kPa/m at

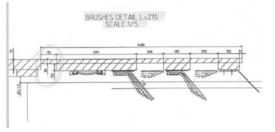


Figure 4. Back-side of the TBM showing tail-grease brushes and play (=jeu) between tunnel lining and back-side of the TBM-shield.

ring 3241, see Talmon & Bezuijen 2008). With the method described in Section 2.2 and 2.3 the vertical grout pressure gradient behind a TBM is calculated as a function of injection strategy and grout properties. This method is a good alternative for the DCgrout model. The advantage is that no numerical simulation is needed. The disadvantage however, is that it only presents the gradient directly after the tunnel. In the calculations that follow we assume that in 8 m this gradient decrease down to 9.3 kPa/m as will be explained later.

#### 2.4 Grout pressures in eccentric tail void

The grout pressure calculation model of Section 2.2 assumes a concentric position of the tunnel lining in the soil cavity, as does the DCgrout model of Talmon et. al. (2001). At Groene Hart Tunnel however some 7 cm upward movement was measured by a water leveling system just before the pressure sensors came into the grout, Talmon & Bezuijen (2008). It is unknown if, or how, surrounding soil moved. A movement of around 2 cm was found in 2-D calculations by Bezuijen & Bakker (2008) for a tunnel of 10 m diameter. However, this result will depend to a large extend on the soil parameters and they were not obtained for this location. The results of these 2-D calculations do indicate that a soil movement of some centimeters is the order that can be expected. From a geometric point of view, the maximum relative movement between the tunnel lining and the rear of the TBM is at max 9 cm before the tunnel lining touches the tail-end of the TBM, see Figure 4.

It is concluded from Figure 4 that the maximum eccentricity between the tunnel lining and soil cavity will be about 45 mm. The existing models do not necessarily have to be modified. An adaptation of inputted grout flow rates is considered a workable alternative. For 45 mm upward eccentricity of the tunnel lining at GHT, the apparent centre of gravity has been calculated at  $z_g = 0.22 \text{ R}$ .

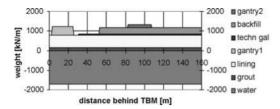


Figure 5. Loading diagram tunnel lining Groene Hart Tunnel: loading per unit length.

Figure 3 shows that for a relative centre of gravity of  $z_g/R = 0.22$ , grout injection produces a vertical pressure gradient of  $dp/dz = 1.6 \tau_y/h = 13.6 \text{ kPa/m}$ for a yield stress of  $\tau_y = 1.5 \text{ kPa}$  and a tail void thickness of h = 0.18 m. Addition of the static grout pressure gradient (at  $\rho = 1850 \text{ kg/m}^3$ ) gives for the calculated vertical pressure gradient behind the TBM: dp/dz = 18.1 - 13.6 = 4.5 kPa/m. This is close to the measured initial vertical gradient of 4 a 6 kPa/m at ring 2117.

#### 2.5 Different positions of injection ports

The present application applies to the layout of the Groene Hart Tunnel in the Netherlands. Similar calculations should be conducted for other positions of the grout injection ports, for instance for a layout such at Botlek Rail tunnel, where there is no injection port at the crest. Expected is that the result (Figure 3) will not differ much.

#### 3 BEAM ACTION CALCULATION

#### 3.1 Tunnelling conditions

The (local) uniform loads in the tunnel are sketched in Figure 5.

The forces exerted by the main jacks of the TBM are calculated from measured hydraulic oil pressures and piston/barrel construction of the hydraulic cylinders. During exit of the first rings of the COB-passage, ring 2117, the point of application of the axial force was 1.5 m below the TBM-axis: the bending moment exerted by the TBM jacks was 79 MNm.

The bending moment exerted by the TBM jacks was found to be virtually identical for each ring, with the exception of those first rings of the COB-passage, see Figure 6. A representative value is 65 MNm. Assuming that the other external influences (transverse force from TBM, vertical grout pressure gradient behind the TBM and longitudinal uniformity of soil) remain constant, a time-distance transformation of axial strains measured in tunnel ring 2117 produces the bending moment-curve of the tunnel lining shown in Figure 7.

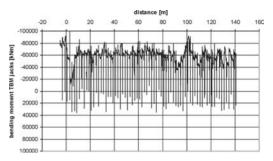


Figure 6. Bending moment exerted by TBM jacks during drilling phase.

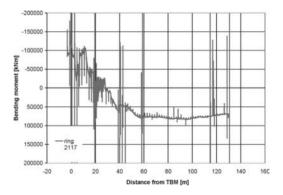


Figure 7. Measured bending moment in tunnel lining (computed from strain-gauges precasted in tunnel segments).

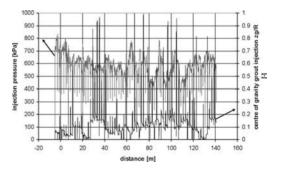


Figure 8. Average injection pressure of six grout injection lines and the vertical position of the centre of gravity of grout injection.

At the beginning of the COB-passage the variation in bending moment exerted by TBM jacks is precisely synchronous with the bending moment calculated from the strain gauges in tunnel ring 2117.

Figure 8 shows the course of grout injection pressure (average of all six injection lines) measured 4 m before the outlet of the injection pipes. Also the centre of gravity of grout injection, calculated by Equation 2,

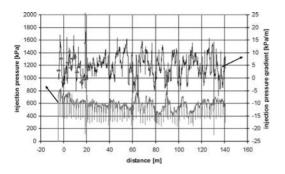


Figure 9. Average injection pressure and vertical pressure gradient of grout injection lines, measured at the inlet of the injection pipes.

is shown. Grout injection pressures vary between 4.5 bar and 7.5 bar. The vertical position of the centre of gravity of grout injection varies between 0 and 0.2R above tunnel-axis.

For a concentric configuration of tunnel lining and soil cavity, at a grout yield stress of 1.5 kPa and twosided friction of the grout layer, this corresponds to a variation of vertical grout pressure gradient at the TBM between 7.1 < dp/dz < 18.5 kPa/m.

Figure 9 shows the vertical pressure gradient calculated from injection pressures. Basically this gradient varies between 0 and 10 kPa/m over the COB-passage. This gradient is lower than measured with the pressure sensors on the lining. It could be that the pressure drop over the four meters pipe length is higher in the upper part of the TBM, because of higher grout flow rates (typical grout flow velocity is 0.3 [m/s]).

TBM jacks may transfer transverse forces between the tunnel lining and TBM. The magnitude of such forces should be limited, because jacks are not designed for that. This unknown transverse force is however important to beam action of the tunnel lining, and has to be determined.

In the tail void the segments are supported by forces from the grout layer. Inside the TBM tunnel rings are unsupported. At the Groene Hart Tunnel the typical length of unsupported tunnel lining in the TBM is 4 meters.

Vertical support by fluid grout is determined from grout pressures measured around the tunnel lining. Upon exit vertical grout pressure gradients as low as 6 kPa/m occurred, that after 3 m progress stabilized to a gradient of 12 kPa/m (Talmon & Bezuijen 2008). This latter value is considered a realistic and representative value for grout loading just behind the TBM.

#### 3.2 Result beam action calculation

The staged beam action model described by Hoefsloot (2008) is used. Input-parameters are the above

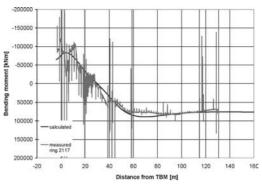


Figure 10. Measured and calculated bending moments compared (Groene Hart Tunnel, the Netherlands).

mentioned bending moment by TBM jacks, transverse force by TBM, vertical grout pressure gradient behind the TBM, loading diagram and unsupported length of tunnel lining in the TBM.

Model-specific is the so called "end of the fluid grout zone". In the staged beam model, this is the location where the reaction-force from the soil is zero: new elements are mathematically added in a stress-less condition. Here the associated vertical pressure gradient across the tunnel lining is 9.3 kPa/m (weight of grout layer has been subtracted from a 10 kPa/m hydrostatic pressure). The measured position of 9.3 kPa/m is positioned about ten meters behind the TBM. The vertical grout pressure gradient is assumed to decay linearly from 12 kPa/m at the TBM to 9.3 kPa/m at the end of the fluid grout zone. Consequently the external loading by fluid-grout connects smoothly to the reaction force from the soil. A further second order condition is that the axial gradient of vertical grout pressure gradient and reaction force from the soil are identical. An "end of fluid grout zone" situated 8 m behind the TBM satisfies these criterion. However, it is not clear how general applicable the implicit boundary conditions are. This will be subject to further research.

The model requires also information on structural properties: bending stiffness of the lining and modulus of subgrade reaction of surrounding soil. These have to comply with fundamental properties of segmented lining and surrounding soil, but also have to comply with the vertical displacements that have been measured (Talmon & Bezuijen 2008). Therefore lower values are used then one would apply on pure theoretical grounds. The following values have been used:  $EI = 3.2 \cdot 10^9 \text{ [kN m}^2 \text{] and } \text{k} = 7.3 \cdot 10^4 \text{[N/m}^2 \text{]}.$ 

A modest downward force of 1.5 [MN] from the TBM is needed to reach perfect agreement between measured and calculated bending moment curve in the tunnel lining, see Figure 10.

At ring 2117 and ring 2118 a total of four inclinometers meters have been attached to lining segments

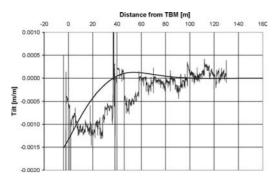


Figure 11. Measured and calculated inclination of the tunnel (Groene Hart Tunnel, the Netherlands).

situated at tunnel-axis-level. These four instruments produced exactly the same results. Figure 11 shows the results of one of these. It shows that after the TBM the tunnel inclination is of the order 1:1000 (compared to its final orientation) and that inclination decreases with distance. The graph shows that there remains a discrepancy between inclination calculated by the beam action model and measured inclination.

#### 4 CONCLUSIONS

#### Grout injection:

The centre of gravity of grout injection, in combination with grout yield stress, is shown to determine the vertical grout pressure gradient immediately behind a TBM.

In case of an eccentric position of the tunnel lining, the influence of eccentricity on grout pressures is also quantifiable.

In future application of beam action models for tunnel linings, the model of Section 2 can be used to calculate beam loading caused by the vertical grout pressure gradient behind the TBM.

#### Beam action tunnel lining

The measured bending moment curve in the tunnel lining is well reproduced by the beam action model. The way how the transition from liquid grout with a constant force on the tunnel to elastic behaviour is incorporated in the model needs further study. Simulations have shown that the buoyancy forces that occur at the location where the tunnel lining is 'floating' in liquid grout are essential to simulate the measured bending moment.

Lower values for the bending stiffness of the tunnel lining and the modulus of subgrade reaction of surrounding seem to apply than are expected on pure theoretical grounds. The construction phase of a bored tunnel leads to axial forces that remain in the tunnel lining after completion (Blom 2002). The construction phase also produces a remaining bending moment, as was found at the Groene Hart Tunnel. The associated point of application of this axial force is situated about 1.5 meter above the tunnel axis.

#### ACKNOWLEDGEMENT

The work was conducted as a co-operation between HSL-South Organisation and Centre for Underground Construction. The authors gratefully acknowledge HSL South Organisation, and their treasurer the Dutch Public Works Department, and Centre for Underground Construction for permission to publish.

#### REFERENCES

- Bezuijen, A. & Bakker, K.J. 2008. The influence of flow around a TBM machine. Proc. 6st Int. Symposium on Geotch. Aspects of Underground Construction in Soft Ground, Shanghai
- Bezuijen, A. & Talmon, A.M. 2005. Grout properties and their influence on backfill grouting. Proc. 5th Int. symp. Geotechnical aspects of underground construction in soft ground, Amsterdam 15–17 June.
- Blom, C.B.M. 2002. Design philosophy of concrete linings for tunnels in soft soil. PhD-thesis, Delft University Press.
- Hoefsloot, F.J.M. & Bakker, K.J. 2002, Longitudinal effects bored Hubertus tunnel in The Haque. 3rd International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground – IS Toulouse, 23–25 october 2002.
- Hoefsloot, F.J.M. 2008. Analytical solution longitudinal behaviour tunnel lining, TC28 Shanghai.
- Koek, A.J., Bakker, K.J. & Blom, C.B.M. 2006. Axial prestresses in the lining of a bored tunnel. 5thint. Symp. Geotechnical Aspects of Underground Construction in Soft Ground, 15–17 June, ISSMGE-TC28, Amsterdam, The Netherlands
- Talmon, A.M. & Bezuijen, A. 2005. Grouting the tail void of bored tunnels: the role of hardening and consolidation of grouts. 5thint. Symp. Geotechnical Aspects of Underground Construction in Soft Ground, 15–17 June, ISSMGE-TC28, Amsterdam, The Netherlands.
- Talmon, A.M. & Bezuijen, A. 2008. Backfill grouting research at Groene Hart Tunnel. Proc. 6st Int. Symposium on Geotch. Aspects of Underground Construction in Soft Ground, Shanghai.
- Talmon, A.M., Bezuijen, A., Aanen, L. & van der Zon, W.H. 2001. Grout pressures around a tunnel lining. *Proc. IS-Kyoto conference on Modern Tunneling Science and Technology*, pp. 817–822.
- Talmon, A.M., Bezuijen, A. & Hoefsloot, FJ.M. 2007. Evaluatie tweede fase Groene Hart tunnel: Ontwerpparameters bij krachtswerking, groutdruk en invloed beweging tunnelbuis. *COB-report*, COB-F512-07-02.

Theme 3: Case histories

# Tunnel face stability and settlement control using earth pressure balance shield in cohesionless soil

A. Antiga Soil S.r.l., Milano, Italy

M. Chiorboli Metropolitana Milanese S.p.A., Milano, Italy

ABSTRACT: Soft ground tunneling, in urban areas are generally bored using pressurized TBM. Tunnel design involves the need to revalue the classical concepts of deformation response to the excavation and the control priorities, focalized on face and cavity, loose their importance; the evaluation of the settlements becomes the main aspects to consider. The definition and the control of the face pressure are fundamental design steps. It is important to give a proper importance to the face pressure relating to settlements control. The pressure increase leads to higher safety factors of the face stability but it is not able to guarantee comparable settlements decrease. We examines the essential parameters in relation to the use of the EPBS in two case histories in Milan: line 1 extension to the new Fair and Passante rail infrastructure. The reference values of the cited parameters are identified for cohesionless soil i.e. fluvio-glacial sand deposit and gravel.

#### 1 INTRODUCTION

In recent years, the growing needs for transportation infrastructure has lead to a sharp increase in the utilization of underground. During 1993 and 1994, an EPBS with a diameter of 8.03 m was used in the area of the "Passante" (underground railway line) in Milan. This method, with a diameter of 6.56 m, was then used in 2004 to extend the Milan subway to the new trade fair of Rho. This document considers the main project aspects relating to the use of the EPBS, with reference to the two cited projects in Milan and so to a geotechnical context of cohesionless soil under high water head. The reference values for the basic tunnel design parameters are defined; a reference value for the parameter "i" (point of inflection in the transverse settlement trough) is identified. The document also describes how, given the values necessary to guarantee the face stability, the tunnel face pressure would not influence the final settlements, and how this parameter could be best defined using only calibration during the work. The theoretical pressure values to use can be determined using simple equilibrium ratios in K0 conditions as initial starting values for tunnelling operations.

## 2 MECHANIZED TUNNELLING DESIGN IN URBAN ENVIRONMENT

Peck (Peck 1969) highlighted the basic points to consider when planning underground works in the case of "soft ground":

- 1 the stability of the tunnel with particular reference to the stability of the tunnel face and of the zone immediately behind;
- 2 the evaluation of the deformations at the face and around the tunnel and, in the case of a tunnel with a low depth, the resulting superficial subsidence;
- 3 the definition of stabilization actions to guarantee compliance with conditions (1) and (2) and of the lining structure to ensure the long-term stability of the tunnel.

With the construction of tunnels in urban environment, the control of superficial subsidence and its effects assumes primary importance with regard to those items listed above. The conditions for stability of the face and of the cavity and the acceptability of respective deformations, with regard to the breaking values, are conditions that are necessary but not sufficient to guarantee the validity of a given construction

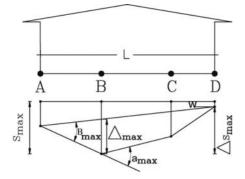


Figure 1. Building subsidence: main parameters.

system because they are not able to guarantee, a priori, the acceptability of the subsidence. This statement is particularly valid in the case of tunnelling systems with shield machines and full-face pressurization for which there is no direct control of the strain state of the face and of the cavity during the construction phase, and for which the most significant quantitative parameters are:

- stabilization pressure at the face
- volume of extracted soil
- superficial subsidence

#### **3 SUBSIDENCE EVALUATION**

The process of design for tunnels in urban areas must be based on the settlement value parameter.

The objectives of the settlement analysis should be as follows:

- definition of acceptable settlement values
- study of the process that generates the subsidence and the control methods
- technical and economical optimization of the project by defining the construction methods to use and any required mitigation actions.

Burland and Wroth identified the parameters to consider when valuating subsidence (figure 1) (Burland and Wroth 1974). Rankin subsequently focussed on the deflection ratio  $DR = \Delta_{max}/L$  and on the horizontal deformation, he moreover showed that in engineering practice, you can rarely rely on the complete data identified by Burland and Wroth (Rankin 1988). Similar to a normal Gaussian probability function, the profile of the transverse settlement trough is consolidated by a mass of literature on the subject and is proven by experience.

The curve that defines the transverse settlement trough is characterized by two parameters: (a) the maximum settlement  $S_{\text{max}}$  (corresponding to the tunnel

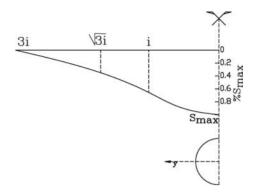


Figure 2. Transverse settlement trough.

axis), (b) the distance "*i*" between the tunnel axis and the point of inflection in the subsidence profile that defines the size of the transverse settlement trough.

The vertical settlement at distance *y* on the tunnel axis is given by:

$$S = S_{\max} \cdot e^{\left(\frac{y^2}{2l^2}\right)} \tag{1}$$

The total volume of the subsidence trough  $V_s$  (for the unit length for the tunnel) can be obtained from the integration of (1) and the result is:

$$V_s = \sqrt{2\pi} \cdot i \cdot S_{\max} \tag{2}$$

The volume of the subsidence trough  $V_s$  is directly dependent on the volume loss  $V_p$  (volume of soil that is excavated in excess of the theoretical volume of the tunnel).

In agreement with the proposal by O'Reilly and New, for z > D (D = diameter of the tunnel), the parameter "*i*" depends on the type of soil and the depth of the tunnel and appears to be independent on the tunnel diameter and the tunnelling mode (O'Reilly and New 1991); giving:

$$i = K \cdot z$$
 (3)

where z is the depth of the tunnel and K is a coefficient that depends on the type of soil.

For cohesive soil, the value of K is generally between 0.4 and 0.6. For sand and gravel, the values for K are more dispersed, however the usual range is between 0.25 and 0.45 (Mair 1997).

From the function (1) you can derive the slope and curve of the transverse settlement trough, factors that are relevant on the pre-existent structures.

To summarize, the value of the settlement is therefore dependent on "i" and on  $V_p$  through  $V_s$ .

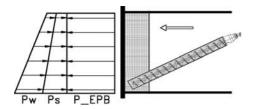


Figure 3. EPB Shield \_ Face stabilization pressure.

In the case of non-complex situations that can be well planned following the hypothesis described above, there are no significant improvements in the reliability of the results after switching from an empirical Gaussian method to a more sophisticated numerical model.

#### 4 FACE STABILIZATION PRESSURE

The definition and the control of the face stabilization pressure represents one of the basic stages in the constructive phase. The determination of this parameter in the planning phase, however, give rise to some uncertainties: despite there being a large amount of literature, there is not yet a generally agreed analysis method and at the same time, there are no normative references. Also, ignoring the uncertainties resulting from the geotechnical characterization of the soil, using the different approaches discussed in literature, you obtain frequently significant differences in the theoretical values to apply.

However, it is necessary to lay proper emphasis to the face stabilization pressure in terms of the final subsidence result.

On one hand, the increase in value of the pressure applied to the face guarantees higher safety coefficients for the face stability; however, on the other hand, a correlated reduction in settlements at the surface are not absolutely guaranteed.

The definition of the stabilization pressure to apply must achieve the objective of guaranteeing the stability of the face while changing the in situ stress as little as possible. In a fully-operational situation, you should try to excavate in *K*0 condition, that is with a constant advance rate.

A study conducted by AFTES identified how the ideal advance condition, for minimizing the deformations and ensuring the face stability, is a balance between the volume of material extracted and the theoretical tunnel volume and how with this condition the pressure remains constant. In this condition, you could advance with a constant volume and the pressure applied to the face would be equal to the earth pressure at rest (i.e. *K*0 conditions); by checking the two identified parameters (volume and pressure) you

would obtain the definition of the correct face stabilization pressure (AFTES 2001). As a result, during construction phase, the research should concentrate on the achievement of the balance between the volumes, both theoretical and extracted, based on stabilization pressure that is sufficiently cautious.

The machines used in recent years, mainly due to the correct definition of the soil conditioning technologies, enable good control of pressures and, therefore, of the stability of the tunnel face, as we will see in the Milan examples. With the control of the pressure, you can influence the situation ahead of the face of the tunnel and this represents a condition necessary. but not sufficient, to guarantee subsidence control. In fact, to greatly increase the pressure would bring the soil ahead of the cutting head in conditions nearer to that of passive pressure with an increase in the total pressure to apply and, as a result, with a series of effects that bring about a reduction in the advance speed and operating conditions that are less favourable and more risky (for example, an increase in the required torque, an increase in energy consumed, an increase in tool wear, an increase of temperature in the tunnel working chamber as a result of increased friction with the possibility of creating material blocks). The soil in front of the face does not record any excess pressure applied to the face in the successive phases in which it is subject to the phenomena of loosening that results from the different annular cavities in the passage of the shield, i.e. over-cut, due to the tools at the periphery of the cutting wheel (the statement applies to incoherent earth).

To summarize, the pressure in the working chamber of the TBM must be maintained at a level capable of ensuring stable working conditions with a safety coefficient that allows the absorption of the fluctuations resulting from a dynamic situation. This should be sufficient to avoid uncontrolled collapses of material within the working chamber but not so that it causes soil deformations and blow-up with a consequent loss of the stabilization conditioned mixture far from the face (AFTES 2001). The definition a priori of the face stabilization pressure would, in fact, be a secondary element only guided in choosing the initial value; it is optimized during the process of work on the basis of the balance of the excavated soil volume. The problems relating to the subsidence require maximum attention also to other parameters, in particular advance rate and backfilling operations behind the face. Research, more technological than theoretical, must also analyse the mechanisms that rise ahead of the face: grouting systems above the shield have been recently introduced, mainly using polymers or bentonite muds. These muds, injected at pressure, hold the soil above the shield for the time period from the passage of the head to the unthreading of the shield and the related final backfilling with cement mortar.

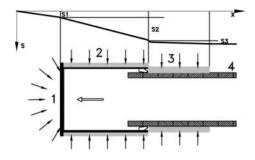


Figure 4. EPB Shield \_ Individual factors of volume loss VL.

#### 5 VOLUME LOSS VL

The "volume loss" VL represents the fundamental index parameter for the valuation of the surface settlement and, consequently, of the disturbance caused by the tunnelling. The total "volume loss" is derived from the sum of various components whose relative contribution depends on the mechanical and physical characteristics of the soil, on the technical characteristics of the tunnelling equipment, on the execution method, with particular attention to the pressures applied at the face and around, and on the advance rate. These factors are essentially (figure 4) (Mair 1997):

- volume loss at the face (face loss): owing to the deformation of the portion of soil ahead of the face of the tunnel and to the loss resulting from localized micro-stability problems. This loss of volume is counteracted by the face stabilization pressure.
- volume loss around the shield essentially due to the convergence of the tunnel profile towards the extrados of the shield machine. This factor can be subdivided into the following terms: (2a) overcutting due to greater diameter of the tunnelling cutterhead compared to the shield machine, necessary to reduce the friction of the shield machine on the soil and to facilitate steering; (2b) planoaltimetric characteristics of the trail; the presence of narrow curve ranges results in over-cutting values higher than those present in the straight sections. This aspect is accentuated by possible misalignments caused by the operator who may let the machine zigzag; (2c) over-cutting caused by possible cone shaped of the shield; (2d) roughness of the shield which can lead to movements and subsidence due to the friction against the soil.
- volume loss around the lined tunnel section due to the gap between the tunnel profile and the extrados of the pre-cast segments; this results in a tendency to converge towards the extrados of the lining.
- volume loss due to deformations of the lining.
- long-term effects/consolidation.

With regard to the five points listed above:

- the component relating to the stress relief of the soil at the face with the use of pressurized face tunneling machines becomes unimportant if the pressure at the face is adequately controlled.
- the volume loss during the passage of the shield is difficult to counteract and can be the weakest element; the limitation of this component is above all to reduce the "technological" over-cutting to the minimum possible. In recent years, shield machines that allow annular grouting for filling the region around the shield have been used. It can also be added that holding an high advance rate brings a significant reduction in this component, preventing that the gap closes completely.
- the volume loss around the lined section of tunnel is minimized by using cement mortar from tail of the shield; however problems may still occur in the execution which render this component critical (insufficient grouting pressures, washing away of the mixture etc.). In soil that has a self-stable capacity adequate to the advance rate, the injection of cement mortar can create a recompression of the ground and allows the recovery of a part of the volume defined in point 2.
- is in general unimportant compared to others.
- is significant in soil subject to consolidation.

Mair showed that, in the case of Slurry Shields or EPB-S, possible references are VL  $\leq 0.5\%$  for sandy soil and VL = 1  $\div 2\%$  in soft clay (Mair 1997). The definition of a reliable value is, however, very difficult in design phases because this parameter depends on factors that are highly dependent on the specific context and cannot be generalized (soil, technological factors of tunnelling equipment, workmanship skill, hydrogeology etc).

#### 6 EPB-S TUNNELLING IN MILAN

Between 2004 and 2005, parallel to the construction of the new trade fair in Rho/Milan, line 1 of the subway was extended. The track, with a length of around 2.1 km, is characterized by the presence of high water table conditions (up to 12 m) and variable tunnel depth between 10 and 20 m; two tunnels with one single platform were excavated using an EPB-S machine. This method of tunnelling was previously used in Milan in 1993/1994 during the construction of the "Passante Ferroviario" (underground railway line) for a section of around 4 km when digging two tunnels with variable depths between 4 and 16 m.

Some details of the two works are reported by Chiorboli and Marcheselli (Chiorboli, Marcheselli 1996) and by Cavagna and Chiorboli (Cavagna,

	LINE 1	PASSANTE FERROVIARIO
	LINE I	FERROVIARIO
CLIENT	MUNICIPALITY	MUNICIPALITY
	OF MILAN	OF MILAN
		REGION OF
		LOMBARDY
BUILDING	TORNO - ICLET	TORNO - CMB -
CONTRACTORS	FIAT	COGEFAR
	ENGINEERING	IMPRESIT -
		LODIGIANI -
		COLLINI – P&C –
		TETTAMANTI
PERIOD	2004–2005	1994–1995

Chiorboli 2004); the next table reports the main data:

The Milan area is characterized by fluvio-glacial deposits consisting of a heavy deposit  $(50 \div 60 \text{ m})$  of gravel and sand with a medium to high density. The typical, grain size distribution, which can be defined as the average granulometric curve is:

 $D < 0.074 \text{ mm} = 10 \div 20\%$ D > 2 mm = 70% $D > 10 \text{ mm} = 10 \div 20\%$ 

#### 7 FACE STABILIZATION PRESSURE

The maximum settlement values are always shown in association with the lowest possible pressure values, in terms of minimum pressure value peaks. These minimum pressure peaks, if just momentary, reach values significantly lower that the values required for the stability and they are, almost always, appeared where the conditions around (exit or entry of the machine in the construction pit,) or operating conditions (initial start phase, mechanical rests) cause a loss of pressure that is difficult to control. If you consider this in terms of average recorded pressure value, you cannot define an effective link between the increase in pressure and the reduction in final settlement. The two following figures show a summary of the average recorded pressure values, the theoretical pressure values in K0 conditions and the measured settlement for the two cited works. You can see that by assuming a pressure value close to the theoretical value in K0 conditions as the start value of the machine, the pressure is subsequently reduced: the measured settlement does not show relative associated increases.

#### 8 TRANSVERSE AND LONGITUDINAL TROUGHS

The transverse and longitudinal settlement troughs have been analysed in both cases. The troughs in areas

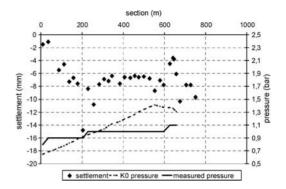


Figure 5. Line 1: settlements - face pressure.

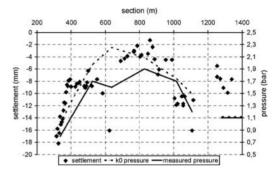


Figure 6. Passante Ferroviario: settlements - face pressure.

"disturbed" by the presence of buildings or grouting treatments have been ignored. With regard to the case of line 1 and the underground railway line, n. 18 and n. 5 transverse settlement troughs together with n. 78 and n. 28 longitudinal settlement troughs have been analysed. The graphic in figure 7 plots the values of the position of the transverse point of inflection "*i*" relating to the tunnel depth and the corresponding interpolating lines; the same graphic also shows the values compared with the range of literature indicated by Mair for sand and gravel (Mair 1997). Note that for both works considered, there is a good concentration of data around an identifiable value such as:

$$i = (0.43 \div 0.46) \cdot z$$
 (4)

It is possible to conclude that for the soil of the Milan area the reference range K for the calculation of settlements with the "Gaussian" formula is:

$$K = (0.43 \div 0.46)$$
 (5)

Figure 8 shows, for both cases, the longitudinal subsidence profiles with settlements normalised to the

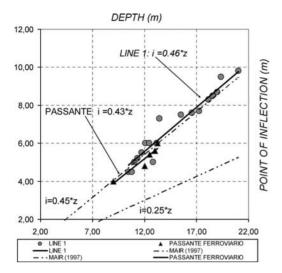


Figure 7. Transverse point of inflection - tunnel depth.

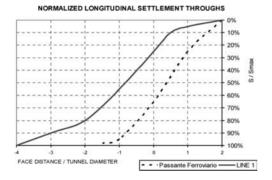


Figure 8. Normalized longitudinal settlement troughs.

maximum settlement (i.e. ratio of settlements to maximum settlement is presented) the abscissa shows the position of the face, normalized to the diameter. In terms of absolute settlements, they show similar values in the two cases while the two percentage curves show different values.

For the "line 1" it is observed that around 40% of the subsidence is due to the passage of the shield (9 m at the back of the head) and, considering that around 25% is ahead of the face, the remaining 35% is behind the machine in grouting phases for the annular space in the extrados of the segments. This would appear to show good management of the pressure in the tunnel working chamber although the subsidence percentage behind the shield appears rather high. For the "Passante Ferroviario", you can see that at the tunnel face an average value of 70%, of the subsidence, is reached; the percentage becomes 94% with the passage of the shield. After the passage of the face, the behaviour, apart from rare cases, was similar for all points irrespective of the coverage and other parameters.

#### 9 CONCLUSIONS

The analysis of the data collected and of the quoted references enables to stress that:

- 1 for the Milan area soil the reference range of the parameter *K* for settlement calculation is  $0.43 \div 0.46$ .
- 2 The net difference in behaviours between the two cases essentially refers to technological aspects of backfilling operations for the gap above the pre-cast segments. In "line 1" a simple system of grouting from 4 nozzles placed around the shield was used. In "Passante" a more complex system was adopted (Hochtief system) which uses injections of a cement grouting from nozzles placed around the shield; however, this bentonite fluid was able to exert a pressure on the soil, because it was confined to a mobile annular mould with transducers. This equipment enables the variation of pressure of the grouting and works in a similar way to "compensation grouting", however, it is extremely taxing in terms of maintenance and the requirement of highly specialized personnel.
- 3 The analysis of the development of the longitudinal subsidence curve over time in relation to the position of the face and the advance rate showed that, in the Milan soil, the strain effects of the tunnel advance spread very quickly, almost immediately, over the surface.
- 4 Given the values necessary to guarantee the face stability, the tunnel face pressure would not influence the final settlements The pressure increase leads probably to higher safety factors of the face stability but it is not able to guarantee comparable settlements decrease. Moreover, as explained by Anagnostou and Kovari (Anagnostou, Kovari 1996), if the support pressure exceeds a certain upper limit some operational problems (high wear of the cutter, excessive torque increase, difficulties in muck discharge) may occur; it may also cause high fluctuations of the distribution of the effective pressure acted on the face that may lead also to local instability. So support pressure should be defined using only calibration during the work starting from an initial value determined using simple equilibrium ratios in K0 conditions (AFTES 2001).

#### REFERENCES

A.F.T.E.S., 2001. Synthese Eupalinos 2000 – EPB Shield Theme B1: Laboratory studies on reduce models.

- Anagnostou G., Kovari K., 1996. Face stability conditions with earth-pressure-balanced shields. *Tunnelling and Underground Space Technology*, 11(2):165–173. Elsevier Science.
- Attewell P. B., 1978. Ground movements caused by tunnelling in soil. *Large Ground Movements and Structures*, 1978 London, 812–948. Ed. Geddes, Pentech Press London.
- Burland J. B., Wroth C. P., 1974. Settlement of buildings and associated damage. *Proceedings of a Conference on Settlement of Structures*, 1974 Cambridge, 611–654. Pentech Press London.
- Burland J. B., 1997. Assessment of risk damage to buildings due to tunnelling and excavation. *Earthquake Geotechni*cal Engineering, 1997 Ishihara, 1189–1201. Balkema.
- Chambon P., Corte J. F., 1994. Shallow tunnels in cohesionless soil: Stability of tunnel face. *Journal of Geotechnical Engineering*, 120, 1148–1165. ASCE.
- Chiorboli M., Marcheselli P., 1996. Analysis and control of subsidence due to Earth Pressure Shield tunnelling in Pas-

sante Ferroviario of Milano. *North American Tunneling*, 1996 Washington DC, 97–106. Balkema.

- Cavagna B., Chiorboli M., 2004. The use of EPB in the Milan subsoil: line 1 extension. *International Congress on mechanized tunnelling*. 2004 Torino, 107–118. Politecnico Torino.
- Mair R. J., Taylor R. N., Burland J. B., 1996. Prediction of ground movements and assessment of risk of building damage due to bored tunnelling. *International Symposium* on Geotechnical Aspects of Underground Construction in Soft Ground, 1996 London, 713–718. Balkema.
- Mair R. J., Taylor R. N., 1997. Theme lecture: bored tunnelling in the urban environment. *Conference on soil mechanics and foundation engineering*, 1997 Hamburg, vol. 4, 2353–2385. Balkema.
- Rankin W. J., 1988. Ground movements resulting from urban tunnelling: predictions and effects. Engineering geology of underground movements, *Geological Society Engineering Special Publication*, 79–92. Geological Society, London.

# Displacements and stresses induced by a tunnel excavation: Case of Bois de Peu (France)

#### S. Eclaircy-Caudron, D. Dias & R. Kastner

INSA Lyon, Civil and Environmental Engineering Laboratory (LGCIE), Villeurbanne cédex, France

ABSTRACT: The tunnel of Bois de Peu is part of a project of the south-eastern Besançon (France) by-pass. An exploration gallery permitted to highlight the presence of eighteen geological units and to distinguish four sorts of materials: limestone, marls, clays and interbedings of marls and limestone. Despite of the important number of laboratory and in situ tests carried out, many uncertainties remained on the mechanical parameters value and on the position of the different geological units. So, it was decided to apply the interactive design method during the construction to adapt the excavation and support to the actual conditions found. In the framework of this method, an important monitoring program was foreseen. This article shows the behavior observed in the different kind of soils. In the clayey zone where the support is the most complex, strains in the ground and in steel ribs are monitored. Finally, the available measurements permit to better understand the behavior of soil-structure interaction problems.

#### 1 INTRODUCTION

The digging of a tunnel induces a modification of the initial stress field in the ground which creates an unbalance state. This unbalance results in movements of soil like convergence of the cavity, pre-convergence ahead of the face, extrusion of the face and settlements.

During the excavation an arching effect is created. If it is not sufficient to stabilize the cavity, lining supports are set up in order to limit soil movements and thus, to avoid the failure of the structure.

The lining support can include shotcrete, ribs and/or radial bolting. In certain cases, when the ground has low mechanical characteristics, additional support like ground reinforcement or ground improvement at some stage of the excavation is necessary. Several reinforcement systems have been developed. Pelizza & Peila (1999) presented the different methods of soil and rock improvement used to permit safe tunneling in difficult geological conditions. Lunardi (2000) divided the support methods into three groups: pre-confinement, confinement and pre-support. Each group exerts a different kind of effect on the cavity.

When the traditional method of digging is used in difficult geological conditions, the main problem is the control of movements. Without support or adapted treatment, the ground tends to sink into the opening (tunnel face failure, tunnel face extrusion): it is the phenomenon of decompression. In order to reduce this phenomenon, an action of pre-confinement may be required. A pre-confinement action is defined as any active action that increases the formation of an arch effect in the ground ahead of the tunnel face. The pre-confinement can be achieved by reinforcement or protective intervention ahead the tunnel face. The umbrella arch method and the face bolting are included in protective interventions or pre-support methods.

The support and forepoling introduce many three dimensional soil–structure interactions. Consequently, it is difficult to understand these phenomena analytically. Moreover, although the umbrella arch method is widely used, there are no simple approximations to simulate this method in numerical analyses. The design of an umbrella arch is still based today on empirical considerations or on simplified schemes (Oreste & Peila 1997). Several numerical studies carried out in 2D and in 3D, were focused on the manner of taking into account umbrella arch (Tan & Ranjith 2003, Bae et al. 2005). In the same way, several numerical modeling were carried out in 2D and in 3D on the manner of taking into account the face bolting (Yoo 2002, Dias 1999).

For some large geotechnical engineering projects, a monitoring program is generally defined in project phase to record the soil movements which really occur during construction and to evaluate the performance of the construction design. In most of the cases, the recorded data are just used to control the construction process. But, these data can be also used to update predictions by using inverse analysis processes on

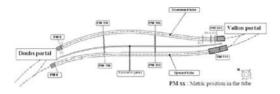


Figure 1. A plan view of the Bois de Peu tunnel project.

these measurements in order to decide of a possible adaptation of the construction process in the case where unsafe values would be predicted. This later practice is a part of the observational method (Peck 1969, Powderham & Nicholson 1996). The AFTES guidelines (2005) related to the monitoring methods of underground works present the major methods and give advices on the measurements frequency.

In this article, monitoring results obtained in the different geological units found during the excavation of the downward tube of the Bois de Peu tunnel are presented. First, this communication introduces the tunnel project. Then, in a second part, the monitoring results are presented. They permit to better understand the soil and the umbrella arch behavior set up in the clayey zone.

#### 2 PRESENTATION OF THE TUNNEL

#### 2.1 General presentation

The tunnel of Bois de Peu (cf. Fig. 1) is part of the project of the south-eastern Besançon (France) by-pass entitled "La Voie des Mercureaux". This project includes several engineering structures: two tunnels and one bridge, and several retaining walls. The tunnel is composed of two tubes of 520 m length. The cover height varies between 8 m and 140 m. The excavation, achieved in September 2006, was carried out full face by drill and blast for the major part of the tunnel.

#### 2.2 Geology and geotechnics

An exploration gallery was dug in 1995 in order to assess the mechanical properties of the ground. It has a width of 3 m and a height of 3.5 m. Various laboratory and in situ tests were carried out. The results lead to conclude that the tunnel is situated in a disturbed area. Eighteen geological units are identified. A geological cross section is showed in Figure 2. Among these eighteen units, four sorts of materials can be distinguished: clays, marls, limestone and interbedings of marls and limestone. Geotechnical properties of these materials defined at the end of site investigations are summarized in Table 1. Two types of characteristics were defined for the marl (probable and

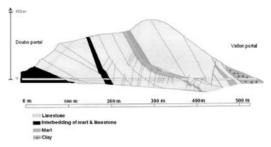


Figure 2. The geological cross section.

Table 1. Geotechnical properties of material.

Material	γ kN/m <sup>3</sup>	E MPa	С	φ MPa
Limestone Marl (probable)	26.3 24.8	3290–10000 1600	_ 0.7	-40
Marl	24.0	750	0.21	36
(exceptional) Clay	23	80	0.25-0.4	13–17

exceptional) because the in situ and laboratory tests leaded to variable values. The Poisson ratio, the dilatancy angle and the earth pressure ratio are the same for the materials and are respectively equal to 0.3,  $0^{\circ}$  and 0.7.

The exact position of the different units is difficult to know before the digging. Finally, following the observation made in this exploration gallery, many uncertainties are remaining. So, it was decided to apply the observational method during the digging in order to adapt the lining support to the real ground conditions. In this framework, an important number of experimental measurements were foreseen and four sorts of support were defined in project phase. During construction, the choice of the adapted support depended on the monitoring results and on the quality of the soils assessed by geological surveys. In this context, two other supports were realized during the excavation. The digging step for each support was defined variable.

#### **3 MONITORING RESULTS**

Four monitored sections are presented: one in interbedings of marl and limestone (entitled D1), one in marls (D2) and two in clay (D3 & D4). Each of them are not circular. For two of them, only convergence and leveling measurements are available. For the others, more specific measurements were carried out such as strains measurements ahead the face by extrusometers.

Table 2. Main characteristics of each studied section.

Section	PM m	Material	H m	D <sub>face</sub> m
D1	9.75	marl/limestone	22	3.1
D2	457	marl	40	13
D3	510.3	clay	15	1.3
D4	493.8	clay	22	3.84

Each section is presented and analyzed. Table 2 reports the localization in the tube (PM, referred to Fig. 1), the type of material, the overburden from the tunnel axis (H) and the distance from the face at the origin of the convergence and leveling measurements (Dface) for each studied section. D1, D3 and D4 are situated near one of the two tunnel portals (see Fig. 1, low overburden). For the sections D1, the wall support is composed of shotcrete and steel ribs set up every 0.75 to 1.75 m. For D2, it is made up by shotcrete and radial bolts. The diging step varies between 3.5 and 4.5 m. For D3 and D4, the support is more complex. It includes a wall support and an arch invert by shotcrete and steel ribs, set up at the tunnel advance every 1.5 m. And a forepoling by umbrella arch and face bolting are realized every 9 m in the general case. In D3, the excavation was made in partial face otherwise it was made in full face in D4.

#### 3.1 Convergence and leveling

Tunnel wall convergences between reference points are realized by optical sights. Five optical reflector targets are installed in the monitored sections where the excavation is made in full face (at the crown, at  $45^{\circ}$ and at the spring line). When the excavation is realized in partial face, the five targets set up after the excavation of the half higher section are completed by two other targets installed after the excavation of the lower part at the side walls. Leveling measurements are less accurate than convergences ( $\pm 5$  mm against  $\pm 1$  mm) because of the use of several reference stations. But, for all studied sections in the downward tube, leveling curves are exploitable.

The maximal convergence and leveling values are obtained in the clayey zone. In the half higher section D3, leveling measurements reached 25 mm for targets 1 and 2 (Fig. 3) while convergences remain lower than 7 mm.

The maximal convergence value is recorded for wire 4. Although the section is closed by a temporary buton, vertical movements are important. As targets 1, 2 and 3 present higher displacements, the deformation of the section is dissymmetric. The side wall located near the other tube presents larger vertical movements.

After the excavation of the lower part, convergences measurements are higher and reach 12 mm for wire 4

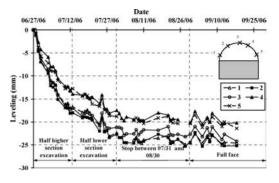


Figure 3. Leveling measurements in the half higher section D3.

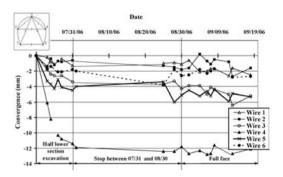


Figure 4. Convergences measurements in the half lower section D3.

(Fig. 4). For the other wires, convergences remain lower than 7 mm. The leveling measurements are not so high than in the half higher section. They remain lower than 6 mm. All targets show similar displacements so the deformation of the section is symmetric after the excavation of the lower part. Measurements in section D3 seem to be stabilized after the return towards an excavation in full face.

In section D4, 25 mm of convergence is registered for wire 4 (Fig. 5) and the speed of convergence is high despite of the distance from the face at the origin which is important (4 m). Vertical movements are important but remain lower than those registered during the digging of the half higher section D3. They reach 14 mm. The section is closed by an arch invert set up at the tunnel advance every 3 m. So, the section is not immediately closed. This can explain the leveling values. Measurements are not stabilized at the end of the excavation.

Displacements recorded in the clayey zone in the downward tube are more important than those measured in the other tube (Eclaircy-Caudron et al. 2007). Consequently, the clayey zone seems to be of better quality in the downward tube. This conclusion is also

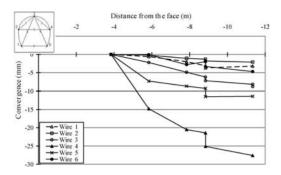


Figure 5. Convergences measurements in section D4.

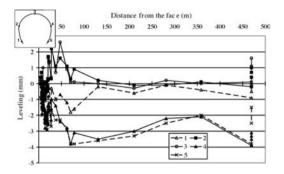


Figure 6. Leveling measurements in section D1.

verified by the strains measurements performed by extrusometers ahead the face.

The section D1 is situated in interbedings of marls and limestone. The convergence measurements show an important dissymmetry confirmed by the leveling measurements (Fig. 6). In fact, vertical displacements are more important for targets 4 and 5 and reach 4 mm. Maximal convergences values are obtained for wires 3 and 4 and reach 8 mm.

This section presents lower measured values than sections situated in a similar soil in the upper tube (Eclaircy-Caudron et al. 2007). This can be explained by the dip of the interbedings which is more favorable in the downward tube (Fig. 7). In fact, the face survey shows almost horizontal interbedings of marls and limestone. Moreover, the face survey presents less fractures and faults in this section than in similar sections in the other tube. So, the soil seems to be of better quality in the downward tube. Measurements are stabilized at a distance from the face equal to 60 m so 9R against 150 m in the other tube.

In section D2, located in marls, the maximal convergence value is obtained for wire 4 as in the section D1 and in marls in the upward tube. It reaches 12 mm before the end of the excavation from the Doubs portal at 03/17/06. Leveling measurements show a dissymmetric deformation of the section as in the upward

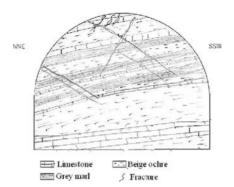


Figure 7. Face survey in section D1.

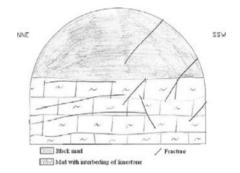


Figure 8. Face survey in section D2.

tube. Maximal vertical movements are recorded for targets 3 and 2 and reach 15 mm before the end of the excavation from the Doubs portal and almost 20 mm after. The side wall situated on the side of the other tube presents more displacements than the other while in section D1 the opposite is noticed. This dissymmetry can be explained by the presence of the exploration gallery and not by the geology which is perfectly symmetric (Fig. 8). Convergences measurements are stabilized before the end of the excavation from the Doubs portal (at 25 m). After the excavation from the Doubs portal and before the beginning of the excavation from the Vallon portal, the measured convergences increase. These displacements increments can be due to the construction phases (drilling of the first face bolts and of the first umbrella arch...). After the beginning of the excavation from the Vallon portal, the convergence measurements stop increasing. So, the digging from the other portal does not influence the convergences measurements of this section while in the other tube the opposite was noticed in this kind of soil. Concerning the leveling, no conclusion can be drawn because after the beginning of the excavation from the Vallon portal, these measurements present some unexplained variations.

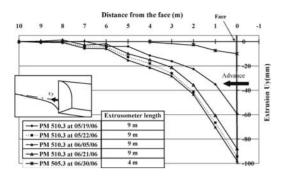


Figure 9. Extrusion evolution versus the distance from the face at different face positions (extrusometer 1).

#### 3.2 Radial displacements in marl at PM 325

These measurements are realized by three borehole extensometers installed radially from the tunnel wall (at the crown and at 45°). Each extensometer has a length of 12 m and included 7 measurements points at 0, 1, 2, 4, 6, 8 and 10 m from the tunnel wall. These measurements can be used to assess the extent of the zone of influence around a tunnel. Movements are measured automatically with an accuracy of 0.02 m. The face was at a distance of 4 m at the origin of the measurements. The point located at 10 m is assumed to be outside of the influence zone. So, the displacement of each point is computed by considering that this point is fixed. The computed displacements can be compared to the convergences measurements realized in marls. Displacements are very low compared to convergences and remain lower than 2 mm. The distance from the face being equal to 4 m at the origin of displacements measurements, low displacements can be explained by the fact that a part of them was lost.

### 3.3 Displacements measured ahead the face in the clayey zone

Two extrusometers of 20 meters length were set up at PM 521 and 501. The feature consists to measure relative displacements between two successive points spaced by one meter. It is destroyed at the tunnel advance. If the anchor point can be considered outside the zone influenced by the excavation then absolute displacement of each point can be computed. Generally, the zone of influence extends until one radius ahead the face. So, to consider that the last point is fixed, the extrusometer has to be longer than one radius. The measurements obtained for each extrusometer are presented.

#### 3.3.1 Extrusometer 1 (PM 521)

Figures 9 and 10 present the extrusion evolution versus the distance to the face and the PM.

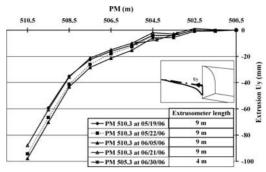


Figure 10. Extrusion evolution versus the PM at different face positions (extrusometer 1).

The length of the extrusometer is indicated in each case. Four measurements were performed when the face was stopped at PM 510 after collapses which took place in May 2006. After the resumption of the excavation in partial face only one measurement was realized and it is not exploitable due to the short length of the extrusometer. The maximal value of extrusion reaches 10 cm (so a strain U/R equal to 1.5% with R = 6.8 m). This value is four times greater than in the other tube. 80% of this extrusion is obtained in the first 4 m so at a distance lower than one radius as in the other tube. Due to a lack of measurements, it is not possible to determine the radius of influence of the face. Measurements should be realized at each digging phases.

#### 3.3.2 Extrusometer 2 (PM 501)

Figure 11 shows the most important extrusion measurements performed with the second extrusometer. Fourteen measurements were recorded. One was made during the excavation of the half higher section. Four were performed between the end of the digging of the half higher section and the beginning of the excavation of the lower part. Four were realized after the beginning of the lower part excavation and three were made after the complete excavation of the part in partial face. Finally, only two measurements were realized after the return towards an excavation in full face. The curves evolution versus the distance from the face is different. This can be explained by a different behavior of soil or of the bolts. The maximal extrusion reaches 4 cm (so U/R = 0.7%). This value is similar to the one obtained in the other tube.

#### 3.4 Strains in the steel rib at PM 493

These measurements are realized by 14 vibrating wires extensioneters installed on steel ribs as shown in Figure 12. They permit to obtain the strain of the steel rib. Movements are measured with an accuracy of  $1 \,\mu$ m/m. Temperatures are also monitored. The face

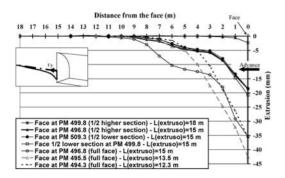


Figure 11. Extrusion evolution versus the distance from the face at different face positions (extrusometer 2).

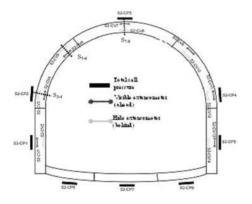


Figure 12. Location of the instrumentation at PM 495.

was at a distance of 1 m at the origin of the measurements. Maximal values are recorded in extensometers S2-CV1 and S2-CV2 and reach 400 µm/m. So, the deformation of the rib is dissymetric. However, the geology is the same in both sides. From the strains measured by a pair of extensometers it is possible to compute the maximum stress induced in the steel rib. For example, the stress  $S_{1-2}$  is determined from the measurements of S2-CV1 and S2-CV2. These stresses are important and the maximal value is obtained in extensometers 1 and 2 and reaches 100 MPa. However, this value remained lower than the limit stress (180 MPa). The computed stresses are shown in Figure 13. The same behavior than in the other tube is observed. Stresses induced in the steel rib permit also to compute the normal force and the bending moment. It appears that the steel rib works essentially in compression as in the upward tube. As regard to the measurements, structural elements are less loaded in the downward tube than in the other tube but displacements are higher.

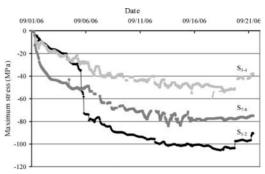


Figure 13. Evolution of the stresses in the steel rib at PM 493.

#### 4 CONCLUSIONS

This paper presents the most representative measurements which occurred during the excavation of the downward tube of the Bois de Peu tunnel in Besançon (France). The measurements permit to better understand the behavior of the complex support set up in the clayey zone and to adapt the support and excavation to the found conditions during the construction. The convergences measurements vary between 6 and 25 mm and the leveling between 4 and 25 mm following the soil. It appeared that structural elements are less loaded in the clayey zone of the downward tube than in the upward tube but displacements are higher. In fact, vibrating wires extensometers set up on the steel rib permit to conclude that this structural element is loaded at 55% of its admissible stress against 70% in the upward tube where the clayey zone seems to be of better quality.

In the upward tube, higher displacements are observed in the geological unit including interbedings of marl and limestone where a heterogeneous geology is observed with many faults. In the downward tube, this unit is less fractured and displacements are reduced. The displacements monitored by extrusometers show that the perturbed zone ahead the face extends to one diameter. And, 80% of the extrusion at the face is obtained at a distance lower than one radius. Comparisons between predictions and measurements permitted to evaluate the actual mechanical parameters of the soils found by the tunnel. Comparisons in the clayey zone showed that this layer had similar properties than those defined at the end of site investigations.

#### REFERENCES

AFTES. 2005. Recommandations relatives aux méthodes d'auscultation des ouvrages souterrains. *Tunnels et Ouvrages Souterrains* 187: 10–47.

- Bae, G. J., Shin, H. S., Sicilia, C., Choi, Y. G. & Lim, J. J. 2005. Homogenization framework for three dimensional elastoplastic finite element analysis of a grouted piperoofing reinforcement method for tunnelling. *International Journal for Numerical and Analytical Methods in Geomechanics* 29: 1–24.
- CETU. 2003. Dossier de consultation aux entreprises. Tunnel de Bois de Peu.
- Dias, D. 1999. Renforcement du front de taille des tunnels par boulonnage – Etude numérique et application à un cas réel en site urbain. *Thesis*, Institut des Sciences Appliquées de Lyon, Lyon, France.
- Eclaircy-Caudron, S., Dias, D. & Kastner, R. 2007. Movements induced by the excavation of the Bois de Peu tunnel in Besancon (France). ECCOMAS thematic Conference on Computational Methods in Tunneling (EURO:TUN 2007 Vienna, Austria, 27–29 August 2007).
- Lunardi, P. 2000. The design and construction of tunnels using the approach based on the analysis of controlled deformation in rocks and soils. *Tunnels and Tunnelling International*: 3–30.

- Oreste, P. P. & Peila, D. 1997. La progettazione degli infilaggi in avanzamento nella costruzione delle gallerie. *Convegno* di Ingegneria Geotecnica; Proc. Intern. Symp., Perugia.
- Peck, R. B. 1969. Advantages and limitations of the Observational Method in applied soil mechanics. *Géotechnique* 19: 171–187.
- Pelizza, S. & Peila, D. 1999. Soil and rock reinforcement in tunneling. *Tunnelling and Underground Space Technology* 8: 357–372.
- Powderham, A. J. & Nicholson, D. P. 1996. The Observational Method in Geotechnical Engineering. *Institution of Civil Engineers*: 195–204. London: Thomas Telford.
- Tan, W. L. & Ranjith, P. G. 2003. Numerical Analysis of Pipe Roof Reinforcement in Soft Ground Tunnelling. 16th International Conference on Engineering Mechanics, ASCE, Seattle, USA.
- Yoo, C. 2002. Finite element analysis of tunnel face reinforced by longitudinal pipes. *Computers and Geotechnics* 29: 73–94.

## Shield tunneling beneath existing railway line in soft ground

#### Q.M. Gong & S.H. Zhou

The Key Laboratory of Road and Traffic Engineering at Tongji University, Shanghai, P.R. China

ABSTRACT: The paper discusses the geodynamic challenges and technical countermeasures in the shield tunnel projects passing under an existing railway line, especially for soft ground conditions. Three main issues needed to be solved by the engineering community: train safety during the period of tunnel construction, accelerated track degradation of track structure in the long term and tunnel structure durability. This paper describes and predicts the nature of the three issues, the countermeasures are also given.

#### 1 INTRODUCTION

Cities and urban population steadily grow, and put stringent requirements for metro transportation. With the development of the metro system, more and more metro tunnels need to pass under an existing railway line, such as metro line 9, 11 and 7 in Shanghai, China. At the same time, the railway lines have been speeded up and are planned for increasingly higher train speeds and higher loads, while existing lines need to be upgraded to allow faster, heavier and more frequent trains. In this context, three main issues need to be considered:

- 1 The "Train Safety" issue: excessive settlement of the track due to the tunnel excavation must be avoided to ensure the permissible deviations in track geometry.
- 2 The "Track Structure Durability" issue: increasing traffic density, loads and metro tunnel structure bring about higher dynamic actions and more intensive track maintenance.
- 3 The "Tunnel Structure Durability" issue: dynamic actions are to be sustained by tunnel structure.

This paper discusses these issues in Shanghai, China. The train safety issue and countermeasures during the metro tunnel construction are concentrated.

#### 2 SITE DESCRIPTION

The site chosen for the project was the Jiading area in Shanghai, where metro line 11 passes under Hu-Ning railway Line. Figure 1 shows their plane and vertical position. The angle between them is 85 degree, almost perpendicular. The overburden soil depth of shield tunnel is 11.08 m, its outside diameter is 6.2 m and the thickness of tunnel lining is 0.35 m, the groundwater level is about 1m below the ground surface.

Table 1.	Soil	parameters.
----------	------	-------------

	Soil layer						
Parameters	21	22	$(3)_1$	5	6		
Gravity ( $\gamma$ ) (kN/m <sup>3</sup> )	18.4	18.1	17.4	17.7	19.3		
Cohesion C(kPa)	13.0	8.0	9.0	13.0	45		
Angle of friction $\Phi(degrees)$	18.5	24.0	17.0	13.0	15		
Lateral ratio of earth pressure K <sub>0</sub>	0.46	0.41	0.50	0.50	0.40		
Compressive Modulus (MPa)	5.25	7.22	5.50	6.77	10.5		

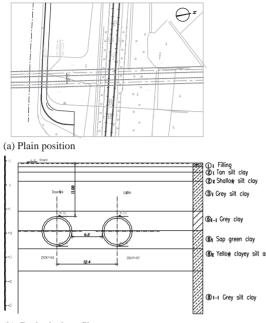
\* In Table 1,  $@_1$  is Tan silty clay,  $@_2$  is Sallow silty clay,  $@_1$  is grey silty clay, @ is grey clay, @ is Sap green Clay.

The 4.5 m high embankment of Hu-Ning railway line has been constructed in 1908. The train speed have increased up to 250 km/h at 18th, April, 2007 and demands for shortening travel times are rising. Figure 1(b) also shows the geological profile of the chosen site, the soil properties are in table 1. According to the geotechnical investigation, the site can be characterized by a 1.5 m weathered crust over a layer of soft clay with a thickness of about 13.0 m. Under these layers lies silty clay whose stiffness is more greater than the overlying soils.

Track-side buildings are almost not existing, so the influences on environment can be ignored.

#### 3 THE SAFETY OF RUNNING OF THE TRAINS DURING TUNNEL EXCAVATION

Tunnel excavation may induce adverse effects on nearby existing structures and services (e.g., deformation on tracks, derail of trains). The accurate prediction



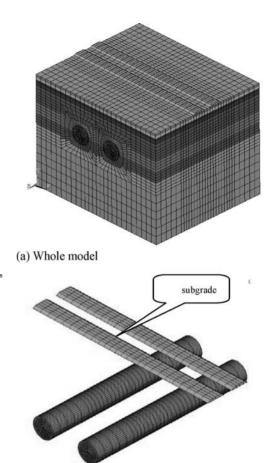
(b) Geological profile

Figure 1. Layout of railway line and shield tunnel.

of tunneling effects poses a major challenge during design.

The effects of tunnel advancement during construction on the ground and railway track are three dimensional and transient. In this paper, a 3D, elastoplastic analysis was performed. Figure 2 shows the 3D finite element mesh. The mesh was 50 m long, 45 m wide and 40 m high. Eight-node brick elements and four-node shell elements were used to model the soil and the concrete lining, respectively. Roller supports were applied on all vertical sides of the mesh, whereas fixed supports were assigned to the base of the mesh. Therefore, the movement normal to all vertical sides of the mesh and the movement in all directions at the base of the mesh were restrained. The water table was located at the ground surface. An elasto-perfectlyplastic soil model using the Mohr-Coulumb failure criterion with a non-associated flow rule was adopted in this study. The tunnel lining was modeled as a linear elastic material. The Young's modulus and Poisson's ratio for the tunnel lining were taken as 30 GPa and 0.3, respectively. The unit weight of the tunnel lining was 24 kN/m<sup>3</sup>.

Figure 3 shows the progressive changes in the tunneling-induced surface settlement under the track as the tunnel advances. With further excavation, the surface settlements continue to increase. Considering the tunneling-induced deformation, the irregularity of the track is shown in Figure 4.



(b) Location relationship between tunnel and railway

Figure 2. Finite element mesh.

The derail coefficient and rate of wheel load reduction were shown as table 2 using locomotive-track dynamic coupling model. According to the protocol published by the Ministry of Railway in China, the stability and safety of the train will be threatened if the running speed of train is above 100km/h (passenger car) and 60 km/h (freight), so the train speed must be limited during the tunnel construction.

#### 3.1 Influence on tunnel lining by running trains

After the construction of the tunnel, the dynamic stresses induced by a running train will act on the tunnel lining for a long time, even until the railway line or the metro line will be abandoned. So the influence of the dynamic stresses must be considered. In general, the dynamic stresses induced by a train running dissipate quickly because of the nonlinear material and the viscous damping. The influence depth is only about 3m under the sub-grade bed. But the values of dynamic stresses will be greater with the tunnel under

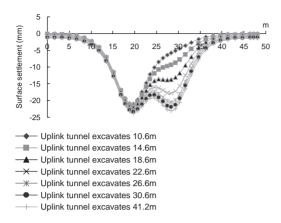


Figure 3. Tunneling-induced surface deformation.

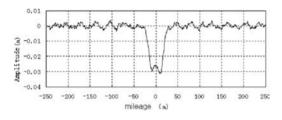


Figure 4. Longitudinal irregularity of the tracks by tunneling-induced deformation.

Table 2. Maximum value of derail coefficient and wheel load reduction.

Vehicle		Passenger car		Freight					
Speed (km/h)		90		100		50		60	
Item		1	2	1	2	1	2	1	2
Pair of wheels	I II III IV	0.63 0.51 0.48 0.60	0.13 0.10 0.08 0.11	0.59 0.56	0.20 0.13 0.11 0.21	0.56 0.53	0.15 0.15	0.78	0.22

\*In Table 2, ① represents Derail coefficient, ② represents wheel load reduction.

the railway line, because the stiffness of tunnel is much greater then the soft soil at the same depth.

The dynamic stresses acting on tunnel lining have been established using a dynamic numerical model. Laboratory tests have shown that soil stiffness and damping change with cyclic strain amplitude under dynamic cyclic loading conditions. To simplify, the soil is modeled using equivalent Linear analysis, the damping ratio also changes with strain. Figure 5 shows the loads acting on the tunnel lining including the

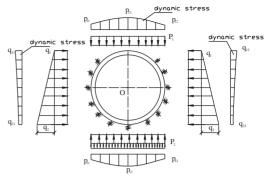


Figure 5. Load-structure model including dynamic stress.

dynamic stresses. It is quite different from the conditions with only static stresses. The long-term actions of dynamic stresses will influence the service life time of tunnel structure.

#### 4 SAFETY COUNTERMEASURES

#### 4.1 Countermeasures to ensure the train safety during tunnel excavation

There are basically three ways to decrease the influence on safety of the trains during tunnel construction: (a) by decreasing the tunneling induced deformation; (b) by increasing the longitudinal bending stiffness of track structure itself; (c) limit the running speed of the train.

Decreasing the tunneling induced deformation may for instance be done by means of improving the stiffness and strength of the soil around the tunnel. This can be done by for example lime-cement piles, jet-piles or other deep-mixing methods. These countermeasures are easier to implement as part of the foundation work of new lines, than as a retrofitting method under existing lines.

Figure 6 shows the reinforcing areas and methods around the tunnel, including jet-piles and grouting. These countermeasures decrease the track deformation during the tunneling. The value of the settlements is only 15% of the original deformation, so the remaining track irregularity is small. At the same time, the bearing capacity of the foundation of the railway track after the tunnel construction will increase.

The construction parameters should be controlled during the metro tunnel excavation to decrease the disturbing of the soil. Table 3 shows the recommended parameters. The controlling and monitoring systems should also be perfect to follow the track deformations. Figure 7 shows the measured deformations at the site during tunnel construction. ① is the preliminary settlement, ② is the settlement induced by shield arrival, ③ is the settlement during tunnel construction, ④ is the

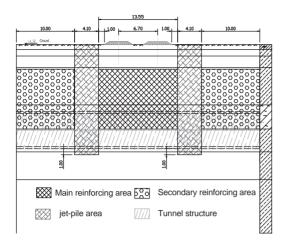


Figure 6. Reinforced soil around the tunnel.

Table 3. Construction parameters.

Construction parameters	Construction site Hu-Ning Railway	
Support pressure (MPa)	$0.20 \sim 0.22$	
Grouting pressure (MPa)	$0.19 \sim 0.21$	
Grouting amount (m <sup>3</sup> )	$3.3 \sim 4.1$	
Grouting speed (L/s)	$1.1 \sim 1.4$	
Construction speed (cm/min)	$2.0 \sim 2.5$	

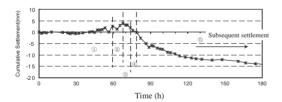


Figure 7. Measured deformation at the site during tunnel construction.

settlement induced by sub-optimal filling of shield's rear, (5) is the subsequent settlement. The results show that the deformation of the track has been controlled effectively. These approaches can be used.

#### 4.2 *Countermeasures to protect the tunnel lining*

Considering the long life time and the exhausted strength, the lining stiffness and strength should be increased. So the reinforcement ratio of the tunnel lining under the railway line has been increased.

#### 5 CONCLUSIONS

Tunnels passing under an existing railway line must ensure the train safety, the track structure durability and the tunnel structure durability. These problems are especially important in soft ground areas. In order to decrease the dynamic stresses acting on the tunnel lining, to decrease the tunneling induced deformation, and to even increase the bearing capacity of the ground, the soft soil and tunnel lining stiffness must be reinforced.

#### ACKNOWLEDGEMENTS

This research was sponsored by National NaturalScience Foundation of China (No.50678131) and Science and Technology Commission of shanghai Municipality (No.06ZR14083). Grateful appreciation is expressed for these supports.

#### REFERENCES

- Esveld, C. 1989. Modern Railway Track. MRT-Productions, ISBN 90-800324-1-7.
- Esveld, C. 1997. Innovation for the control of infrastructure maintenance. *Rail International/Schienen der Welt*.
- Suiker, A.S.J. 1996. Dynamic behaviour of homogeneous and stratified media under pulses and moving loads. TU Delft, Report 7-96-119-1.

### Case history on a railway tunnel in soft rock (Morocco)

A. Guiloux, H. Le Bissonnais & J. Marlinge

TERRASOL, Geotechnical consulting engineers, France

H. Thiebault, J. Ryckaert, G. Viel & F. Lanquette *SETEC TPI, France* 

A. Erridaoui MAROC SETEC, Morocco

M.Q.S. Hu TEC Engineering, P.R. China

ABSTRACT: Ras R'Mel Moroccan tunnel is a 2.6 km long and  $60 \text{ m}^2$  section single track railway tunnel. It was excavated in a complex geological context of highly heterogeneous and highly deformable soft rock. The purpose of this paper is first to describe the methodology used for specific design of support and lining. The deformation monitoring process applied during construction will then be detailed, and an analysis of the deformations measured will be given, making a comparison with design calculations results. Main concerns were potential face instability and high tunnel deformations.

#### 1 PROJECT DESCRIPTION

The Ras R'Mel tunnel presented in this case-history is a single track railway tunnel, part of the new railway line being constructed in Morocco between the town of Tangier and its new Mediterranean harbour, located on the coast about 30 km north-east of the city. The tunnel itself is located some 15 km east of Tangier, near the location of Ras R'Mel. Western portal is called "Tangier portal", eastern one is named "Ras R'Mel portal". Kilometric points on the railway line increase from Tangier portal (PK 26+841) to Ras R'Mel portal (PK 29+445).

Chinese company TEC Engineering won the contract for construction of the tunnel, which also included construction of a second tunnel, shorter (600 m long) but in a similar geotechnical context, and construction of the railway platform in between the two tunnels. For both tunnels, following a previous collaboration on the Meknes tunnel project (another Moroccan railway tunnel, the study of which was described in a previous paper for AFTES October 2005 international congress), TEC Engineering chose French engineering companies SETEC TPI and TERRASOL, subsidiaries of French group SETEC, as its consulting engineers for specific design of support and lining, and for deformation analysis during construction. This paper focuses on design and construction issues regarding Ras R'Mel tunnel.

The tunnel, of a 2600 m length and a  $60 \text{ m}^2$  section, was excavated in highly heterogeneous flyschs, with up to 150 m depth in its central part (Cf. Fig. 2). The excavation was given a horse-shoe shape of a 8.5 m height and a 7.6 m to 7.8 m width, depending on the lining thickness in order to preserve the ultimate circulation clearance (Cf. Fig. 1). Four zones with provisional enlarged sections were excavated, in order to allow entering and exiting trucks to cross during construction. The enlarged sections are of a 10.2 m width and a 9.4 m height. At final stage, the over-excavation of those zones will be filled with lining concrete.

The entrance portals were at a depth of only 10 m. As those portals were almost vertical, and as the quality of the pelites, classified as soft rocks, is rather poor, it was decided to stabilise the portals building two 22 m long "false tunnels" (one at each portal) using a cut and cover technique.

Indicative tunnel profile is given on Figure 2.

From each portal, and on a total length of around 1600 m, the excavation is at a depth lower than 50 m: great surface settlements were expected in this zone. In the central part of the tunnel, on a length of 1000 m, excavation is at a depth of about 150 m: low surface settlements but high convergence and tunnel settlements

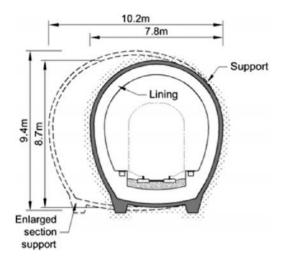


Figure 1. Type profiles (basic and enlarged cross-sections).

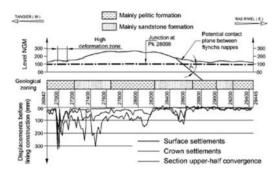


Figure 2. Tunnel profile and deformations measured before lining construction.

were expected in this zone. As the tunnel is in a countryside environment with no construction at surface, surface settlements were not expected to be of a major concern, but tunnel convergences were.

#### 2 GEOTECHNICAL CONTEXT

Ras R'Mel tunnel is located in a geologically complex and highly tectonised zone. Two flyschs formations can be found, the older one (the Tisiren nappe, alternating levels of sandstone and pelite) being on top of the younger one (the Beni Ider Nappe, softer clayeycalcareous flysch) being a thrust-nappe. The contact plane between those formations slightly dips towards east. It was expected to be intersected by the tunnel but could not be firmly identified during construction. This contact is known to be intersected by several sub-vertical faults disturbing locally the lithology.

Table 1.	Geomechanical	characteristics.
----------	---------------	------------------

Formation	Mainly pelite	Mainly sandstone	Fully sandstone
$\gamma$ (kN/m <sup>3</sup> )	24	24	24
UCS (MPa)	0.66	_	46
C <sub>u</sub> (kPa)	200	500	_
φ <sub>u</sub> (°)	0	0	_
C' (kPa)	20	100	1000
φ' (°)	28	30	35
$E_0$ (MPa)*	500	1000	1500
E <sub>∞</sub> (MPa)*	250	_	_
ν	0,3	0,3	0,3
$\sigma_{\rm g}$ (kPa)	200	0	0

 $^*E_0$  and  $E_\infty$  are respectively short and long term Young's moduli.

Furthermore, the geological formations are complex themselves: both flyschs formations consist of alternating levels of pelites (soft rock with Unconfined Compressive Strength (UCS < 1 MPa) and highly resistant sandstones (UCS = 30 to 40 MPa). Four (4) kinds of formations were then expected:

- non-altered, highly fractured pelites, with occasional breccia zones, Rock Quality Designation (RQD) ranging from 50% to 75%, and sandstones levels RQD, expressed as a percentage, is the summed length of core pieces greater than 10 cm measured for a 1 m long core pass; once excavated, the pelites get quickly altered;
- blocky sandstones, with occasional pelite levels;
- alternating levels of pelites and sandstones of low thickness, that can be found intricated in blocky sandstones;
- claystone breccia.

As blocky sandstones layers where not persistent, no continuity for those levels could be drawn between the borehole logs, and no accurate geological profile could be built for the project. It has then been decided, to propose an indicative geological zoning along the axis of the tunnel, for support and lining design of the project, pointing at three main categories of formations:

- formations mainly pelitic;
- formations mainly comprising sandstones;
- formations fully comprising sandstones;

Table 1 sums up the average mechanical characteristics given to these equivalent formations. Short term cohesion values for mainly pelitic formation are the updated values following analysis of first measures of tunnel deformations.

Potential swelling pressure  $\sigma_g$  in pelites is evaluated to 200 kPa, applying under lining invert.

Because of the thrust nappe context, high values for horizontal stresses were expected, which was taken into account in the design calculations setting parameter  $K_0$  to 1 (horizontal stress equal to vertical one).

During construction, sandstone levels were confirmed non-persistent, which meant that the "Fully Sandstone" formation was non relevant. The two other formations describe soft rocks with rather weak characteristics: under expected geostatic stress state, the stability ratio  $\sigma_0$ /UCS ranges from 2 to 3,5 (>1) meaning that stress level is high enough for excavation to be unstable without support.

From a hydrogeological point of view, captive water of limited extent had been identified in the boreholes, sheltered by sandstone layers. The surrounding pelite matrix is little permeable. However, the highly fractured pelites and sandstones might allow limited water flows to reach the tunnel. It was then decided to protect tunnel lining with a watertight membrane on crown and shoulders.

#### **3 PROJECT DESIGN**

TERRASOL and SETEC TPI performed specific design studies for support and lining of the tunnel, as well as stability analysis of tunnel face. In order to control tunnel and surface deformations, a full face excavation method was considered.

Based on the indicative geotechnical zoning proposed for the tunnel, six calculation profiles were defined, named P1 to P6. Those profiles differed by the formation and depth taken into account for calculation (each profile is supposed to be composed of one single formation). Five additional profiles were studied, corresponding to potential enlarged section zones. The precise location of those zones would be defined precisely during construction, depending on real geological conditions.

#### 3.1 Tunnel face stability analysis

Tunnel face stability analysis was performed using TERRASOL convergence-confinement code *TUN-REN*. The code uses a C-Phi reduction method, based on the analytical model EXTRUSION developed by Wong et al. (1999). Hypothesis are the ones of convergence-confinement method (circular section, isotropic, homogeneous and infinite medium, uniform stress field), with two additional ones:

- tunnel face is supposed spherical, and stress-strain field follows a spherical symmetry;
- tunnel face heading is modelled by taking into account a decreasing radial pressure at tunnel face.

Material behaviour is supposed elasto-plastic, following Mohr-Coulomb or Tresca criterion.

From initial  $(c, \phi)$  short-term values, mechanical characteristics of the ground are progressively reduced

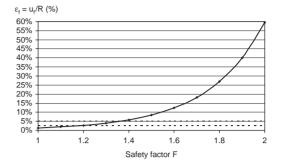


Figure 3. Tunnel face stability analysis for P1 profile (mainly pelitic formation at 50 m depth).

by a security factor F. For each set of mechanical characteristics, average radial strain is calculated at tunnel face, and results are presented via a chart giving tunnel face relative deformation ( $\varepsilon_f = u_f/R$ , where  $u_f$  is extrusion value and R excavation radius) versus security factor F. Stability is evaluated considering security factor values  $F(\varepsilon_f = 2,5\%)$  and  $F(\varepsilon_f = 5\%)$  giving relative deformations of respectively  $\varepsilon_f = 2,5\%$  and  $\varepsilon_f = 5\%$ . Figure 3 shows an example of result chart, where  $F(\varepsilon_f = 2,5\%) = 1.2$  and  $F(\varepsilon_f = 5\%) \approx 1.4$ .

Calculation showed that tunnel face stability depends on tunnel depth. Whatever the formation considered, both security factors  $F(\varepsilon_f = 2,5\%)$  and  $F(\varepsilon_f = 5\%)$  were greater than 1 for depth lower than 50 m. For maximal depth (150 m), security factors where lower than 1. Sensitivity analysis showed that tunnel face became unstable under 60 to 70 m overburden. Ground behaviour appears to be limit-plastic at 50 m depth, and totally plastic at 150 m depth.

In order to perform construction using full-face excavation method, TERRASOL-SETEC TPI advised face reinforcement would then be necessary.

#### 3.2 Support and lining design

The construction method chosen was a half-sections method. Support and lining, as defined in the contract, were as follows:

- support: all over the section, including invert, 5 cm shotcrete confinement layer, plus HEB 180 steel ribs settled in a 18 cm thick shotcrete layer; steel ribs spacing had to be defined by design calculations, and was expected to range from 0.75 m to 1.5 m depending on geological conditions;
- *lining*: watertight layer on crown of the tunnel, and 40 to 60 cm thick concrete reinforced at lateral walls and invert junction; reinforcement sections had to be defined by design calculations.

In order to estimate tunnel deformations and solicitations of lining, a staged calculation was run with finite elements model (FEM). Calculation

Table 2. FEM support calculations (after calculation profile name, P and S are respectively for "Mainly Pelite" and "Mainly sandstones", figure indicates depth).

Calculation	Surface settlement	Crown ground settlement	Horizontal convergences (mm)		
profile	(mm)	(mm)	Ground	Support	
P1 (P, 50)	3.2	13	44	9	
P2 (S, 50)	1.1	4.5	11	4	
P3 (S, 150)	_	25.8	60	16	
P4 (P, 150)	_	97.6	190	12	
P5 (P, 10)	0.4	1.8	6.5	3	
P6 (P, 30)	0.7	3	8	3	

procedure was lead through TERRASOL convergenceconfinement code TUNREN, and through French LCPC finite elements code CESAR. For each profile, calculation steps were as follows:

- Convergence-confinement calculations: estimation of the deconfined rate  $\lambda_0$  after tunnel full-face excavation, using TUNREN code;
- FEM calculations: full-face excavation (deconfinement up to  $\lambda_0$  value previously estimated, load applied to excavation walls set to ( $\sigma_N = (1 - \lambda_0)\sigma_0$ ); completion of support and final deconfinement ( $\sigma_N = 0$ ); completion of lining (mechanical characteristics are set to long-term values, swelling pressure is applied under invert and ground creeping is taken into account by option "EFD" of the calculation code).

Results were expressed as deformations and stress fields for support, and only as stress fields for lining. Table 2 sums up displacements results after support calculations.

Results showed that surface settlements were expected to be lower than 5 mm.

For a given formation, tunnel deformations were expected to grow with tunnel depths. For formations with mainly sandstones, maximal convergences in support and crown settlements are respectively 16 mm and 25.8 mm, which is rather limited. For mainly pelitic formations, they are of respectively 12 mm and 97.6 mm, which shows that at maximal depth, tunnel behaviour is completely plastic and deformations can become important. Numerical model indicates crown ground settlements about half as high as ground horizontal convergences for depth greater than 10 m.

Support calculations indicated that great deformations (of about 100 mm) could occur in support at depth greater than 50 m. Support completion would then be of a major concern regarding control of those deformations.

Lining calculations showed that only minimum reinforcement would be necessary at junction between lateral walls and invert.



Figure 4. Upper half-section excavation with central part of the face left in place (Tangier face, around PK 27+960).

#### 4 CONSTRUCTION

Works began in February 2006, with access cut excavation. Tunnel excavation began in March 2006 at both portals. Junction of the two faces occurred at PK 28+098 on July the 5th of 2007, after 15 months. Average heading was of about 80 m per month at Tangier face, and of about 90 m per month at Ras R'Mel face. Ras R'Mel face appeared to be in better geological conditions than the other face, which allowed higher heading rate and lighter support: steel ribs could be spaced from 1,2 to 1,5 m in average, against 0,75 m in average at Tangier face.

TEC Engineering decided not to apply face reinforcement treatment. As tunnel-face was feared to be unstable, it was then decided to apply the same halfsections excavation method as used for Meknes tunnel. In order to control tunnel face stability, during excavation of section upper-half, and in zones with poor quality ground, central part of the face was left in place until upper-half support completion (Cf. Fig. 4).

Furthermore, as lower-half excavation and support completion occurred in average 4 days after upper-half excavation, behaviour of tunnel face was almost equivalent to a full face excavation, which helped control deformations.

Opening ranged from 0,75 m in zones with mainly pelites, to 1,5 m in zones with mainly sandstones, and was adapted to the deformations measured. A provisional formwork of wooden boards hold by steel bars prevented shotcrete loss during shotcrete projection, as can be seen on Figure 3.

A total of 4 crossing zones with enlarged section was excavated, 2 at each face.

As expected, main concern during construction was tunnel deformations due to excavation of soft deformable rocks under high stress level.

#### 4.1 Surface deformations monitoring and analysis

From each portal, and on a total length of 1500 m, excavation is at a depth lower than 50 m, Ras R'Mel face even having to cross a zone at a depth of only 15 m. Surface settlements where then monitored. Every 20 m along tunnel axis, a monitoring cross-section was set using 5 surveyor's rods set perpendicularly to the tunnel axis (numbered from 1 to five, with 10 m to 15 m spacing, central rod number 3 being above tunnel axis). For a given monitoring section (named following the corresponding kilometric point of the tunnel), measurements started 2 to 4 weeks before tunnel face reached the given kilometric point. The frequency was one measure per day up to stabilisation, and one measure a week once stabilisation reached. Settlements in the centre part of the tunnel, at greatest depth (150 m), were not monitored.A chart showing final surface settlements measured before lining construction is presented on Figure 2.

#### 4.1.1 Ras R'Mel face

At Ras R'Mel face, settlements for a given monitored section began when tunnel face was approximately 20 m ahead of the corresponding PK. Stabilisation was reached as tunnel face was 80 m further (approximately after 1 month). At tunnel axis, final settlements are of about 20 mm for the first 100 m after portal. Further on, they are inferior to 5 mm. No relation could be established between tunnel depth and surface settlements. The low settlements did not allow observation of relevant transverse settlement throughs, and the crossing of the very low depth zone, between PK 28+924 and PK 28+870, did not induce any increase of surface settlements.

Surface settlements measured were as expected after FEM calculation.

#### 4.1.2 Tangier face

At Tangier face, settlements for a given monitored section began as tunnel face was about 20 to 40 m ahead of the corresponding PK. Stabilisation was reached when tunnel face was 80 to 120 m further (1 to 1.5 month later). Above tunnel axis, final settlements are rather high in the first 100 m after the portal (from 20 to 50 mm). These values, higher than the ones calculated, could be explained by the proximity of the access cut to the portal, and by the low depth of the tunnel in this zone. Nevertheless, as a 45 m overburden was reached, and settlements seemed to decrease, tunnel face hit a rather weak zone (from PK 26+978 to PK 27+140), in which surface settlements were much higher than expected (greater than 20 mm at tunnel axis, reaching up to 190 mm).

Figure 5 shows settlements evolution at PK 27+000. Tunnel face upper-half reached the corresponding section, when already 20 mm settlements had occurred at surface. The effect on surface

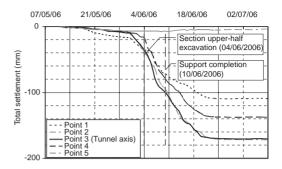


Figure 5. Settlements at PK 27+000 (Weak zone at Tangier face).

settlements of tunnel face lower-half reaching the section, can hardly be seen as no stabilisation had occurred at that moment. Stabilisation was reached, two weeks after completion of tunnel support. The final settlement value above tunnel axis is of 175 mm. Those settlements can be related to high tunnel deformations as will be discussed further on.

Final settlements for points 5 and 4, respectively compared with those for points 1 and 2, are lower. Almost no settlement occurs at point 5. This dissymmetry in transverse settlement through is relevant of what could be observed at Tangier face: surface terrain is naturally dipping south, so that the cross-section monitored is in direction of the dip. Points 1 and 2 are on the upper side of the dip, whereas points 4 and 5 are on the lower side. Crossing a weak zone exaggerated this phenomenon.

As no building was constructed above the tunnel, such high settlements were not of a major concern.

After this zone of high deformations, settlements are lower than 20 mm. From pk 27+140 to pk 27+340, surface settlements at tunnel axis increase from 7 mm to 17 mm, following depth increase from 40 m to 90 m. For greater PK, settlements decrease while depth increase. As the centre part of the tunnel is reached (with depth of 150 m), final settlements are of a 5 mm average, which fits the settlements calculated by the FEM simulations, and is compatible with measures relative to Ras R'Mel face.

Surface settlements were then partly dependent on tunnel depth, and partly on lithology (greater settlements in highly deformable soft rock).

#### 4.2 Tunnel deformations monitoring and analysis

As said before, tunnel deformations were expected to be of a major concern. Every 20 m along tunnel axis, a monitored cross section was established after support completion, equipped with 5 measuring targets. It allowed measurements in the support of crown and side wall settlements, and of section upper and lower-half convergence. Due to construction constraints, convergence measurements are of questionable reliability. However, it clearly appears that final measured convergences are highly variable from one section to another, depending on lithology rather than on tunnel depth. The convergence values reached range from 20 mm to more than 300 mm, which is far greater than estimated by calculation. Furthermore, differences appear between the two tunnel faces, Ras R'Mel face once again showing lower deformations than Tangier's. Figure 2 shows tunnel deformations.

At both faces, tunnel deformations appeared to stop as soon as support was completed, closing the excavated sections. Later creeping could be identified as lining was being constructed.

#### 4.2.1 Ras R'Mel face

From Ras R'Mel portal, up to junction PK, tunnel deformations were limited, section upper-half deformations ranging from 10 mm divergence to 30 mm convergence, crown settlements being of a comparable size. Those values are limited, but higher than calculated for corresponding profiles (P1, P2, P3, P5 and P6).

From PK 28+620 up to PK 28+400, tunnel deformations are higher, reaching 130 mm convergences and 78 mm crown settlements. In this zone, crown settlement is about half of convergences. This local increase in deformations should be related to decreasing quality of excavated rock rather than to an increase of overburden. The contact plane between the two flyschs formations might have been crossed along with this change of rock quality, but could not be identified firmly.

Approaching PK of faces junction, the deformations increased and reached values consistent with the ones measured at the other face.

#### 4.2.2 Tangier face

As shown for surface settlements, after a zone with rather high deformations due to perturbations induced by the vicinity of Tangier portal, tunnel suddenly hit a zone of very high deformations at limited depth (around 50 m), located between PK 26+978 and PK 27+140.

High convergences in the section upper-half occurred, greater than 100 mm and reaching 330 mm. Crown settlements were of values half as high. Trying to secure the zone, it appeared that deformations stopped as soon as section's support was completed. Final stabilisation was reached within one month after section opening, although later creeping could be identified locally causing support deformations. The values measured overcome by far the values calculated for support deformations. They are thought to be linked with local mechanical characteristics for the rock lower than expected.

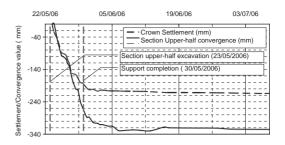


Figure 6. Greatest tunnel deformations, measured at PK 26+978.

It was feared that deformations would become unacceptable when reaching maximum depth and excavation would become highly unstable. As soon as it became obvious that support section completion blocked further deformations, it has been decided to go on construction with the same method, operating excavation carefully, and keeping opening and steel-ribs spacing at only 0.75 m as long as necessary.

Figure 6 shows the evolution of tunnel greatest deformations, which occurred at PK 26+978 (Stabilisation was reached by mid-June).

The high deformations zone goes up to PK 27+780, where greatest depth is reached. From this PK, and up to the junction of tunnel faces, deformations are still important (about 90 to 100 mm in section upper-half convergences, and 50 mm in crown settlements) but lower than the previous zone. This is supposed to be linked with depth increase, and a better stiffness of soft rock mass. The deformations measured in this zone, are higher than convergences calculated for the corresponding profile (P4).

Deformations values increased as junction section approached, reaching values compatible with the ones observed at Ras R'Mel face.

From a general point of view, it is thought that part of the deformations calculated as "ground deformations before support completion" (theoretically non-measurable), are measured as upper-half support convergences, because of the half-sections excavation method used, (measures begin as section excavation is not completed, and upper-half support is not totally blocked as no invert is constructed at that stage). This could explain why measured tunnel deformations are greater that the ones calculated. Section behaviour is then intermediate between full-face (as calculated in specific design studies) and half-section excavation.

Face potential instability had been pointed at by calculation. The cautious excavation method chosen allowed no collapse to occur on Ras R'mel tunnel works. The other tunnel for the railway project (Sidi Ali tunnel), operated by the same team of company in similar context but with slightly better geological conditions did not receive such cautious treatment at tunnel face. Tunnel face collapsed on 31/07/2006, which shows that TEC Engineering method was appropriate regarding Ras R'Mel tunnel face.

#### 4.2.3 Zones with enlarged section

The four zones with enlarged section did not induce greater deformation compared to surrounding basic sections.

#### 5 CONCLUSION

Ras R'Mel tunnel had to be excavated in highly variable geological formations, highly deformable soft rocks, and at a high stress level. To control tunnel deformations and tunnel face potential instability, it was then necessary to use half sections excavation and, when necessary, divided section excavation for upper-half. This method derived from a method previously used at Meknes tunnel (Marocco), operated by the same team of companies. During construction, sections behaviour was intermediate between full-section and half-sections excavations. Design studies gave a rough size range for surface and tunnel deformations, but several zones with tunnel high deformations were crossed, which required cautious excavation. Nevertheless, the excavation method used allowed to control deformations, that were blocked as soon as support was completed.

At time this paper was written, tunnel completion was programmed for mid-September 2007.

#### REFERENCES

- Guilloux, A., Le Bissonnais, H. & Pre, M. 2005. Tunnel de Meknes (Maroc): conception et réalisation d'un tunnel en terrain meuble sous faible couverture, AFTES, Les tunnels, clé d'une Europe durable; Proc. Intern. Congress., Chambéry, 10–12 October 2005. Lyon: Spécifique.
- Terrasol, Tunren. Tool for tunnel conception (lining and tunnel face stability), *convergence-confinement and extrusion calculation code*.
- Wong, H., Trompille, V. & Dias, D. 1999. Déplacements du front d'un tunnel renforcé par boulonnage prenant en compte le glissement boulon-terrain: approches analytique, numérique et données in situ, *Revue Française de Géotechnique* n°89: 13–28.

### Observed behaviours of deep excavations in sand

B.C.B. Hsiung

Department of Civil Engineering, National Kaohsiung University of Applied Sciences, Kaohsiung, Taiwan, R.O.C.

H.Y. Chuay

Mott MacDonald, Kaohsiung, Taiwan, R.O.C.

ABSTRACT: In this paper, structural and ground behaviours of several excavations in Kaohsiung, Taiwan were described and examined. Based on behaviours observed from these excavations, it was found that the maximum lateral wall displacement ( $\delta_{hmax}$ ) in relation to the maximum excavation depth (H<sub>e</sub>) is approximately 0.034 to 0.3%. The ratio of maximum surface settlement ( $\delta_{vmax}$ ) to  $\delta_{hmax}$  varies from 0.5 to 0.7 for the excavation constructed by bottom-up method and from 1.3 to 1.8 for the excavation using a semi-top-down method. The zone at ground level affected by the excavation is up to 3 times the maximum excavation depth behind the diaphragm wall which retains the earth during construction. This finding is different from previous conclusions reported by Clough & O'Rourke. It is apparent that chemical churning pile does not effectively reduce influences of the excavation on adjacent buildings.

#### 1 INTRODUCTION

Ground deformations induced by deep excavations in clays have been explored widely (Wong et al., 1997, Hsieh & Ou, 1998, Hsiung, 2002, Liu et al., 2005) but studies regarding observed behaviours of deep excavations in sand are comparatively limited (Burchell, 2000 and El-Nahhas, 2006). In this paper, case histories from excavations in Kaohsiung City, Taiwan provide an opportunity to explore structural and ground behaviours induced by excavations in sand. Empirical approaches for evaluating lateral wall movements and surface settlements were studied and discussed. Further, the effectiveness of chemical churning pile for house protection and prediction of prop load were also discussed in this study.

#### 2 THE SITES

Several deep excavations for the Orange Line of the Kaohsiung Rapid Transit Systems (KRTS) have recently been carried out in an area where the ground conditions are relatively uniform. Among them, excavations of three underground stations, O6, O7 and O8 were selected for this study. The sites are located at the centre of Kaohsiung City, Taiwan and the maximum excavation depth varies from 19.6 m to 20.9 m. Lengths and widths of these excavations are in the range of 178 to 215 m and 22 to 24 m, respectively. All excavations are retained by 1.0 m thick reinforcement concrete diaphragm walls. Construction sequences of

393

Table 1. Construction sequence at O6.

Construction activity	Period (day/month/year)
Excavate to 3.4 m below ground level	24/10/03-27/10/03
Install 1st level prop at 2.5 m below ground level	02/11/03-06/11/03
Excavate to 6.8 m below ground level	08/11/03-12/11/03
Install 2nd level prop at 5.9 m below ground level	13/11/03-16/11/03
Excavate to 10.0 m below ground level	24/11/03-27/11/03
Install 3rd level prop at 9.1 m below ground level	04/12/03-07/12/03
Excavate to 13.5 m below ground level	14/12/03-16/12/03
Install 4th level prop at 12.6 m below ground level	28/12/03-31/12/03
Excavate to 17.0 m below ground level	06/01/04-08/01/04
Install 5th level prop at 16.1 m below ground level	09/01/04-12/01/04
Excavate to 19.6 m below ground level	25/01/04-02/03/04

O6, O7 and O8 are listed in Tables 1 to 3. H-type steel sections were selected for horizontal props and the levels of props at these sites are also described in Tables 1 to 3. The horizontal spacing of props is similar at O6. O7 and O8 and is approximately 4.5 m. In addition, angle bracing and waling were constructed using H-type steel sections in order to strengthen the strut system.

The main soil stratum of these sites is similar and consists of medium to dense silty sand with several

#### Table 2. Construction sequence at O7.

Construction activity	Period (day/month/year)
Excavate to 3.1 m below ground level	09/12/03-13/12/03
Install 1st level prop at 2.5 m below ground level	26/12/03-03/01/04
Excavate to 6.9 m below ground level	08/01/04-17/02/04
Install 2nd level prop at 6.3 m below ground level	18/02/04-22/02/04
Excavate to 10.7 m below ground level	06/03/04-08/03/04
Install 3rd level prop at 10.1 m below ground level	09/03/04-14/03/04
Excavate to 14.6 m below ground level	15/04/04-18/04/04
Install 4th level prop at 14.0 m below ground level	19/04/04-21/04/04
Excavate to 18.4 m below ground level	22/04/04-26/04/04
Install 5th level prop at 17.8 m below ground level	27/04/04-29/04/04
Excavate to 21.7 m below ground level	31/05/04-31/05/04

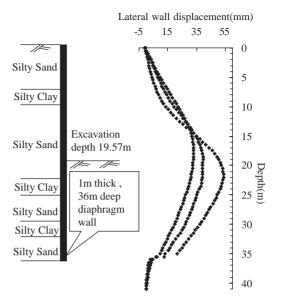
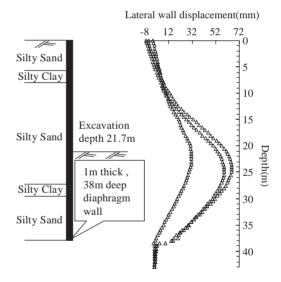


Table 3. Construction sequence at O8.

Construction activity	Period (day/month/year)
Excavate to 1.6 m below ground level	27/02/04-02/03/04
Install 1st level prop at 1.0 m below ground level	03/03/04-07/03/04
Excavate to 5.2 m below ground level	18/03/04-22/03/04
Install 2nd level prop at 4.6 m below ground level	23/03/04-27/03/04
Excavate to 8.3 m below ground level	30/03/04-02/04/04
Install 3rd level prop at 7.7 m below ground level	03/04/03-07/04/04
Excavate to 14.6 m below ground level	08/04/04-10/04/04
Construct concourse-level slab (0.8 m thick) at 14.2 m below ground level	11/04/04-15/05/04
Excavate to 17.0 m below ground level	14/08/04-16/08/04
Install 4th level prop at 16.4 m below ground level	17/08/04-19/08/04
Excavate to 19.0 m below ground level	08/09/04-15/09/04
Install 5th level prop at 18.4 m below ground level	15/09/04-18/09/04
Excavate to 20.9 m below ground level	19/09/04-22/09/04

Figure 1. Lateral wall deformations and ground profile at O6.



thin layers of clay. The SPT-N value of ground is from 5 to 30. The initial groundwater level was 3–6 m below ground level and it remained hydrostatic before commencement of excavation. Most excavations are surrounded by 4-storey to 7-storey buildings but some high-rise buildings (up to 12-stories) are also observed on site.

#### 3 OBSERVATIONS

Monitoring instruments installed on site included inclinometers inside the diaphragm wall and soils,

Figure 2. Lateral wall deformations and ground profile at O7.

observation wells, standpipe/electrical piezometers, vibration wire gauge on struts, tiltmeter and précision levelling on buildings.

Figures 1 to 3 show the lateral wall deformations observed from O6, O7 and O8 at the final excavation stage together with their ground profiles, respectively. It is evident that the maximum lateral wall movement varies from 32 to 64mm. The shape of lateral wall deformation is a cantilever at the shallow excavation

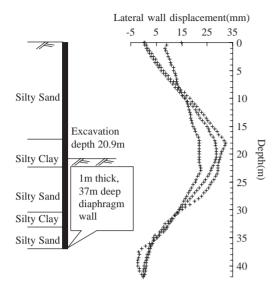
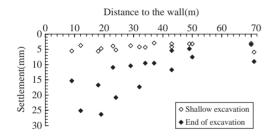
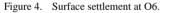


Figure 3. Lateral wall deformations and ground profile at O8.





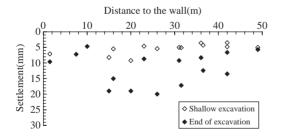


Figure 5. Surface settlement at O7.

level but tends to become prop-mode the deeper the excavation. The depth where the maximum lateral wall movement occurs is generally at the same depth of excavation.

Figures 4 to 6 present induced surface settlement and the observed maximum surface settlement is up to 20 to 28 mm at the completion of excavation. However,

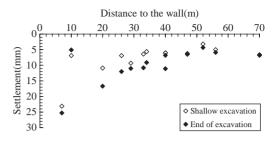


Figure 6. Surface settlement at O8.

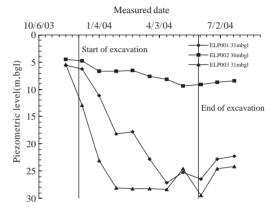


Figure 7. Piezometric levels inside the excavation at O7.

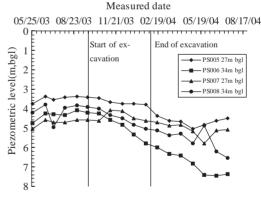


Figure 8. Piezometric levels outside the excavation at O6.

as shown in Figures 4 to 6, limited data was available for the area close to the excavation.

Since piezometers inside the excavation at O8 were all broken, only piezometric levels inside the excavation at O6 and O7 were measured, as indicated in Figure 7. The piezometric levels continue to decrease as excavation progresses. Figure 8 indicates the piezometeric level outside the excavation. It is

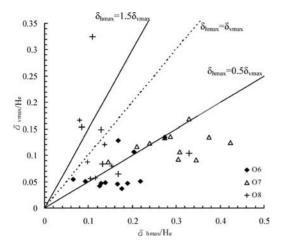


Figure 9. The relationship of  $\delta_{vmax}$ ,  $\delta_{hmax}$  and  $H_e$ 

apparent that it is affected by a change of pore pressure inside the excavation therefore it shows a slight drop as excavation progresses.

The loads on struts were also measured and the maximum observed measured load is in the range of 220 to 320 tons.

#### 4 DISCUSSIONS

#### 4.1 Maximum movements

Figure 9 presents the relationship of maximum lateral wall movements and excavation depth at O6, O7 and O8. It is found that the maximum lateral wall displacement ( $\delta_{hmax}$ ) in relation to the maximum excavation depth (H<sub>e</sub>) is approximately 0.034 to 0.3%. The ratio of  $\delta_{hmax}$  /H<sub>e</sub> is comparatively high at O7 and this could be accredited to less stiffness of the supporting system. Mana & Clough (1981) described the ratio of  $\delta_{hmax}$  /H<sub>e</sub> mainly in the range of 0.4 to 2.0%. Ou et al. (1993) reported that the same ratio was approximately from 0.2 to 0.5% for excavations in Taipei. The ratio obtained from this study tends to be lower and it is considered that varying ground conditions could be the reason for the difference.

As shown in Figure 10, the depth where maximum lateral wall movement occurs is in a range of 0.86 to  $1.26H_e$ . The maximum lateral wall movement is close to the excavation level.

The lateral wall movement at O8 is smaller than the others and it is assumed a semi-top-down construction method might provide a greater strut stiffness and reduce lateral wall movement in the excavation.

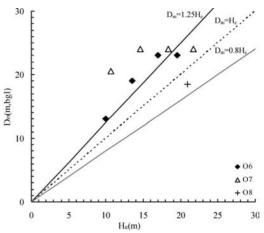


Figure 10. The relationship of the depth of the maximum lateral wall displacement  $(D_m)$  and excavation depth  $(H_e)$ .

#### 4.2 Surface settlement troughs

As shown in Figure 9, the maximum surface settlement  $(\delta_{vmax})$  is approximately 0.05–0.13% of H<sub>e</sub>. Wang (2003) suggested that  $\delta_{vmax}$  induced by the excavation in Kaohsiung might reach 0.04% to 0.25% of H<sub>e</sub>. Observations in this study appear to be consistent with Wang's conclusion.

Further, the relationship between  $\delta_{\text{vmax}}$  and  $\delta_{\text{hmax}}$ was explored and Figure 9 presents the distribution of the calculated ratio of  $\delta_{vmax}$  to  $\delta_{hmax}$ . It appears that the ratio of  $\delta_{\text{vmax}}/\delta_{\text{hmax}}$  falls in a range of 0.3 to 0.75 at O6 and O7 but tends to be greater at O8 (approximately from 1.3 to 1.8). Since different construction methods (bottom-up for O6 and O7 but semi- topdown for O8) were employed this is considered to be the main reason for the difference. As additional lateral wall movements at O6 and O7 were generated by the delayed installation of struts at several stages, this might reduce the ratio of  $\delta_{vmax}$  to  $\delta_{hmax}$ . Some data from O6 and O7 were therefore ignored, hence it is suggested that the ratio of  $\delta_{vmax}/\delta_{hmax}$  varies from 0.5 to 0.7 at O6 and O7. However, only limited data were collected for areas near the excavation (0 to 5 m from the excavation). This might affect the accuracy of the measurement of ground surface settlement.

Clough & O'Rourke (1990) suggested that the influence zone at surface level affected by the excavation in sand was two times the excavation depth for the ground behind the wall retaining the soil, but it tends to be greater (up to three times the excavation depth) at O6, O7 and O8. Pumping from deeper levels under ground is thought to be the reason for that.

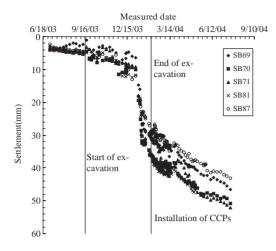


Figure 11. Building settlement at O6.

#### 4.3 Use of chemical churning pile

Chemical churning piles (CCP) are commonly used in Taiwan in order to reduce the influence on buildings resulting from adjacent excavation. Materials used for CCP generally include cement and water. Woo & Moh (1990) reported that the movements of the building could be prevented if the tip of the chemical churning pile was installed deeper than the potential failure surface. CCPs were used at O6 in order to protect structures nearby. For the design of CCPs, piles have to penetrate at least 2 meters below the 45° active failure surface of the excavation (Maeda- Longda Joint Venture, 2003). CCPs were installed between the excavation and buildings, approximately 7.5-9.0 m from the excavation. The diameter of the pile is 0.35 m and the length of the pile varies from 12.0 to 14.5 m. During the construction, pressure used for jet-grouting was kept at 19.6 MPa. The rate of rotation and raising of the rod were 20 rpm and 4.0 min/m, respectively. Although CCPs were installed around O6, the buildings adjacent to O6 continued to settle after installation of CCPs, as shown in Figure 11. CCPs may only provide very limited influence on reducing continual settlement of buildings and this is consistent with observations that Hsiung (2002) reported. Insufficient CCPs stiffness could be a possible reason.

#### 4.4 Prop load

Peck (1969) suggested that the prop load could be estimated associated with the apparent earth pressure method. Consideration of case histories from different projects, Twine & Roscoe (1997) revised the apparent earth pressure method. Figures 12 and 13 present the comparison of strut load from field measurement

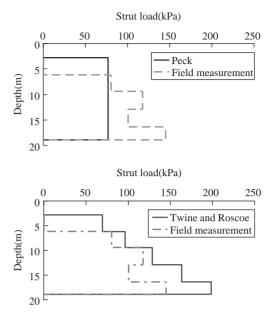


Figure 12. Comparison of prop load from field measurement and apparent earth pressure at O6.

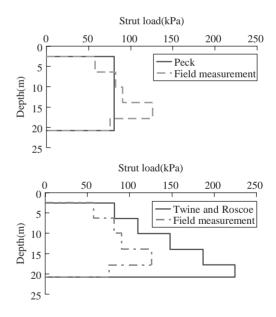


Figure 13. Comparison of prop load from field measurement and apparent earth pressure at O7.

and apparent earth pressure suggested by Peck and Twine and Roscoe. It suggests that Peck's method obviously underestimates the actual load on the prop but the method suggested by Twine and Roscoe provides a more accurate prediction in practice. Such conclusion is similar to those by Ou et al. (1998) and Hsiung (2002).

#### 5 CONCLUSIONS

This study provides an opportunity for exploration of ground behaviours induced by excavation in sand. Based on this study, conclusions can be made, as follows:

- 1. The lateral wall movement induced by excavations selected in this study is up to 0.3% of the excavation depth. Such displacement is less than those found from several related previous literatures. It is suspected the varying ground conditions could be a factor in the difference.
- 2. The maximum lateral wall displacement is 0.86 to 1.26 of the excavation depth. The depth where the maximum lateral wall movement occurs tends to be close to the excavation level
- 3. Associated with observations in this study, the ratio of  $\delta_{\text{vamx}}$  to  $\delta_{\text{hmax}}$  varies from 0.5 to 1.8 and the use of construction method (bottom-up and semitop-down) might induce the obvious change in this ratio.
- 4. The influence zone caused by excavation can be up to three times the excavation depth.
- 5. The installation of CCPs does not provide an effective solution for reducing the settlement of buildings near the excavation. Insufficient CCPs stiffness might be a reason.
- 6. The apparent earth method suggested by Peck underestimates the prop load. The method suggested by Twine and Roscoe might provide a more acceptable result in practice.

#### ACKNOWLEDGEMENTS

The authors would like to thank for the financial support from National Science Council, Taiwan for this project (Project Number: NSC92-2218-E-151-003). Data for this research provided by Maeda- Longda Joint Venture and comments on paper content and English from Mr. Tony Shield are also appreciated.

#### REFERENCES

- Burchell, A. J. 2000, Cairo metro line 2- construction problems and their solutions, *Tunnelling* Asia' 2000, New Delhi
- Clough, G. W. and O'Rourke, T. D. 1990, Constructioninduced movement of insitu walls, Design and performance of earth retaining structure, *ASCE*, special publication, No. 25, pp. 439–470
- El-Nahhas, F. M. 2006, Tunneling and supported deep excavations in the Greater Cairo, *International symposium on utilization of underground space in urban area*, Sharm El-Sheikh, Egypt
- Hsieh, P. G. and Ou, C. Y. 1998, Shape of ground surface settlement profiles caused by excavation, *Canadian Geotechnical Journal*, Volume 35, No. 6, pp. 1004–1017
- Hsiung, B. C. 2002, Engineering performance of deep excavations in Taipei, *PhD thesis*, University of Bristol
- Liu, G. B., Ng, C. W. W. and Wang, Z. W., 2005, Observed performance of a deep multistrutted excavation in Shanghai Soft Clays, *Journal of Geotechnical and Geoenvironmental Engineering.*, volume 131, issue 8, pp. 1004–1013
- Maeda- Longda Joint Venture, 2003, Working proposal for house protection at O6 station (in Chinese)
- Mana, A. I. and Clough, G. W. 1981, Prediction of movements for braced cut in clay, *Journal of the Geotechnical Division*, Proceeding of the American Society of Civil Engineers Volume 107, No. GT6, pp. 759–777
- Ou, C. Y., Hsieh, P. G. and Chiou, D. C. 1993, Characteristics of ground surface settlement during excavation, *Canadian Geotechnical Journal*, Volume 30, pp. 758–767
- Ou, C. Y., Liao, L. T. and Lin, H. D. 1998, Performance of Diaphragm Wall Constructed Using Top-Down Method, *Journal of Geotechnical and Geoenvironmental Engineering*, volume 124, issue 9, pp. 798–808
- Peck, R. B. 1969, Deep excavation and tunneling in soft ground, proceeding of the 7th international conference on soil mechanics and foundation engineering. Mexico City, state of art volume, pp. 266–277
- Twine, D. and Roscoe, H., 1997, Prop loads guidance on design, Funders Report FR/CP/48, 224 pages, CIRIA, London
- Wang, C. C. 2003, the study of engineering characteristics of deep excavations in Kaohsiung, NSC research report, Taiwan (in Chinese)
- Wong, I. H., Poh, T. Y. and Chuah, H. L. 1997, Performance of Excavations for Depressed Expressway in Singapore, *Journal of Geotechnical and Geoenvironmental Engineering*, Volume 123, Issue 7, pp. 617–625
- Woo, S. M. and Moh, Z. C. 1990, Geotechnical characteristics of soil in Taipei basin, *Proceedings of the 10th Southeast Asian Geotechnical Conference*, Volume 3, Taipei, pp. 51–65

# Environmental problems of groundwater around the longest expressway tunnel in Korea

S.M. Kim, H.Y. Yang & S.G. Yoon Bau Consultant CO. LTD., Seoul, Korea

ABSTRACT: Long tunnels are usually located in mountainous areas with limited infrastructure. Therefore very little is known about the geological, hydrogeological and geotechnical conditions: the longer the tunnel, the higher the probability of encountering adverse conditions for tunneling ; the greater the cost and duration for tunnel construction. Inje tunnel will be the longest expressway tunnel in Korea. In case of Inje tunnel, Special problems related to the length of the tunnel are for example typically the logistics, ventilation and environmental impacts. Even though modern excavation methods of tunnels have been developed, various types of problems such as change in groundwater distribution and transformation of geographical features still remain. It is not uncommon that private wells and small streams are used for daily life in the regions where mountain tunnels are located. Then serious social problems such as well water level fall, being attributable to tunnel excavation occurs. In the design stage, we evaluated that the quantity of leakage water into tunnels and groundwater drawdown area was simulated using numerical modeling such as MODFLOW and MAFIC to reduce adverse effects on life environment around tunnels.

#### 1 INTRODUCTION

Nowadays road construction projects in Korea have shown a tendency toward linear route designs, for the purpose of increasing running speed and reducing transportation time. Moreover, a growing number of projects are designed with tunnels and bridges in order to minimize damage to the natural environments, which can be caused by slope cutting.

However, some road construction projects, which were started after the year 2000, provoked the delay of projects and creating a lawsuit battle. Such a problem precipitated because the impacts of groundwater outflow during tunnel excavation were dealt with superficial experiences rather than scientific verification. Since 1998, there have been four important environmental conflicts related to tunnel groundwater outflow in Korea. Every tunnel construction that was mentioned above has passed legal environmental impact assessment (EIA) procedures. Nevertheless, these construction projects were delayed or stopped, within a span of 2 or 3 years on average, due to strong objections from civil environmental NGOs and local residents near the project sites. The NGO insisted that the groundwater outflow during tunnel excavation accompanied more serious secondary environmental impacts such as abrupt changes 4 Mitigation Plan for Environmental Impacts of Groundwater in Tunnels in the fauna and flora ecology near the upper part of the tunnel sites. (Lee, J. et al 2005).

Generally speaking, the movement of groundwater is based on meteorology conditions, surface vegetation, hydrogeological conditions and so on. Because a tunnel is a linear structure, the possibility of changing hydrogeology conditions is high and it is difficult to avoid adverse effects to life environment in regions where small-scale water usage remains. Therefore, many studies on the quantity of leakage water into tunnels and groundwater drawdown area are executed to reduce adverse effects on life environment around tunnels as well as retain safety and workability in tunnel constructions.

In this study, we performed the following investigation and numerical analysis to evaluate the environmental influence of groundwater with excavation of tunnel in mountainous area;

- Ground Investigation: Borehole logging, geophysical survey, BIPS, Lugeon test, groundwater level monitoring, water well survey and so forth
- Numerical Analysis for groundwater flow: Continuous and fracture media model
- Evaluation of influence: drawdown of groundwater level, groundwater inflow rate into tunnel Etc.

#### 2 OVERVIEW OF THE TUNNEL

The longest tunnel in Korea will be the Inje tunnel with the length of 11 km. The tunnel consists of two parallel double-lane tube each with a width of 14.5 m. The tubes are connected by cross tunnel as emergency facility every 250 m and 750 m. The gradient of the tunnel from Chuncheon to Yangyang is 1.95% downward. An Incline tunnel with the length of 1.5 km was designed on the purpose of access for excavation face and escape tunnel in emergency. Table 1 shows general overview of the tunnel.

#### 3 GROUND INVESTIGATION

# 3.1 General overview on the topology and geological conditions

The dominant topographic features of the area are rugged mountains and a few of streams. The western part of the area is lower altitude and more relief than the eastern part. This area shows early mature stage in geomorphologic cycle.

This area consists of Pre-Cambrian porphyroblastic gneiss, banded gneiss, and Jurassic biotite granite, two-mica granite, Cretaceous basic and acidic dikes, and Quaternary alluvium and diluvium as shown in Figure 3.

#### 3.2 Ground investigation

We carried out geological and geotechnical survey as shown in Figure 2.

#### 4 NUMERICAL ANALYSIS

#### 4.1 General aspects of groundwater flow model

The environmental assessment of tunnel groundwater aims to forecast the drawdown and variation of domestic groundwater near planned routes due to tunnel excavation. As usual, numerical methods of groundwater flow modeling are used for the environmental impact assessment of tunnel groundwater.

The program packages used for the modeling of tunnel groundwater outflow are different from each other, depending on the regional groundwater level variation in whole tunnels or small-scale groundwater level variation in narrow fracture zones. Normally, continuous numerical model packages are commonly used for modeling large-scale groundwater flow variations, while fracture media models are applied for

Table 1. Layouts of Inje Tunnel.

Length	10,965 m
Alignment	$R = 2000 \sim R = 4991.3$
Gradient	-1.95%
Shape of Portal	Arch wall type
Traffic Type	2 tube (2- lane)
Ventilation	Jet Fan : 80 Ventilation Shaft : 4
Excavation Method Emergency Facilities	Drill and Blast Escape connecting tunnel : 44 – Interval : 750 m for vehicle : 250 m for human

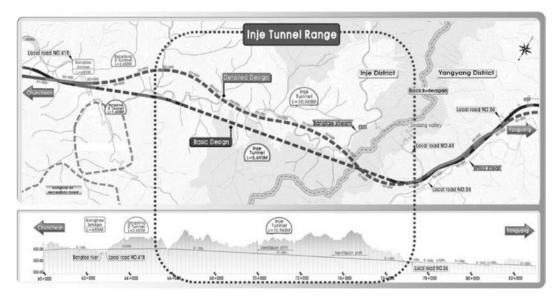


Figure 1. Location of Inje tunnel and its vertical section profile.

groundwater flow modeling in smaller zones composed of jointed rock aquifers.

Similarly, when we carry out the modeling of groundwater outflow, the MODFLOW package is commonly used for numerical analysis of regional groundwater flows in whole areas of tunnel and drainage, and MAFIC is applied for detailed groundwater modeling in small discrete zones in tunnel areas.

In this study, two representative models are used to forecast the tunnel groundwater outflow pattern as following Table 2.

The ground conditions of modeling section are Very-good(rock type 1)~ Moderate(rock type 3) grade

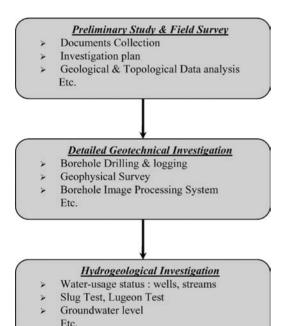


Figure 2. Flow chart of Ground Investigation.

in rock classification and the rock covers of tunnel under stream and valley area are about 100 m or higher.

#### 4.2 Continuous media modeling

#### 4.2.1 Setup of the model

MODFLOW was used to simulate groundwater flow as continuous media model in the studied area. The whole area is  $4 \text{ km} \times 3.5 \text{ km}$ .

The boundary of the model is set at the summit of northern mountains and southern mountains in the upper and lower end, respectively.

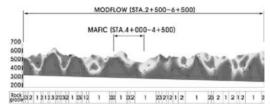
Hydraulic conductivities are classified into fifteen groups according to the permeability of the regions from Lugeon test. The range of value is  $4.5E-5 \text{ cm/sec} \sim 7E-6 \text{ cm/sec}.$ 

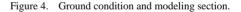
#### 4.2.2 *Steady state simulation*

We calibrated the value of groundwater head between the calculated and observed one in order to modify

Table 2. Layout of groundwater flow modeling.

Program	MODFLOW	MAFIC
Model	3D-Continuous Model	3 D-Discontinuous Model
Section	Sta. $2 + 500 \sim 6 + 500$	Sta. $4 + 000 \sim 4 + 500$





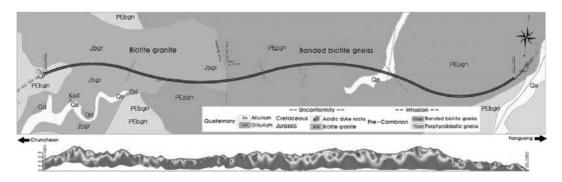


Figure 3. Geological features of Inje tunnel.

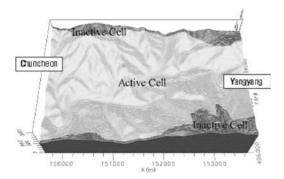


Figure 5. Boundary condition of the model.

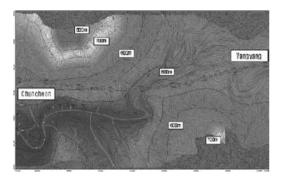


Figure 6. Distribution of equivalent head in steady state condition.

the hydraulic model. Figure 6 shows distribution of equivalent head in steady state condition.

We can see that groundwater inflow was concentrated at tunnel alignment and flow from the summit areas of northern and southern mountain to Bangtae stream.

#### 4.2.3 Transient state simulation

Result of Transient flow simulation is used to evaluate variation of groundwater level with tunnel excavation. We can know that the change of groundwater level with time is very small amount of  $0.008 \text{ m} \sim 0.015 \text{ m}$  before tunnel excavation.

# 4.2.4 Change of groundwater level due to tunnel excavation

We can see the distribution of groundwater level and drawdown at the check point due to the tunnel excavation as following Figure 8 and Table 3. The check points in model are located every 500 m from Sta. 3 + 000 to Sta. 6 + 500.

The drawdown of groundwater due to tunnel excavation is  $1.92 \text{ m} \sim 3.29 \text{ m}$  as you can see the Figure 8. It is small amount in general, but we worried about the influence on groundwater systems around tunnel. Therefore, we carried out simulation for waterproof

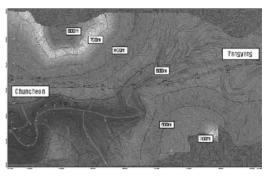


Figure 7. Distribution of equivalent head in transient state condition.

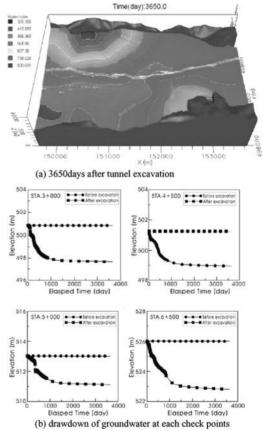


Figure 8. Distribution of groundwater level.

grouting with the aim of reducing water inflow into tunnel in rock mass.

Groundwater level at Sta. 5 + 000 will fall down to 2.03 m after tunnel excavation and water inflow rate per km is  $0.127 \text{ m}^3/\text{min}$ . In case of using waterproof grouting to the ground near tunnel, drawdown of groundwater level is 0.79 m and water inflow rate

Check point	Drawdown of groundwater level (m) (without grouting)	Drawdown of groundwater level (m) (with grouting)		
Sta. $3 + 000$	3.20	0.94		
Sta. $3 + 500$	2.94	0.87		
Sta. $4 + 000$	2.32	0.58		
Sta. $4 + 500$	1.92	0.73		
Sta. $5 + 000$	2.03	0.79		
Sta. $5 + 500$	2.98	1.04		
Sta. $6 + 000$	3.20	0.77		
Sta. $6 + 500$	3.29	1.05		
Water inflow	0.127 m <sup>3</sup> /min	0.060 m <sup>3</sup> /min		
rate per km	(730.75 m <sup>3</sup> /day)	$(343.27 \text{ m}^3/\text{day})$		

Table 3. Groundwater level and water inflow rate into the tunnel.

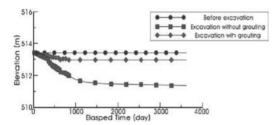


Figure 9. Drawdown groundwater level in case of applying waterproof grouting at Sta. 5 + 000.

per km is  $0.060 \text{ m}^3/\text{min}$ . It can be notified that if waterproof grouting into the ground is performed in advance, the amount of groundwater inflow into the tunnel will decrease as shown in Figure 9 and Table 3.

#### 4.3 Fracture media modeling

MAFIC (Matrix And Fracture Interaction Code) determines solutions of flow and pressure by conjugate gradient finite element methods, and solves solute transport by particle tracking discrete fracture network models. Finite element meshes are based on fracture network analysis and geometric modeling.

#### 4.3.1 Set up of the fracture model

Using the package of MAFIC, we simulated groundwater flow through fracture model in the studied area (Sta.  $4 + 000 \sim 4 + 500$ ). Input data was estimated based on joint data from detailed surface and borehole survey. In facture model, three of major joint set was estimated as following Table 4.

Figure 10 shows three dimensional fracture network and tunnel.

#### 4.3.2 Steady state condition

As the stage of setting boundary condition in steady state simulation, we estimated groundwater level and

Table 4. Major joint set and fault in this area.

Joint set	Orientation	Length (m)	S.D of Length (m)	Distribution of length
Set 1 Set 2 Set 3 Fault	80/194 78/264 30/124 85/105	15.36 12.93 34.47	10.38 9.63 13.18	Lognormal Lognormal Lognormal

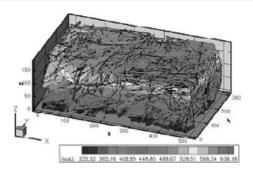


Figure 10. three dimensional model of fracture network and tunnel for steady state simulation.

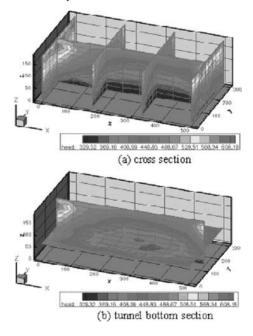


Figure 11. Distribution of water head in steady-state condition before tunnel excavation.

transmissivity(T) based on results of MODFLOW in steady-state flow condition, water well survey and borehole test and so on.

Especially, average transmissivity evaluated from the result of Legeon test and BIPS is  $1.24 \text{ m}^2/\text{sec.}$ 

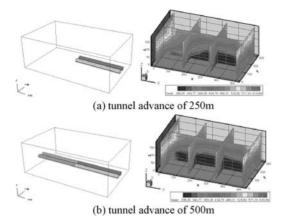


Figure 12. Distribution of water head in transient-state condition before tunnel excavation.

Figure 11 shows distribution of water head in steady-state condition before tunnel excavation. It was set as the value of initial water head in transient flow condition.

#### 4.3.3 Transient state condition

We simulated the groundwater flow due to tunnel excavation in the condition of transient state. As tunnel excavation was performed, groundwater flowed into tunnel and water head fell down suddenly and afterward gradually.

In the case of 500 m tunnel advance, water inflow rate per km into tunnel is  $0.10 \text{ m}^3/\text{min}$ . That was similar to the result of MODFLOW (0.127 m<sup>3</sup>/min).

#### 5 CONCLUSIONS

In spite of the difficulty for estimation of water flow characteristics into tunnel before excavation, it is very important to predict the variation of hydraulic system due to the tunnel.

In this study, two representative models are used to forecast the tunnel groundwater outflow pattern such as MODFLOW and MAFIC.

Based on the result of numerical modeling, it can be notified that if waterproof grouting into the ground is performed in advance, the amount of groundwater inflow into the tunnel will decrease compared to nongrouting case.

#### REFERENCES

- Kim, S. & Yoon, S. 2006a. Environmental Problems of Groundwater around the mountain tunnel (2), *Tunnel Tech*nology Journal of Korean Tunnel Association, Vol. 8, No. 2, 82–98.
- Kim, S. & Yoon, S. 2006b. Environmental Problems of Groundwater around the mountain tunnel (1), *Tunnel Technology Journal of Korean Tunnel Association*, Vol. 8, No. 1, 64–78.
- Kim, S., Yoon, S. & Yoon, J. 2005. Influence of Groundwater in the Railway Tunnel Construction Korea Society for Rock Mechanics 2005 Fall National Conference, October 27, 2005, Chuncheon, Korea.
- Lee, J. & Lee, Y. N. 2005. A Study on Mitigation Plan for Environmental Impacts of Groundwater in Tunnels. *Korea Environment Institute*, 5–36.

## Measurements of ground deformations behind braced excavations

T. Konda Geo-Research Institute, Osaka, Japan

H. Ota & T. Yanagawa Osaka Municipal Transportation Bureau, Osaka, Japan

#### A. Hashimoto

Kotsu Service Co., Ltd., Osaka, Japan

ABSTRACT: The deformation behavior of earth retaining wall and backside ground due to braced excavation were measured at the construction sites of Osaka Subway Line No. 8. It was observed that backside ground moved to excavation & down side along a circular slip following the wall deflection, and its influence spread to the ground surface. The backside ground surface settlement near the wall was suppressed due to the friction between wall and surrounding ground, and maximum settlement occurred at some distance from wall in the 45–60 degree area from the bottom of the wall. The backside ground deformation area decreased exponentially with the distance from the wall. As a result each observed data from monitoring sites was analyzed statistically according to the soil characteristic, wall deflection and surface settlement in prominent soft clay layer were larger than those in prominent sand and gravel layer. The relation between the wall deflection area  $A_{\delta}$  and backside ground settlement area  $A_s$  was that  $A_s \cong (0.2–0.3) \times A_{\delta}$  in prominent soft and sensitive clayey layer. The backside ground settlement of this observed data converged smaller than past data.

#### 1 INTRODUCTION

There are many neighboring constructions in recent subway works. In order to guarantee the construction safety and control the influence on the surrounding environment to the minimum, it is necessary to use the observational method and prediction results effectively. For example, in the case of cut and cover method, which is usually used to construct the subway station, it is necessary to evaluate the backside ground deformation of earth retaining wall and groundwater level fluctuation, to carefully monitor some particular concern points, and of course to estimate the safety of timbering of a cut and the excavation bottom.

In this paper, earth retaining wall deflection and backside ground deformation due to braced excavation were evaluated circumstantially in open cut construction sites, which main excavation soil was the alluvial soft clay, in Osaka Subway Line No. 8. All observed data in 11 monitoring sites were got together and analyzed with the same idea. The evaluated results were described for each soil characteristic. Some behaviors of ground deformation due to braced excavation based on these monitoring results were considered.

#### 2 CONSTRUCTION SUMMARY

Osaka Subway Line No. 8 was constructed at the east side of Osaka city as north–south rail route in underground. Every station was built by open cut method. In order to control the settlement at the ground surface and the influence on the neighboring ground environment, seepage control method was adopted in all of the station construction sites with extending earth retaining walls, where the most were Soil Mixing Walls, to the Pleistocene clayey layer under the artesian aquifer.

It is possible to classify three areas roughly based on the difference of soil characteristic as Figure 1 shows. This new rail route located at the East side of Uemachiplateau, and this area was an inland sea in ancient times.

#### (1) North Area

This area interleaved the Yodo-river, and constitutes the alluvial layer, upper and lower Pleistocene. Alluvial layer is composed of a fine sandy layer having low uniformity coefficient Uc, and soft and sensitive clayey layer having unconfined compressive strength  $q_u = 40-100 \text{ kN/m}^2$ . Tenma-gravel layer accumulated

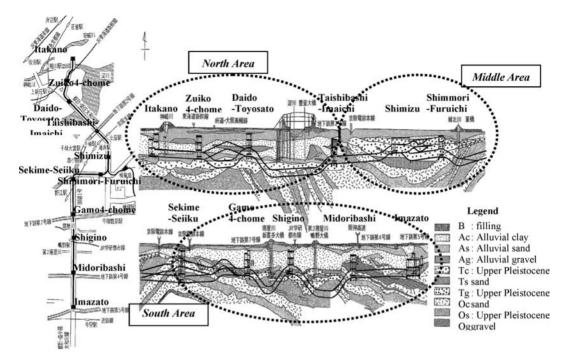


Figure 1. Plain view and soil crossing view of Osaka Subway Line No. 8 (Unei et al., 2002).

continuously with just about uniform thickness as upper Pleistocene layer with high permeability. Lower Pleistocene layer Osaka Group is laminated ground with stiff clayey layer of cohesion  $c = 300-600 \text{ kN/m}^2$ , and dense sandy layer of Standard Penetration Test N-value is more than 60. Each Osaka Group dips in the south and east direction neighbor the Yodo-river.

#### (2) Middle Area

This area is constituted of the thin alluvial layer, upper and lower Pleistocene. Upper Pleistocene layer is constituted with stiff clayey layer (N-value = 5–10) and very dense sand and gravel layer with high permeability. Lower Pleistocene layer Osaka Group is laminated ground with stiff clayey layer ( $c = 200-400 \text{ kN/m}^2$ ) and dense sandy layer (N-value > 60). The ground of this area is harder than other areas comparatively.

#### (3) South Area

This area located in Neyagawa-lowland and it is different from the rest area for that the very soft and sensitive alluvial clayey layer (N-value = about zero,  $q_u = 40 100 \text{ kN/m}^2$ , liquid limit  $I_L$  = about 1.0) deposited within about 15 meters. Under the Alluvial layer, upper Pleistocene layer (N-value = 22–60) with high permeability and lower Pleistocene layer Osaka Group, with stiff clayey layer (c = 200–400 kN/m<sup>2</sup>) and dense and slightly cohesive sandy layer (N-value = 30 – and above 60). Each Osaka Group dips in the south and east direction parallel to Uemachi-plateau.

#### 3 BEHAVIOR OF BACK GROUND DEFORMATION DUE TO BRACED EXCAVATION

Here, some monitoring results of the earth retaining wall deflection and backside ground deformation due to braced excavation at the A-site are represented and the dispersed behavior in backside ground from wall to ground surface is described as Figure 2 shows (Ito *et al.*, 2006).

Alluvial layer and upper & lower Pleistocene layer are formatted from the ground surface near the A-site. Alluvial layer is composed of a fine sandy layer (1st aquifer) with 2 m thickness having low Uc and N-value equals to about 2, and a soft and sensitive clayey layer with N-value = 0-3,  $I_L = 0.4-1.0$ ,  $c = 20-100 \text{ kN/m}^2$ . This Alluvial clay layer is a typical soft layer in this construction site. On the other hand, upper Pleistocene sandy and gravel layer Tsg (2nd aquifer) with partially scattered gravel, lower Pleistocene clayey layer Oc3 ( $c = about 400 \text{ kN/m}^2$ ) and lower Pleistocene sandy layer Os3 (3rd aquifer, N-value >60) existed continuously under the Alluvial layer.

Seepage control method was adopted in this construction site with extending the wall to the low

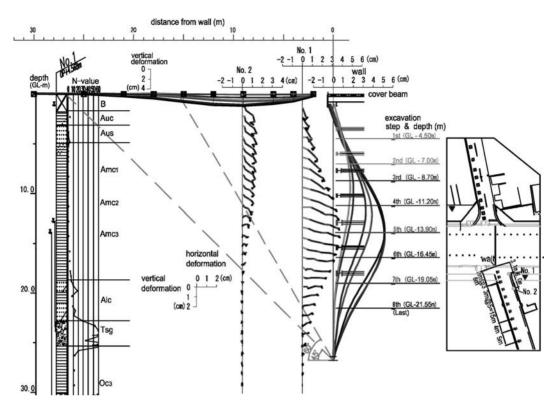


Figure 2. Earth retaining wall deflection, backside ground deformation vector and distribution of backside ground surface settlement at the A-site.

permeable layer Oc3 (GL-26.3 m). The excavation width was 16.2 m, and the final excavation depth was GL-21.5 m, in short, the wall penetration depth is 4.8 m. Because the soil improvement work was carried out near the monitoring site after GL-1.5 m excavation depth to protect the deficient part of wall, initial value of each observed data was set the value after 1st excavation to GL-4.5 m.

The maximum wall deflection occurred at a slightly deeper depth from the excavation bottom due to the braced excavation till the 6th excavation step, and this influences reached the backside ground and surface. For example, the maximum wall deflection was 38 mm at the excavation bottom, which was the middle of Amc layer (GL-12.5 m), in the 5th step. According to this influences, the maximum horizontal ground deformation of the No. 1 was about 29 mm at the slightly above the maximum deformation depth of wall. The maximum horizontal ground deformation of the No. 2 was about 12 mm in Auc layer and Aus layer. It was known that the wall deflection caused the back side ground deformation like the circular slip and this influence reached the ground surface.

On the other hands, the maximum surface settlement at the No. 2 was 6 mm, and the settlement

distribution was convex downward with 0 mm settlement at the point, 20 m outside from the wall. It was thought that the friction between the wall and surrounding ground supported the surface settlement. These ground movements happened in the 45–60 degree area from the bottom of the wall.

The maximum wall deflection increased to 51 mm at just above from the excavation bottom, which is the lower of Amc layer (GL-14 m), in the 6th step. Since the 6th step, wall deflection did not increase any more. It was believed that the cause of this inhabitation phenomenon was from the subgrade reaction in the Tsg and Oc3 layer was large and the effects of pre-load on the wall.

Figure 3 shows the definition of symbols used in this paper. The relation between the excavation depth to the maximum wall deflection ratio  $\delta_{max}/Z_e$  and the maximum deformation ratio  $S_{max}/\delta_{max}$  are shown in Figure 4.

East and west wall deflection became large due to the braced excavation in the middle of Amc layer till the 4th step, and  $S_{max}/\delta_{max}$  was tend to decrease. However, the wall deflection was decreased gradually, and  $S_{max}/\delta_{max}$  was tend to increase adversely.

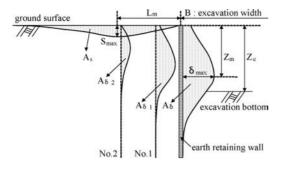


Figure 3. Definition of each symbol.

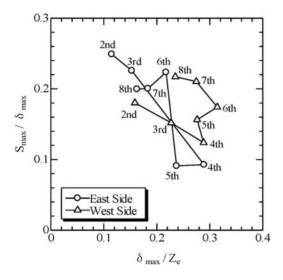


Figure 4. Relation between  $\delta_{max}/Z_e$  and  $S_{max}/\delta_{max}$ .

The relation between the excavation depth to width ratio  $Z_e/B$ , the maximum deformation ratio  $S_{max}/\delta_{max}$  and the deformation area ratio are shown in Figure 5.  $Z_e/B$  is the non-dimensional value for excavation size and increase due to the braced excavation.

As the increase ratio of the wall deflection and surface settlement is small till the 4th step due to the braced excavation in the middle of Amc layer,  $S_{max}/\delta_{max}$  was tend to decrease. However,  $S_{max}/\delta_{max}$  was increased conversely due to the braced excavation after the 4th step. This tendency occurred in the same way with the relation of  $A_s/A_{\delta}$ .

Figure 6 shows the relation between the wall deflection area  $A_{\delta}$  and the backside ground surface settlement area  $A_s$  at the beginning of the excavation and after removing all struts. More or less  $A_{\delta}$  and  $A_s$ tend to increase from the start to the end. Both side wall deflections have  $A_s \cong 0.2 \times A_{\delta}$  relationship in the 8th step, and shifted to  $A_s \cong 0.3 \times A_{\delta}$  relationship due to removal of struts. Unalterably, the west side wall deflection was larger than the east one.

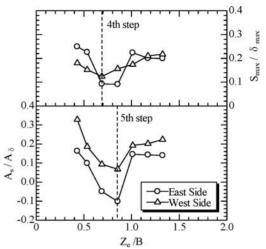


Figure 5. Relation between  $Z_e/B$ ,  $S_{max}/\delta_{max}$  and  $A_s/A_{\delta}$ .

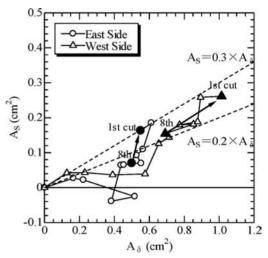


Figure 6. Relation between  $A_{\delta}$  and  $A_{s}$ .

The relation between the distance from the wall to backside, wall deflection area  $A_{\delta}$  and backside ground deformation  $A_{\delta 1}$  &  $A_{\delta 2}$  are shown in Figure 7. Broken line is the approximate curve based on the calculation result by least-square method with three measurement data in each excavation stage.

The ground deformation area attenuated gradually as getting away from the wall position till the 6th step. On the other hand, as the increase of the wall deflection became small, each deformation area changed little. The attenuation rate of the backside ground deformation area on wall deflection area was just about steady, about 70% at the No. 1 and about 25% at the No. 2.

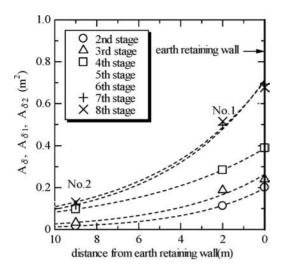


Figure 7. Influence propagation aspect to backside ground due to braced excavation.

#### 4 STATISTICAL COMPARISON BETWEEN WALL DEFLECTION AND BACK GROUND SURFACE SETTLEMENT

Figures 8 and 9 show the relation between the wall deflection area  $A_{\delta}$  and backside ground surface settlement area  $A_{\delta}$  at the beginning of the excavation and after removing all struts of all monitoring site for excavation soil characteristic. The broken line shows the observed data from final excavation stage to all struts removal stage.

The case of soft and sensitive clayey layer is shown in Figure 8. For example there were different tendencies between east and west wall deflection behavior, however,  $A_s \cong (0.2-0.3) \times A_{\delta}$  relationship was given. Additionally, the wall deformation and surface settlement increased due to the removal of struts.  $A_{\delta}$  and  $A_s$  were larger than one in the other different grounds. The case of alluvial sandy layer is struts.  $A_{\delta}$  and  $A_s$ were larger than one in the other different grounds. The case of alluvial sandy layer is shown in Figure 9. There are great variances in this reference, however,

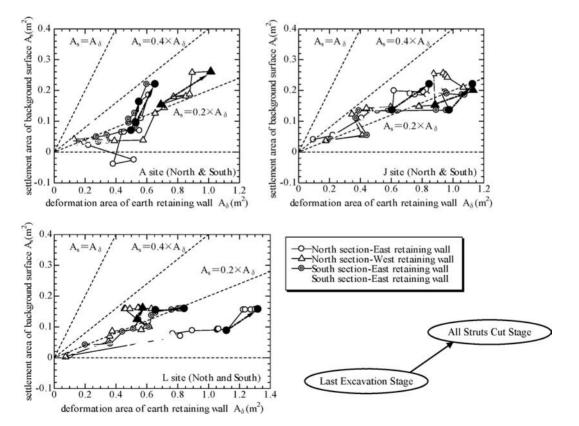


Figure 8. Relation between  $A_{\delta}$  and  $A_{s}$  in the case of soft and sensitive clayey layer.

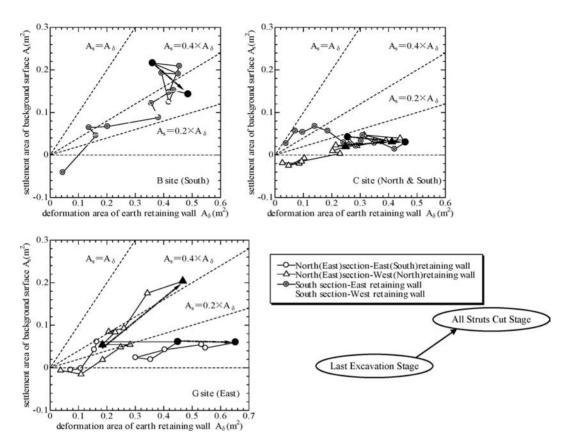


Figure 9. Relation between  $A_{\delta}$  and  $A_{s}$  in the case of alluvial sandy layer.

 $A_s \cong (0.1-0.4) \times A_{\delta}$  relationship was given. The deformation area of wall and surface increased due to removal of struts the same as the behavior of the soft and sensitive clayey layer case. But the amount of deformation was relatively small.

Figure 10 shows the predictive method of backside ground surface settlement based on the relation between  $A_{\delta}$  and  $A_{s}$  due to the braced excavation.  $A_{\delta} = A_{s}$  means that the influence of the wall deflection passes to the surface settlement directly without volume change (volume loss = 0%).

In the case of including the consolidation settlement,  $A_s$  was larger than  $A_\delta$  according to the literature. On the other hand,  $A_s$  was smaller than  $A_\delta$  in other cases (JSCE, 1993). Some observed data in the case of alluvial soft clay ground were added to past examples shown as Figure 10. Because the seepage control method was adopted in No. 8 Line construction, and actually the change of ground water level was small, it is possible to estimate the small amount of degradation for the consolidation settlement caused by the groundwater level. However past observed data were distributed along the  $A_{\delta} = A_s$  relation, some monitoring data in No. 8 Line construction were located along the  $A_s \cong (0.2-0.3) \times A_{\delta}$  relation and under the  $A_s \cong 0.4 \times A_{\delta}$  relation. The backside ground settlement of this observed data converged smaller than the past data and the volume loss from  $A_{\delta}$  to  $A_s$  was 70–80%. One of the reasons for this phenomenon is thought to be the support to the neighboring ground based on the arching effect developed in backside ground.

#### 5 CONCLUSIONS

The conclusions are drawn as follows:

- It was found that the earth retaining wall deflection caused the back side ground deformation like the circular slip and spread influences to the ground surface in A-site.
- 2. The distribution of backside ground surface settlement occurred as convex downward with the maximum value at some distance from the wall. It was thought that the friction between the wall and around ground supported the surface settlement.

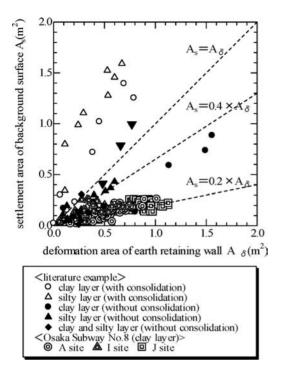


Figure 10. Relation between  $A_{\delta}$  and  $A_{s}$ .

These ground movements happened in the 45–60 degree area from the bottom of the wall.

The attenuation rate of the backside ground deformation area on wall deflection area was just about steady, about 70% at 2 m distance and about 25% at 9 m distance from the wall.

- 4. The relation between the wall deflection area  $A_{\delta}$ and backside ground settlement area  $A_s$  was that  $A_s \cong (0.2-0.3) \times A_{\delta}$  in prominent soft and sensitive clayey layer, and  $A_s \cong (0.1-0.4) \times A_{\delta}$  in prominent Alluvial sandy layer depended on the construction condition.
- 5. Some monitoring data in No. 8 Line construction were located under the  $A_s \cong 0.4 \times A_\delta$  relation. The backside ground settlement of this observed data converged smaller than past data, and the volume loss from  $A_\delta$  to  $A_s$  was 70–80%. One of the reasons for this phenomenon is thought to be the support to the neighboring ground based on the arching effect developed in backside ground.

It is necessary to carry out the evaluation preliminary in detail and use the observational method to void some kinds of risk. It is believed that this study will become useful for predicting the influences of neighboring construction in future.

#### REFERENCES

- Ito, H., Yanagawa, T., Konda, T. & Hayakawa, K. 2006. Relation between braced wall deflection and deformation behavior of backside ground due to braced excavation, *Proc. of Tunnel Engineering*, *JSCE*, vol. 16, 439–446 (in Japanese).
- JSCE. 1993. Design Calculation Example of Makeshift Structure based on the Japanese Standard for Cut and Cover Tunneling, *Tunnel Library* 4, 107 (in Japanese).
- Unei, N., Sakaguchi, Y. & Yamaguchi, H. 2002. Challenge to Close Approach to or Crossing with Shinkansen Line and Other Existing Important Structures – Construction plan for subway station on Osaka Municipal Line No. 8, *Tunnels and Underground*, Vol. 33, No. 12, 31–40 (in Japanese).

# Research on the effect of buried channels to the differential settlement of building

#### D.P. Liu, R. Wang & G.B. Liu

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: Great and differential settlement occurs under the building during an adjacent metro deep foundation pit construction. Factors influencing the building settlement in different construction stages are analyzed based on the monitoring data and geology distribution condition. The result shows that the large lateral deformation of retaining wall and time-depended deformation of soft clay under the building are the reasons that make the major settlement occur. The main factor inducing differential settlement of the building is local distribution of adverse geological condition. Some feasible means of improvements in practice are presented, which can reduce effectively the influence of adverse geological condition such as buried channel on excavation and surroundings. Though the paper the author hopes that engineers can attach more importance to the negative influence of adverse geological condition, and the results could be valuable reference to other engineering.

#### 1 GENERAL INSTRUCTIONS

Shanghai has the representative soft ground in China. The stratum of Shanghai mainly made up of saturated clay, silty clay and sand. Most metro stations and tunnels of Shanghai lie about 20 m beneath the ground surface. Soil of such depth mostly is soft clay which characteristics such as water content, degree of sensitivity, compressibility and rheology are notable, and the unit mass, strength and permeability are bad. Except those, the adverse geological factors such as buried channel, shallow-buried methane, underground barrier etc. may be exist which will do harm to the constructing of deep excavation and adjacent buildings. So the adverse geological factors should be paid enough attention.

During the construction of a metro excavation in Shanghai, great and differential settlement occurs on an adjacent building. Factors influencing the building settlement in different construction stages are analyzed based on the monitoring data and geological distribution condition. Reasons which effect the major settlement are discussed and the results could be valuable reference to other engineering.

#### 2 GENERAL SITUATION

The metro station of this study is designed as a two layers underground frame structure. The total length of the station is 364.7 m and the width of its standard

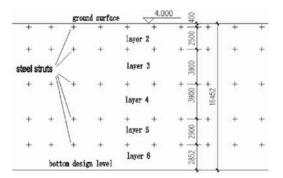


Figure 1. Strut profile of standard segment.

segment is 24.5 m. The buried depth of the bottom plate is 17 m, and the thickness of retaining wall is 800 mm. As for the edge wells of the station ,the buried depth is about 18.6 m, the thickness of its retaining wall is 800 mm, Pre-stressed steel pipes of 609 mm in diameter (external) and 16 mm in thickness were installed at each levels (standard segment: level 1 to level 5; edge wells : level 1 to level 6) to support the retaining wall. The protective grade of the excavation is grade 1. The cross section of the standard segment is shown in Figure 1. Soil profile and geological description of the soils under the adjacent building of the excavation are given in Table 1.

Table 1.	Profile and	geological	description of	of the soils.

Serial			Bottom	Water	Unit	Void	Shear strength (peak)	
number	Name	Thickness/m	level /m	content /%	weight $/kN \cdot m^{-3}$	ratio	C/KPa	$\Phi/^{\circ}$
1 (1)	filled soil	1.2	2.93	_	_	_	_	_
21	clay	2.6	0.33	34.6	18.2	0.99	21	17.5
3	silty clay	4.2	-3.87	43.0	17.3	1.21	13	17.0
<b>(4</b> ) <sub>1</sub>	silty clay	8.0	-11.87	49.1	16.8	1.39	14	11.0
<b>5</b> <sub>1-1</sub>	clay	3.0	-14.87	38.9	17.6	1.12	16	14.0
<b>5</b> <sub>1-2</sub>	silty clay	6.5	-21.37	34.9	17.9	1.02	15	18.5
<b>5</b> <sub>2-2</sub>	sandy silt	14.5	-35.87	32.2	18.0	0.94	4	29.0
$   \overline{\mathcal{O}}_2 $	silty sand	No penetrated	No penetrated	26.3	18.8	0.77	1	32.0

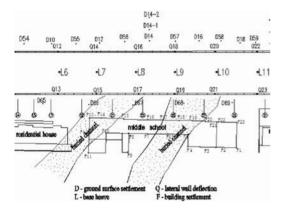


Figure 2. Plan of monitoring points and buried channel.

#### 2.1 Adjacent buildings and surrounding condition

The buildings around the excavation are relatively dense. The prime protection object is the nearer buildings on the south side of the excavation. Great settlement occurred under a teaching building belonged to a middle school which is sited  $14 \sim 16$  m away from the excavation on the south side. Total length of the building parallel to the excavation is 87.882 m and its width is 15.188 m on the east, 20.883 m on the west. The building was constructed at the end of 1970s, and it is the reinforced masonry structure with ring beam, shallow strip foundation.

According to the geological documents, there are two buried channels under the building (Figure 2). Influenced by the adverse geological factor, there has been about 400 mm total settlement happened before the excavation construct.

#### 2.2 Building settlement

The curves of building settlement developed with time are shown in Figure 3 after the excavation constructed August, 2005. Figure 4 shows the accumulative total

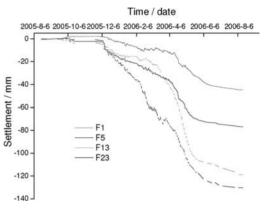


Figure 3. Building settlement with time.

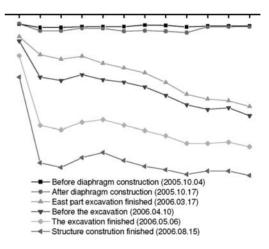


Figure 4. Accumulative total settlement of building during different stage.

building settlement of several main monitoring points during different construction stages.

From Figure 4, monitoring points of F13 and F23 have the large accumulative and differential

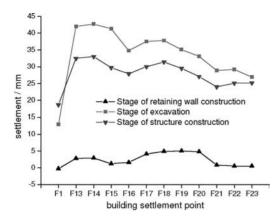


Figure 5. Building accumulative total settlement in different stages.

settlement. Until March, 17, the differential settlement in longitudinal direction between F13 and F23 has reached to 45 mm and inclining slope is 0.0005; and which is 60.7 mm and 0.0041 respectively transversely. Inclining slope of the two directions have exceeded 0.003 which is the allowable value according to national criterion. Many cracks appeared in walls of the building, some local width of cracks is about 10 mm. so did the parallel crakes on the ground surface between the building and the excavation.

#### 3 BUILDING SETTLEMENT ANALYSIS IN DIFFERENT STAGES

Great and obvious differential settlement of the building is the result affected by several factors, which have dissimilar effect in different construction stages. Figure 5 and 6 respectively shows the accumulative total settlement and settlement velocity with different parts of the building during the main construction stages.

Based on the main construction stages of the part of excavation near the building, settlement of different parts of the building in different construction stages was analyzed.

#### 3.1 Stage of retaining wall construction

Generally, deformation of the surrounding induced by retaining wall construction is relatively small and the effected area is limited. The building lies outside the influencing area of the retaining wall construction so that the effect can be neglected when the building settlement is analyzed.

Field data of the building in retaining wall construction stage is analyzed. Construction of the part of retaining wall began on Oct.4, 2005 and finished on Oct.17. During the period building subsided little and

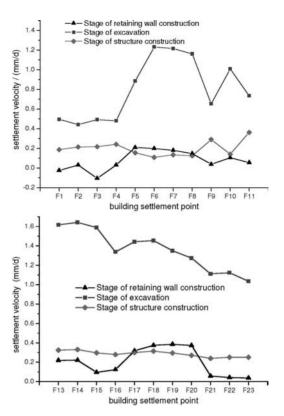


Figure 6. Building settlement velocity in different stages.

the average settlement is 2.7 mm. Settlement of F21 to F23 are less than 1 mm and that of F15 to F16 are 1.2 to 1.6 mm, as shown in Figure 4. Take the normal fluctuation of monitoring data into account, the construction of retaining wall has little influence on building settlement, which tests the conclusion of other correlative studies.

In Figure 5, settlement in different location of the building is of variation from the minimum 0.5 mm of F23 to the maximum 5 mm of F19. Because of the existing of the buried channel, building settlement of the location under which is the buried channel are greater than that of normal subgrade soil. At that time, there is no other construction, so the main reason leading to the settlement can be ensured as the existing of the high rheology of the soft soil.

#### 3.2 Stage of excavation

Sequence of the excavation is from the east to the west. Differential settlement between the two ends was 45 mm till March, 17. In Figure 7, the lateral deflection of the retaining wall at Q17 is gathered according to each stage of the excavation. So do the deformation of excavation and the surroundings as shown in Table 2.

In Figure 7, large deformation produced at the early stage of the excavation. There has been 32 mm lateral deformation after the 3rd level strut installed which is mainly related to high time-depended deformation of the 3rd level soft soil. In addition, long exposure time without struts and long laying up time after the installing of struts also made for the lateral deflection of the retaining wall increasing.

Cooperating with the excavation, basal heave and the ground surface settlement caused by lateral deflection of the wall are increasing, which also accelerate the settlement of the building. (Figure 5 and Figure 6)

Settlement velocities of points  $F6 \sim F8$  in the location of buried channel are double greater than those

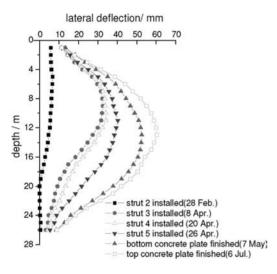


Figure 7. Lateral deflection of the wall at Q17.

Table 2. Deformation of excavation and the surroundi	ngs.
------------------------------------------------------	------

of points F1 ~ F4. Settlement velocities of points F13 ~ F15, F17 ~ F20 are also greater than the others. (Figure 8) The maximum local inclining slope of the building in longitudinal direction is 0.0006, and which is 0.0052 in transverse. They all exceeded the allowable value 0.003. So the reason of differential settlement of the building could be mainly attributed to the local distribution of buried channel.

#### 3.3 Stage of structural construction

Compared with the excavation stage, lateral deflection of the retaining wall, basal heave and building settlement in this stage are steady relatively because of the strengthening of the structural stiff. But the building and ground surface surround the excavation still sank a little at the rate  $0.3 \sim 0.4$  mm/d. This can be explained

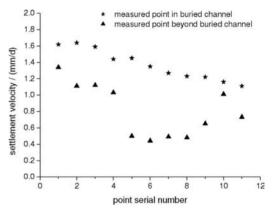


Figure 8. Comparison of settlement velocity between the parts in buried channel and the others.

		Accumulativ moment / m	ve total on different m	t	Stage of exca	vation	Stage of structure construction	
Item		Before the excavation	The excavation finished	Structure construction finished	Increment / mm	Velocity / (mm/d)	Increment/ mm	Velocity / ( mm/d)
Lateral	013	31.8	59.1	76.4	27.3	1.05	17.9	0.15
deflection	Q17	32.7	47.2	60.3	14.5	0.60	13.1	0.13
of the wall	Q19	49.7	61.6	68.6	11.9	0.70	7.0	0.07
Basal	L6	16.5	45.9	51.1	29.4	1.13	5.2	0.18
heave	L7	7.7	34.2	35.2	26.5	1.02	1.0	0.03
	L8	7.7	49.6	41.0	41.8	1.67	-8.6	-0.28
	L9	24.4	63.0	63.3	38.6	2.27	0.3	0.01
	L10	32.1	50.4	60.3	18.3	1.66	9.9	0.23
Ground	D65	28.2	52.3	83.2	24.1	0.93	30.9	0.30
surface	D66	35.3	76.9	118.8	41.6	1.60	41.9	0.41
settlement	D67	49.6	93.6	130.5	44.0	1.76	36.9	0.35
	D68	52.3	86.3	128	34.0	2.0	41.7	0.37
	D69	70.6	82.2	130.0	11.6	1.05	47.8	0.41

as the reason of time-depended deformation of the soil around the excavation.

#### 4 REASONS ANALYSIS

- Relatively great lateral deflection of the retaining wall causes the whole settlement of ground surface. Time-depended deformation of the soft soil disturbed from the construction, overload of the building, long exposure time without struts and long laying up time after installing the struts made the lateral deflection of the retaining wall increasing.
- 2. Time-depended deformation of the soft soil under the building increased the whole building settlement.

Soft soil has obvious rheology and sensitivity. Time-depended curve of typical soft soil in Shanghai is shown in Figure 9. High time-depended deformation because of the disturbing of excavation construction causes the sustaining settlement of the building.

 Differential settlement of the building mainly attributes to the local distribution of adverse geological action.
 Settlements in different location of the building are obviously of variation because of the local distri-

obviously of variation because of the local distribution of buried channel, the building settlements of the part under which is the buried channel are greater than those located on normal subgrade soil.

#### 5 CONCLUSION

1. Optimizing the design of the retaining system of the excavation in complex geological condition, shortening the exposure time of retaining wall during excavation should be done to control the lateral deflection of retaining wall, which in turn could reduce the effect on the surroundings.

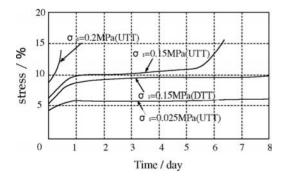


Figure 9. Dynamic triaxial rheology test of soft clay.

- 2. The adverse geological condition such as buried channel should be paid enough attention. The buried channel should be strengthened or separated from the building to decreasing the disadvantageous effect on the building in practice.
- Monitoring data can give feedback to the designers about the construction so that they can adjust design parameters in time to instruct the construction and ensure the safety of the excavation and the surroundings.

#### REFERENCES

- Gao, Y.B., Wu, X.F. & Ye, G.B. 2003. Study on the Influence of Diaphragm Wall Construction on adjacent Building Settlement, Underground Space. 23(2):115~118.
- Liu, G.B. & Lu, H.X. 2004. Study on the Influence of Building Settlement Diaphragm Wall Trench Construction, *Chinese Journal of Geotechnical Engineering*. 26(2):287~289.
- Liu, J.H. & Hou, X.Y. 1997. Manual of Excavation engineering, Chinese Construction Industry Press, Beijing.
- Tang, H.F. & Li, B. 2004. Analysis of measured environmental influence and protection of specially big and deep excavation engineering in soft soil areas, *Rock and Soil Mechanics*. 25(s1):553~558.
- Xiong, J.H. & Lou, X.M. 2004. Monitoring and Analysis of displacement of a Foundation Pit in Soft Soil, Underground Space. 24(3):354~358.
- Zhang, L 2004. Analysis of the Settlement of Buildings Caused by the Construction of Metro Pit under Special Geological Condition, Urban Railway Traffic Study. 23(5):38~41.
- Zhu, R.J., Gao, Q & Qi, G. 2006. Settlement Analysis/n Building near Reining Pile of Deep Foundation Pit, Journal of Chongqing Jianzhu University. 28(2):52~55.

### Performance of a deep excavation in soft clay

G.B. Liu & J. Jiang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China.

#### C.W.W. Ng

Department of Civil Engineering, the Hong Kong University of Science and Technology, HKSAR

ABSTRACT: The observed response of diaphragm wall and the surface settlement of a deep excavation for a metro station in Shanghai soft clay are presented, compared with the similar excavations (retained by a diaphragm wall) in soft clay areas in Asia and five metro station excavations (similar depth) in Shanghai. Results show that the maximum lateral wall deflection  $(\delta_{hm})$  and ground settlement  $(\delta_{vm})$  are 0.32% and 0.1% of final excavation depth (H<sub>e</sub>), respectively. Lateral wall deflections are near the average magnitude value in Shanghai, smaller than those in Singapore. The largest lateral wall deflection is near the excavation center and the ratio of  $\delta_{hmcor}/\delta_{hmcor}$  of the maximum wall deflection is 0.39–0.74. The ground settlement is relatively small and falls in the Zone I limit described by Peck (1969). The adoption of prestressed steel struts, short excavation sections, fast workmanship sequences and compaction grouting may contribute to the deformation characteristics of deep excavations for metro stations in Shanghai.

#### 1 GENERAL INSTRUCTIONS

Excavations for metro stations are in general deep, long and narrow, and located in urban environments. In Shanghai, China, some metro lines have been built and several lines are under construction or planned, to cross the large and congested city. Thick soft-to-medium alluvial and marine deposit strata exist in this area. Under such circumstances, movement control, rather than stability at the deep excavation site becomes the governing factor for design and construction (Liu & Hou 1997).

Research shows that field monitoring and performance analysis are essential to deep excavations in soft soils. Monitoring the field performance of the excavation and the magnitude of movements in the surrounding soil (Peck 1969; O'Rourke 1976; Clough et al.1989) along with estimating the effects of such movements and field performance on adjacent structures (Burland & Wroth 1974, Boscardin & Cording 1979) are important to design engineers. The observed performance of deep excavations and case histories are very useful to verify design assumptions and reduce the construction risk in the excavation process (Whittle et al. 1993; Ng 1998, Ou 1998, Long 2001, Finno & Bryson 2005, 2007, Leung & Ng 2007).

The observed responses of diaphragm walls and the ground surface of a deep multistrutted metro station

excavation in Shanghai soft clay are presented here. Diaphragm wall deformations and ground movements are compared with some case histories in Shanghai and excavations in soft clay in other locations in Asia. From the measured data in this study, some recognition of the characteristics of deep excavations can be obtained and can offer references for back analysis and case studies.

#### 2 SITE CONDITION

#### 2.1 Introduction

Shanghai is located in the front fringe of the Yangtze River Delta in China. The Da Muqiao excavation is situated in the southwest of Shanghai and there is an interchange for two lines, Metro 4 and Metro 9. The site plan is shown in Figure 1. Note that the excavation width is variable.

The site crosses two roads, Da Muqiao Road and Ling Ling Road. The shadowed area in Figure 1 shows the interchange excavation  $(35 \text{ m} \times 25 \text{ m} \times 20.7 \text{ m})$ , in the shape of "cross". The excavation depth is 3.8 m deeper than the excavation depth in other parts (16.9 m). There are several concrete, three-story to six-story buildings along the sides of the excavation.

Figure 2 shows a cross section of section 1-1 of the interchange part (INP); including the support system,

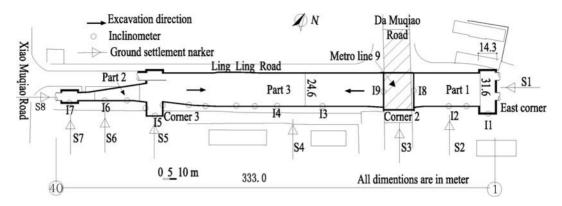


Figure 1. Site plan of the metro station and excavation direction.

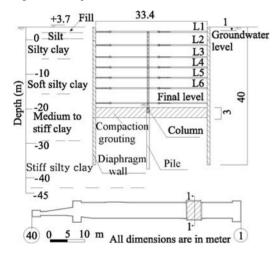


Figure 2. Cross section of the INP (Section 1-1).

the strut level and excavation depth. The concrete diaphragm wall for the INP is 40 m deep and 0.8 m thick; the 16.9 m deep excavation is retained by a 34 m deep and 0.8 m thick perimeter concrete diaphragm wall. The ground level of the site is 3.75 m above the Shanghai City Datum (SCD). There is 3 meter compaction grouting below the final excavation to increase that soil strength in this part. Six steel struts were used at the INP in vertical plane while there were five steel strut levels in other excavation parts. All struts are steel tubes and the first one has a 580 mm diameter and other ones are 609 mm in diameter and are 16 mm thick. Two steeltubes (12.45 m) were inter-joined to form the struts (24.6 m). The struts have a 3 m lateral spacing. The specific installment of struts was described by Wang (2005).

#### 2.2 Geology

The strata of Shanghai are thick soft soils comprising Quaternary alluvial and marine deposits. A high water

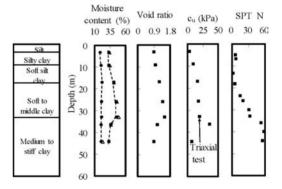


Figure 3. A typical soil profile and soil parameters.

content, low shear strength, high compressibility and low ground bearing capacity are typical characteristics of the soft soil in Shanghai.

The ground is underlain by relative soft to medium– soft marine deposits, which generally consist of uniform bedding planes. A typical soil profile from up to down obtained during site investigation is shown in Figure 3. The top layer is artificial fill, 1.6 m in thickness. About 45 m thick clay underlies the fill. The inside of the excavation is among the clay, which is a quaternary alluvial and marine deposit. In general, groundwater conditions are approximately hydrostatic 1.0 m below ground level.

#### 3 CONSTRUCTION PROCEDURE OF THE EXCAVATION

From installing the diaphragm wall to the completion of casing the bottom slab, the working period is divided into eight main construction stages, listed in Table 1. The excavation depth of the INP is 3.8 m deeper than other parts (see Figure 2) so there is one more excavation stage indicated by italics Table 1.

Table 1. Summary of the main construction.

Stage (1)	Construction operation (2)	Day (3)
1	Construction of diaphragm wall	81
2	Construction of bored pile	116
3	Reduce level dig to L1 & installation of L1 strut	122
4	Excavation to L2 & installation of L2 strut	129
5	Excavation to L3 & installation of L3 strut	136
6	Excavation to L4 & installation of L4 strut	143
7	Excavation to L5 & installation of L5 strut	151
8	Excavation to L6 & casting bottom slab	160
9	Excavation to L7 & casting the bottom slab	168

The excavation of this long metro station was divided into several short sections in the horizon plane. Note that the excavation was from the two short sides to the middle part. The INP and the east part were synchronously excavated, indicated as Part 1. Part 2 was at the west end and was excavated one month after Part 1 was started. After finishing these two excavations, Part 3 was excavated from both ends. A short section of soil excavation was adopted in Part 3 also. Each excavation depth was dug to 0.3 m below the strut level to offer space for installing the struts.

#### 4 INSTRUMENTATION

Inclinometer tubes were fixed to the steel reinforcement cages and concreted in the retaining walls. The casing was installed with a pair of grooves oriented in the expected direction of movement. To avoid twisting of the casing, the grooves were checked before installment to ensure that their twist angle was no more than 0.1%. The casing was fixed tightly to the steel cages of the retaining wall. The groove direction was checked again after lowering the cages into the bentonite. The twist angles of the grooves were measured after the concreting. The allowable value of each twist angle was 0.2%. Probes with a resolution of 0.02 mm/50 mm and temperature of  $-20^{\circ}$  to  $+50^{\circ}$  were used in the inclination monitoring.

Ground surface settlement monitoring markers were located on the sides of the excavation. The markers were located about 1.5–35 m perpendicularly away from wall at each rectangular box station and were secured 0.5 m below the ground surface. The section settlements were monitored with a leveling instrument with a stated accuracy of 0.5 mm/km.

#### 5 OBSERVED WALL DEFLECTION

#### 5.1 Lateral diaphragm wall deflection

Along the south side of this deep and long excavation, seventeen inclinometers were placed to monitor

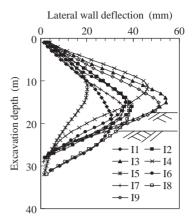


Figure 4. Maximum lateral wall deflection at the final excavation depth.

the lateral wall deflection. Only nine of these inclinometers are pictured, as I1 to I9, in Figure 1, representing three typical locations of the diaphragm walls. (1) Inclinometers I1, I2, I5, I6 and I7 were at or near the corners; (2) Inclinometers I3 and I4 were at the centers of the excavation; (3) Inclinometers I8 and I9 were set for the INP. These nine inclinometers measured the data that are analyzed here for lateral wall deflection.

Figure 4 shows the lateral wall deflection at I1 to 19 at the end of the final excavation (Stage 8). The upper final excavation depth is that of the general part while the lower one is that of the INP. As indicated in this figure, the profiles of lateral wall deflection at the inclinometers are similar but with different deformation magnitudes.

The lateral wall deflection of type (1) is smaller than type (2). This finding shows that wall deflections were affected by the stiffness of the corners. The maximum wall deflection of this deep excavation was at the center of excavation I3, with a value of 54.5 mm; while the minimum wall deflection was at I5, with a value of 20.2 mm. I5 is at Corner 3 and the excavation width on the west side (see Figure 1) is significantly shorter than that on east side. This varied excavation width may contribute to the smallest wall deflection at I5.

The lateral wall deflection of type (3) (INP) is between type (1) and type (2). The maximum values for I8 and I9 were 38.2 mm and 51.0 mm, respectively. Note that the locations of all lateral wall deflections were above the final excavation level.

## 5.2 Relationship between maximum lateral wall deflection and excavation depth

Lectures on lateral retaining wall deflection caused by deep excavations in Asian soft clay were reported by some researchers (Ou et al. 1993, 1998, Wong et al. 1996, Tamano et al. 1996, Lee et al. 1998, Wang et al.

Table 2. Summary of the six metro excavations.

Name	H <sub>e</sub> (m)	Strut No.	Ks	FOS <sub>base</sub>	$\frac{\delta_{\rm hm}}{H_{\rm e}}$ (%)
Pudian	16.3	4	626.4	2.58	0.42
Yangshupu	16.5	4	626.4	2.59	0.57
Pudongdao	16.5	4	626.4	2.61	0.15
Luban	18.2	5	777.6	2.35	0.40
Pudong	17.3	5	1101.6	2.53	0.28
Damuqiao	16.9	5	1414.3	2.43	0.32

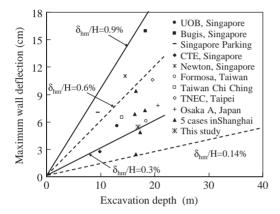


Figure 5. Relationship of the maximum lateral wall deflection to the excavation depth.

2005, Wallace, 1992, Hulme et al. 1989,) Figure 5 shows the measured lateral wall deflection compared to the excavation depth of this deep excavation. Data of five other deep excavations in Shanghai (listed in Table 2.) and several excavations in Taiwan, Japan and Singapore soft clay are plotted in Figure 5 for comparison. Note that all the excavations were in soft clay and the retaining walls were all diaphragm walls.

Figure 5 shows that the magnitude of the maximum wall deflection in Shanghai soft clay is smaller than that in Singapore soft clay while it is similar to that in Taiwan and Japan soft clay. The ratio of  $\delta_{\rm hm}/{\rm H_e}$ in metro excavations in Shanghai is between 0.14%-0.6%, while that in Singapore is between 0.3%-0.9%, and that in Taipei and Japan is in 0.3%-0.6%. The ratio of  $\delta_{\rm hm}/{\rm H_e}$  in this study is 0.32%. A comparison of geotechnical parameters of the soil in Shanghai clay and Taiwan clay shows that undrained strength and water content are similar, but the average measured deflections in the Shanghai soil were smaller than in the Taiwan soft soil. The use of the multistrutted support system, the prestressed steel struts, the short excavation sections and the fast workmanship sequences in Shanghai probably account for these differences.

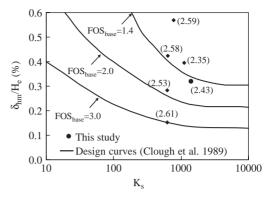


Figure 6. Normalized lateral wall deflection  $(\delta_{hm}/H_e)$  versus system stiffness (K<sub>s</sub>).

## 5.3 Effect of the stiffness of the support system on the maximum wall deflection

The effect of the stiffness ( $K_S$ ) of the support system on lateral wall movement was proposed by Clough et al. (1989).  $K_S$  is expressed as follows:

$$K_{S} = \frac{E_{W}I}{\gamma_{W}h^{4}} \tag{1}$$

where  $E_W$  is the Young's modulus of the wall; *I* is the second moment of the area of the wall section,  $I = t^3/12$ , *t* is the wall thickness; *h* is the average vertical prop spacing of the multistrutted support system and  $\gamma_W$  is the unit weight of water.

Figure 6 shows the normalized lateral wall deflection of the long excavation in the chart as suggested by Clough et al. (1990). Another five metro excavations with the same retaining wall type and support system in Shanghai with excavation depths from 16.3 m to 18.2 m, are chosen for comparison. A summary of these metro excavations is listed in Table 2. The retaining structures of all these excavations were 0.8 m width diaphragm walls and the value of  $E_W$  was 30 kN/mm<sup>2</sup>. This form by Clough (1990) is for excavation in soft-to-medium clay to account for system stiffness and FOS<sub>base</sub>. The value of FOS<sub>base</sub> is calculated by the definition proposed by Clough et al. (1989).

The  $K_s$  value ranges from 626.4 to 1414.3 and the ratio of  $\delta_{hm}/H_e$  varies from 0.15 to 0.57 with the value of FOS<sub>base</sub> from 2.35 to 2.61. However, for a given  $K_s$ , there is a relatively scatter in  $\delta_{hm}/H_e$  values, there may be no very strong correlation between measured  $\delta_{hm}/H_e$  and  $K_s$  value. It can be seen that measured lateral wall deflection decreases with increasing FOS<sub>base</sub>. With similar system stiffness, the larger value of FOS base the smaller the ratio of  $\delta_{hm}/H_e$ . And the measured data do not seem to correspond very well with the design curves. The suggested chart only could give

1

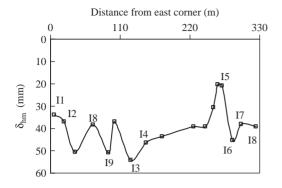


Figure 7. Maximum lateral wall deflection versus.

preliminary predictions of wall movement in six metro excavations in Shanghai.

#### 5.4 Three-dimensional deformation of excavation

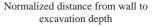
Researches on three-dimensional responses of excavations by measured data or finite analysis (Bono et al.1992, Ou et al. 1993, Wong 1993, 1996, Lee et al. 1998, Finno et al. 2007) showed that ratios of  $\delta_{corner}/\delta_{center}$  in deeper excavations are smaller in general. Figure 7 shows the measured maximum lateral wall deflection ( $\delta_{hm}$ ) at the end of the final excavation in this study. The lateral wall deflection at and near the center of the excavation is larger than the wall deflection near or at the corners, which means that the stiffening effect of the corners affected the deformation of the diaphragm walls. Note that the reduced lateral wall deflection at I5 is significant. The ratio of each maximum wall deflection to the maximum wall defection at the center,  $\delta_{hmcor}/\delta_{hmcen}$ , of the wall was 0.39-0.74.

#### 6 SURFACE SETTLEMENTS

In some cases, settlement was measured after the starting of construction of the diaphragm wall; some were measured at different times. Making consistent comparisons is therefore difficult. In this study, the settlement of the ground surface during and after the final excavation was measured.

Eight groups of ground settlements were measured along one of the long sides (south side) and two ends of the excavation (see Figure 1). All data were selected from one side of the site. Figure 8 shows the relation of the maximum surface settlements to the distance from the diaphragm walls at the final excavation depth of this study.

In comparison with surface settlements in Singapore and Taiwan soft clay, the measured data in this deep excavation fell into a relatively small range, despite the weak ground conditions in Shanghai.



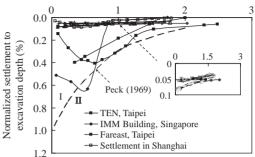


Figure 8. Distribution of the ground surface settlement normalized by the excavation depth.

Wang (2005) summarized the surface settlements of some metro excavations in Shanghai and also indicated that the magnitude was relative small ( $\delta_{hm}/H_e$  is less than 0.1%), plotted in Figure 8 also.

The 3 m thick compaction grouting below the final excavation level, which improved the ground condition, and the high-stiffness diaphragm wall and prestressed multistrutted steel struts and fine workmanship may contribute the small surface settlements.

#### 7 DISCUSSION OF THE MOVEMENT MECHANISM

Compared with the lateral wall deflection and ground settlement caused by excavation in soft clay in Singapore, Japan and Taiwan, the measured deformations in this deep excavation for a metro station in Shanghai is relatively smaller (see Figure 7). The 3 m thick compaction grouting below the final excavation level, usage of prestressed multi struts and a stiffing diaphragm wall, the short soil excavation sections and quick and fine construction are main factors associated with this measured result. Among the above reasons, one that ought to be noted is that the vertical spacing of the struts in the Shanghai excavation was smaller than that in Singapore and Taiwan, which contributed to the small wall deflection. Generally in 12 m to 18 m deep Shanghai metro excavations, four to five struts are set to support the system, while in the 9.9 m deep excavation of CTE in Singapore, for example, only two struts were set in the vertical plane. The system stiffness in the Shanghai excavation was larger; it is not surprising to observe the smaller deformation in Shanghai.

#### 8 CONCLUSIONS

Observed data from a deep and long metro excavation, with an interchange metro station  $(35 \times 25 \text{ m}^2)$  crossing it, in Shanghai soft clay is presented. On the basis of the interpreted observed field data, the following conclusions are drawn:

- 1. The wall deflections along the long side and three corners are different, but with similar profiles. The ratio of  $\delta_{\rm hm}/H_{\rm e}$  in this deep excavation is less than 0.32%. The measured maximum wall deflection is near the center. Three-dimensional analysis of this deep excavation shows that the corner effect is found. The  $\delta_{\rm hmcor}/\delta_{\rm hmcen}$  of the wall was 0.39–0.74, though the aspect ratio was large.
- 2. Compared with the excavations in soft clay in Taiwan, Japan and Singapore retained by diaphragm walls and another five similar metro excavations in Shanghai, this maximum lateral wall deflection falls near the average magnitude for Shanghai and is at the low limit line for Singapore. The undrained strength and water content of Shanghai clay and Taiwan clay is similar, but the average measured deflections in Shanghai are smaller than in Taiwan. The use of prestressed steel struts, a support system, short excavation sections, a fast workmanship sequence, and compaction grouting below the final excavation level in Shanghai may contribute to this smaller deflection.
- 3. The influence of support stiffness on wall movement was analyzed. A smaller lateral wall deflection of excavation with large Ks values was observed. Though scatter exists in the chart proposed by Clough et al. (1990), the suggested chart seems to allow preliminary predictions of wall movements in excavations for metro stations in Shanghai soft soil.
- 4. The settlement along the corners and the long side is relatively smaller ( $\delta_{hm}/H_e$  is less than 0.1%) than that in other Asian soft clay, though the soft clay strata is thick in this deep excavation. This settlement is located in the zone I as proposed by Peck (1969), which is consistent with the case study in Shanghai (Wang et al. 2005).

#### ACKNOWLEDGMENTS

The authors would like to thank colleagues who contributed to the field monitoring in Shanghai and acknowledge the earmarked research grant 618006 provided by the Research Grants Council of the HKSAR.

#### REFERENCES

Bono, N.A., Liu, T.K. & Soydemir, C. 1992. Performance of an internally braced slurry-diaphragm wall for excavation support. Slurry walls: Design, construction, and quality control, ASTM STP 1129, D.B.Paul, R.G.Davidson, and N. J. Cavalli, eds., ASTM, Philadelphia. pp.169–190.

- Boscardin, M.D. & Cording, E.J. 1979. Case studies of building behavior in response to adjacent excavation. University of Illinois Rep. for the U.S. Department of Transportation, Rep. No. UMTAIL-06-0043-78-2, Washington D. C.
- Burland, J.B. & Worth, C.P. 1974. Settlement of buildings and associated damage. Proceeding of conference on settlement of structures, Cambridge. pp. 611–654.
- Clough, G.W., Smith, E.M. & Sweeney, B.P. 1989. Movement control of excavation support systems by iterative design. In proceeding of current principles and Practices on foundation and engineering, ASCE, New York. Vol. 2, pp.869–884.
- Clough, G.W. & O'Rourke, T.D. 1990. Construction induced movement of in-situ walls. In proceeding of design and performance of earth retaining structures. ASCE special conference, Ithaca, N.Y. pp. 439–470.
- Finno, R.J., Bryson, L.S. & Calvello, M. 2005. Performance of a stiff support system in soft clay. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 128(8). pp. 660–671.
- Finno, R.J., Blackbum, J.T. & Roboski, J.F. 2007. Threedimensional effects for supported excavation in clay. Journal of Geotechnical and Geoenviromental Engineering, ASCE, 133(1). pp. 30–36.
- Hulme, T.W., Potter, J. & Shirlaw, N. 1989. Singapore MRT system: Construction. *Proc.*, *Instn. of Civ. Engrs.*, Vol. 86, London, 709–770.
- Lee, F.H., Yong, K.Y., Quan, K.C. & Chee, K.T. 1998. Effect of corners in strutted excavations: field monitoring and case histories. Journal of Geotechnical and Geoenviromental Engineering, ASCE, 124(4). pp. 339–349.
- Leung, E.H.Y. & Ng, C.W.W. 2007. Wall and ground movements associated with deep excavations supported by cast in situ wall in mixed ground conditions. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 133(2). pp. 129–143.
- Liu, J.H. & Hou, X.Y. 1997. Excavation engineering hand book. Chinese Construction Industry Press, Beijing, P.R. China.
- Long, M. 2001. Database for retaining wall and ground movements due to deep excavation. Journal of Geotechnical and Geoenviromental Engineering, ASCE, 127(3). pp. 203–224.
- Ng, C.W.W. 1998. Observed performance of multi-propped excavation in stiff clay. Journal of Geotechnical and Geoenviromental Engineering, ASCE, 124(9). pp. 889– 905.
- O'Rourke, T.D., Cording, E.J. & Boscardin, M.D. 1976. The ground movements related to braced excavation and their influence on adjacent structures, Univ. of Illinois Rep. for the U.S. Dep. of Transportation, Rep. No. DOT-TST-76T-22, Washington D.C.
- Ou, C.Y., Liao, Hsied. P.G. & Chiou, D.C. 1993. Characteristics of ground surface settlement during excavation. Canadian Geotechnical Journal, 30(5), pp. 758–767.
- Ou, C.Y. Liao, Hsied. & Lin, H.D. 1998. Performance of diaphragm wall constructed using top-down method. Journal of Geotechnical and Geoenviromental Engineering, ASCE, 124(9). pp. 798–808.
- Peck, R.B. 1969. Deep excavation and tunneling in soft ground. In proceeding of the 7th international conference on soil mechanics and foundation engineering, Mexico City. Vol. 1, pp. 225–281.

- Tamano, T., Fukui, S., Mizutani, S., Tsuboi, H. & Hisatake, M. 1996. Earth and water pressures acting on a braced excavation in soft ground. Proc., Int. Symp. Geo Aspects of Underground Constr. in Soft Ground, City University, London, 207–212.
- Wallace, J.C., Ho, C.E. & Long, M.M. 1992. Retaining wall behavior for a deep basement in Singapore marine clay. *Proc., Int. Conf. Retaining Struct.*, Thomas Telford, London, 195–204.
- Wang, Z.W., Ng, C.W.W. & Liu, G.B. 2005. Characteristics of wall deflections and ground surface settlements in Shanghai. Canadian Geotechnical Journal, 42(10). pp. 1243–1254.
- Whittle, A.J., Hashaha, Y.M. & Whitman, R.V. 1993. Analysis of deep excavation in Boston. Journal of Geotechnical Engineering, ASCE, 119(1). pp. 69–90.
- Wong, L.W. & Patron, B.C. 1993. Settlements induced by deep excavations in Taipei. 11th Southeast Asian Geotechnical Conf., Singapore. pp. 787–791.
- Wong, L.W., Poh, T.Y. & Chuash, H.L. 1996. Analysis of case histories from construction of the central expressway in Singapore. Canadian Geotechnical Journal, 33(1). pp. 732–746.

#### SYMBOLS

 $E_W$  = Young's modulus of the wall;

 $FOS_{base} = factor of safety against the basal heave;$ 

h = average vertical prop spacing of multi-strutted support system;

 $H_e = final$  excavation depth;

I = second moment of inertia of the wall section;

INP = interchange part excavation;

Ks = system stiffness;

t = the wall thickness;

 $\gamma_W$  = unit weight of water;

 $\delta_{\text{corner}} = \text{deformation}$  at corner of excavation

 $\delta_{\text{center}} = \text{deformation}$  at center of excavation

 $\delta_{hm}$  = magnitude of maximum horizontal diaphragm wall deflection;

 $\delta_{\text{hmcor}} = \text{maximum}$  lateral wall deflection at corner;  $\delta_{\text{hmcen}} = \text{maximum}$  lateral wall deflection at center;

 $\delta_{\rm vm}$  = maximum ground surface settlement;

# Deformation monitoring during construction of subway tunnels in soft ground

#### S.T. Liu

Department of Surveying and Geo-Informatics, School of Civil Engineering, Tongji University, Shanghai, P.R. China Department of Civil Engineering, Henan Institute of Engineering, Zhengzhou, P.R. China

#### Z.W. Wang

Fourth Team of Henan Province Coal Field Geology Bureau, Xinzheng, P.R. China

ABSTRACT: Monitoring the deformations of subway tunnels in soft ground is a principal means for selecting the appropriate excavation and support methods in the design. In this paper, briefly describes the basic deformation monitoring requirements, analyzes the tunnel deformation monitoring accuracy difference between mountain area and urban area, discusses the measurements of deformations at ground surface, measurements of deformations on the ground and measurements in the tunnel, especially gives an example of vertical convergence monitoring, introduces the data processing method and the quality evaluating method based on the measured data. Mainly explores the technique of tunnels' crown settlement monitoring, convergence monitoring and the Automatic Deformation Monitoring methods. It is put forward that the monitoring job during the construction of subway tunnels can be done with both traditional and modern surveying instruments and the job will be significant to the same kinds of projects.

#### 1 INSTRUCTIONS

Determining shape and position changes occurred in engineering is one of an application area of geodetic surveys. Temporary and permanent deformations occur in engineering structures such as dams, bridges, tunnels, viaducts and towers due to natural and artificial forces. Causes of these deformations are usually physical properties of ground, weight of structure, active external forces etc. (Clough 1960). The need to upgrade and further develop transportation infrastructure has lead to the on-going construction of large-diameter, long tunnels under difficult conditions.

Such conditions usually arise from a combination of adverse ground and groundwater regimes, very high overburden pressures or, in the case of urban tunnels, the existence of sensitive structures within the zone of influence of the tunnel. It is imperative to provide accurate and frequent monitoring of tunnel linings to detect any movements that could pose a safety hazard.

Deformation monitoring in tunneling usually includes some of the following measurements (Dunnicliff 1993):

1. Convergence of the tunnel wall, and usually crest settlement and spring-line closure.

- 2. Deformations at ground surface including settlements and tilts of surface structures.
- 3. Deformations in the ground around the tunnel.

Tunnel deformations can be monitored with geodetic or geotechnical methods. Geodetic measurements provide absolute coordinates of the target locations in time, while geotechnical measurements usually provide relative displacements of the target locations with respect to an initial condition (at the time when the initial measurement is recorded) (Hisatake 1999).

Geotechnical measurements can provide absolute coordinates of the target locations in time, if the initial positions of the targets are obtained using geodetic means. Depending on the tunneling application, deformation measurements can be recorded, processed and evaluated in real-time using digital recording and telecommunication systems, or can be recorded manually and processed later in batch mode.

Real-time processing of ground deformations offers the possibility of rapid response to upcoming situations but requires advanced technology and an appreciably higher cost. Thus, real-time monitoring is limited to cases where rapid response is absolutely necessary, i.e., mainly in urban tunnels near sensitive structures. In this paper, briefly describes the deformation monitoring requirements, and then discusses the deformation monitoring at ground surface, in the ground and in the tunnel separately, furthermore gives an example of traditional vertical convergence monitoring, at last mainly introduces the Automatic Deformation Monitoring method.

#### 2 BASIC REQUIREMENTS

The basic requirements are: lay out monitoring benchmarks, monitoring targets timely, take measurements regularly, analyzes the survey data and feedback information to relative departments in time (Kavvadas 1999).

Measurement accuracy depends on the purposes of the deformation observation. In order to ensure monitoring accuracy, Operating personnel must be familiar to the purposes and equipment operating rules, survey team members should corporate and work carefully.

Each monitoring step should abide the "four fixed" principle (Liu 2006). The so-called "four fixed" principles namely: instruments and rods fixed, observing staff fixed, monitoring method fixed and monitoring environment fixed.

The first step in deformation monitoring is to carry out a basic survey, consisting of individual surveys inside a narrow time frame. All following surveys will be compared to the results of this survey. It is therefore important that this survey is carried out before the deformations to be observed can be expected to occur and it is also important that the basic survey is carried out with a high degree of reliability regarding results, as it is obviously impossible to follow up with checksurveys after deformations have taken place or can be expected to have taken place. It is recommended that the first elevation result should be figured out in the average of at least two times of monitoring (Liu 2007).

The equipment being used should be tested before the first time monitoring, after continuously use for 3–6 months the equipments should be tested again (Kaiser 1993).

Benchmarks and stations and Monitoring points inside the tunnels should be set in area where rocks are stable. Benchmarks and stations inside the tunnel are installed in special brackets, which forced centered (Kontogianni & Stiros 2002, 2003).

#### 3 MONITORING ACCURACY DIFFERENCE BETWEEN MOUNTAIN AREA AND URBAN AREA

#### 3.1 Objectives difference

The objectives of ground deformation monitoring are different in mountain and urban tunnels. In mountain

tunnels, the main objective of deformation measurements during construction is to ensure that ground pressures are adequately controlled, i.e., there exists an adequate margin of safety against collapse, including roof collapse, bottom heave, failure of the excavation face, yielding of the support system, etc. (Mihalis & Kavvadas 1999).

Adequate control of ground pressures ensures a safe and economical structure, well adapted to the inherent heterogeneity of ground conditions. This procedure is compatible with modern tunnel design methods which include a range of excavation and support systems to cover the anticipated spectrum of conditions along the tunnel, with selection of the applicable system in each case relying on the encountered geology at the tunnel face, experience on tunnel behavior at previously excavated sections under similar conditions and, on accurate deformation measurements, i.e., by applying the so-called "observational method".

This method of construction can ensure adequate safety and, at the same time, an economical construction. On the contrary, in urban tunnels, the main objective of ground deformation monitoring is to limit ground displacements to values sufficiently low to prevent damage to structures and utilities at ground surface. Thus, the fundamental difference in deformation monitoring stems from the fact that in mountain tunnels the objective is to guard against an ultimate limit state (i.e., collapse) while in urban tunnels the objective is to guard against serviceability limit states (i.e., crack initiation) for structures and utilities at ground surface.

#### 3.2 Monitoring accuracy difference

As a result of these differences in objectives, design philosophies, and construction techniques, the types and required accuracy of the measured ground deformations vary between the two classes of tunnels, as follows.

1. In mountain tunnels, considerable ground deformations are deliberately permitted (and often provoked) in order to reduce the initially very large "geostatic" loads on the temporary support by increasing ground de-confinement. Such reduction of ground loads on the tunnel support can be appreciable and, thus, extremely beneficial provided that excessive "loosening" of the rock mass is prevented (such "loosening" can cause roof failures and an eventual increase of the ground loads). De-confinement is achieved by controlled inward ground deformation at the excavation face (facetake), controlled delay in the completion of the temporary support measures (by increasing the distance from the face where the tunnel invert is closed), a relatively flexible temporary support system (e.g. long passive rock-bolts and thin sprayed concrete liners) and, finally, by installing the permanent lining at a later time when evolution of the long-term (creep) ground deformations has practically stopped. In extreme cases of strongly squeezing ground conditions, sliding supports may be installed to permit tunnel wall convergences of several tens of centimeters. In all these cases, control of ground deformations depends strongly on efficient and timely deformation measurements. However, due to the large ground deformations (several centimeters and even several tens of centimeters), the required level of precision of these measurements needs not be excessive; typically, accuracy of the order of one centimeter is sufficient in mountain tunnel applications.

2. In urban tunnels, the main objective is limiting ground deformations around the tunnel and thus causing the minimum possible movement and disturbance at ground surface and the structures founded there. This is achieved by (a) limiting inward ground deformation at the excavation face (face-take), e.g. by face pre-reinforcement using fiber-glass nails, stiff steel beams (fore-poles), cement- or jet-grouting techniques, (b) by installing a stiff temporary lining, usually including invert closure, as early as possible and (c) by installing the final lining as quickly as possible, especially when tunnel wall convergences continue to evolve with time. The above "stiff" construction methods tend to reduce ground de-confinement and thus the ground loads on the tunnel lining are a significant fraction of the initial "geostatic" loads are much smaller than those in deep mountain tunnels. Due to the small ground deformations induced by tunneling (usually less than 10 mm at ground surface and occasionally less than 5 mm), measurement precision and the early installation of the measuring devices is of utmost importance.

## 4 DEFORMATION MONITORING AT GROUND SURFACE

Measurements of deformations at ground surface are crucial in urban tunneling projects where damage to surface structures and utilities should be prevented. These measurements typically include settlements (and heaves) of structures as well as tilting. Such measurements are performed with surveying instruments (Precise Geodetic Level, total stations and GNSS), or with geotechnical instruments like Electronic Liquid Level Gauges, Electrolytic Tilt Sensors (electro-levels) surface clinometers/tilt meters, precise taping, and crack-meters.

Precise leveling and façade monitoring are the most common methods for monitoring displacements at ground surface. The accuracy of these measurements is typically 0.2 mm (over about 100 m lengths) for precise leveling and 1" for angles and  $(1 \text{ mm} + 2 \text{ mm} 10^{-6}\text{D})$  for distances in the case of façade monitoring with total stations. Façade monitoring can be automated and measurements can be obtained and transmitted in practically real-time.

Inside buildings and in areas with limited visibility for the application of the above geodetic measurements, geotechnical measurements can be performed using the following precision instruments (Moaveni 2003):

- Electronic Liquid Level gauges, for the measurement of settlements at several locations. The method consists of installing a number of liquid filled pots, hydraulically connected to a reference pot located in a stable area. The elevation of the liquid in the reference pot is maintained constant by means of a mini-pump, reservoir and an overflow unit. LVDT float sensors monitor the height of the liquid in each pot. When settlement or heave occurs, the sensor detects the apparent change in the height of the liquid and transmits the signal to a data logger for continuous monitoring and real-time processing. The accuracy of the system is 0.3 mm.
- 2. Electrolytic tilt sensors are precision bubble levels that are electrically sensed as a resistance bridge. The bridge circuit outputs a voltage that is proportional to the tilt of the sensor. The sensors are usually attached on metal beams, one to three meters long, with their ends mounted on the structural elements to be monitored. Chains of such tilt sensors are often installed in sequence in the horizontal direction to monitor differential settlements along long walls or beams. The precision of the instrument is typically1".
- Surface clinometers (tilt meters), precise taping using invar tape between fixed anchor points and various types of crack-meters are also used in measuring deformations at ground surface.

## 5 DEFORMATION MONITORING IN THE GROUND

These measurements record the deformation of target positions in the ground, either around the tunnel or deep below the ground surface. They are often used to calculate strains or control the deformation of characteristic points in the ground (e.g. below the foundation of buildings, below utility lines, etc.) (Sakurai 1981).

Such measurements are performed with geotechnical instruments including single- or multi-point borehole rod extensometers, magnetic extensometers, sliding micrometers, inclinometers, probe deflect meters (often called sliding curve meters) and deep settlement plates. These instruments can be installed either from the ground surface (before the tunnel face reaches the area of the instrument) or from inside the tunnel (radically from the tunnel wall or along the tunnel axis ahead of the excavation face).

#### 6 DEFORMATION MONITORING IN THE TUNNEL

Tunnel wall convergence (closure) between references points (hooks) bolted on the tunnel walls is usually measured with standard metal tape extensometers. For distances up to about 10–15 meters, the accuracy of such measurements is typically 0.2 mm. The method is easy to use and maintain but it only offers the magnitude of the deformation along the line of measurement.

Because of this disadvantage, in most present-day tunneling applications, deformations of the tunnel walls are obtained in three dimensions, by routine geodetic surveying using total stations with integrated distance measurement. In such applications, optical reflector targets are installed at regular distances along the tunnel axis (e.g. at sections every 15–20 m) and, on each section, at selected locations of the tunnel wall (e.g. five reflectors per section: at the crest, at 45 degrees and at the spring-line). As tunnels are usually long, the fixed (stable) reference positions are typically located outside the tunnel, often at distances exceeding one kilometer and usually out of sight from inside the tunnel.

Thus, measurements of the targets inside the tunnel are obtained by placing the total station at pre-defined rugged stations1 (bolted on the tunnel wall) and successively moving the instrument forward (towards the tunnel excavation face) while measuring the coordinates of the visible targets from each station. The theoretical accuracy of these measurements over a distance of about 100 m) is about 2–3 mm for lengths and  $\pm 2^{"}$  for angles. For long tunnels, the accuracy of the measurements is usually reduced by unclean atmosphere and due to the multiple positions of the instrument (especially if these positions are not stable due to creep deformations of the tunnel).

A recent development of measuring the geometry of tunnel walls in cross section (and thus assess the deformation in the interval between two measurement epochs) is the Tunnel Profile Scanners (profile meters). In addition to measuring tunnel wall convergences in time, profile meters are also employed for a variety of other purposes like comparing the actually excavated tunnel cross section with the design requirement and for measuring the volume of shotcrete placed on the excavated rock surface (by measuring the profile before and after shotcreting). Tunnel profile meters are fully digitized photogrammetric measuring devices.

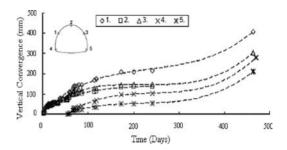


Figure 1. An example of settlement monitoring result (Villy Kontogianni et al. 1999).

A typical such system consists of two CCD cameras which are mounted on a portable frame. The cameras produce stereoscopic digital images of the tunnel surface. The position of the camera frame is automatically determined by a total station with automatic target recognition placed up to a maximum distance of 100 meters.

For this purpose, three reflector targets are permanently mounted on the frame. Digital images are automatically stored in a laptop computer and can be processed to provide the 3-D coordinates of the surveyed tunnel wall surface with an accuracy of  $\pm 5$  mm for each coordinate.

Although the accuracy of this method is low compared to routine geodetic surveying, the advantage of recording a very large number of points on the tunnel wall outweighs the low accuracy in many applications.

#### 6.1 Traditional geodetic measurements

Three dimensional coordinates of object points (deformation points) which will be constructed on ceiling, sole, and side walls of cross- section determined at different intervals on tunnels were measured by electronic instruments. The coordinates of object points which were constructed on ceiling, sole, and side walls were measured at determined period intervals. By using coordinates measured at different periods, forming movements were determined. (Figure 1 is an example of settlement monitoring result).

#### 6.2 Automatic deformation monitoring

## 6.2.1 Automatic deformation monitoring requirement

High-quality precision optical monitoring targets mounted on support of excavation and tunnel linings, with fully automated motorized total stations under computer control to monitor remotely the three components of movement. Also required was a measurement precision of 1 mm for sight distances up to 100 m, with wireless data links to the control site. The robotic total stations would have to be totally automatic and operate unattended 24/7 under all weather conditions, and also be insensitive to refraction effects caused by temperature or pressure variations. Each target would have to be "hit" at least every 30 minutes.

Multiply TCA2003s, located where they could monitor hundreds of target prisms to be affixed at predefined intervals to the structures to be monitored. The measurement data would be transmitted at specified intervals via radio data links to a central location, situated miles from the actual instrument monitoring stations. The total network would be tied together and controlled by GeoMoS (Geodetic Monitoring Software) system.

#### 6.2.2 Installation challenges

There were two TCA2003 total station sites on each side, for a total of four. The total stations were permanently mounted on fixed location with vented heavy-duty glass enclosures to protect the system from the elements. Each installation included an intercom radio and modem with directional antenna to transmit data from the site to the controller. Band pass filters were added to overcome the high levels of RF activity in the airport environment. The pedestals were isolated to eliminate vibration and movement.

The GeoMoS software was installed on the computer network, and was configured so that data can be accessed via a secure IP link by authorized personnel and the resident engineer. Protecting the remote sites from wind and weather presented special challenges. The glass enclosures had to be robust and rugged, capable of withstanding heavy winds, rain, snow and ice, and they had to be non-reflective so as not to distort the EDM signal. Necessity being the mother of invention, they experimented with a number of different types of enclosures including clear round dog igloos until they found the right solution.

#### 6.2.3 Monitoring software

The GeoMoS software is a powerful tool for controlling the network of the remote sites, as well as collecting data, providing alarms, post-processing, reporting and visualizing data. The software represents the data and results in graphical or numerical format. You can select a time-line graph showing the trends of movement over selected time periods (Burland 2001). Multiple points can be viewed simultaneously in the same graph. Alternatively, you can select a vector view that shows displacement for a selected area, to easily see where the greatest movement has occurred.

Senior-level project engineers and other authorized personnel can access the GeoMoS data through a realtime web-based portal from any PC or laptop. They can log onto the secure GeoMoS site and download system status and reports. Measurement tolerances are established and loaded into the GeoMoS system, If any

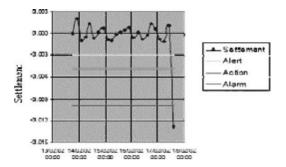


Figure 2. An example of tabular report of Automatic Deformation Monitoring for Hong Kong KSL Railway (Tang et al. 2007).

of these tolerances are exceeded, an automatic alarm is activated (Figure 2 is an example of tabular report).

#### 7 CONCLUSION

In order to determine deformations of the subway tunnel in soft ground, deformation monitoring must be taken on the ground surface, in the ground and in the tunnel. Practice shows that the traditional deformation measurement can be achieved with 0.5–1 mm displacement accuracy, but deformation measurements can also be recorded, processed and evaluated in real-time by digital recording and telecommunication systems. Real-time processing of ground deformations offers the possibility of rapid response to upcoming situations but requires advanced technology and an appreciably higher cost. So real-time monitoring is limited to cases where rapid response is absolutely necessary, i.e., mainly in urban tunnels near sensitive structures.

#### REFERENCES

- Burland, J. B. & Standing, J. R. 2001. Building Response to Tunneling Case Studies from Construction of the Jubilee Line Extension. London: Thomas Telford publishers.
- Clough, R.W. 1960. The Finite Element Method in Plane Stress Analysis, Proceedings of American Society of Civil Engineers, 2nd Conference on Electronic Computations. 23: 345–378.
- Dunnicliff, J. 1993. Geotechnical Instrumentation for Monitoring Field Performance. U.S.A: John Wiley & Sons Inc.
- Hisatake, M. 1999. Direct Estimation of Initial Stresses of the Ground Around a Tunnel. *Proceedings of the Numerical Methods in Geomechanics*: 373–377.
- Kaiser, P. 1993. Deformation Monitoring for Stability Assessment of Underground Openings. *Compressive Rock Engineering*: 607–630.
- Kavvadas, M. 1999. Experiences from the construction of the Athens Metro project, *Proceedings of 12th European Conference of Soil Mechanics and Geotechnical Engineering*: 1665–1676.

- Kontogianni, V. & Stiros, S. 2002. Shallow Tunnel Convergence Predictions and Observations. *Engineering Geology* 63(3–4):333–345.
- Kontogianni, V. & Stiros, S. 2003. Tunnel Monitorig During the Excavation Phase:3-D Kinematic Analysis Based on Geodetic Data. Proceedings of 11th FIG Symposium on Deformation Measurements. Santorini, Greece.
- Kontogianni, V., Tesseris, D. & Stiros S. 1999. Efficiency of geodetic data to control tunnel deformation. *Proceedings* of The 9th FIG International Symposium on Deformation Measurements. Olsztyn, Poland:206–214.
- Liu, S. 2006. Deformation Measurements During the Construction of Large Dam Projects. *Chinese Journal of* Underground Space and Engineering 06(Z2):1346–1348.
- Liu, S. & Zhao, Z. 2007. Deformation Monitoring of 70 m Span Box Girders of Hang-Zhou Bay Sea-Cross Bridge at Construction Stage. World Bridge 07(2): 58–60.

- Mihalis, I. & Kavvadas, M. 1999. Ground Movements Caused by TBM Tunnelling in the Athens Metro Project. Proc. Int. Symp. on the Geotechnical Aspects of Underground Construction in Soft Ground, Tokyo, Japan, June 1999: 269–274.
- Moaveni, S. 2003. *Finite Element Analysis*. New Jersey: Pearson Education.
- Sakurai, S. 1981. Interpretation of Displacement Measurements. Proceeding of the International Symposium on Weak Rock, Tokyo: 751–756.
- Tang, E., Lui, V. & Wong, A. 2007. Application of Automatic Deformation Monitoring System for Hong Kong KSL Railway. *Monitoring Strategic Integration of Sur*veying Services. Proceedings of FIG Working Week 2007, Hong Kong SAR, China.

### The construction and field monitoring of a deep excavation in soft soils

#### T. Liu & G.B Liu

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

#### C.W.W. Ng

Department of Civil Engineering, the Hong Kong University of Science and Technology, HKSAR

ABSTRACT: This paper describes the construction of a 40 m multi-propped deep excavation in the downtown area of Shanghai, China and the interpretation of the monitoring data. The entire excavation was 263 m in length, 23 m wide and 38–41 m deep. The excavation was divided in three sections, i.e., the eastern, the middle, and the western pits. Noticeable characteristic of this project are the excavation of a nearly 60 m wide section embedded in the Huangpu River and the presence of many sensitive buildings nearby. In this paper, some construction details of the excavation and the monitoring results are presented and discussed.

#### 1 INTRODUCTION

#### 1.1 Excavations for underground railway networks in Shanghai

With the rapid development of Shanghai and the city's underground railway networks, metro construction has become a multidisciplinary and multifaceted project. During construction, regular service of neighboring underground railway lines must remain open. Figure 1 shows the distribution of completed and ongoing excavations for underground railway networks in Shanghai as of 2007. 92% of the metro stations are deeper than 15 m and 31% are over 20 m in depth. Zhao & Yang (2004) have suggested that any excavation, which is over 20 m in depth, will impose severe geotechnical challenges in soft soils. This paper describes the construction of a 40 m deep excavation in the downtown area of Shanghai, China and the analysis of its construction impacts to the environment.

#### 1.2 Engineering background

The tunnel between the South Pudong Road Station and the Nanpu Bridge Station is a cross-river section on Line 4 of Shanghai's underground railway system (see Figure 2). On 1 July 2003, a failure occurred resulting in flooding of the tunnel from a sub-artesian aquifer. This led to the collapse of some sections of the tunnel, excessive ground settlements and the closure nearby buildings because of the potential danger to the public.

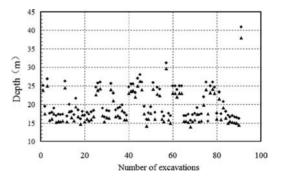


Figure 1. Statistics of depth of completed and on-going excavations for underground railway networks in Shanghai.

An in-situ restoration program was implemented in which an open cut was made in the damaged sections of the tunnel to expose the collapsed sections, clear the debris, and then to construct a new open structure.

As the extent of the collapse was large, the restoration work was constructed both on shore and in the river. Using a cofferdam, a piled steel platform was constructed above the river to excavate the collapsed tunnel after it had been backfilled. Two undamaged tunnel sections, which were flooded, were de-watered by high-pressure pumps and then reinforced. The entire restoration project included five major components: three deep excavations at the eastern, middle, and western parts of the tunnel, clearance work for

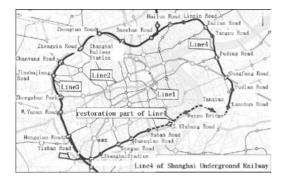


Figure 2. Line 4 of Shanghai's underground railway.

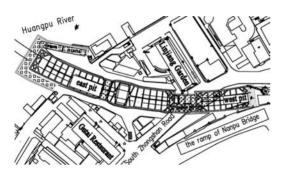


Figure 3. General layout of the restoration project.

the undamaged tunnel in Pudong (1003 m) and in Puxi (760 m), and the connection work at the two ends of the damaged and undamaged tunnel sections. The general layout of the restoration project is shown in Figure 3.

A 65 m deep 1.2 m thick concrete diaphragm wall was used to retain the three excavations (or so-called pits). The total length of the excavations was 263 m in length and 23 m wide (see Figure 3). The depth of the excavations varied from 38 m to 41 m. Ten and nine levels of concrete propping slabs were installed to support the diaphragm wall at the eastern and the western pits, respectively (see Figure 4). In the middle pit, 35 m long, 1.2 m in diameter cast-in-place piles were constructed to support the steel columns whose cross-sectional area is  $650 \times 650 \text{ mm}^2$ . In turn, these columns were used to carry the horizontal propping slabs.

#### 2 GEOLOGICAL CONDITIONS

The restoration project was located adjacent to Outer River Road, Dongjiadu Road and Zhongshan South Road, where the terrain was quite flat. Figure 5 compares the current geological profile obtained from a shift put down after the tunnel had collapsed and the previously surveyed data obtained before the tunnel

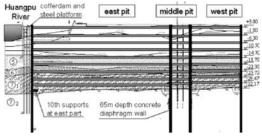


Figure 4. Ground profile at the restoration project site.

Former sur		Current survey data 3. 35		
1	2.23 ± 0.00	1	± 0.00 -4.29	
② 2 ⑤ 1	- 11. 37 - 16. 47	2	- 15. 09	
52 6	-20. 37	(5) (6)	-22.19 -25.69	
1	- 33. 27	7)1	- 37. 99	
2	-45.77			
	-	⑦ 2	-60.85	
	_	9	- 73. 65	

Figure 5. Comparison of strata on site.

collapse. The physical and mechanical properties of the soil layers are summarized in Table 1.

Based on the ground investigations, groundwater was expected along and above the tunnel. The groundwater table is 0.5–1.0 m below the ground surface. The ground comprises clay and silty clay layers at shallow depths and more sandy materials at greater depths. Layer 7 is the first aquifer in Shanghai, while layer 9 the second aquifer. They are connected hydraulically on site but apparently they are not connected to the Huangpu River. The groundwater and the Huangpu River have no obvious hydraulic link on site.

#### 3 ENGINEERING DIFFICULTIES AND CONSTRUCTION TECHNOLOGY

The scale of the restoration work was large and the depth of excavations was the deepest in Shanghai at that time. Around the excavation site, there were sensitive buildings nearby such as the Linjiang Garden Building and the Nanpu Bridge which required protections. Ground conditions were very complex owing to the tunnel collapse and the materials left from the

Layer No.	Soil Type	Thickness (m)	Depth below ground (m)	Water content, w (%)	Unit weight, γ (kN/m <sup>3</sup> )	Void ratio, e	Cohesion, (kPa)	Friction angle (°)	Average SPT N values
1	Fill	7.70	-4.29	33.5	18.7	0.98	14	27.5	_
22	Silt clay	1.8	-15.09	30.9	18.7	0.90	7	32.0	10
\$ <sub>1&amp;2</sub>	Gray clay	7.10	-22.19	43.3	18.2	1.24	14	12.5	_
6	Green silty clay	3.5	-25.69	24.4	20.2	0.70	43	15.5	_
$\overline{\mathcal{O}}_1$	Silty sand	12.30	-37.99	35.1	20.3	1.04	0	33.0	40
$\overline{\mathcal{O}}_2$	Silty fine sand	22.86	-60.85	28.2	19.8	0.78	0	37.0	50
9	Silty fine sand	_	_	25.2	_	0.71	0	35.5	50

Table 1. Physical and mechanical properties of the soils at the restoration project site.

rescue of the tunnel including slurry, polyurethane, freezing agents, rails, and reinforced concrete pipes. Since the depth of excavations reached the confined water aquifer at layer 7 (see Figure 4), dewatering was therefore required. As the pumping rate of a single well from dewatering tests was found to be about  $90 \text{ m}^3/\text{h}$ , the loss of groundwater was expected to high during pumping. Special measures were taken to minimize ground settlements and hence to protect the surrounding buildings. Several important construction processes of this restoration project are described below.

#### 3.1 Treatment of underground materials

The restoration work covered the main areas affected by the tunnel collapse. Within the collapsed areas, there were materials from building foundations and abandoned underground pipelines, emergency rescue materials, construction waste, sand backfill, previously abandoned buildings (e.g. the Wenmiao Pumping Station), and the septic tanks of the Linjiang Garden building. In the deep strata, there were two refrigeration units, tracks, sleepers, tunnel reinforced steel supports, a large number of water pipes and the collapsed reinforced concrete tunnel debris.

Materials at 10–15 m depth were excavated and removed using a heavy plant. The materials further below were removed from excavation pits. Cutting equipment was used to remove any obstructions, which were taken away with the earthworks. When deep underground obstructions affected the construction of the diaphragm wall, a boring machine was used to clear the obstructions (see Figures 6 and 7). The diaphragm wall was then constructed after backfilling.

#### 3.2 Construction of the diaphragm wall

To retain the deep excavation pits, which varied from 38 m to 41 m, a record depth of 65 m and 1.2 m thick concrete diaphragm wall in Shanghai was required. The construction of such a deep wall was expected to encounter many difficulties. The available equipment

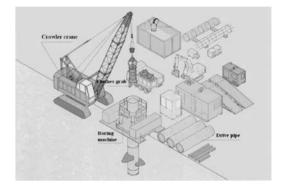


Figure 6. Cutting of the former tunnel by boring machine.

Cut the former tunnel by the boring machine

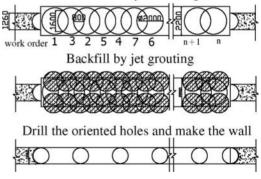


Figure 7. Barrier removal at the position of the diaphragm wall.

and technology had to be improved for the construction. As shown in Figure 8, the "two-drill-one grab" method was used. The method involves drilling two oriented holes first with a rotary drilling machine and then grabbing the soil between the holes effectively. Using this method, the verticality of the holes was better than 1/300 and the slurry trench excavation in

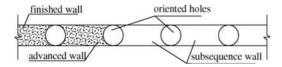


Figure 8. The two-drill-one grab method for the construction of the diaphragm wall.

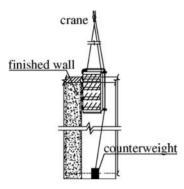


Figure 9. Brushing work for the diaphragm wall.

the strata with obstructions could be carried out much faster.

To achieve better efficiency in cleaning the soil at the previously installed diaphragm wall panel, a counterweight was placed at the bottom of the diaphragm wall trench (see Figure 9). This generated horizontal forces that made the brushing machine cling to the joints by its directional bearings. In addition, the method enhanced the waterproofing performance of the wall.

#### 3.3 Cofferdam and steel platform

The cofferdam and the steel platform in the Huangpu River were important temporary structures, which contributed to the success of the restoration work in the river (Figure 10). The foundation of the platform was supported by 0.8 m diameter of steel pipes with a wall thickness of 12 mm and 1.2 m in diameter cast-in-place piles. The superstructure consisted of H700 steel pile cap beams, H700 steel stringers, 18# I beam bridge plan distribution beams, 10 mm bridge deck steel, and  $\Phi$ 45 steel pipe grates. The platform was +3.5 m above ground and its width was 8–9 m.

#### 3.4 Strengthening of the ground by jet grouting

In order to reduce the deflection of the diaphragm wall during excavation and construction of the concrete propping slabs, rotary jet grouting (see Figure 11) was carried out below each level of slab, starting from the fourth propping level and around the excavation.

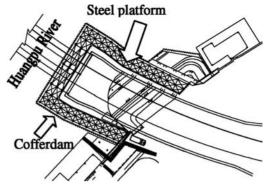


Figure 10. The cofferdam and steel platform in the Huangpu River.

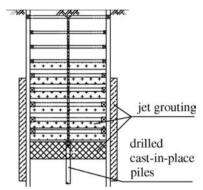


Figure 11. Jet grouting construction.

The grout formed a frame-like earth beam to enhance the shear strength of the soil in the passive zone. Jet grouting was also carried out at the bottom of the excavation. The grouting was combined with dewatering to protect the excavation from any adverse effects such as inflow of water or sand and base heave. Two jet-grouted piles were installed at each joint between diaphragm wall panels to minimize the ingress of ground water into the excavation (see Fig. 7).

#### 3.5 Dewatering and monitoring

The excavation depth of the two open cut pits was more than 38 m, which reached the aquifer of layer 7. Dewatering became one of the most vital activities on site. Since the dewatering was at great depth, ground settlement had to be strictly controlled to protect the neighboring buildings such as the Linjiang Garden and the ramp of the Nanpu Bridge from any damage. Based on an optimized design using results from in-situ dewatering tests, 56 wells of 61 m deep were sunk and recharge wells were installed at appropriate positions outside the pits. In addition to monitoring the ground settlements, water level observation

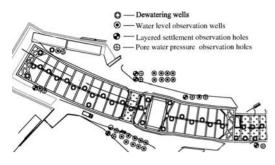


Figure 12. Dewatering monitoring points of the eastern pit.

wells, sub-surface settlement monitoring points, and pore pressure observation holes were installed (see Figure 12).

#### 3.6 Three excavation pits

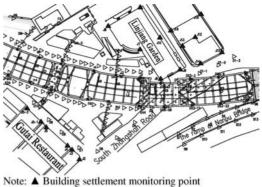
The excavation was 41.2 m deep at the two end sections and 38 m at the middle section (see Figure 4). The pit was 22.3 m wide at the middle section and 23.7 m wide at the two end sections. Due to constraints imposed by the presence of the deep obstructions, a partition wall for dividing the entire excavation, which was 174.1 m long in the eastern pit, and 62.5 m in the western pit, could not be built. Because it was a very long and narrow excavation, the stability of the underground supporting system was very important. A three-dimensional structural supporting frame with high rigidity, formed by upright column piles, reinforced concrete supports and purlins, was used to provide the required stiffness to control the lateral deformations.

#### 4 INTERPRETATION OF MONITORING DATA

Excavation for the eastern pit began on 1 March 2006 whereas that for the western and middle pits on 6 October 2006. The lateral deflections of the diaphragm wall, settlement of the surrounding ground and buildings, and bottom heave at each stage of the excavation were measured at the monitoring points installed before excavation (see Figure 13). Due to the page limit, only selected data are presented.

#### 4.1 Lateral deformations of diaphragm wall

Thirty-nine inclinometers were placed in the diaphragm wall to measure its lateral deflections during excavation. A significant increase in the measured deflection of the diaphragm wall was used as a warning signal. Figure 14 shows the measured wall profiles by the inclinometer (I12) at some key stages of excavation in the middle zone of the eastern pit.



△ Ground settlement monitoring point
 △ Bottom heave monitoring point

Inclinometer

Figure 13. Plan showing the instrumentation installed on site.

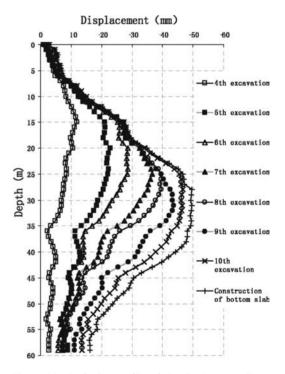


Figure 14. Deflecion profile of the diaphragm wall (at inclinometer I12) at each stage of excavation at the eastern pit.

As shown in Figure 14, the measured deflection profiles of the wall are typical for a multi-propped excavation in Shanghai (Liu et al. 2005; Wang et al. 2005). As the excavation went deeper, the magnitude of inward wall deflection increased as expected. The largest deflection of the wall was 49.6 mm recorded at a depth of 28 m. Figure 15 compares the measured

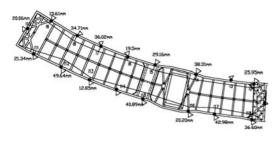


Figure 15. Comparisons of the largest wall deflections measured by all inclinometers.

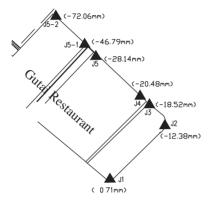


Figure 16. The largest settlement values measured at all the monitoring points installed at Gutai Restaurant.

maximum deflections at all inclinometers. The measured maximum deflections ranged from 20 mm to 50 mm, which are fairly small for a 40 m deep excavation as compared with other shallower excavations in soft clays in Shanghai (Wang et al. 2005). This was because of the large rigidity of the retaining system provided and the early installation of props prior to each stage of excavation.

#### 4.2 Settlement monitoring

The adjacent buildings and structures around the excavation such as the Gutai Restaurant, Linjiang Garden and the ramp of the Nanpu Bridge (see Figure 3) required major protective measures and their settlement had to be monitored. Figure 16 shows the largest settlement values measured at the Gutai restaurant. It can be seen that largest recorded settlement of the restaurant was 72.1 mm at the corner J5-2. This was because dewatering was carried out near this corner region. Figure 17 shows the measured settlement at J5-2 versus time. As expected, the measured settlement increased as the excavation progressed. However, upward movement (heave) was recorded after the 9th stage of excavation and the rate of upward movement seemed to accelerate after the construction of



Figure 17. The measured settlement at J5-2 of the Gutai Restaurant versus time.

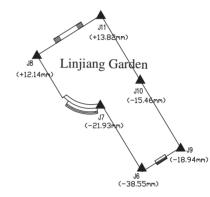


Figure 18. The largest settlement values measured at Linjiang Garden.

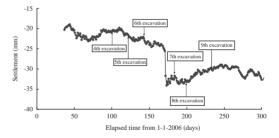


Figure 19. The measured settlement at J6 of the Linjiang Garden versus time.

the bottom slab. This was probably of the increase in pore water pressure and hence a reduction in effective stress due to the recharging of groundwater after dewatering.

Figure 18 shows the measured maximum settlement values at the Linjiang Garden. As expected, the measured maximum settlement of 38.9 mm was at the corner closet to the excavation (i.e. at J6). Figure 19 shows the measured settlement at J5-2 versus time. Similar to that shown in Figure 17, soil swelling was recorded by J6 after the 8th stage of excavation but the rate was smaller than that recorded by J5-2.

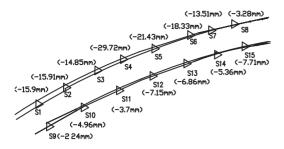


Figure 20. The largest settlement values measured at the ramp.

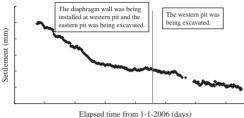


Figure 21. The measured settlement at the middle of the ramp (S4) against time.

Figures 20 and 21 show the measured maximum settlement at all monitoring points at the ramp of the Nanpu Bridge and the settlement history of S4, respectively. The measured maximum settlement ranged from 2.2 mm at S9 to 29.7 mm at S4, which was located at the middle of the ramp. As expected, the measured settlement along the ramp closer to the west excavation pit was much larger than the values recorded away from the pit. However, no soil swelling was recorded at S4. This was probably because the duration of monitoring at this point was not long enough after the completion of the western pit.

#### 5 CONCLUSIONS

This paper provides a case history which illustrates that a complex multi-propped deep excavation in saturated soft soils can be effectively engineered by proper design and construction. The maximum lateral deflection (49.6 mm) of the 1.2 m thick diaphragm wall is relatively small for a 40 m deep excavation in soft clays in Shanghai.

#### ACKNOWLEDEMENTS

The authors would like to thank many colleagues who have contributed to the construction and field monitoring of the project and to acknowledge the earmarked research grant 618006 provided by the Research Grants Council of the Hong Kong Special Administrative Region.

#### REFERENCES

- Liu, G.B., Ng, C.W.W. & Wang, Z.W. 2005. Observed performance of a deep multi-strutted excavation in Shanghai soft clays. Journal of Geotechnical and Geoenvironmental Engineering, ASCE 131(8): 1004-1013.
- Wang, Z.W., Ng, C.W.W. & Liu, G.B. 2005. Characteristics of wall deflections and ground surface settlements in Shanghai. Canadian Geotechnical Journal 42(5): 1243-1254.
- Zhao, X.H. & Yang, G.X. 2004. Practice and theory for specially big and deep excavation engineering. China Communication Press: 152-156.

## Excavation entirely on subway tunnels in the central area of the People's Square

Y.B. Mei

Shanghai No. 7 Construction Co. Ltd., P.R. China

X.H. Jiang, Y.M. Zhu & H.C. Qiao Shanghai No. 1 Construction Co. Ltd., P.R. China

ABSTRACT: The Open and Go-down square is located at the center of the city where the constructional surroundings are very complex. It's a great challenge to the excavation because metro line 1 and 2 tunnels just underlie the pit forming a shape of double crossing "#". The detailed excavation process and many technical measures are stated in the paper which may be referred by coming similar projects.

#### 1 INTRODUCTION

With the rapid development of the state economy and acceleration of urbanization, many domestic big cities are facing problems such as land limitation, population expansion, traffic jam, environment pollution et al. which restrict the continuable development of the cities. To exploit the underground space can find new space for the continuable development of the city, and to develop the rail traffic is an effective way for solving the problem of traffic jam.

With the large-scale exploitation of the underground space and formation of the city rail traffic network, more and more deep excavations will be very close to the existing rail traffic facilities, including 1) sharing the same retaining wall with existing subway stations, 2) excavation at the side of running subway tunnels, 3) excavation above running subway tunnels, 4) excavation between the columns of the elevated light-rail, and all these cases are challenges to the new excavation engineering.

#### 2 GENERAL CONDITION OF THE CASE PROJECT

The Open and Go-down square is located at the crossing point of West Nanjing Road and Middle Xizang Road, neighboring New World Mansion and World Trade Building, and it's one part of the key project – the pivotal rail traffic project of the People's Square. The area of the excavation is about 3300 square meter with depth about 4 meter. The tunnels of running metro line 1 and line 2 underlie the pit forming a shape of double crossing "#", and the minimum distance from the bottom of the pit to the top of existing tunnels of metro line 1 is only 3.3 meter. The features of this project are mainly as follows:

- 1. The site is at the center area of the city, so the surrounding traffic is very busy, and the organization of constructional vehicles is difficult.
- 2. Environment protection is very strict for the control of the noise, vibration, dust, and wastewater caused by construction activities.
- Time limitation is critical because many construction activities can not be carried out before the underground rail traffic stopped running at midnight.
- 4. For the safety of the running metro lines, the deformation of the underlying subway tunnels should be controlled within certain scope, which makes it very difficult for the construction.
- The deformation of the nearby roads and underground pipelines should be strictly controlled.
- 6 Many concrete obstructions underlie and should be demolished first.

#### 3 GEOLOGICAL CONDITION

As well-known, Shanghai is one of the classical areas that have deep soft soil which has poor mechanical characteristics such as high compression, large deformation and obvious rheology. Therefore, the soft soil is easily disturbed and excavations shall display sharp "Time and Space Effect".

According to the geotechnical report, within the scope that excavation affects, the soil layers are mainly

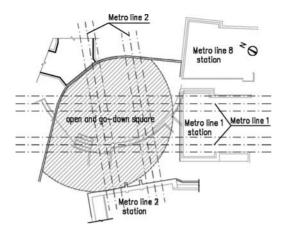


Figure 1. Location of Open and Go-down square.

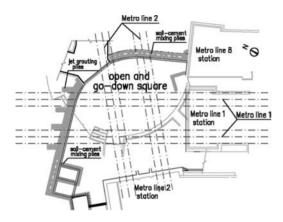


Figure 2. Plan of retaining structures.

Table 1. Soil layers.

Soil layer	Description	Thickness m
① <sub>1</sub>	Fill	1.2
① <sub>2</sub>	Fill	0.6
2	Silty clay	1.1
3	Muddy and silty clay	3.2
4	Muddy clay	7.4
51	Clay	8.5
53	Silty clay	10.0
54	Silty clay	1.9
72	Fine sand	12.3

composed by muddy silty clay and muddy clay with thin layer of fine sand between them. Because of some certain geological reasons, the soil layers ()) and  $\mathcal{O}_1$  are absent. The soil layers from top to down are listed in table 1.

The groundwater belongs to phreatic water type and is supplied mainly by rainfall. The water table varies from 0.5 m to 1.2 m below the ground surface.

#### 4 RETAINING SYSTEM AND REQUIREMENT OF DEFORMATION CONTROL

The area of Open and Go-down square is about 3300 square meter with length of 66 m, 55 m in longitudinal and latitudinal direction respectively. The slab of Open and Go-down square ramps from ground surface to 3.8 m below in west-east direction.

For the excavation is not deep, a dam formed by soilcement mixing piles is adopted as temporary retaining structure which has a perimeter of 137 m, width of 3.2 m. The length of soil-cement mixing piles varies from 6 m to 11 m, and its strength should be no less than 1.2 MPa. To reduce the impact on the subway tunnels, high pressure jet grouting piles with same parameters are used to substitute for soil-cement piles above and between metro line 1 tunnels. The jet grouting piles have diameter of 800 mm, overlapped by 150 mm.

The life of subway facilities is 100 years for its essentiality, and its safety during excavation is the key point of this project. The deformation requirements of station structures and tunnels are regulated as below by metro-managing unit.

- 1. Differential deformation of subway rails in transversal direction should be less than 2 mm,
- 2. Differential deformation of subway rails in longitudinal direction should be less than 2 mm per 10 m,
- 3. The radius of deformation curvature of subway structures should be greater than 15000 m,
- 4. The relative deformation curvature of subway structures should be less than 1/2500,
- 5. The final absolute settlement and displacement of subway structures should be less than 10 mm,
- 6. The rate of structures' deformation caused by excavation should be less than 0.5 mm per day,
- 7. The width of newly generated crack should be less than 0.2 mm,
- 8. The final accumulated settlement of station and tunnels should be less than  $\pm 10$  mm,
- 9. The final accumulated displacement of station retaining walls should be less than  $\pm 2.5$  mm,
- 10. The accumulated settlement of structures in longitudinal direction should be less than  $\pm 4$  mm per 10 m,
- 11. The distance change between subway rails should be limited within the scope of -2 mm to +6 mm.

For the particularity of this project, although the excavation depth is not large, the pit engineering is still designated to the highest grade, so the control of

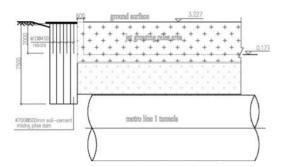


Figure 3. Section of retaining structure.

deformation indices are rather strict than before which are listed below,

- 1 The accumulated horizontal displacement of temporary retaining structure should be less than 0.1%H (H is excavation depth),
- 2 The settlement of ground nearby the pit should be less than 0.14% H.

#### 5 TECHNICAL MEASURES OF EXCAVATION

#### 5.1 Treatment of soft soil

Because of its poor mechanical characteristics, Shanghai soft soil makes it rather more difficult in underground engineering than in other areas. For the soft soil is easily disturbed and easily deforms, usually certain ways are adopted to improve its poor mechanical characteristics. To reduce the lateral deformation of retaining structure during excavation, jet grouting piles with diameter of 1200 mm at spacing of 800 mm are used for soil treatment. Nearly the soft soil in the pit are all treated from the ground surface to 3 m below the bottom, and the total area reaches 2880 m<sup>2</sup>. Over the metro 1 tunnels, the jet grouting piles extend from the ground to 500 mm to the top of tunnels. For the treated soil is hard to remove, so the requirements below and above the pit bottom are different. The strength of treated soil below and above the bottom should reach 1.2 MPa and 0.6 MPa respectively.

Usually, when the jet grouting piles is being processed, there is fairly high pressure acting on the very close soils, thus those soils would be compressed and disturbed. To avoid the high pressure acting on the subway tunnels, semicircular other than circular jet grouting piles are used adjoining the metro line 1 tunnels (see fig. 4). The semicircular piles are formed by jet grouting only at opposite side of tunnels which is called directional jet grouting technology, and this technology is developed during the past years in Shanghai.

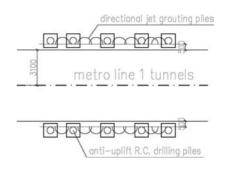


Figure 4. Plan of directional jet grouting piles.

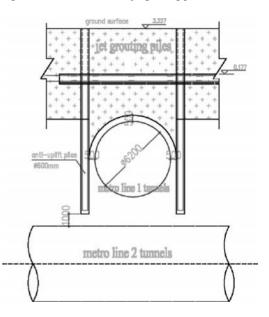


Figure 5. Special anti-uplift structure.

#### 5.2 Special anti-uplift structure

The existing running subway tunnels are in balance by the soil below and over them. If part of the soil over them is removed, then the balance state will be broken. and there seems to have a force dragging the tunnels upward, so the tunnels may uplift. To prevent the tunnels from uplifting, new balance must be established. In this project, a new kind of anti-uplift structure is invented which is indicted in figure 5. At each side of tunnel, reinforced concrete (abbreviated as R.C. hereafter) bored piles with diameter of 600 mm are set up close to the tunnel by 500 mm. The end of drilling pile is only 1 m from the underlying metro line 2 tunnels at crossing point. When a small area of soil over the tunnel is removed, the R.C. slab in same area will be constructed quickly. At the same time, rebars of bored piles are anchored into slab and welded together with its rebars. Thus, the uplift capacity of bored piles can

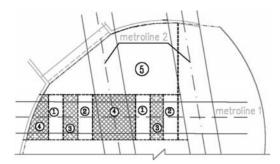


Figure 6. Excavation step 1.

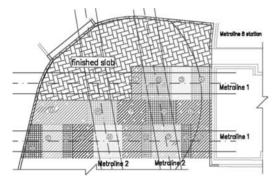


Figure 7. Excavation step 2.

balance the uplifting force acting on tunnels which equals the weight of removed soil.

#### 5.3 Dividing excavation into pieces

It is not difficult to be understood that more soil are removed from the top of the tunnels at one time, the uplifting displacement will be larger. To control the uplifting displacement within certain scope, the pit is divided into three parts which are divided into pieces either, so the procedure of excavation is fairly complex. The whole excavation includes tree steps corresponding to the tree parts. The numbers in figure 6 indicate the sequence of excavation in step 1. Excavation over the metro 1 tunnels is relatively shallower which is arranged to be carried out first. Usually, the width of each excavation piece is 3 m which is determined by experience. After the soil is excavated to the bottom, the bedding cushion layer and R.C. slab will be constructed on time. The main rebars in the slab of different pieces would be connected by mechanical connector pre-embedded in the concrete. Figure 7 and figure 8 demonstrate excavation in step 2 and 3, and the numbers do not indicate the excavation sequence.

#### 5.4 Loading on the finished slab

Another measure is taken to assure the safety of subway tunnels during excavation. That is, one piece of

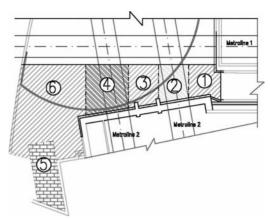


Figure 8. Excavation step 3.



Figure 9. Loading on slab.

slab will be loaded with heavy materials soon after being finished and reaching certain strength. The loading must be equivalent to the weight of soil removed from the tunnels' top.

#### 6 INFORMATION-BASED MONITORING

It is well-known that information-based monitoring is necessary during the construction in underground engineering. Due to vagueness and variability of mechanical characteristics of soft soil, it is impossible to predict the deformation of retaining structure accurately during excavation, so monitoring is the only measure that can see the state of safety. Figure 10 shows the layout of monitoring points of metro line 1 tunnels.

Figure 11 demonstrates the deformation curve of metro line 1 tunnel (upper one) during the first excavation step. The accumulated maximum uplifting displacement of underlying tunnel is 3.05 mm after the complement of excavation in step 1. It is obvious that

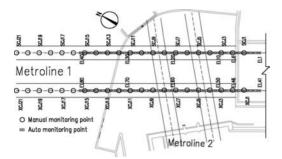


Figure 10. Monitoring points of subway tunnels.

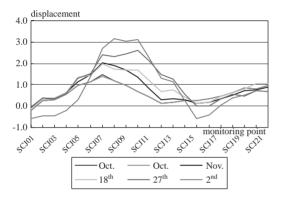


Figure 11. Uplifting displacement of subway tunnels during excavation step 1.

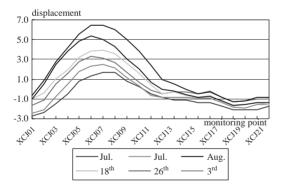


Figure 12. Uplifting displacement of subway tunnels during excavation step 2.

the deformation is fairly small and the safety of running tunnel is guaranteed. Figure 12 demonstrates the deformation curve of metro line 1 tunnel (lower one) during the second excavation step. The accumulated maximum uplifting displacement of underlying tunnel is 6.47 mm in step 2 which includes displacement generated in step 1. Generally speaking, the accumulated deformation is fairly small during the construction of underground engineering.

#### 7 CONCLUSION

With the rapid development of the urban rail traffic, more and more excavations are to be restricted by existing underground subway facilities such as excavation under, above or between subway tunnels et al. One case excavation completely on the running subway tunnels is introduced in detail in this paper. The technical measures employed during the process of excavation and the successful experience can be referred by similar projects in future.

#### ACKNOWLEDGMENTS

This research was supported by Science and Technology Commission of Shanghai Municipality (No. 062012002 and 07QB14019)

#### REFERENCES

- Garske E., Kauer H., & Von Soos P. 1989. Excavation lining and foundation for the new Kreissparkasse building above existing subway tunnels in Munich, *Bauingenieur* (*German*), 64(11): 505–511.
- Lo K.Y & Ramsay J.A. 1991. Effect of construction on existing subway tunnels, *Tunnelling and Underground Space Technology* (6)1: 287–297.
- Eisenstein Z. Dan, Martin Geoffrey R., & Parker Harvey. 1997. Challenges of tunneling for the Los Angeles subway, ASCE Construction Congress Proceedings, Managing Engineered Construction in Expanding Global Markets: 281–289.
- Ookado, & Nobuyuki. 1998. Large-scale excavation in Tokyo for the construction of the Namboku subway line, *Geotechnical Special Publication* 86:124–143.
- Zhu Z. F., Tao X. M., & Xie X. S. 2006. The influence and control of deep excavation on deformation of operating subway tunnels, *Chinese Journal of Underground Space* and Engineering (1):128–131.
- Zhang J. L., Liu G. B., & Liu H. 2006. Study on construction technology of excavation on subway tunnels, *Construction Technology* (4): 85–90.

# The benefits of hybrid ground treatment in significantly reducing wall movement: A Singapore case history

N.H. Osborne, C.C. Ng & C.K. Cheah Land Transport Authority, Singapore

ABSTRACT: The first major use of hybrid ground treatment, Jet Mechanical Mixing (JMM), in Singapore, was in the construction of the new Nicoll Highway Station (NCH) for the Circle Line Project (CCLP). This system played a major role in the success of the project. The retaining wall design was a 1.5 m thick diaphragm wall to provide support of the ground during the station construction and also as the permanent walls of the station. The proposed construction scheme is that of top-down with the use of a 7 m thick JMM below the base slab level, which comprises a large diameter deep soil mixing method comprising a central core combined with a jet-grouted outer circumference to form a large diameter of improved soil mass. This paper aims to demonstrate the effectiveness of the excavation support system and benefits of JMM in reducing wall movement of deep excavation in soft ground with 35 m thick of Marine Clay.

#### 1 INTRODUCTION

#### 1.1 Background

The construction works for the original Nicoll Highway Station (NCH) on the Circle Line Project (CCLP) was halted when a collapse of the cut and cover tunnels leading to the station occurred in April 2004. Following the collapse, several options were studied for the recommencement of the works. The option to realign part of the project to avoid the collapsed site was eventually adopted. As a consequence of this realignment, NCH was relocated approximately 100 m to the south, as shown in Figure 1, with the station design and excavation restarting afresh.

### Provides Necel Highway Station Collapse Area New Miccel Highway Station Revined Turnel Alignment

Figure 1. Tunnel Alignment Drawing.

#### 1.2 Ground condition

At NCH, the ground consists of man-made fill, fluvial sands, fluvial clay and the Marine Clay of the Kallang formation, underlain by the Old Alluvium, as shown in Figure 2. The thickness of the fill is typically 3 to 6 meters. Underlying the fill is a layer of fluvial sand. Beneath this, it is the very soft to soft Marine Clay. The thickness of the sand layer is 3 to 7 m. The depth of the Marine Clay varies from 30 m to 40 m below ground level.

Locally, the Marine Clay is separated by a layer of laterally discontinuous fluvial deposits. The fluvial sands found at NCH are typically described as loose to medium dense gray sands or silty sands. The properties of the fluvial sands are described by Chu, et al. (2000). The properties of the Singapore Marine Clay and problems associated with it from a tunnelling and deep excavations perspective have been well established in Singapore; see Tan (1972), Shirlaw & Copsey (1987), Chang (1991) and Tanaka et al (2001), and generally relate to its softness. The Marine Clay is normally consolidated or slightly over-consolidated, with a undrained shear strength (Cu) starting at about 20 kPa and increasing slowly with depth. The compression index is typically in the range of 0.6 to 1.0. The permeability is low and is in the order of  $10^{-9}$  to  $10^{-10}$  m/s. The Old Alluvium is typically described as sandy silt or clayey silt. At depth the material is generally found to have some cementation. However much of the cementation has been lost due to weathering at shallow depth. The permeability of the Old Alluvium

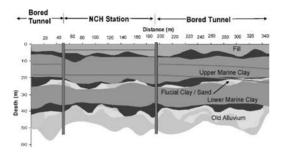


Figure 2. Ground conditions at the new NCH Station.

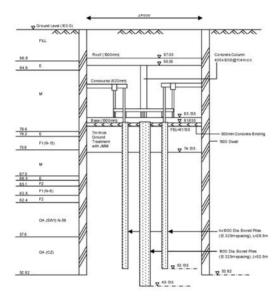


Figure 3. Typical cross section of NCH Station.

depends on weathering and grain size distribution. It typically ranges between  $10^{-6}$  to  $10^{-9}$  m/s.

#### 1.3 Section details

The new NCH was designed as a top down structure with a 1.5 m thick permanent diaphragm wall anchored into the Old Alluvium strata. Toe level is at 60 m below ground level. This also reduces the need for temporary struts as the permanent concrete roof and concourse slabs are constructed during excavation. The excavation depth is 20 m below ground level and the width of excavation is 24 m. The cross section of the station box is shown in Figure 3.

#### 1.4 Hybrid type of ground treatment

Ground treatment underneath the base of the station is often used to limit the wall deflection, act as a working platform and prevent uplifting of these soft clayey



Figure 4(a). JMM Machine.

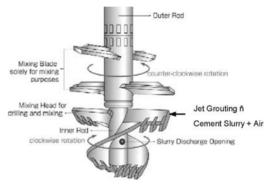


Figure 4(b). Schematic diagram of the drilling rod showing the mixing arm of the JMM machine.

soils. The application of ground treatment such as jet grout piles (JGP) for deep excavations in Marine Clay in Singapore has been presented by Page et al. (2006). For this project, the ground treatment option was Jet Mechanical Mixing (JMM), a hybrid of jet grouting and deep soil mixing. A proprietary name, RASJET is given to it by the specialist contractor from Japan, Raito Kogyo. This was the first time such a system had been used in large scale in Singapore.

JMM is a combination of soil mixing and jet grouting that produces overlapping columns with an internal column of mixed soil by the auger and an external column created by a slurry jet into the in-situ soil. The process of forming the columns is similar to the method of forming JGP columns with the addition of dual and counter rotation mixing blades on the drill rod to ensure intensive soil mixing. Figures 4(a) & (b) show the JMM machines, the drilling rod and the mixing arm of the machine. The rod/auger had a large diameter of 457 mm as compared to the traditional JGP rod of 200 mm. The high stiffness of the drill

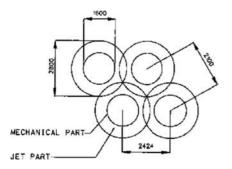


Figure 5. Typical layout of the JMM columns.



Figure 6. Columns created during withdrawal.

rod contributes to a more accurate drilling verticality. Combined with the rod were the mixing blades which created an inner mechanical soil mixing column of 1.6 m diameter. A jet grout nozzle on the mixing blade introduces cement slurry mix with pressurized air into the soil and adds a further 0.6 m of jet grouting around the soil mixing column, creating a 2.8 m column within the ground. These columns are then designed with appropriate overlap to provide a full coverage of the treated areas. Figure 5 shows the layout of the mechanical mixing part and jet grouting part of the JMM column.

There are numerous advantages to this system (Page et al. 2006 & Ueda et al. 2007), principally the benefits of mixing and grouting are experienced. From the mechanical soil mixing, a known treated area is assured and from the jet grouting, a sizeable overlap and penetration into any shadow areas close to the retaining system is achieved.

To install a JMM column, the auger is first drilled to the base level of the JMM column with water injection, and withdrawn to the top level of the JMM column with mechanical mixing without any injection. It then descends with slurry injection and mechanical mixing to form the internal soil mixing column up to base level. After which, it ascends with jetting to form the external jet grouting perimeter. The whole process is automated and monitored real time by data loggers to ensure that a high level of quality control. It should be noted that a further benefit is gained during the withdrawal the auger after completing the JMM slab. As the auger is withdrawn, lower quantities of cement are added and the ground is mixed. This creates a 1.6 m diameter treated column all the way to the surface, as shown in Figure 6. Consequently the strength of the soil above the treated ground is significantly enhanced.

#### 2 INITIAL DESIGN OF NCH STATION TEMPORARY WORKS

#### 2.1 Initial design assumption

The initial design of NCH was carried out using moderately conservative parameters for the original ground and JMM. A conservative approach was adopted for the initial design and therefore no additional strength attributed to the soil cement mix above the JMM layer was considered. The Undrained Shear Strength (Cu) and Young's Modulus (E) of the JMM layer was assumed to be 300 kPa and 90 MPa respectively, based upon traditional design parameters used for jet grouting in Singapore. Table 1 shows the parameters of the ground and JMM used in the initial design.

#### 2.2 Geotechnical analysis

The geotechnical analysis of the excavation sequence was done using a two-dimensional analysis of deformation and stability with PLAXIS (Version 8), a geotechnical finite element program. The non-linear and stress-dependent stress-strain properties of the soils are modeled as elastic perfectly plastic using the Mohr Coulomb model. The undrained behaviour of clavs and cohesive materials is simulated in PLAXIS using Mohr-Coulomb soil model in terms of effective stresses with undrained strength parameters (commonly known as Method B in Singapore). This was done by specifying the effective Young's Modulus (E')and Poisson ratio ( $\nu'$ ) with Cu under undrained setting. In this method of analysis, the Cu of the clays and cohesive materials were capped at the input values given by the user. The input soil parameters were based on the moderately conservative values shown in Table 1.

Figure 7 shows the cross section of temporary works for the excavation. The 1 m and 1.8 m diameter bored piles were modeled as a fixed-end anchor with horizontal spacing of 12.325 m centre to centre to avoid possible ill effect of soil-beam interaction in a 2-D plane strain analysis. The equivalent length of the anchor is taken as half the pile penetration length, which is 16 m. Temporary steel plunge-in column is modeled using the node-to-node anchor to simulate the behavior of an axially loaded member to avoid possible ill effect of soil-beam interaction. Though large

Stratum	Standard penetration test SPT – N	Bulk density, γ (kN/m <sup>3</sup> )	Undrained shear strength, Cu (kN/m <sup>2</sup> )	Drained shear strain, C' (kN/m <sup>2</sup> )	Friction angle, ′(°)	Elastic modulus, E (kN/m <sup>2</sup> )
Fill	$6\pm4$	19.0	25	0	30	10000
М	0~1	16.0	Design: 15 (0 to 10 m) Worst Credible: 13 (0 to 10) Design: $15 + 1.2$ ( $z - 10$ ) Worst Credible: $13 + 1.1$ ( $z - 10$ )	0	22	Design: 300 Cu Worst Credible: 200 Cu
F2	$11\pm5$	19.0	20 (0  to  10  m) 20 + 1.14 (z - 10)	0	22	Design: 300 Cu Worst Credible: 200 Cu
F1	$11 \pm 7$	20.0	0	0	30	1500 N
E	0~4	15.0	15 (0  to  10  m) 15 + 1.2 (z - 10)	0	18	Design: 300 Cu Worst Credible: 200 Cu
OA(W) N < 30	25	20.0	,	Sandy: 0 Clayey: 10	Sandy: 32 Clayey: 25	Design: 2000 N Worst Credible: 1000 N
OA(SW-2) 30 < N < 50	$35\pm 6$	20.0	Design: 5 N (max. 500 kPa) Worst Credible: 3 N	Sandy: 5 Clayey: 20	Sandy: 32 Clayey: 25	
OA(SW-1) 30 < N < 100	$70\pm13$	20.0		10	30	
OA(CZ) N > 100	>100	20.0		25	32	
JMM		16	300			90000

Table 1. Soil parameters and strength of treated soil used in the initial design.

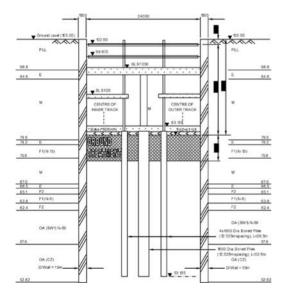


Figure 7. Cross section of temporary works design.

voids are present at concourse level at reduced level of 90.92 m, the diaphragm wall panels are supported by the concourse slab abutting the diaphragm wall acting in-plane as a waler beam. This waler beam support is modeled as a fixed-end anchor in lateral direction.

As shown in Figure 7, there were 2 layers of temporary steel struts above the permanent roof slab during the excavation. The first and second layers of struts (S1 & S2) were H-beam of size  $414 \times 405 \times 232$  at spacing of 5.7 m. The first and second layers of struts were installed at reduced levels of 102 m (1 m below the ground level) and 99.87 m respectively. The struts were modeled in the program as fixed end anchors with effective length of 12 m, which is half of the strut length between the two diaphragm walls support. The pre-stressing on the struts was typically 50% of the design strut load.

On comparing the predicted finite element model and the actual maximum wall deflections measured by in place inclinometers, a very significant difference of 60 mm of movement is observed. A maximum measured deflection of 25 mm compared to a predicted 85 mm wall deflection is observed, as shown in Figure 8. This is a very notable discrepancy. This difference rapidly became apparent as the excavation progressed. Although it was evident that much of this disparity could be attributed to the JMM, as suggested by the strength and stiffness results from the site testing of the JMM, back analysis was undertaken to quantify these differences. Although a change in deflection from 0 mm to 20 mm was observed over 5 m length of the diaphragm wall, no visible crack was observed. The deflection of diaphragm wall could be less. This could be due to an inherent problem of the inclinometer installed in steel pipes in the diaphragm wall with cement bentonite backfill.

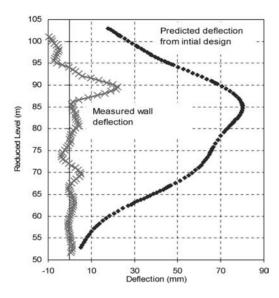


Figure 8. Predicted wall deflection of the original design and the measured all deflection.

#### 3 ACTUAL STRENGTH PARAMETERS FOR JMM AND SOIL CEMENT MIX ABOVE

#### 3.1 JMM strength parameters

Post installation and prior to the commencement of excavation, an extensive quality check on the strength parameters of the JMM layer as well as the soil cement mix above was carried out. Figure 9 shows the summary of the analysis of the test results from the 7 m thick layer directly beneath the base slab. It shows the comparison between the average strength of the JMM layer and the parameters assumed in the original design. Also included is a strength factored by a mass correction factor to account for any variation caused during the construction process, for use later in further analysis (refer to Section 4.1 for details of correction factor). The average strength, Cu, of the JMM layer is about 1845 kPa, with very consistent strengths achieved in the samples tested, ranging from a lowest of 1150 kPa to the highest of 2370 kPa. The average strength is more than five times higher than that originally assumed value of 300 kPa in the design.

#### 3.2 Soil cement mix strength parameters

Similar findings are experienced on analysing the results from the stiffness testing, Figure 10 shows the comparison of the stiffness of JMM from test results, the original design value and the factored value taking into account the mass correction factor. The average stiffness of the JMM from test results is 572 MPa,

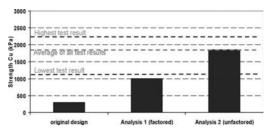


Figure 9. Comparison of JMM strength used for initial design and actual values from test data.

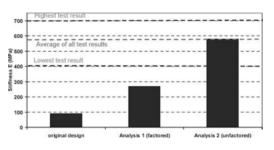


Figure 10. Comparison of JMM stiffness used for initial design and actual values from test data.

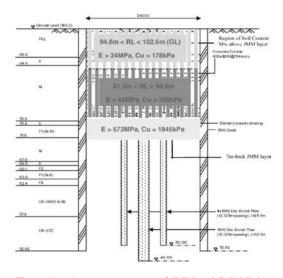


Figure 11. Average parameters of JMM and Soil Mixing above.

with a lower bound of 400 MPa and an upper bound of 700 MPa. Again the tested average stiffness is significantly higher than the 90 MPa assumed in the original design. The various strength of the treated ground with JMM and soil cement mix above the JMM is summarized in Figure 11.

#### 4 BACK ANALYSIS

#### 4.1 Accounting for better strength parameters

As it was evident in the early stages of the excavation that the retaining wall performance, in terms of movement, was appreciably better than that design prediction. The review of the JMM test results suggested that they were the most plausible reason for the reduced movement. Back analysis of the performance was undertaken for two main objectives: (i) to model the performance of the cofferdam with the JMM modeled more accurately; (ii) to justify the omission of the third temporary strut, which was originally proposed between the concourse and base slab. In principal, the analysis was first done through modifying the initial finite element model using most probable parameters to match the actual wall deflection and strut load of the second stage of excavation. It was refined thereafter as the excavation proceeds. At the time of the back analysis, the second stage of excavation was the current excavation progress.

The most probable parameters include the most probable soil parameters, JMM parameters and the improved parameters of the soil mixing columns above the JMM. The most likely parameters for 7 m thick JMM slab and the soil mixing column above the JMM slab were based on the actual core tests results multiplied by a factor (0.725), which coincides with most probable or mean mass correction factor for the JMM. The correction factor is applied to account for the potential non-uniformity of the ground treatment. This correction factor is derived from the estimated coefficient based on the square of the ratio of seismic wave velocity, obtained from cross hole seismic testing (Vsf), to the seismic wave velocity obtained from laboratory Ultrasonic test (Vsc), i.e.

## Correction Factor = (Vsf/Vsc)2

The results of the wave velocities (Vsf and Vsc) and the rational of deriving the correction factor from the wave velocities is outlined in a paper by Ueda et al. 2007. Based on these parameters, back analysis was carried out to match the actual wall deflection and strut forces, as close as possible, up to second stage excavation. Table 2 shows the most probable parameters for the JMM and soil above the JMM.

## 4.2 Interpretation of the back analysis results

The results from the various back analysis are summarized in Figure 12. The wall deflection from the back analysis (Analysis 1) using the most probable parameters at second stage excavation, with the mass correction, is able to reasonably match the reading of the in wall inclinometers at the same stage. This is supported by the fact that the reading of strut force at first level strut is quite similar to the strut force obtained from back analysis. However, the wall deflection obtained from the Analysis 2 using unfactored parameters gives a much closer match to the actual wall movements. This suggests that the quality, strength and stiffness of the JMM are consistent throughout the entire JMM slab and no correction factor is needed to be applied.

#### 5 CONCLUSIONS

This case history demonstrates that JMM, if properly installed, has major benefits in controlling the stability and movements induced by deep excavations in soft ground. The reasons can be attributed to the fact that the inner soil column is comprehensively mixed, combined with the attributes of the outer jet grouted column with sufficient overlapping. The whole process undergoes tight quality control and rigorous testing to ensure a continuous and comprehensive slab. In addition to the JMM slab, there is the major benefit of

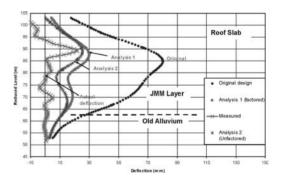


Figure 12. Wall deflection for the back analyses

Table 2. Most probable JMM parameters for back analysis.

Most probable JMM parameters (Correction Factor = 0.725)		Most probable parameters for soil cementmix above the JMM (Correction Factor = 0.725)94.6 m < RL < 102.5 m(GL)81.3 m < RL < 94.6 m				
Cu' (kPa)	$0.725 \times 1845 = 1337.6$	$0.725 \times 178 = 129$	$0.725 \times 339 = 245.8$			
E <sub>50</sub> (MPa)	$0.725 \times 572.1 = 414.8$	$0.725 \times 24 = 18$	$0.725 \times 44 = 31.9$			

the discrete soil mixing columns formed during the withdrawal of the auger.

Based on the limited usage to date it is difficult to suggest what parameters should be used for future design. However it is clear that the key is in the quality control of the process in ensuring a total and uniform treatment. With today's engineering sophistication, this can be achieved. The strength, stiffness and quality of the JMM is significantly higher than jet grouting, but the choice of actual design parameters to be used is complex. It is recommended that they should be determined on a case by case basis with local trials specific to the ground conditions, but considering the strengths and stiffness already achieved. The benefits of JMM should not be ignored and this technique will be a future benefit to the industry in controlling ground movements.

## REFERENCES

- Chang, M.F. 1991. The Stress History of Singapore marine clay. J. Geotechnical Engineering, Vol. 22, 1991.
- Chu, J., Wen, D., Kay, R.E. & Tay, T.H. 2000. Engineering Properties of fluvial sands at Race Course Road. Proc. of the International Conference on Tunnels & Underground Structures, Singapore, 2000.

- Page, R.J., Ong, J.C.W, Osborne, N. & Shirlaw, J.N. 2006. Jet Grouting for Excavations in Soft Clay – Design and Construction Issues. Proc. of International Conference on Deep Excavations, 2006.
- Shirlaw, J.N. & Copsey, J.P. 1987. Settlement over Tunnels in Singapore Marine Clay. Proc. of the 5th International Geotechnical Seminar "Case Histories in Soft Clay", NTI, Singapore, 1987.
- Tan, S.B. 1972. Foundation Problems in Singapore Marine Clay, Asian Building and Construction 1972.
- Tan, S.B., Tan, S.L. & Chin, Y.K. 1985. A Braced Sheetpile Excavation in Soft Singapore Marine Clay. Proc. of 11th Conference on Soil Mechanics and Foundation Engineering, San Francisco, 1985.
- Tanaka, H., Locat, J., Shibuya, S., Tan, T.S. and Shiwakoti, D.R. 2001. Characterization of Singapore, Bangkok and Ariake Clays, Can. Geot. J. 38:378–400.
- Ueda, Y., Furusone, T., Sato, Y. & Imamura, S. 2007. Design, Construction and Quality Control of the Ground Improvement Method Adopted for Singapore Marine Clay – Mechanical Soil Mixing with Cement Slurry Jet Grouting. Proc. of 4th Civil Engineering Conference in the Asian Region 2007.

# 3D deformation monitoring of subway tunnel

# D.W. Qiu, K.Q. Zhou & Y.H. Ding

Beijing University of Civil Engineering and Architecture, Beijing, P.R. China

# Q.H. Liang & S.L. Yang

Beijing Jiaotong University, Beijing, P.R. China

ABSTRACT: As a kind of modern vehicle, subway has shown us the advantages of safety, speediness, low power consumption, low pollution etc. It has been the main part of the urban high-capacity public traffic system. More and more subway lines appear in order to meet the need of the society. The civil engineering construction especially in Soft Ground must lead to the deformation of the adjacent subway tunnel, and it causes the severe influences to the stabilization and safety of the tunnel. This paper puts forward a kind of three-dimension deformation monitoring method. It can precisely monitor the deformation of the tunnel liner real-time which do not interrupt the subway transport, and expediently provide mechanics analysis on the deformation of tunnel construction. The paper discusses how to build the 3D mathematical model for the subway tunnel by the ground lidar surveying technology. Use the Georobot to survey the deformation of the tunnel cross section and the rail automatically. According to data processing and analysis using a model interpolation method, the 3D digital model of the deformation and displacement for the whole metro is fitted finally. Thus we can obtain the 3D image of the subway deformation in a precision, real-time, stereo and visual way.

## 1 INTRODUCTION

In order to alleviate the pressure of the ground transportation, urban subway has been programmed or built in big cities. As a kind of modern transportation facilities, subway has shown us the advantages of safety, speediness, low power consumption, low pollution etc. It has been the main part of the Beijing's high-capacity public traffic system. Along with the development of city in Beijing, more and more civil engineering construction must lead to the deformation of the adjacent subway tunnel. It causes the severe influences to the stabilization and safety of the tunnel, thus endanger the whole urban transportation system. During the construction of periphery foundation ditch engineering and tunnel engineering, how to guarantee the safety of the subway tunnel has been the severe difficulty needed to be solved.

The traditional monitoring method is to set monitoring cross sections on the deformation district of tunnel. By surveying these monitoring points with convergence rule, total station and leveling, we can monitor the deformation of the tunnel structure. Some drawbacks exist here. First, the number of the monitoring points is limited, which can not reflect the deformation tendency. Load analysis of the deformation of tunnel structure is constricted and the corresponding reinforce measure is difficult to carry out. Second, the traditional one has not used the remote monitor method, so it disturbs the transportation of the subway to some extent. Third, the dim light, narrow space and the complicate environment do disturb the safety monitoring.

A method of three-dimension safety deformation monitoring for urban subway is put forward here. It can obtain the 3D digital data of the subway tunnel deformation, which do not interrupt the subway transportation. It can not only precisely monitor the deformation of the tunnel structure and get the tendency of subway deformation, but also provide mechanics analysis on tunnel structure and the rail. This method has been applied to the safety monitoring of the Beijing subway line 1 (Babaoshan Station – Bajiao amusement park Station).

# 2 THREE-DIMENSION SAFETY MONITORING METHOD

# 2.1 Model building

Lidar technology is also called three-dimension laser scanning technology, which is a new kind of non-touch surveying method. It can obtain the array geometric

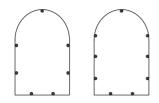


Figure 1. Reflection target setting.

image of survey object from laser point clouds, which were emitted by scan prism and the quick laser ambulator. So the three-dimension space model can be made. This technology can obtain the three-dimension coordinates of one point without reflecting prism, and the speed can reach 100,000 points/second. This technology is well suited for many applications: industrial, architectural, civil surveying, urban topography, reverse engineering, archaeology.

The detailed monitoring project is introduced here:

- 1. Within the deformation district, set the annular closed survey control network along the middle line of the up-down rail.
- 2. Put a set of cross section every 3 m following the middle line of tunnel. Set the reflection target on the arch, vertical wall, subgrade of the rail (As Fig. 1 shows), and collect the point clouds information by three-dimension lidar scanner.
- Based on the NURBS Curved face function (refer with: Eq. 1), the three-dimension model of tunnel is established by the data process such as data joint and registering.

Let the order of polynomial is  $p \times q$ , NURBS Curved face function can be written as:

$$S(u,v) = \frac{\sum_{i=0}^{m} \sum_{j=0}^{n} w_{i,j} d_{i,j} N_{i,p}(u) N_{j,p}(v)}{\sum_{i=0}^{m} \sum_{j=0}^{n} w_{i,j} N_{i,p}(u) N_{j,p}(v)}$$
(1)

where  $d_{i,j}$  ( $i = 0, 1, \dots, m$   $j = 0, 1, \dots, n$ ) represent the control vertex, and  $w_{i,j}$  denotes the weight factor of vertex, and  $N_{i,p(u)}, N_{j,q(v)}$  is gage B spline primary function.

#### 2.2 Real-time safety monitoring system

The 24 hour safety monitoring is necessary to guarantee the safety of both the tunnel structure and transportation of the subway. Considering the high density of the subway's transportation, we adopt the remote automatic monitor system to real-time monitor the tunnel structure, vertical wall, and subgrade of the rail by the Georobot.

### 2.2.1 Measurement principle of Georobot

Georobot is often called automatic electronic total station (ETS). It is a kind of intelligent electronic total station, which is able to search target automatically, recognize, trace, collimate precisely and obtain the 3D coordinates.

Target points observation of tunnel deformation adopts free stationing principle of Georobot. Reflection sheets are set on the target points. To achieve higher resolution and improve reliability of observation data, Georobot can finish redundant observation automatically under the control of on-board software. Then adjustment of observation data of different periods, 3D coordinates values in different periods will be done by post-software of computer, finally we can get 3D coordinate displacement of target points: ( $\Delta X$ ,  $\Delta Y$ ,  $\Delta Z$ ). For observation networks of free stationing, we choose indirect adjustment model to process data. Let t be necessary observation number and n be total observation number (n > t). Then adjustment model is:

$$B_{\text{poin noise node rod}}^T P B \delta x + B_{\text{poin noise nod}}^T P l = 0.$$

$$\text{taking } N = B_{\text{poin noise nod}}^T P B, U = B_{\text{poin noise nod}}^T P l .,$$

$$(2)$$

we get 
$$\underset{t \ge 1}{N} \frac{\partial x}{\partial t} + \underbrace{U}_{t \ge 1} = 0$$
, so that  $\frac{\partial x}{\partial t} = -\underbrace{N^{-1}U}_{t \ge 1}$ .

The corresponding error equation by the matrix is:

$$V_{n \times l} = B_{n \times t} \frac{\delta x_{l}}{\lambda \times l} + \frac{l}{n \times l}.$$
(3)  
where  $V_{n \times l} = [v_{1}, v_{2}, \dots, v_{n}]^{T}, \delta x_{t \times l} = [x_{1}, x_{2}, \dots, x_{t}]^{T},$   
 $l_{n \times l} = [l_{1}, l_{2}, \dots, l_{n}]^{T} = \frac{d}{n \times l} - \frac{L}{n \times l}.$ 

The above-mentioned method is that gets displacement by comparing coordinates of observation points. In the course of subway tunnel deformation analysis, we also consider lateral vector after observation value adjustment of different periods:

$$d_{ij}^{(k)} = [(x_i - x_j)^2 + (y_i - y_j)^2 + (z_i - z_j)^2]^{1/2}.$$
  
$$\Delta d_{ij}^{(k)} - \Delta d_{ij}^{(k-1)} = \Delta_{k-1,k}.$$
 (4)

where,  $x_i$ ,  $y_i$ ,  $z_i$  are adjustment value. k is observation times of tunnel deformation,  $\Delta_{k-1,k}$  is the convergence of measurement lines, they do not include error from possible displacement of base points, so they can accurately reflect tunnel deformation. By regressive analysis of these adjustment deformation values of different periods, we can conclude forecasting results of tunnel structure deformation.

#### 2.2.2 The setting of Georobot

The specially made instrument pier is put outside the right of the first rail. Georobot is forced to be fixed

on the instrument pier through the pedestal and is protected with the glass cover. 6–16 reflecting sheets are installed for each monitoring station, which is distributed in the arch, vertical wall, orbit drainage ditch, rail fastener and so on. With the monitor program, Georobot collects the coordinates automatically and transports the data to the control server via data wire.

# **3 ENGINEERING APPLICATIONS**

The thermal pipeline engineering of Babaoshan south road crosses over the structure of Beijing subway line 1 (Babaoshan Station – Bajiao amusement park Station). Engineering construction causes the subway tunnel deformation. The kilometer post of the tunnel deformation district is from K3+770 to K3+810.

The safety monitoring result is shown as follows: the accumulative deformation value of the max deformation point on the tunnel structure is +1.90 mm; the max accumulative value of the rail deformation is +1.86 mm; the max differential settlement of the rail subgrade is -0.29 mm. So such conclusion can be made that the accumulative displacements of both tunnel structure and rail caused by the engineering construction are less than 2 mm, which is within the allowed deformation range and put no influences on the subway transportation.

#### 4 CONCLUSIONS

This method of three-dimension safety monitoring has the advantages of high automation and three-dimensional measurement. It can be applied to the safety monitoring of high-rise building, side slope, deep foundation ditch engineering and so on. Of course this method is not so mature. Its theory needs to be further researched through engineering experience.

# ACKNOWLEDGEMENTS

The authors wish to acknowledge the support of the Key Technologies R&D Programme of China (No. 2006BAJ15B01) and the Beijing University of Civil Engineering and Architecture Science Research Foundation of China (No. 100701205, No. 100601905).

# REFERENCES

- Bassett, R.H., Kimmance, J.P. & Rasmussen, C. 1999. Automated electrolevel deformation monitoring system for tunnels. Proceedings of the Institution of Civil Engineers, Geotechnical Engineering: 117–125.
- Wu, Z.A. 1989. Processing Deformation Data of Engineering Construction. Beijing: Surveying Publishing House.
- Xu, W.P. 2000. Exploring on Feasibility of New Way to Monitor Tunnel Displacement. *Railway Engineering Transac*tion 66(2).
- Yu, L.F. & Duan, D.Q. 1996. Real-time Theodolite Industrial Surveying System. Beijing: Surveying Publishing House.

# Challenging urban tunnelling projects in soft soil conditions

# H. Quick, J. Michael & S. Meissner

Prof. Dipl.-Ing. H. Quick, Ingenieure und Geologen GmbH, Darmstadt, Germany

# U. Arslan

Univ.-Prof. Dr.-Ing. U. Arslan, Technische Universität Darmstadt, Germany

ABSTRACT: Different challenging tunnel projects in the downtown area of the city of Mainz in Germany are presented. These tunnels run parallel in a distance of 4 m to max. 50 m. The tunnels were built in soft soil conditions consisting of filling, clay and marl layers of the Tertiary. The paper presents the different construction techniques, the calculation methods for the two tunnels as well as the results of measurements for the New Tunnel Mainz. The experience for the construction of this tunnel and the results of the measurements were the basis for the chosen construction and calculation method for the rehabilitation of the Old Tunnel Mainz, which is currently under construction.

# 1 INTRODUCTION

The New Tunnel Mainz had been constructed in the years 1998 to 2001 directly adjacent to the existing Old Tunnel Mainz. This old tunnel built in 1884 is currently being rehabilitated, converted and enlarged during the next years. Due to the small overburden of both tunnels and sensitive structures on the ground surface challenging and unique tunnelling techniques were chosen to guarantee the stability and serviceability of the tunnels and of sensitive structures. In addition calculation methods and results from geotechnical measurements are presented in the following. The situation of the tunnels is shown in figure 1.

## 2 GROUND CONDITIONS

The geological condition is mainly characterized by the tertiary strata (Miocene) of the Mainzer Basin. The Tertiary strata sequence consists of an alternating sequence of marly clays, chalk marl, sandy silts (hydrobia silts, hydrobia oyster shells) and sands in an alternating sequence with chalkstone banks (fig. 2). The chalkstone banks are partly compact/massive to weathered. The consistency of the in-situ ground is stiff to semi-solid, turning into soft/paste-like if water intrudes. The groundwater can be found up to the level of the floor/upper edge of track; otherwise there is only local stratum water of little importance.

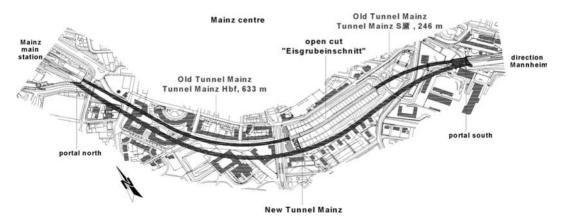


Figure 1. Plan view tunnel situation (Quick et al. 2001).

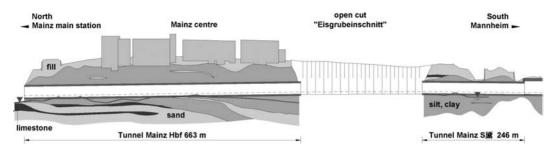


Figure 2. Geotechnical longitudinal section – Old Tunnel Mainz.

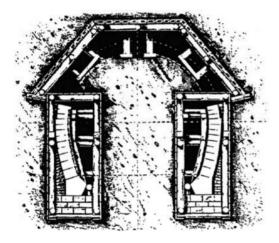


Figure 3. German core centre technique – Old Tunnel Mainz (Maidl 2004).

# 3 OLD TUNNEL MAINZ

#### 3.1 Construction

The Old Tunnel Mainz was erected in the years 1881 until 1884 as one continuous double track tunnel. The tunnel has an horseshoe shape and the lining consist of sandstone with a thickness of approx. 0.9 m. The German core center tunneling technique was used to built the Old Tunnel Mainz with an overall length of 1200 m. The German core center technique (fig. 3) uses partial drivings, mostly sidewall drivings. Due to the bad condition of the tunnel, especially the masonry and to improve the smoke venting system, a 300 m long and up to 26 m deep open cut was built in the early 30ies of the last century, which divides the tunnel nowadays into the Tunnel Mainz Central Station and the Tunnel Mainz south (fig. 1/2).

The Old Tunnel Mainz consists of the following structures:

- Tunnel Central Station: 663 m
- Open cut: 300 m
- Tunnel Mainz South: 246 m

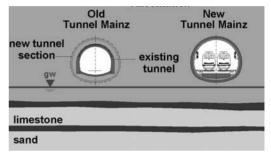


Figure 4. Geological cross section.

## 4 CONSTRUCTION OF THE NEW TUNNEL

#### 4.1 Construction techniques

Parallel to the existing Old Tunnel Mainz, the New Tunnel Mainz was built in the years 1998 to 2000. The new 1250 m long double track railway tunnel with a low overburden of 10 m to 23 m runs under buildings including a hotel with basements up to 10 m under ground level. Moreover, there are old (Roman) underground hollow spaces (gallery systems) to be undercrossed.

The clearance between the Old Tunnel Mainz and the new one varies between 4 m and 50 m (fig. 4).

Regarding ground conditions, existing settlementsensitive structures and the possible influence on the Old Tunnel Mainz the excavation of the New Tunnel Mainz had to be carried out only with little deformation. Hence, an universal shotcrete tunnelling method with side wall drifts followed by the excavation of the calotte and core/bench was chosen as construction method (fig. 5).

The distance between the side wall faces and the final lining (ring closure) was limited to less than 100 m and in particularly sensitive parts to 50 m. The distance between the calotte face and the ring closure of the preliminary support was restricted to 30 m. Apart from the usual measurements in tunnelling additional securing measures were applied in

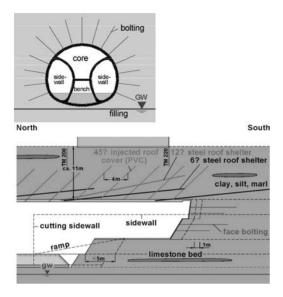


Figure 5. Undercrossing of a hotel; driving concept and securing measures (Steiger et al. 1999).

areas of settlement-sensitive structures. They are as follows:

- Horizontally injected steel pipe roof shelter (strengthening of the longitudinal rock bearing arch) (fig. 5). The roof shelter is placed in the upper area of the face. The length of the drilling is 20.5 m. The minimum overlapping to the next roof shelter is 3 m. The advantage of this technique is quite very obvious; a widening of the roof area to place the drillings is not necessary.
- In order to prevent any dilatational effect above the tunnel roof, 45 degree inclined, grouted PVC-fans are installed in continuous distances of about 5 m.
- Injections from the ground surface, pre-installed injection systems under the foundation of buildings as well as systematic face boltings within the tunnel are applied additionally in order to minimize deformations. All these measures together in connection with prior determined combinations of available measures were the basis of a successful driving with little deformations under settlement-sensitive structures.

#### 4.2 Calculation method

For proof of the stability and serviceability 2D- and 3D-numerical calculation were carried out by means of the Finite-Element-Method with the program Abaqus (figs 7, 8). Continuum elements were used for the soil, where as beam elements for the lining. For the realistic simulation of the soil an elastoplastic soil behaviour was chosen (Quick et al. 2001). The modified

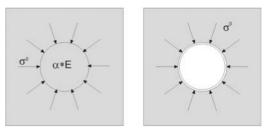


Figure 6. Principle of the alpha-method (Quick et al. 2001).

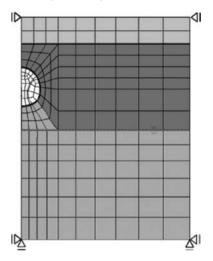


Figure 7. 2D-finite element mesh of the New Tunnel Mainz.

Drucker-Prager material law with cap was implemented. The yield surface of this elastoplastic model is not constant in the principal stress space. It can expand due to plastic straining. Furthermore it distinguishes between different stiffness for loading, unloading resp. reloading.

For the calculation of the preceding deformations as well as to account for three dimensional arching effects around the unsupported tunnel the alpha-method was applied (fig. 6). The principle of this method is to reduce the stiffness of the finite elements which are to be removed in the next calculation step. The reduction causes changes in the initial stress field and therefore leads to preceding deformation. In case of the sidewall and calotte drivings the factor is set to 0.5. This assumption which controls mainly the preceding deformations was verified by measurements (fig. 11).

The 3D-finite element model was created by extruding the 2D-mesh. Under respect of the construction procedure the length was chosen to 100 m (fig. 8).

#### 4.3 Monitoring

Regarding the extraordinary situation to undercross several settlement-sensitive structures with only low

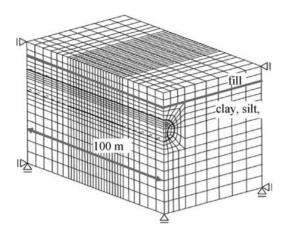


Figure 8. 3D-finite elements mesh of the New Tunnel Mainz.

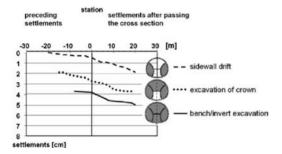


Figure 9. Measured surface settlements due to drivings (TM 117).

overburden in soft ground an extensive geotechnical monitoring program had been carried out. At the surface the deformations due to tunnelling are measured in close distances by levelling as well as deformation monitoring systems, working on the principle of corresponding tubes. Figure 9 shows the measured surface settlements due to the driving at station TM 117. The surface settlement adds up to 5 cm. The settlements measured at the ground surface along the tunnel (fig. 10) were in most areas between 1.5 cm and 2.5 cm in average; at the very beginning of the driving additional securing measures – as described prior – have not been applied; the settlements at surface reached up to unacceptable 11 cm.

Figure 10 shows the surface settlements under respect of the different drivings (sidewall drift, excavation of crown etc.) at station TM 117. This station is close to the portal north (fig. 1). Most of the measured surface settlements are related to excavation of the sidewall drifts and the crown, while only a smaller amount of settlements arise from the bench/invert excavation.

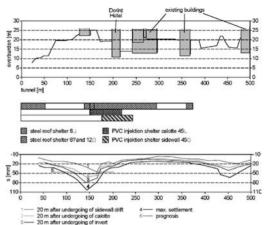


Figure 10. Surface settlement at surface due to tunneling.

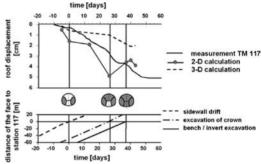


Figure 11. Comparison of measurements and calculations (roof displacements).

Figure 11 shows the comparison between the measured roof displacements of the tunnel and the calculated roof displacements at station TM 117. The results of the 2D-calculation show a good correspondence with measurements regarding the preceding displacement as well as the displacements due to the excavation of the sidewall drifts and the crown. However the calculated heave of the roof due to the excavation of the bench/invert does neither match the measurement nor the expected ground behaviour (fig. 11). For such soft soil conditions it is therefore recommended to increase the stiffness of the finite elements below the tunnel.

## 5 REHABILITATION OF THE OLD TUNNEL

### 5.1 *Construction technique*

The rehabilitation and enlargement of the Old Tunnel Mainz is going to be done under respect of the experiences of the New Tunnel Mainz. The partial loose ground (old back filling) around the Old Tunnel is improved by injections. In the first step the old filling around the tunnel is injected. Voids will be filled, improved by injections (bulk filling). For this 10 drillings up to a length of 2.5 m in a longitudinal distance of 1.5 m are carried out (fig. 12).

In the second step up to 8.5 m long injection drillings around the tunnel to activate a support ring are drilled. These two ground improvement steps are done under protection of the existing Old Tunnel. Subsequently the calotte driving in shotcrete method with an additional forepiles takes places. For the stability of the preliminary lining of shotcrete with a thickness of 0.30 m a widening of the calotte footing was established. In the final construction step the invert is excavated and the preliminary ring closure is achieved.

The distance between the face of the calotte driving and the preliminary ring closure with shotcrete is limited to less than 12 m.

# 5.2 Calculation method

For the design and the proof of the serviceability of the tunnel and the mentioned structures 2D- and 3D-numerical calculation are carried out with the program Plaxis. The numerical calculations regard all construction phases as well as the former excavation of the Old Tunnel Mainz and the New Tunnel Mainz. In order to create realistic results the material law of Hardening Soil with a yield surface, which is not fixed in the principal stress space is used. The yield surface expands due to plastic straining. In addition the material law can distinguish between different stiffness for loading and unloading resp. re-loading.

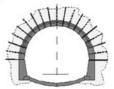
To account for the three dimensional arching effect of the unsupported enlargement of the Old Tunnel Mainz the beta-method is applied under respect of the used calculation program. The principle of this method is described in 3 steps (fig. 13):

- 1. Generation of the initial stress field  $-\sigma^{\circ}$ .
- 2. De-activation (excavation) of the tunnel clusters without activation of the tunnel lining and generation of (1-beta) reduced forces, which can be done by a reduction of the ultimate level of the full calculation step.
- 3. Activation of the tunnel lining.

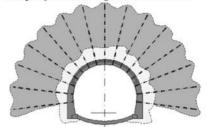
The beta-value was obtained by an iterative back analysis of the New Tunnel Mainz. The predicted surface settlements of the 2D calculation for the ground surface amount to approx. 2 cm (fig. 14).

#### 5.3 Monitoring

The rehabilitation of the Old Tunnel Mainz is accompanied by an extensive monitoring program within the 1. step: injection of the old back filling



2. step: injection drillings around the tunnel





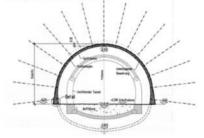


Figure 12. Tunnelling technique - Old Tunnel Mainz.



Figure 13. Principle of the beta-method.

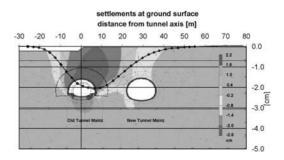


Figure 14. Calculation results - Old Tunnel Mainz.

tunnel and on ground surface. For a quick interpretation of the data three levels of settlement limits for different areas under respect of the overburden and the existing structure were defined. On the basis of pre-defined measures such as an additional temporary shotcrete invert settlements can be slowed down and reduced to guarantee the stability of the existing structures.

# 6 CONCLUSION

The presented tunnels in the inner city of Mainz in soft soil conditions show a variety of different measures in the ground for the drivings in order to meet the requirements of the serviceability of existing buildings. 2D or 3D calculations were carried out to predict the displacements. It is shown that the evaluation of the input parameters and the calculation methods is complicated but decisive for the calculated results. Hence, every tunnelling project must be accompanied by an intelligent monitoring program to observe the impact on the environment due to the drivings and to ensure the stability and serviceability of the tunnel and neighboring structures as well as to generate data for possible back-analysis in order to improve the calculated results and to verify the assumptions. The paper shows also numerical approaches with different material laws and calculation methods. These different approaches can all lead to tolerable results, if methods are used properly and the input parameters are evaluated appropriately.

# REFERENCES

Emeriault, F., Bonnet-Eymard, T. & Kastner, R. 2005. Movements induced on existing masonry buildings by the excavation of a station of Toulouse subway line B – 5TH international symposium, Amsterdam, Geotechnical Aspects of Underground Construction in Soft Ground. June 2005

- Kovacevic, N., Edmonds, H.E., Mair, R.J. & Higgins, K.G. 1996. Numerical modelling of the NATM and compensation grouting trials at Redcross Way – Geotechnical Aspects of Underground Construction in Soft Ground, Rotterdam, 1996
- Maidl, B. 2004. *Handbuch des Tunnel- und Stollenbaus*. 3. Auflage. Verlag Glückauf GmbH, Essen
- Quick, H., Michael, J., Arslan, U. 1999. Tunnelling for German High Speed Railway Lines – Proc. Civil and Environmental Engineering Conference – New Frontiers and Challenges, Bangkok, Vol. 2, part I, pp. II~117–128
- Quick, H., Michael, J., Arslan, U. 2001. About the effect of preliminary measures on ground movements due to tunnelling – Response of Building to Excavation-Induced Ground Movements, London, 17–18. July 2001
- Quick, H., Meißner, S., Michael, J., Arslan, U. 2001. Vergleich von Ergebnissen numerischer Berechnungen mit in-situ Messungen am Beispiel eines Tunnelvortriebes – STUVA-Tagung, München, 19–22. November 2001, Studiengesellschaft
- Quick, H., Michael, J., Schöttner V., Arslan, U., Katzenbach, R. 2000. *Tunnelling and Deep Excavation in Soft Ground* – GeoEng2000, Melbourne, 19–24. November 2000
- Steiger, H., Theissen-Wenzel, C., Quick, H. 1999. Neuer Mainzer Tunnel – Behandlung geotechnischer Grenzfälle in der Planung und Ausführung – Vorträge zum 6. Darmstädter Geotechnik-Kolloquium, Darmstadt, 11. März 1999, Mitteilungen des Institutes und der Versuchsanstalt für Geotechnik der Technischen Universität Darmstadt, Heft Nr. 44

# Supervision and protection of Shanghai Mass Rapid Line 4 shield tunneling across the adjacent operating metro line

R.L. Wang & Y.M. Cai

Shanghai Metro Operation Co. Ltd., P.R. China

# J.H. Liu

Shanghai Municipal Engineering Administration Bureau, P.R. China

ABSTRACT: Shanghai Mass Rapid Line 4 shield tunnels cross the operating Line 2 with 1.03 m distance beneath the operating line 2, small angle and small turning radius. The tunneling-across was carried out without ground improvement. This project took great challenge to the operating line. The influence of the tunnel crossing was predicted and a scientific construction scheme and measures were arranged to guarantee safe operation of Metro Line 2. The construction was performed based on the following principle strictly "to push step by step slowly, to turn equably with short step, to maintain stable pressure, and to improve the foundation with lower pressure and small amount." Thanks to these measures, the shield tunnel crossed Metro Line 2 successfully.

# 1 GENERAL INSTRUCTIONS

"Zhangyang Road—Pudian Road" tunnel of Metro Line 4 was constructed with shield method. The shield tunnel crossed under the operating Line 2 the operating line with a small angle and small turning radius. Figure 1 shows you the "#" shape of the relation between Metro Line 2 and 4. Following difficulties were encountered in this project.

- 1. Distance. The new tunnel cross blow the existing tunnel with minimum distance 1.03 m.
- 2. The tunnel at the crossing area is curved with small turning radius. The radius of Line 4 tunnel is only 380 m. For the shield, to advance with a small radius curve would influence the surroundings more seriously than advancing along straight line.
- 3. Influence area. For Line 2, the shield advance will directly influence 60 m at the up direction line and 94 m at the down direction line. Length of the influenced range, in which the distance between the upper and lower tunnel is less than the tunnel diameter, is above 300 m.
- 4. No ground improvement. The construction point was located in a place where several roads cross, so that the ground traffic is very heavy. It is impossible to conduct any ground improvement.
- 5. Bad geological condition. Line 2 tunnel is located in the fourth layer stratum, which exhibits high compressibility and obvious rheological behavior.
- 6. No relevant experience. Prior to this project, the tunnel of Line 2 had ever crossed below Line 1

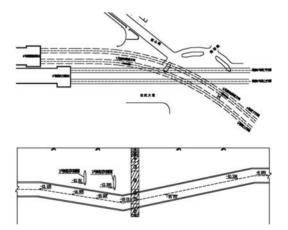


Figure 1. Plane and profile of the cross section.

when constructing, but in that case the ground improvement was carried out. To sum up, tunnel crossing without soil improvement is highly risky.

# 2 PROCESS ANALYSIS AND COUNTERMEASURE OF THE SHIELD TUNNELING

# 2.1 Process analysis

The influence of construction to metro operation was predicted and the change of construction character was

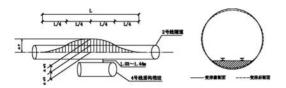


Figure 2. Deformation of tunnel Line 2.

analyzed. The theoretical analysis shows: before the shield-advance reach the centre line of Line 2 tunnel, it would disturb the equilibrating stratum stress and change the stress distribution. It would induce upward and forward deformation to Line 2. When the shield face passed through, the Line 2 tunnel would deform continuously due to the reasons including excess pore water pressure dispersing, consolidation settlement and secondary consolidation settlement and operation trains' vibration. In order to control the deformation of Line 2 tunnel, it is necessary to conduct synchronizing grouting and second grouting to Line 4 tunnel. The grouting could give a pre-heaving to Line 2 and the further settlement could be smaller. If the operating tunnel settlement approached the limitation, a re-grouting would be needed. Meanwhile ground improvement could be carried out between two tunnels to reduce settlement of Line 2. Consequently, when the shield crossed the Line 2 tunnel, longitudinal deflection in the horizontal and vertical direction might be occurred. It is composed with tunnel horizontal displacement, vertical heave or settlement and radial convergence deformation. It is shown in figure 2.

# 2.2 Main disturbing factors in the shield advance process and its control measure

Indeed, various factors would cause Line 2 tunnel deformation, the factors, however, could be reduced to the following three categories:

- 1. During the shield advance process, the shield shell and some projecting parts on the shield shell would cause stratum loss. The stratum loss volume, marked as Ve1, is closely relevant with the shield advance velocity. In order to reduce the stratum loss, we must limit the shield advance velocity strictly and try our best to make it suitably slow and constant. According to the theoretical analysis and the existing experience, the shield advance velocity was set as  $v = 5 \sim 10 \text{ mm/min}$ . Furthermore, we request the shield advance must pause for 10~30 min every 10 min advance. According to the metro protection standard mentioned above and the monitoring data from automatic equipments, we would adjust and re-determine the pause time at any moment.
- 2. Volume loss caused by curved shield advance is marked as  $V_{e2}$ . When the shield is advancing in

curve, in every 10 cm advance, an angle deviation, marked as  $\alpha$ , should occur at the shield axes. Volume of Ve2 varies directly as the shield length square and inversely as the curve radius. To reduce the volume loss, articulated equipment of shield machine must be used correctly, to reduce the shield effective length. The deviation-correcting degree per time must be limited strictly to ensure V<sub>e2</sub> as small as possible. Theoretically speaking, deviation-correcting at every ring advance with 1200 mm ring width is 17.6 mm. When it is divided into 12 times, however, the deviation-correcting volume per time is only less than 1.5 mm. That is to say the small degree and multi-times deviationcorrecting would influence only 1/12 area as influenced by the original way. The disturbed area is therefore considerably reduced and the volume loss volume, Ve2, is also reduced consumedly.

Volume loss caused by unbalanced pressure at the 3. shield face, marked as Ve3. Ve3 is the volume loss caused by difference between shield face pressure and stratum original static pressure  $\Delta P$ . During the course that the shield advance is approaching Line 2 tunnel, in order to reduce the metro tunnel settlement caused by shield tail grouting lack and consolidation settlement,  $\Delta P$  should be a positive volume, namely the shield face pressure should be  $0\sim5\%$  larger than stratum original static pressure. Because the Line 4 tunnel is in a small radius curve in this section and it crosses the operating Line 2 with a small angle, pressure on the shield cutter head is unbalanced at left and right sides during the shield crossing through process. And sudden pressure change might occur during the process. To maintain the pressure at shield face stable (especially when the segment is erecting, the jack would be getting loose) and to make the soil be a little uplifting is very beneficial to Line 2 tunnel settlement control. Consequently, whether the construction is successful or not is depend on whether the soil pressure at the shield face could be controlled and maintained at a suitable level. What should be adjusted in the construction process is the parameter mentioned above: the shield advance velocity, v; pressure at shield face; and axes deviation occurred in every 10 cm advance,  $\alpha$ , (generally, this index is defined as the predetermined volume). General effectiveness caused by these three factors, when reflected in the Line 2 tunnel horizontal displacement, vertical uplift or settlement and tunnel radial convergence, is  $\delta_v \delta_h$  and the tunnel radial convergence deformation.

According to the metro tunnel deformation specification and metro safety operation requirement, for the operating Line 2 tunnel, the maximum tolerance of the longitudinal deflection in the horizontal and vertical level planes are determined as  $\pm 5$  mm; and

the tunnel radial convergence deformation tolerance is strictly controlled within 20 mm.

# 2.3 Real time information construction

During the shield advance process, the vertical deflection of the Line 2 tunnel, both in vertical level horizontal level, marked as  $\delta_{v_{x}}$  and in the vertical plane, marked as  $\delta_h$  must be controlled strictly. Whenever the shield advances a step, the change rate of  $\delta_{v}$ and  $\delta_h$  must be analyzed and the construction parameters must be adjusted within  $\pm 5 \text{ mm}$  tolerance, so that to control the tunnel deformation. In the shield advance process, high-automatic and high accurate monitoring system was adopted to supervise Line 2 tunnel status lively. High-precise automatic electrical level supervision system was adopted to measure the tunnel longitudinal settlement and transverse subsidence difference between two tracks; and Bassett convergence system adopted to supervise the tunnel convergence status. After the train service finished, we adopt additional manual supervision to monitor settlement, displacement, and convergence status of the Line 2 tunnel (to set 3 settlement monitoring points at each profile; 1 on the track bed and the other 2 on the segment the monitoring profiles are set every 2 m along the tunnel.)

# 2.4 Analysis on grouting and foundation improvement

After the shield cross over the Line 2 tunnel centre line, whether the interspace at the shield tail could be refilled timely and suitably is a key factors that cause tunnel longitudinal deformation. In this process, the shield jack stress on the tunnel segment would cause an opposite longitudinal deflection, but control volume of  $\delta_h$  must be less than -5 mm. After the shield passed, the consolidation settlement of the layer beneath the tunnel is also an unneglectable factor. Therefore, after the shield passed below the Line 2 tunnel, several timely grouting with low pressure and small amount is very important and directly related to the success of Line 2 tunnel settlement control.

# 3 TECHNOLOGY KEYS OF THE SHIELD ADVANCE AND PRE-CONTROL OF MAIN CONSTRUCTION PARAMETER

Based on the deep analyses and clear understanding on the construction difficulties, the operator conducted a 45 kph speed limit in the relevant section of Metro Line 2.

We comply with the following principle carefully and strictly in the construction – "to push step by step slowly, to turn equably with short step, to maintain stable pressure, and to improve the foundation with lower pressure and small amount." The technology keys are decomposed into the following points:

- 1. Settlement or uplift control volume of the Line 2 tunnel is  $\pm 5$  mm. When the uplift volume reaches 50% and settlement, 30%, of the control volume, the monitoring system will alarm. That makes the tunnel always in a slight deformed condition.
- 2. The shield advance slowly and constantly; the shield advance speed is maintained at  $5 \sim 10 \text{ mm/min}$ . And the shield advance must pause for  $10 \sim 30 \text{ min}$  every 10 min advance. The pause time length depends on whether the Line 2 tunnel has "uplift ~ fall back" to the protection standard.
- 3. The shield face soil pressure should be adjusted to a correct volume so that it would cause minimum influence onto the Line 2 tunnel. To maintain a stable face soil pressure is extremely important. The shield face pressure should be  $100 \sim 105\%$  of stratum original static pressure, however, the volume is flexible and should be adjusted based on the live monitoring data. When the segments are erecting, there should be effective precaution to prevent the face soil pressure from descending; and the erecting time should be as short as possible.
- 4. Shield Advance Deviation-correcting. The deviation-correcting degree per time is the smaller the better. The deviation-correcting is required to taken every 10 cm advance. And the articulated equipment of shield machine must be used correctly, to reduce the shield effective length.
- 5. Synchronizing grouting pressure should be less than 0.4 mpa or even lesser. Grouting amount is  $1.1 \sim 1.8$  times of the interspace cubage. Detailed grouting amount depends on the monitored Line 2 tunnel data.
- 6. Second liquid grouting would be adopted in the second grouting. The grouting conduct principle is: low pressure, multi-times, moderate and timely. The pressure should be maintained at 0.2 mpa. Grouting amount per time per ring is  $50 \sim 100$  liter. We request the grouting should be conducted every other ring, to reduce its influence to Line 2 above. Generally, we request the grouting only be conducted when the train service finishes.
- 7. Foundation improvement is necessary to prevent the Line 2 tunnel from settling continuously. In this process, all relevant parameters such as, grouting pressure, grouting amount, grouting time, grouting pipe withdrawing velocity, and grout mix proportion and consistency, should be limited carefully to reduce the harmful influence to the tunnel above as much as possible.
- 8. Set automatic and high-precise electrical level system along the Line 2 tunnel. The electrical level is set to acquire data every 5 minutes and deliver the data to shield operation control centre. The operation centre would adjust construction parameter and

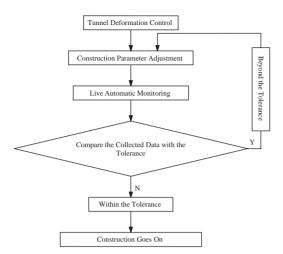


Figure 3. Pre-control system sketch.

guide shield construction based on these data. The construction technology keys and quantified construction parameter are together formed an effective and reliable pre-control system, which is shown in figure 3.

# 4 SUPERVISION ON THE SHIELD CONSTRUCTION AND THE OPERATING TUNNEL DEFORMATION ANALYSES

#### 4.1 Construction process

The shield machine is 8.625 m long and the outer diameter  $\Phi = 6.34$  m. The segments are 350 mm thick and single segment longitudinal length is 1.2 m. The whole construction process is as follows:

- 1. The first tunneling across process of the shield. Shield at Line 4 down line tunnels across Line 2 up line first, and then tunnels cross Line 2 down line.
- 2. After the shield machine arrived Zhangyang Road Station, soil between the tunnels at the crossed section was improved to ensure the metro Line 2 operation safety.
- 3. The second tunneling cross process of the shield. The shield turned back to advance in the Line 4 down line. It cross Line 2 down line first, and then tunnels cross Line 2 up line.
- 4. After the tunneling finished, soil of this section was improved. Four tunnels at the cross through section formed a "#" shape with small angle on a projection plane. The project therefore influenced very large range of Line 2. Length of the range, in which the distance between the upper and lower tunnels is less than the tunnel diameter, is above 300 m.

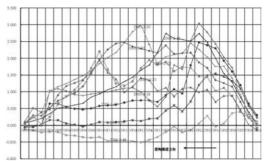


Figure 4. The Line 2 up line tunnel settlement/uplift data during the first tunneling cross process.

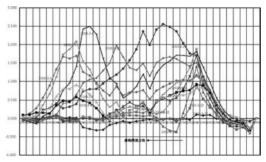


Figure 5. The Line 2 down line tunnel settlement/uplift data during the first tunneling cross process.

# 4.2 Process description and site monitor of the first tunneling cross

The first shield tunneling cross process was started from Pudian Road Station. The first tunneling cross process lasted for 15 days.

At the very beginning of the shield advance, the preset construction parameter makes the Line 2 tunnel to maintain a slight uplifting trend.

When the shield cutterhead was approaching below the Line 2 up line tunnel projection, the shield advance velocity and face soil pressure were adjusted, but the Line 2 tunnel still maintained uplifting trend and the uplift volume was up to  $0.2 \sim 0.5$  mm. During the shield advance process, soil at different points had different stress, thus caused different influence in the Line 2 tunnel. When deviation-correcting conducted, soil at head and left of the shield machine was stressed but stress of soil at head and right was released; meanwhile, at the shield tail, soil at the right side was stressed but stress of soil at left side was released.

When the shield advanced just below the axis of Line 2 tunnel (the shield was only 1.44 m away below Line 2 tunnel then), the face pressure onto soil had been adjusted speedily from the previous  $0.23 \sim 0.27$  mpa to 0.166 mpa, much smaller than the previous pressure.

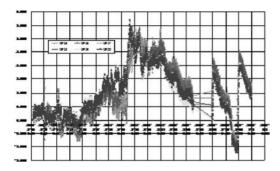


Figure 6. Settlement/uplift data of the closest point from the Line 2 up line tunnel during the first tunneling cross process.

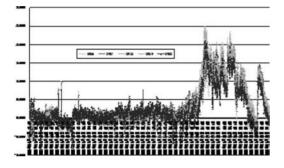


Figure 7. Settlement/uplift data of the closest point from the Line 2 down line tunnel during the first tunneling cross process.

At that time, the up line of Line 2 tunnel had uplifted for  $2.5 \sim 3.1$  mm.

When the shield cutterhead was escaping from the projection of Line 2 tunnel, because of incorrect and unsuitable deviation-correcting and synchronized grouting, instantaneous uplift volume of Line 2 tunnel was up to 3.3 mm. Figure 4 and figure 5 show the settlement and uplift condition during shield cross through Line 2 up line tunnel and down line tunnel respectively.

Figure 6 and Figure 7 show the settlement and uplift condition of the closest settlement point during shield cross through Line 2 up line tunnel.

# 4.3 Process description and site monitor of the second tunneling cross

The second shield tunneling cross process was started from Zhangyang Road Station. By June 24, 2003, when the shield of Line 4 construction arrived 2 m away from the projection line of the Line 2 down line tunnel, the shield advance had caused settlement and uplift at Line 2. On June 30, the shield cutterhead entered the projection line of Line 2 down line tunnel, then it crossed over below the Line 2 down line and up line one after another. By July 22, the shield

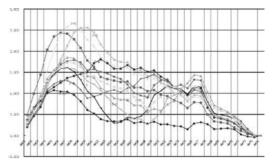


Figure 8. The Line 2 down line tunnel settlement/uplift data during the second tunneling cross process.

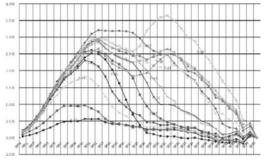


Figure 9. The Line 2 up line tunnel settlement/uplift data during the second tunneling cross process.

escaped from Line 2 projection completely. The second shield tunneling cross process totally lasted for 23 days.

The shield crosses below the Line 2 down line tunnel first with the minimum vertical distance 1.03 m. Its influence on the Line 2 tunnel is obvious, although we have made very strict limit on the construction parameter. By July 1, the Line 2 down line tunnel has uplifted up to 2.5 mm. Because the up line tunnel of Line 4 is very closed to Line 2 tunnel, the Line 2 tunnel assumed obvious dynamic settlement during cross through process. It can hardly be controlled except by synchronized grouting and second grouting method. Figure 8 to Figure 11 shows detailed condition.

#### 4.4 Grouting and foundation improvement issues

In the shield advance process, in order to control the Line 2 tunnel settlement after shield advance, unreactive-grout-synchronized grouting was conducted, so that the grout would refill interspace at the shield tail. Meanwhile, synchronized grouting and second grouting was conducted at the cross section where dynamic settlement occurred. Double liquid grout was used in second grouting to improve the stratum's mechanical properties. 16 grouting holes are

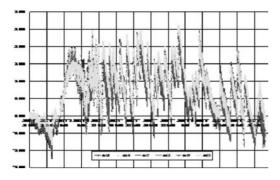


Figure 10. The Line 2 down line tunnel settlement/uplift data at every time during the second tunneling cross process.

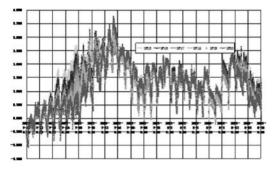


Figure 11. The Line 2 up line tunnel settlement/uplift data at every time during the second tunneling cross process.

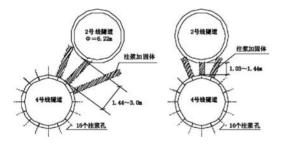


Figure 12. Schema of the Foundation Improvement.

reserved at every segment in the cross section, so that it would be easy to conduct stratum grouting improvement after the shield escaped from the section. The arrangement of the foundation improvement grouting point is shown in figure 12. After the shield passed, by repeating foundation improvement in the recent year, the Line 2 tunnel settlement situation is now well in hand.

## 5 CONCLUSION

After the elaborate construction and strict site supervision and protection, the Line 2 tunnel settlement situation is now well in hand and the safe metro operation during the project are also realized. Valuable experience, which could be reference for the future construction, was accumulated in this case.

- When the shield advancing in curve with small radius, because of different stress condition at different point of the shield, it would cause various influence to the upper metro tunnel.
- The shield advancing velocity is an important factor that causes upper tunnel uplift. The influence, however, could be reduced by suitable construction parameter adjustment.
- Shield deviation-correction is another important factor that causes upper tunnel uplift. The uplift or settlement condition is dependent on relative position of the two tunnels.
- 4. Shield tail grouting would influence quite a large range, up to  $10 \sim 15$  m. It is necessary to maintain low pressure, small amount and correct grout in the grouting process, to reduce the influence to the surroundings.
- 5. Dynamic settlement caused by train running is very dangerous. Synchronizing grouting and second grouting could be adopted to correct that. Foundation improvement and double liquid grouting are important method to control tunnel settlement.
- 6. The principles "to push step by step slowly, to turn equably with short step, to maintain stable pressure, and to improve the foundation with lower pressure and small mount" are proved correct technical keys.
- Strict high-precise automatic monitoring and information based construction is one of the most important technical support.

# ACKNOWLEDGEMENTS

I would like to extend grateful appreciation to Mr. BaiTinghui, Mr. Ge Shiping, Mr. Shen Chengming, Mr. Sun Lianyuan and Mr. Ni Chengyu for their assistance during composing this thesis.

# Kowloon Southern Link – TBM crossing over MTR Tsuen Wan Line tunnels in HKSAR

K.K.W. Wong, N.W.H. Ng & L.P.P. Leung Kowloon-Canton Railway Corporation, HKSAR

Y. Chan MTR Corporation Limited, HKSAR

ABSTRACT: The Kowloon Southern Link (KSL) is a 3.8 km underground railway project in a busy urban area with complex geological and alignment constraints. A key challenge for the project is to construct a critical section of the tunnels above the existing MTR tunnels and below a heavily trafficked road junction, which is termed as the MTR Crossing. Cut and cover method was originally envisaged to construct the MTR Crossing. In view of the complex ground conditions and other physical site constraints, the contractor proposed to extend the bored tunnels using a tunnel boring machine (TBM) across the existing MTR tunnels as an alternative. Adopting the TBM method for the MTR crossing introduced specific challenges which called for comprehensive engineering studies, extensive ground treatment, tailored protective measure for the MTR tunnels, as well as sophisticated instrumentation in order to ensure that the railway operations and the public safety are not compromised. This paper is to discuss the design development of the scheme and the methodology of the MTR Crossing.

# 1 INTRODUCTION

The Kowloon Southern Link (KSL), running 3.8 km underground, will link West Rail from its current terminus at Nam Cheong Station to the East Rail's terminus at East Tsim Sha Tsui Station, with a new West Kowloon Station in between. By joining East Rail with West Rail, it will provide commuters with a convenient interchange between these two railway corridors.

The KSL construction works are basically divided into two sections, northern section and southern section. The northern section covers a twin 2.1km running tunnels between Nam Cheong Station and West Kowloon Station, mainly on reclaimed land where the conventional cut and cover construction is the prevailing method. The 1.7 km southern section comprises a new station and twin 1.1 km TBM bored tunnels located in a busy tourist and commercial area between West Kowloon Station and East Tsim Sha Tsui Station. The project includes the crossing perpendicularly above the existing Mass Transit Railway (MTR) Tsuen Wan Line (TWL) tunnels which run in the north-south direction underneath the heavily trafficked Nathan Road (Figure 1).

This paper focuses on the TBM crossing over the MTR TWL tunnels.

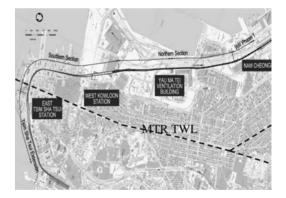


Figure 1. The KSL Alignment.

# 2 SITE CONDITIONS AND REGIONAL GEOLOGY

The solid geology in the Kowloon peninsula and along the KSL tunnel route and at the MTR crossing comprises intrusive igneous bedrock belonging to the Kowloon Granite Formation. Salisbury Road is 4 m above sea level and constructed over old reclaimed fill underlain by superficial deposits. The deposits overlie

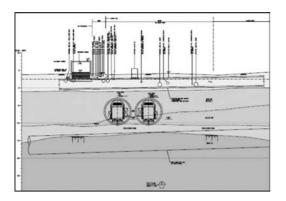


Figure 2. Geological sections.

a relatively thin layer of completely decomposed granite (CDG) above bedrock. Despite close proximity of the harbour the ground water table in Salisbury Road was typically 2.5 m below ground level with minor influence from tidal fluctuations.

The geological interpretation indicated that the MTR down track tunnel lies within the granite bedrock. For the MTR up track tunnel, as-built record indicates the bedrock and completely decomposed granite interface to be at the axis of the MTR up track tunnel. However, based upon additional geotechnical information and borehole results, this interface was interpreted to be higher above the tunnel axis (Figure 2).

#### **3 TUNNEL DESIGN**

### 3.1 Original design based on cut and cover

The original scheme design of the KSL tunnels along Salisbury Road was based on the conventional cut-and-cover construction method. KCRC, the project proponent, submitted its original proposal based upon the original scheme design to the HKSAR Government.

However, MTR raised an objection to the proposed scheme for the reason that the proposed cut-and-cover method at the MTR crossing would give rise to the risk of floatation due to removal of the overburden above the MTR tunnels at the crossing. Subsequently, an alternative micro-tunnelling technique was further proposed for the MTR crossing.

The proposed micro-tunnelling method required hand mining in soft ground within a pre-installed horizontal pipe piled enclosure around the perimeter of the tunnel structure. It was contended that this alternative method would ameliorate the risk of floatation of the existing MTR tunnels compared to the cut-and-cover method. As such, MTR agreed to allow authorization of the scheme by the HKSAR Government based on the micro-tunnelling method.

Despite MTR's no objection to the use of the microtunnelling method from railway protection point of view, there were drawbacks of this method. It is recognized that hand mining in soft ground, in particular in marine deposits and below groundwater table, is a high risk activity. The risks of hand mining include the indirect support system, instability of mined face, potential for seepage and piping through the gaps between horizontal pipe piles, and the likelihood of large ground movements resulting in a risk of damaging existing utilities. To mitigate these risks, substantial ground treatment would be required for the micro-tunnelling construction at the MTR crossing. Furthermore, excavation by hand mining with staged installation of temporary supports and construction of the permanent tunnel structure would be time consuming.

During the KSL tendering stage, some of the tenderers proposed the use of TBM method for the MTR crossing. Due to the limited time for the tendering, none of the tenderers was able to obtain MTR's agreement on adopting the TBM method. As such, the TBM crossing as a potential alternative remained uncertain at the time of the tendering.

#### 3.2 Selling the TBM crossing alternative

Immediately after the contract award, a value engineering workshop was held among KCRC, the contractor and its designer. Consensus was reached that the TBM crossing alternative should be seriously considered. The contractor's designer undertook a detailed engineering study of the TBM crossing over the MTR TWL tunnels. The study included a two-dimensional finite element numerical modeling analysis using the software PLAXIS Version 8.2 and a three-dimensional analysis using the software FLAC 3D Version 3.0.

In both the 2-D and 3-D analyses, the geotechnical parameters adopted were based on the geology determined from existing site investigation information and the MTR construction records. Due to the close separation of the two KSL tunnel bores, ground treatment using jet grouting was recommended by KCRC and its contractor (Figure 3). The analyses showed that the movements and distortions of the MTR tunnel lining due to the KCRC's TBM crossing would be within the required structural tolerance. Furthermore, a parametric study was carried out with varying jet grout strengths between 0.5 MPa and 1.5 MPa and it indicated that the impact on the MTR tunnel lining movement would be satisfactory and the jet grout strength was not a critical factor.

Despite the favorable results of the design assessment, MTR expressed a specific concern regarding the presence of boulders above the crown of the MTR tunnels and below the invert level of the KSL tunnels. MTR considered that there was a risk of the TBM

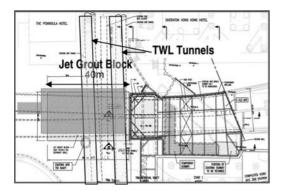


Figure 3. Layout of Jet Grout Block.

dislodging a boulder which may be connected to the MTR tunnel lining, resulting in damage to the MTR tunnel lining. In order to mitigate this risk, it was proposed to drill closely spaced horizontal pipe piles to physically isolate the MTR tunnels from the material through which the TBM would bore. Furthermore, jet grouting the entire area above the MTR tunnels was proposed to form a consolidated block of soil and grout matrix. On this basis, MTR agreed in principle to the TBM method of crossing the MTR tunnels.

However, the alternative TBM proposal still needed to undergo the statutory process of the amendment to the authorized scheme, which also involved consultation with relevant Government authorities, stakeholders, as well as local communities. The merits of the TBM method compared to the original scheme of cut and cover tunnels were obvious from the public and social benefit points of view, apart from the construction benefits. The TBM method minimizes disturbance to the traffic and existing utilities with much less environmental impacts in terms of noise, dust and visual impact, in particular in such a busy commercial and tourist area where there are nearby prestigious hotels, shopping malls and distinguished cultural facilities. As such, the alternative proposal for adopting the TBM method was well received by all the relevant parties without any objection.

#### 4 TECHNICAL CHALLENGES FOR TBM CROSSING

KCRC and its contractor needed to revise the design details of both the permanent works and temporary works to cater for the revised TBM crossing method. This included addressing the vertical alignment constraints imposed by the existing MTR tunnels, the connection into the existing KCRC overrun tracks of the ETS Station, as well as the ground conditions for TBM tunnelling (Figure 4).

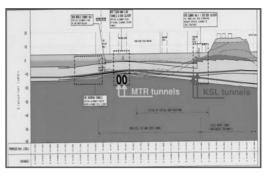


Figure 4. KSL Alignment constraints.

In order to maintain the minimum 1.5 m clearance of any KSL works from the extrados of the MTR tunnels, the depth of the ground cover to the TBM at the MTR crossing was as shallow as only 6.8 m which was less than the diameter of the TBM (viz. 8.05 m). Given the geology at the crossing with boulders present at the tunnel invert and marine deposits within the tunnel horizon and above the tunnel crown, the shallow cover is considered a challenge to the TBM tunnelling in controlling the ground settlements on one hand and minimising the risk of ground heave or slurry leakage on the other. There were existing critical utilities above the crossing. The consequence of damaging these utilities would be very serious. Also, any significant disturbance to the surface such as excessive ground movements or slurry leakage would inevitably cause disruption to the traffic at this busy road junction. These kinds of incidents would have major impact on the road users and KCRC's corporate image and therefore measures were taken to mitigate the risks of their occurrence.

Due to the above vertical alignment constraint, the structural clearance between the KSL tunnels and the MTR tunnels was only 2 m. The presence of boulders at the KSL tunnel invert had been a cause for concern on potential damage to the MTR tunnel lining. The MTR TWL carries more than a million passengers daily. Apart from the public safety which is of paramount importance, any disruption to the railway services would have substantial impact to the public and is unacceptable to MTR and Government authorities. Careful planning, control and monitoring of the TBM drives crossing above the MTR tunnels were deemed to be critical and essential in order to ensure that there would be no adverse impact on the MTR tunnels. These called for the need of the development of wide range of additional protective measures and geotechnical instrumentation.

The existing overrun tracks of the ETS station to the east of the MTR crossing had been operational prior to the KSL construction. The alignments of these overrun tracks had been based on the original cut and cover

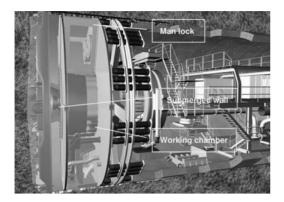


Figure 5. The mixed ground TBM.

tunnel scheme of the KSL with a very narrow separation between the up track and down track. he structural clearance between the two KSL tunnel bores at the MTR crossing was only 900 mm. This extremely close separation of the tunnels gives rise to a technical challenge for the TBM tunnelling, in particular on how to avoid causing excessive movements to the first completed KSL tunnel during the adjacent second TBM drive.

# 5 TUNNEL BORING MACHINE

A purpose-built mixed ground TBM, using mixshield/ slurry technology, was used to excavate the tunnels through both rock and soil deposits (Figure 5). The 8.05 m diameter Herrenknecht TBM, named "Little Dragon Girl" (Xiaolognu), was designed to suit the prevailing ground and groundwater conditions and so limited the potential for face instability and ground settlement when excavating in soft and mixed ground conditions. The TBM cutter head was also equipped with heavy-duty cutting tools for excavation in competent rock. A hydraulically powered jaw type rock crusher was installed in front of the suction grill behind the cutter head and was capable of crushing blocks such as boulders up to 500 mm in size.

The operating principle of the mixshield TBM is to utilise the pressurized bentonite slurry at the confined tunnel face to withstand the hydrostatic pressure and ground pressure in all instances. This system enables the tunnel face stability to be maintained at all times, even in a mixed soil/rock interface.

The bentonite suspension for the tunnel face support is pressurised by an air cushion or "air bubble". "Little Dragon Girl" was equipped with a compressed air regulation unit which accurately controlled the required "air bubble" pressure with a sensitivity of +0.05 bar. Pressure levels were calculated for various sections of the TBM tunnel drives according to the different geological and groundwater conditions. Extensive tests on the marine deposits in the area of the MTR crossing were carried out in order to obtain sufficient data for calculating the "air bubble" pressure to be adopted by the TBM when crossing over the MTR tunnels.

# 6 ADDITIONAL PROTECTION TO MTR TUNNELS

# 6.1 Ground treatment

There are multiple objectives of the ground treatment above the MTR tunnels as follows:

- To stabilize the ground whilst the TBM passes above the MTR TWL tunnels and beneath sensitive utilities,
- To provide water cut-off and prevent lowering of ground water and loss of soil when the TBM breaks into the reception shaft, and
- To cater for TBM low ground cover and narrow separation

Ground treatment in the form of 1.5 m diameter jet grout columns were adopted using cement based grout (including microfine cements) and silicates. For quality control purpose the unconfined compressive strength of post-drilled cores was set at 1 MPa minimum at 28 days. Jet grout columns generally terminate at 1.5 m from the crown of the MTR tunnels and 2.5 m above the crown of the future KSL tunnels. The total number of jet grout columns designed is 690.

The jet grouting employed a double fluid jet grouting system utilizing a swivel head that directs two different fluids of grout and air through concentric double tube rods. Grout and air exit horizontally through the two concentric nozzles attached to the bottom of the grout string. The air and grout are injected at high pressures of 6 bars and 200 bars respectively. The air and grout injection breaks up and mixes with the soil matrix resulting in a grout and soil slurry. The diameter of the grout column is controlled by the rate of rotation and withdrawal of the string. In general, a rotation rate of 6 to 7 rpm and withdrawal rate of 18-20 min/m were adopted to suit the soil stratum. In the cohesive clay layer, precutting by water jet and grouting was carried out at a slower drill rod withdrawal rate of 20-22 min/m. Excess sludge and slurry is removed through the annulus between the boreholes and the drill rods by air lifting effect. The completed column is formed as a hardened cylindrical body of soil cement mixture.

Working space and access were severely restricted due to the requirement to maintain traffic flow through a busy junction as well as numerous underground utilities (Figure 6). It was necessary to divert the traffic in multiple stages to facilitate jet grout column construction within the footprint of the available traffic management schemes. A combination of vertical and



Figure 6. Jetgrouting in progress.

raking holes were constructed, depending on the traffic and utility constraints.

Quality control testing was carried out to prove the effectiveness of the ground treatment. Core samples were drilled and collected from 5% of the columns for unconfined compressive strength testing. Permeability tests were also carried out in vertical drill holes from ground surface and water inflow measurements were made in horizontal probe holes from the TBM retrieval shaft to verify the water-tightness of the completed jet grout block. The results proved to be satisfactory.

#### 6.2 Horizontal pipe piles

Horizontal pipe piles were installed through the jet grouted zone above the MTR TWL tunnels to provide physical separation between the TBM and MTR TWL tunnels, and to isolate any core stone boulders which might be disturbed by the TBM bore.

Conventional Down-the-Hole (DTH) techniques were adopted for the drilling operation using an ODEX bit. The horizontal pipes were 273 mm diameter 6.3 mm thick pipe piles installed at 500 mm centre to centre spacing. The total length of each pipe pile is about 23 m in order to extend over both MTR tunnels and 5 m beyond the centre line of MTR up track tunnel. Structural clearance of the horizontal pipe piles to the MTR tunnels varies from a maximum of 2.0 mm to a minimum of 1.397 m. Tight structural clearance is a combined result of the vertical alignment constraints as described in section 4 above and the downward deviation of the horizontal pipe piles. Drilling of isolated piles which infringed the railway protection zone were permitted however this was conditional on additional requirements for working during non-traffic hours, detailed survey checks and comprehensive monitoring.

Controlling the alignment of the horizontal pipe piles is critical to ensure the pipe piles do not deviate downwards within 1.2 m of the MTR tunnels or upwards into the TBM bore. Alignment control was



Figure 7. Horizontal pipe piles installed.

achieved using a down-the-hole survey method. A survey prism is inserted into the far end of the piles, followed by survey checks every 3 m as the survey prism is withdrawn from the pipe. The survey check is carried out at 5 intermediate stages for each pile as follows: (1) before drilling and after the set-up of the drilling mast; (2) at 4 m depth before arriving at the MTR down track tunnel; (3) at 10m depth between the MTR tunnels; (4) at 16m depth before arriving at the MTR up track tunnel; and (5) after reaching the final length of 23 m.

Various contingency measures were developed in the event of recording unacceptable upward or downward deviation. These included replacing the pipe piles with bundled rebar within a bentonite cement mix and down-sizing the pipe piles to 219 mm in diameter.

# 7 INSTRUMENTATION AND MONITORING

The instrumentation and monitoring for the MTR tunnels was required to meet statutory and specific contract requirements. In addition, Alert, Action and Alarm Response levels (AAA) for building and structure movement, utilities, ground settlement and groundwater change were developed.

It is a pre-requisite requirement for the works within the MTR railway protection zone that the MTR railway and its continuous operation must not be jeopardized and be safely maintained during the TBM crossing. As revealed by the numerical modelling analyses, some movements of the MTR tunnel lining may be caused by the construction of the TBM tunnels directly above. To ensure the safe and continuous operation of the MTR railway at all times, an extensive regime of instruments and monitoring programme was established both inside and outside of the tunnels. The monitoring programme was a large part of the mitigation measures for the crossing to minimize the risks of damage to the MTR tunnel lining and consequential interruption to the railway services and include the following types of instruments and method of monitoring. Instruments and monitoring within the MTR tunnels:

- Automatic Deformation Monitoring System (ADMS): real-time deformation monitoring of the tunnel lining by automatic total stations within the influence zone at the crossing
- Vibrating Wire Strain Gauges: real-time monitoring of changes in the stress of tunnel lining
- Seismograph: real time monitoring of ground borne vibrations induced by the works
- Tape extensioneters: manual convergence survey of the tunnels
- Manual track level offset surveys

Instruments and monitoring of the ground surrounding the MTR tunnels:

- Vibrating Wire piezometers: real-time monitoring of changes in pore water pressure
- Manual and automated in-place vertical inclinometers: monitoring deformation of the ground and cofferdam walls
- In-place horizontal inclinometers: installed inside selected horizontal pipe piles above MTR tunnels monitoring real time ground movement
- Magnetic probe extensioneter: manually monitoring of deep ground settlement.

All monitoring data was uploaded directly to an instrumentation database system called "GEOMON" which is a proprietor product from the specialist instrumentation subcontactor. GEOMON also managed the data by automatically alerting designated recipients of AAA response value exceedances via e-mails and SMS, and could also be viewed and downloaded remotely via the Internet.

Upon exceeding the AAA levels, procedures outlined in the Exceedance Plans were implemented accordingly. During excavation for the first of the two KSL tunnels the TBM successfully crossed over the MTR tunnels. Insignificant movement and ground borne vibration was recorded in the MTR tunnels.

# 8 CONCLUSION

The change from traditional cut-and-cover construction to TBM method enabled two new railway tunnels to be constructed directly above one of the world's busiest operating railways with minimum clearance, but no train speed restrictions or interruptions to the services and with an excellent safety record.

The site conditions and the existing railways presented an unusually onerous range of technical challenges. The success of the MTR Crossing has established a new benchmark for the TBM tunnelling technique and with such recognition received would no doubt lead to its further potential use in complex urban environment similar to those found in HKSAR.

The project has strongly demonstrated that, for particular spatial and operational constraints, the TBM tunnelling method can provide the best overall value and performance in constructing new underground infrastructures.

# ACKNOWLEDGEMENT

The MTR Crossing required also the detailed planning and substantial cooperation and partnering interaction between the two railway corporations. The proactive involvement from MTR in the selection of an appropriate day for the crossing minimizing all possible adverse impacts, the provision of stand-by team and engineering trains during the crossing nights reacting to incidents, if any, instantaneously, and the carrying out advanced improvement works inside TWL tunnels before the crossing, are all contributing factors to the success of MTR crossing which was delivered safely and without interruption to the train services.

The authors would also like to thank the contractor Link 200 JV and their design team Mott Meinhardt JV for the invaluable input in developing and finalizing the MTR Crossing scheme.

# REFERENCES

HKSAR Works Bureau, Environmental Transport and Works Bureau, Technical Circular No. 19/2002 – Mass transit Railway Protection, May 2002.

# Application of pile underpinning technology on shield machine crossing through pile foundations of road bridge

# Q.W. Xu

Shanghai University of Science and Technology, Shanghai, P.R. China Shanghai Jiaotong University, Shanghai, P.R. China

# X.F. Ma

Tongji University, Shanghai, P.R. China

# Z.Z. Ma

Shanghai Shentong Metro Co. Ltd., Shanghai, P.R. China

ABSTRACT: Shanghai's subway construction has entered a new period of rapid development. A notable characteristic of subway network construction is that the construction environment is increasingly complex. Take Shanghai subway line 10 as an example, in the position of Shajinggang Bridge on Siping Road, the shield machine will have to pass through the bridge's pile foundation. Since the embedded depth of piles has already run through the entire tunnel section, these piles have to be pulled out or cut off. But, as one of the main arteries of Shanghai city, Si Ping Road has big traffic flow, and it does not allow the traffic volume to be affected during the construction period. What's more, there are intensive residential quarters around the bridge and the construction space is limited. Therefore, this paper will mainly introduce how the pile underpinning technology can be used in the subway construction of Shanghai area for the first time.

## 1 INTRODUCTION

Along with the continuous construction of metro lines in cities and unceasing improvement of metro network, there are more and more cases of new tunnel lines passing through old nearby buildings  $^{[1]\sim[3]}$ .

In the past 20 years, Shanghai has been experiencing the unprecedented climax of city infrastructure construction. There are total 17 lines planed in the urban rail transit network, of which 11 lines are subway with length of 385 km, presently, 5 lines have been completed, 5 lines are being constructed. By 2010, total 10 lines will be put into use with length of 250 km.

A notable feature of the new period of metro construction is that the construction environment is increasingly complex, that is, examples of intercrossing between new line and old line, crossing through various existing structures etc., are becoming more and more. Take Shanghai subway line 10 as example, it spans from Hongqiao Airport to New Jiangwan Town with total length of 28.8 km, which forms a convenient channel between the north and the west part of Shanghai city. During its construction process, the main pile foundations that impede the advance of shield machine are located at Shajinggang Bridge, Zoumatang Bridge,

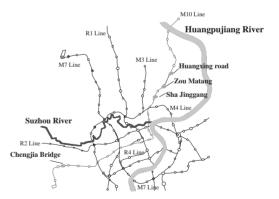


Figure 1. The general picture of subway Line 10.

Chenjiaqiao Bridge and Huangxin Road Bridge, as shown in Figure 1.

Regarding to the situation of new line crossing through existing structures, generally speaking, the line should be chosen to avoid existing buildings in the stage of planning. In fact, there are various practical situations often make it difficult to avoid such structures. As to the examples of tunnel crossing the bridge foundation mentioned in this paper, when the line can not avoid existing pile foundation, the general construction method is: construct temporary alternative bridge  $\rightarrow$  change traffic course, remove the old bridge, extract pile  $\rightarrow$  drive the shield machine forward  $\rightarrow$  construct a new bridge  $\rightarrow$  restore the original traffic  $\rightarrow$  demolish the temporary bridge.

If follow this method, it results in long construction period, high construction cost, and great impact on the society. But if we adopt the pile underpinning technology, that is, based on the premise of keeping the existing structure can be worked normally, while a series of construction technology are used to extract or truncate piles, and finally reach the purpose of driving shield machine forward. This can not only shorten construction period, reduce construction cost, minimize its impact on the community, but also can promote Shanghai's subway construction level.

## 2 PROJECT OVERVIEW

#### 2.1 Status of Shajinggang Bridge

According to the plan of Shanghai subway Line 10, the interval tunnel from Livang Road to Ouyang Road will cross through the piles of Shajinggang Bridge on Siping Road. The bridge is a simple beam structure with three spans; its spans are 6 m, 12 m and 6 m respectively. This bridge includes two piers and two abutments. Each pier uses 23 quadratic reinforced concrete piles as its foundation, with dimension of  $400 \times 400 \times 26000$  mm; while each abutment uses 14 quadratic reinforced concrete piles as its foundation, with dimension of  $400 \times 400 \times 27000$  mm. Since the elevation of tunnel vault beneath the bridge is about  $-6 \sim -7$  m, in the process of shield drive, there are about 4 piles at each pier and  $3 \sim 4$  piles at each abutment will affect tunneling excavation, as shown in Figure 2 and 3.

Around Shajinggang Bridge, there are intensive buildings and underground pipelines, which should be taken into account during the construction process. The important buildings near the bridge are a high-rise building in the southwest, a single story pump station in the south, a 24-storey building in the southeast, a reinforced concrete and brick building with 4 floors in the northeast, the pit of Quyang Road Station in the north and the Huaxi Stock Exchange Building in the northwest. The important underground pipelines are a group of power cable, a water supply pipe with diameter of 1500 mm and a group of telephone cables to the east of the bridge; while two gas pipes with diameter of 300 mm and 700 mm respectively, a water supply pipe with diameter of 300 mm and two groups of telephone cables to the west of the bridge.

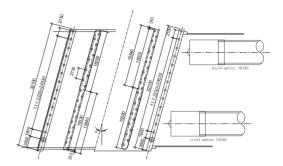
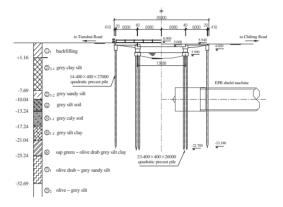
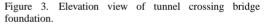


Figure 2. Plan view of tunnel crossing bridge foundation.





#### 2.2 The geological conditions of the project

According to engineering geological survey report, the geological conditions at Shajinggang Bridge can be divided into the following layers. The first layer is miscellaneous backfilling with thickness from 1.9 to 3.7 m, containing cinder, stone, etc. The second layer is grey and yellow clayey mingled with 1.0 m thick black humus in it. The third layer is grey silt soil containing clay in it. It belongs to medium compression soil, about 10 m thick. The fourth layer is grey clay with sand body trapped in it. It is high compression soil, including white stiff block and black humus in it, about  $5 \sim 12$  m thick. The fifth layer is dark green hard clayey. It belongs to medium compressed soil, about  $2.4 \sim 2.8$  m thick. The sixth layer is olive drab hard clay, which is medium compression soil also. The seventh layer is olive drab silt soil.

#### 2.3 Existing problems and solutions

In this project, the uniqueness and difficulties is not only reflected on complicated site topography and more intensive surrounding buildings, but also can be seen from the following aspects.

- Since Quyang Road subway station is very near to the bridge pile foundation, coupled with the restriction of line slope in longitudinal section, the tunnel can not cross beneath the pile foundation directly.
- 2. Since the outer diameter of shield machine is approximately 7 m, in the crossing process, 4 piles at each pier and 3 piles at each abutment should be removed.
- 3. Since Siping Road is one of main roads of Shanghai city, its traffic flow is big at day and night, it is not allowed that the traffic volume to be affected during the construction period.

In view of the above considerations, under the premise of existing traffic not to be affected, the proposed construction strategies include: ① adopting pile foundation underpinning technology at each pier and abutment; ② removing piles affecting shield tunneling. Since Shanghai has not yet had the precedent of pile under-pinning in subway construction, it is necessary to conduct series comprehensive technical studies on this technology and its accessory method, thus provide a common technical guidance for future similar projects.

# 3 ACCESSORY CONSTRUCTION METHODS IN PILE UNDERPINNING

#### 3.1 Methods of foundation reinforcement

The purpose of foundation reinforcement is to ensure the stability of existing structures, such as pier and abutment, and to provide stable foundation.

The main methods of foundation reinforcement can be concluded as:

- Grouting method: the main role of grouting is to seal water, but it is not applicable to cohesive soil.
- Deep mixing method: it can ensure foundation intensity and its sealing characteristics. This method includes two categories, that is, vertical construction method and transverse construction method.
- Freezing method: it can guarantee foundation strength and sealing effect; but it is difficult to resolve the ground deformation when defrosting.

## 3.2 Method of removing existing piles

There are two methods, that is, extracting piles from ground surface and demolishing piles within layers.

## (1) Extracting piles from ground surface

Deep excavation method: it is very simple, cheap and the required operation space is small also. The main procedure is shown in Figure 4.

Extracting piles with casing pipe: according to some overseas related experiences, this method includes processes of extracting piles with casing pipe and

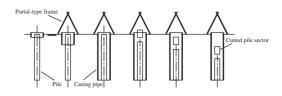


Figure 4. Sketch map of deep excavation.

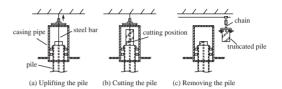


Figure 5. Sketch map of extracting pile with casing pipe.

cutting piles, only needing  $5 \sim 6$  m construction space in height, as shown in Figure 5.

(2) Demolishing piles within layers

This method can also be divided into two types: demolishing piles manually inside the chamber and cutting piles directly by shield machine. For the first type, it should reinforce the foundation firstly, and then remove piles manually in the chamber. For the sake of safety, compressed air can be used as supplementary measure to stabilize the excavation face. For the second type, piles can be cut directly by the special device installed on the cutter disc of shield machine without human operation. But, the shield machine used here is relatively more expensive.

## 4 PROPOSAL FOR SHAJINGGANG BRIDGE

Based on overseas experience and some successful instances [4], two feasible schemes are provided here. A scheme is reinforcing foundation and eliminating piles inside the shield chamber, while B scheme is reinforcing foundation, adding new piles and extracting piles with casing pipe.

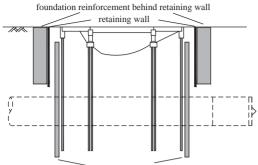
#### 4.1 A scheme

(1) Foundation reinforcement and construction of retaining wall

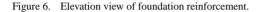
Foundation reinforcement and construction of retaining wall is to form operation space which is used to reinforce bridge structure, as shown in Figure 6 and 7. Since the river flows perennially under the bridge, cofferdam should be built firstly.

### (2) Pit excavation behind the abutment

Since the foundation of pier and abutment can not be reinforced on road surface, so it is necessary to



foundation reinforcement behind abutment



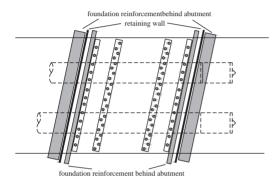


Figure 7. Plan view of foundation reinforcement.

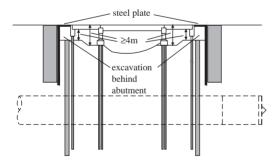
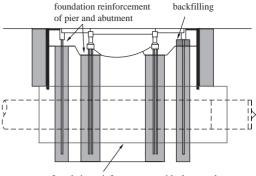


Figure 8. Elevation view of pit excavation behind abutment.

dig pit between the abutment and retaining wall. It is noteworthy that the excavated depth below the road surface should be no less than 4 meters so as to reinforce the foundation smoothly. After excavation, the pit top should be covered with steel plate to ensure that the ground traffic will not be affected, as shown in Figure 8.

### (3) Foundation reinforcement of pier and abutment

Once the foundation of pier and abutment had been reinforced, as shown in Figure 9 to 11, it can not



foundation reinforcement outside the tunnel

Figure 9. Elevation view of structure foundation reinforcement.

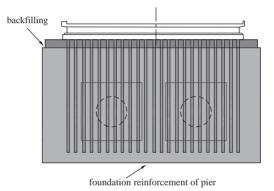


Figure 10. Foundation reinforcement of pier.

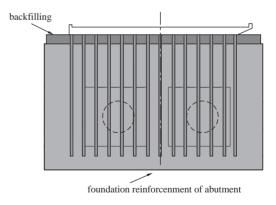
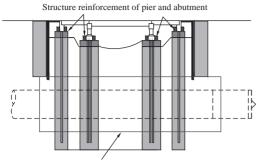


Figure 11. Foundation reinforcement of abutment.

only transfer the load borne by the removed piles to the new added piles and the remaining piles, but also integrate the foundation and piles as a whole body. In addition, the tunnel lining located here should be redesigned as high stiffness segment to withstand the load transferred from the removed piles. Considering



foundation reinforcement outside the tunnel

Figure 12. Elevation view of reinforcing pier and abutment.

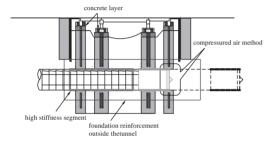


Figure 13. Demolishing piles and propelling shield machine.

the security of manually operation and stability of tunneling face, the foundation outside the tunnel should also be reinforced firstly, and the reinforced scope is indicated by the blue line in following figures.

#### (4) Structure reinforcement of pier and abutment

The purposes of reinforcing pier and abutment structure is to transfer the load borne by the removed piles to the reinforced foundation and remainder piles, and improve the capacity to resist shear force and moment perpendicular to the axial of pier and abutment. During reinforcement, steel beam can be used to support the structure, as shown in Figure 12.

(5) Demolishing piles and propelling forward the shield machine

After all the reinforcement work had been done, it will be able to demolish piles, propel forward shield machine and assemble segments subsequently. Since the work of demolishing piles is done inside the chamber, the excavated soil in it should be discharged firstly. Thereafter, tunneling face is in the unsupported status, so compressed air can be used to stabilize the face. The whole sketch map is shown in Figure 13. If necessary, high stiffness segment may be used within the whole

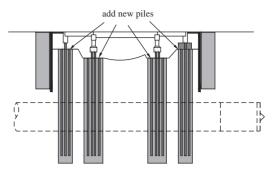


Figure 14. Elevation view of foundation reinforcement.

length below the bridge, which is indicated by the purple line in the figures. Meanwhile, the deformation of pier and abutment should be monitored strictly.

(6) Maintaince of steel beam and restoration of road surface

To prevent the corrosion of steel beam, it should be coated with a layer of concrete. After the obstacle piles had been removed and the shield machine had passed by, in order to guarantee regular transportation running, it is necessary to demolish the retaining wall, backfill the pit and restore the road surface. But, during these processes, more attention should be paid to monitor the deformation of structure.

# 4.2 B scheme

(1) Foundation reinforcement and the construction of retaining wall

Foundation reinforcement and the construction of retaining wall are the same as that in A scheme, as shown in Figure 6 and 7.

(2) Pit excavation behind the abutment

This process is also identical with that in A scheme, as shown in Figure 8.

(3) Foundation reinforcement of pier and abutment and the construction of adding new piles

Compare with that in A scheme, there are two differences in this process altogether, first is the reinforced scope, second is adding new piles, as shown in Figure 14 to 17. After the foundation reinforcement, the foundation at the topside around the piles should be excavated to form working space for adding new piles. Therefore, the required vertical clearance under pier and abutment should be not less than 3.5 meters.

#### (4) Reinforcement of pier and abutment structure

Reinforcement of pier and abutment can better improve its capability to resist exterior loads. Therefore, steel beam can be used to support the structure;

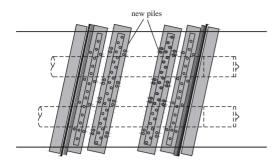


Figure 15. Plan view of foundation reinforcement.

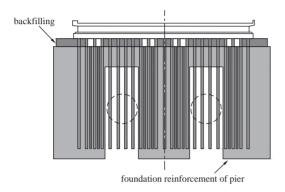


Figure 16. Foundation reinforcement of pier.

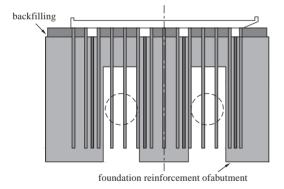


Figure 17. Foundation reinforcement of abutment.

the main process is identical to that in A scheme, as shown in Figure 18.

# (5) Foundation excavation and pile extraction

The purpose of further excavation of foundation below the bridge is to ensure that the working space for extracting pile can be satisfied. But, only those places, where the piles need to be extracted, should be excavated down. Since casing pipe method is used to extract

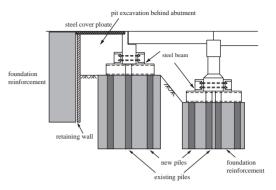


Figure 18. Reinforcing bridge structure and adding new piles.

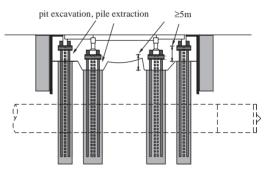


Figure 19. Further excavation of foundation.

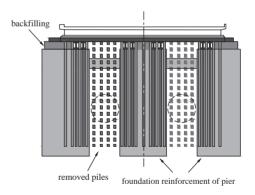


Figure 20. Extracting pile with casing pipe at pier.

piles, the required vertical clearance under the bearing platform should be no less than 5 m, as shown in Figure 19 to 21. During the construction of extracting pile, more attention should be paid on the monitoring of pier and abutment, especially for structure deformation.

(6) Propelling shield machine and installing segments

After the impeded piles had been removed, the shield machine can then move forward and pass

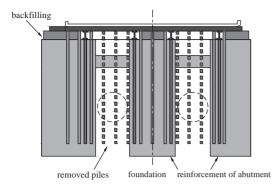


Figure 21. Extracting pile with casing pipe at abutment.

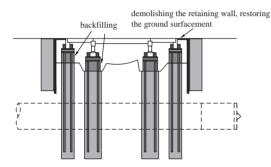


Figure 22. Backfilling and restoration of road surface.

through bridge foundation, followed by installation of segments. During the crossing process, deformation of pier and abutment structure should be monitored primarily.

(7) Backfilling and restoration of road surface

Once the shield machine had passed through the pile foundation, it is necessary to demolish the retaining wall, backfill the pit and restore the road surface to guarantee the regular traffic running, as shown in Figure 22. During such process, it is important to put emphasis on the deformation monitor of bridge structure.

# 5 CONCLUSIONS

Combined with the practical situations of in Shanghai area and the example of shield machine crossing through the pile foundations of Shajinggang Bridge, this paper mainly study how the underpinning technology can be used in subway construction. Based on concrete analytical studies, two main construction schemes are presented here. One is 'reinforcing foundation and eliminating piles inside the chamber'; the other is 'reinforcing foundation, adding new piles and extracting piles with casing pipe'. Since the pile underpinning technology has never been used in the subway construction of Shanghai area, the two schemes provided in this paper can not only be used as reference of design and construction, also as precious experience for future similar projects.

# REFERENCES

- Ishimura, T., Metoki, M. & Shimizu, M. 2006. Development of removed pile method with cutting. *Tunnelling and Underground Space Technology* 21: 411–412.
- Iwasaki, Y., Watanabe, H., Fukuda, M., Hirata, A. & Hori, Y. 1994. Construction control for underpinning piles and their behavior during excavation. *Geotechnique* 44(4): 681–689.
- Sharma, J.S., Hefny, A.M., Zhao, J. & Chan, C.W. 2001. Effect of large excavation on deformation of adjacent MRT tunnels. *Tunnelling and Underground Space Technology* 16: 93–98.
- Yamaguchi, I., Yamazaki, I. & Kiritani, Y. 1998. Study of ground-tunnel interactions of four shield tunnels drive in close proximity, in relation to design and construction of parallel tunnels. *Tunnelling and Underground Space Technology* Vol.13, No.3: 289–304.

# Characteristics of tunneling-induced ground settlement in groundwater drawdown environment

C. Yoo & S.B. Kim Sungkyunkwan University, Suwon, Korea

Y.J. Lee *RIST, Yongin, Korea* 

ABSTRACT: This paper presents the results of an investigation on the characteristics of tunnelling-induced ground settlement in groundwater drawdown environment. The dynamics of the effect of groundwater drawdown on the ground settlements are first investigated using a case history concerning a conventional tunnelling situation in which the interaction between the tunnelling and the groundwater induced excessive ground surface settlements. A 2D stress-pore pressure coupled finite element analysis is then conducted on a tunnelling case with groundwater drawdown, aiming at investigating ground surface settlement characteristics. The results indicated among other things that significant portion of ground settlement can occur before tunnel face reaches, and that the error function approach does not provide a good fit to the settlement troughs for tunnelling cases with groundwater drawdown.

# 1 INTRODUCTION

Tunnelling beneath the groundwater table causes changes in the state of stress and the pore water pressure distribution. In such tunnelling problems, the tunnelling work inevitably causes water inflows into excavated area, thus causing the change in the pore water pressure distribution. The direct environmental consequence of water inflows during tunnelling is the drawdown of groundwater level in the surrounding aquifer (Yoo 2005). The related ground subsidence occurring as a result of the reduction in water pressures in the soil layers can damage nearby structures/utilities (Figure 1). One of the major case histories illustrating damage due to ground settlement associated with tunneling-induced groundwater drawn is perhaps the Romeriksporten tunnel in which the highspeed railway tunnel construction caused more 1 m of ground subsidence due to groundwater drawdown, raising significant technical and political issues pertaining to the effect of tunnelling on surrounding environment (NSREA 1995).

Surprisingly studies concerned with the tunnelinginduced ground movements in groundwater drawdown are scarce as indicated by Yoo (2005). Although a number of studies on the ground subsidence caused by groundwater pumping from an aquifer have been conducted (Shen et al. 2006; Qiao & Liu 2006;

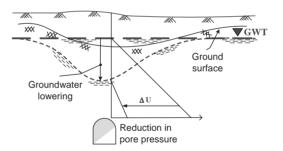


Figure 1. Illustration of ground settlement associated with tunnelling-induced groundwater drawdown.

Xu et al. 2006). The results of their studies cannot be directly applied to the tunnel excavation problems as they focused only on the groundwater drawdown due to groundwater pumping. As urban tunnelling projects tend to involve potential problems related to groundwater drawdown ground movements during tunnelling, there is an urgent need for better understanding on the mechanisms involved in tunnelling-induced ground movements associated with groundwater drawdown.

This paper presents the results of investigation on the characteristics of tunnelling-induced ground settlement in groundwater drawdown environment. The

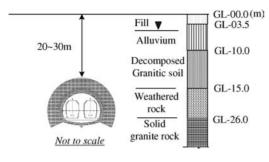


Figure 2. Typical ground profile.

Table 1. Geotechnical properties of soil/rock layers.

Туре	γ (kN/m <sup>3</sup> )	c' (kPa)	$\phi'$ (deg)	E (MPa)	ν	k (cm/sec)
fill	18	0	27	5	0.40	$3.8 \times 10^{-4}$
alluvial	20	15	30	10	0.40	$3.8  imes 10^{-4}$
weathered soil	25	15	30	50	0.33	$2.4 \times 10^{-4}$
weathered rock	25	60	35	120	0.30	$8.8  imes 10^{-5}$
hard rock	26	100	35	200	0.25	$5.0  imes 10^{-5}$

Note:  $\gamma =$  unit weight, c' = cohesion,  $\phi =$  internal friction angle, E = young's modulus,  $\nu =$  poisson's ratio, K = coefficient of permeability

dynamics of the effect of groundwater drawdown on the ground settlements are first investigated using a case history concerning a conventional tunnelling situation in which the interaction between the tunnelling and the groundwater induced excessive ground surface settlements. A parametric study using a calibrated 2D stress-pore pressure coupled finite element model is then conducted on a number of factors influencing the tunnelling-induced ground settlements with groundwater drawdown. Based on the results, the interaction mechanism between the tunneling, groundwater lowering, and ground settlement is identified.

# 2 GROUND SURFACE SETTLEMENT CHARACTERISTICS – FIELD MONITORING DATA

#### 2.1 Tunnelling condition

A case history concerning the conventional tunnelling, i.e., NATM, was considered. The tunnel has excavation width and height of approximately 10.5 m and 8.7 m, respectively, with a cover depth ranging approximately  $20 \sim 30 \text{ m}$ , and constructed in a multi-layered ground including a fill, alluvium, and a weathered zone as illustrated in Figure 2. The geotechnical properties of the ground are given in Table 1.

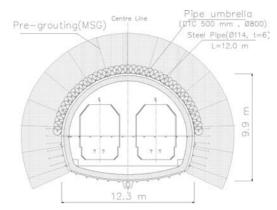


Figure 3. Support pattern (typical).

#### 2.2 Tunnel design

Figure 3 shows a typical tunnel support pattern used for a 100 m long section for the ground profile given in Figure 2. On account of the difficult ground condition the ring cut excavation method was adopted to promote the tunnel face stability during excavation. The primary support system consisted of a 0.2 m thick steel fibre reinforced shotcrete (SFRS) layer with 4 m long system rock bolts at 1.0 and 1.2 m, respectively, longitudinal and transverse spacing. The pipe umbrella technique using 800 mm diameter grout injected 12 m long steel pipes was additionally implemented to promote the face stability through improving the load carrying capacity of the ground ahead of the face. Also adopted was a trumphet shaped micro cement injection (MSG) pre-grouting around the tunnel periphery to create a 5 m thick watertight shell for sections in which the weathered soil layer extended to the tunnel crown level. The pre-grouting scheme was later extended to cover the face after the settlement problem had become an issue.

#### 2.3 Measured ground surface settlements

Figure 4 shows the progressive development of settlements during the tunnel advancement at various monitoring stations. In this figure the measured settlements are plotted against the relative distance between the tunnel face and the monitoring stations normalized by the tunnel diameter (D). These data, measured using the conventional leveling technique, thus indeed represent the settlement history during the tunneling process for the monitoring stations.

A total of five settlement curves are presented in this figure. As can be seen in this figure the five curves are similar both in qualitative and quantitative terms, despite the vertical extent of the decomposed soil relative to the tunnel varies along the route, showing the maximum converged settlements in the range of

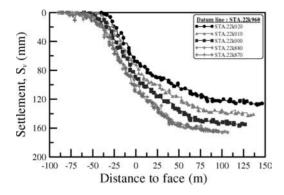


Figure 4. Progressive development surface settlements at various monitoring stations.

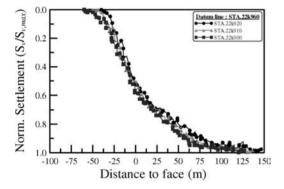


Figure 5. Normalized settlement history curves.

 $1.6\% \sim 1.8\%$ D. Of salient feature that can be observed in this figure is the tendency of settlement increase during which the tunnel advancement was halted, suggesting time dependent (tunneling activity independent) settlement development (to be discussed later). Another of interest trend is the resemblance of the settlement curves with a typical log t curve from a consolidation test, being characterized by three zones as an initial compression, primary, and a secondary zone. Such a trend strongly suggests a possible cause being the volume change effect due to tunneling-induced groundwater drawdown.

The data in Figure 4 are further analyzed by normalizing the settlement values with their respective maximum values ( $S_{v,max}$ ) in Figure 5. As seen in this figure, the normalized curves tend to collapse into one curve. A further inspection of the normalized settlement history curves shows that the settlements started to develop when the tunnel face was approximately 6D away from the monitoring stations. The settlements tend to accelerate when the tunnel face reached 3D away from the monitoring stations, and decelerate after the tunnel advanced  $5 \sim 6D$  beyond the monitoring stations. Also shown are that approximately  $60 \sim 70\%$  of the final settlement ( $S_{v,max}$ ) was completed before the face passed a monitoring station with the remaining 30 ~ 40% occurred after the full passage of the tunnel face. Such a percentage of settlement ahead of the face is considerably larger than the typical value of 40 ~ 50%, suggesting a larger portion of the converged settlement occurred prior to the arrival of tunnel face in this tunneling condition than a tunneling condition without the groundwater drawdown. Moreover, the settlements tend to converge to a constant value after the tunnel face advanced to a distance of 6 ~ 7D beyond the monitoring stations, suggesting slower settlement convergence than a normal condition.

These results in fact are somewhat different from typical trends that can be observed in tunneling conditions without significant groundwater lowering, and led to a conclusion that factors other than the unloading effect due to the tunnel excavation may have played a role. Such a tendency is directly linked to the groundwater drawdown as will be shown in a subsequent chapter.

# **3 PARAMETRIC STUDY**

#### 3.1 Stress-pore pressure coupled analysis

A commercial finite element package ABAQUS (Abaqus, Inc. 2002) was used for the parametric study. A 2D stress-pore pressure coupled effective formulation was adopted in order to realistically capture the interaction mechanism between the tunnelling and the groundwater.

In ABAQUS a porous medium is approximately modelled by attaching the finite element mesh to the solid phase. Equilibrium is expressed by writing the principle of virtual work for the volume under consideration in its current configuration at time *t*:

$$\int_{V} \sigma : \delta \varepsilon \, dV = \int_{S} \mathbf{t} \cdot \delta \mathbf{v} \, d\mathbf{S} + \int_{V} \mathbf{\hat{f}} \cdot \delta \mathbf{v} \, dV \tag{1}$$

where dv is a virtual velocity field,  $d\varepsilon$  is the virtual rate of deformation,  $\sigma$  is the true (Cauchy) stress, *t* are surface tractions per unit area, and **f** body forces per unit volume. **f** includes the weight of the wetting liquid **f**<sub>w</sub> defined as Eq. (2)

$$\mathbf{f}_{w} = (sn + n_{t})\boldsymbol{\rho}_{w} \,\mathbf{g} \tag{2}$$

in which *s* is the degree of saturation, *n* is the porosity, and  $n_t$  is the volume of trapped wetting liquid per unit of current volume. Eq. (1) can then be rewritten as

$$\int_{V} \boldsymbol{\sigma} : \delta \boldsymbol{\varepsilon} \, dV = \int_{S} \mathbf{t} \cdot \delta \mathbf{v} \, dS + \int_{V} \mathbf{f} \cdot \delta \mathbf{v} \, dV + \int_{V} (sn+n_r) \rho_{*} \mathbf{g} \cdot \delta \mathbf{v} \, dV$$
(3)

Table 2. Geotechnical properties.

	γ	E	c	φ	k
	(kN/m <sup>3</sup> )	(kPa)	(kPa)	(°)	(cm/sec)
soil Weathered rock	25 25	50,000 100,000			$5.8 \times 10^{-3}$ $1.3 \times 10^{-4}$

where **f** are all body forces except the weight of the wetting liquid.

The continuity equation is satisfied approximately in the finite element model by using excess wetting liquid pressure as the nodal variable (degree of freedom 8), interpolated over the elements. The backward Euler approximation is used to integrate the equation over time and the Newton iterations are used to solve the nonlinear, coupled, equilibrium and continuity equations. Fundamentals of the stress-pore pressure coupled formulation adopted in ABAQUS can be found in the ABAQUS user's manual (Abaqus, Inc. 2005).

#### 3.2 Condition analyzed

A tunnelling condition frequently encountered in urban situations was considered in the analysis. The tunnel considered is a 10 m diameter horseshoe shaped tunnel with a cover depth of 3.0D, excavated by the bench cut method. The primary support system consists of a 20 cm thick shotcrete lining with system rock bolts installed at 1.5 m center-to-center spacing. A 1.5D thick soil layer was assumed to exist above a weathered rock layer through which the tunnel is excavated. Tables 2 summarizes geotechnical properties of the ground.

#### 3.3 Finite element model

Figure 6 shows the finite element model adopted in this study. The finite-element mesh extends to a depth of two times the tunnel diameter (D) below the tunnel spring line and laterally to a distance of 15D from the tunnel center depending on the cover depth H. The lateral location was selected based on a series of preliminary analysis as it has a significant influence on the results of a stress-pore pressure coupled analysis. At the lateral boundary displacements perpendicular to the boundaries were restrained whereas pin supports were applied to the bottom boundary.

With regard to the hydraulic boundary conditions and with reference to Figure 6, a no-flow condition was assigned to the vertical boundaries perpendicular to the tunnel drive. At the lateral vertical boundary the groundwater table was assumed to be at the ground surface and constant throughout the analysis.

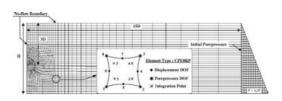


Figure 6. Finite-element model used in the analysis.

The ground and the shotcrete lining were discretized using 8-node displacement and pore pressure elements with reduced integration (CPE8RP). The rock bolts were modeled using the 2-node truss elements. With regard to the material modelling, the soil and rock layers were assumed to be an elasto-plastic material conforming to the Mohr-Coulomb failure criterion together with the nonassociated flow rule proposed by Davis (1968), while the shotcrete lining and the rock bolts were assumed to behave in a linear elastic manner. The geotechnical properties for the ground given in Table 1 were used for analysis. The young's moduli of the shotcrete and rock bolts were chosen as 15 GPa and 21 GPa, respectively.

The actual tunnelling process consisting of a series of excavation and support installation stages was closely simulated in the analysis by adding and removing corresponding elements at designated steps. After establishing the initial stress and pore pressure conditions with appropriate boundary conditions, the stepby-step tunnelling process pertinent to the bench cut excavation method, was then simulated. The 3D effects of advancing a tunnel heading was taken into consideration using the stress relaxation method in which the boundary stresses arising from the removal of excavated elements are progressively applied to simulate the progressive release of the excavation forces as the tunnel heading advances.

#### 3.4 Ground settlement characteristics

Figure 7 presents the relationship between the maximum surface settlement ( $S_{\nu,max}$ ), directly above the tunnel crown, obtained during various stages of tunneling. As seen the maximum surface settlement  $S_{\nu,\text{max}}$  tends to linearly increase with the increase in the groundwater drawdown level  $H_D$ . The settlement occurred after the completion of tunnel is in fact twice that during the tunnel excavation. It should be noted that the plot given in this figure represent those caused by the groundwater inflow into the tunnel after the completion of tunnel excavation, until a steady state condition is achieved. This suggests a direct link between the ground settlement and the groundwater drawdown, thus demonstrating the importance of creating a watertight shell for tunnelling cases where the controlling ground surface settlement is of concern.

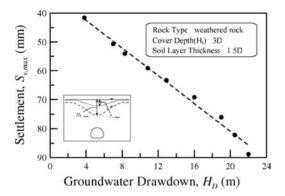


Figure 7. Variation of  $S_{\nu,\max}$  with  $H_D$  after completion of tunnel excavation.

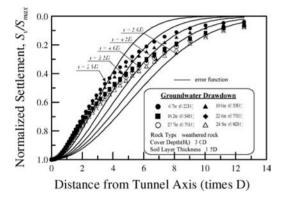


Figure 8. Normalized surface settlement troughs.

In urban tunnelling situations, characteristics of a ground surface settlement trough, such as slope and width of inflection point, are important as they, together with lateral displacements, determine potential for damage of adjacent structures. Figure 8 shows normalized ground surface settlement troughs for different groundwater drawdown levels together with error functions constructed using different values of inflection point. Of interest trends are two fold. First, the extent of ground settlement trough is significantly greater than typical tunnelling conditions without groundwater drawdown. In fact, for the particular tunnelling condition considered, the ground settlement zone extends more than 10 times the tunnel diameter from the tunnel centerline. Second, the error function approach (Attewell et al. 1986; Peck 1969) known to well describe the surface settlement trough for tunnelling cases without ground water drawdown does not provide a good fit to the settlement troughs for cases with groundwater drawdown. Another important observation is that the computed settlement troughs

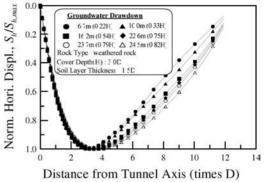


Figure 9. Normalized surface horizontal displacement profiles.

tend to collapse into one curve despite some discrepancies in the region father away, i.e.,  $\geq 4D$ , from the tunnel center.

Normalized horizontal displacement ( $S_h$ ) profiles are shown in Figure 9 for different levels of groundwater drawdown. As seen, the maximum horizontal displacements ( $S_{h,max}$ ) tend to develop at locations 3D away from the tunnel center with decreases in magnitudes thereafter. Again the  $S_h$  profiles tend to collapse into one curve although some discrepancies are observed in the region away from the tunnel center. The results obtained in this study suggest that the settlement trough as well as the horizontal displacement profile may be constructed using normalized curves when relationships between the surface movements and other factors can be established.

#### 4 CONCLUSIONS

This paper presents the results of an investigation on the characteristics of tunnelling-induced ground settlement in groundwater drawdown environment using the measured surface settlement for a site where the tunnelling-induced groundwater drawdown caused significant surface settlement. A stress-pore pressure coupled finite element model was additionally conducted aiming at identifying the ground movement characteristics when tunnelling induces a significant level of groundwater drawdown. Based on the results the following conclusions can be drawn.

 For tunnelling cases in which tunnel excavation causes significant groundwater drawdown, the percentage of settlement that develop prior to the tunnel face arrival to the final settlement is significantly larger than for cases without groundwater drawdown.

- 2. Continued groundwater drawdown after the completion of tunnel excavation may can cause settlement larger than that occur during excavation.
- 3. The error function does not provide a good fit to the settlement troughs for cases with groundwater drawdown.
- 4. Normalization can hold for the surface settlement and horizontal displacement profiles for tunnelling cases with groundwater drawdown.

#### ACKNOWLEDGEMENT

This research is supported by Korea Ministry of Construction and Transportation under Grant No. C4-01. The financial support is gratefully acknowledged.

#### REFERENCES

- Abaqus users manual, Version 6.5. 2005. Hibbitt, Karlsson, and Sorensen, Inc., Pawtucket, Providence, R.I.
- Attewell, P.B., Yeates, J. & Selby, A.R. 1986. Ground deformation and strain equations. Soil movements induced by

tunneling and their effects on pipeline and structures, Blackie, Glasgow: 53–66.

- Norwegian Soil and Rock Engineering Association (NSREA). 1995. *Norwegian urban tunnelling*. Publication No. 10., Norway.
- Peck, R.B. 1969. Deep excavations and tunneling in soft ground Proc., 7th Int. Conf. on Soil Mech. And Found. Engrg.: 225–290.
- Qiao, S. & Liu, B. 2006. Prediction of ground displacement and deformation induced by dewatering of groundwater. Underground Construction and Ground Movement: 73–79.
- Shen, S.L., Tang C.P., Bai, Y. & Xu, Y.S. 2006. Analysis of settlement due to withdrawal of groundwater around an unexcavated foundation pit. *Underground Construction* and Ground Movement: 377–384.
- Xu, Y.S., Shen, S.L. & Bai, Y. 2006. State-of-art of land subsidence prediction due to Groundwater Withdrawal in China. Underground Construction and Ground Movement: 58–65.
- Yoo, C. 2005. Interaction between Tunnelling and Groundwater-Numerical Investigation Using Three Dimensional Stress-Pore Pressure Coupled Analysis. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE* 131(2): 240–250.

# Effect of long-term settlement on longitudinal mechanical performance of tunnel in soft soil

H.L. Zhao, X. Liu, Y. Yuan & Y. Chi

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: This paper introduces the method for analyzing settlement data of tunnel in soft soil. The method is applied to the settlement monitoring analysis of some tunnels across the river. In the case study, the long-term monitoring data for shield tunnel settlement is discussed concretely. Firstly, unitary longitude settlement curves are gained, and the corresponding longitudinal curvature is calculated to investigate the rules of absolute settlement and uneven settlement varying from time. Secondly, based on the uneven settlement magnitude, the mechanical properties of shield tunnel are discussed by means of the equivalent axis stiffness continuous model of shield tunnel. In this way, the relationships between longitudinal curvature and crack of shield tunnel segment joint, the stress of tunnel concrete segment, and the stress of bolts for segment joint are found generally. So the crack and leakage of shield tunnel can be inferred consequently. Finally, the results gotten from analytical solution are compared with the field inspection information, and the favorable agreement is shown. Analysis of effect of long-term settlement on longitudinal mechanical performance of tunnel in soft soil offers guidance to evaluate the safety and serviceability, structural protection and waterproof design of shield tunnel in soft soil.

#### 1 INTRODUCTION

The dramatic longitudinal settlements occur in soft soil after many years' operation because of a number of factors, including vibration induced by trains, leakage, geotechnical conditions, and depth of embedment, construction methods, surface loads and mutative environment actions.

Due to the great span of time and huge inspecting cost, the investigation for tunnel long-time settlements has been limited. By far, there are few special reports for long-term settlement research of the tunnel. All of these factors restrict the development of safety and serviceability evaluation for tunnels in long-term operating process.

The settlement reflects the longitude deformation of tunnels. Generally, the tunnel in soft soil is weak relatively to resist deformation caused by extraneous factors, and the crack of circumferential segment seam is prone to appear, which leads to leakage and segments damage under longitudinal tension or compression. Therefore, it's crucial and significant to investigate and analyze some rules for shield tunnel such as the relationship of the segment inner force caused by uneven deformation, crack of segment joint, and stress of bolts. In this paper, the tunnel longitude deformation curvature is obtained from monitoring data of long-term settlement. Then, the relationship between longitude deformation curvature with the segment inner force and crack of segment joint are concluded. This paper aims at evaluating the performance of tunnel after many years' service according to the corresponding longitude deformation curvature. The analysis results offer important foundational materials for structural safety and durability evaluation of shield tunnel.

#### 2 LONG-TERM SETTLEMENT DATA ANALYSIS FOR SHIELD TUNNEL

This paper focuses on a highway tunnel across the river. This tunnel located in soft soil has serviced for more than 30 years. Tunnel settlement monitoring has being executed in the service stage. In the settlement monitoring, there are two elevation benchmarks set in both ends of tunnel. Settlement monitoring points between the two base points are arranged in both sides of way respectively along tunnel length, so it is a closed circuit style for inspection.

The investigation of this paper is based on the settlement data of this tunnel for recent ten years.

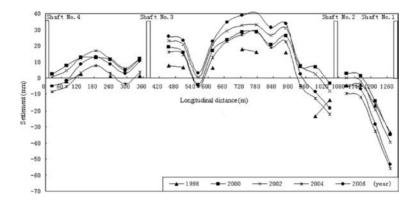


Figure 1. Longitudinal settlement curve of tunnel.

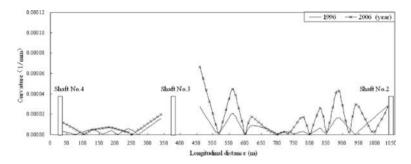


Figure 2. Longitudinal curvature of tunnel from Shaft No. 4 to No. 2

#### 2.1 Longitudinal settlement curve

There is certain rigidity for the structure between two adjacent monitoring points, and local structural deformation will affect adjacent structural deformation and internal force. In order to reflect integral tunnel longitude settlement comprehensively, the settlement analysis should adopt unitary analysis method. A unitary longitude settlement curve can be drawn based on monitoring results, shown as Figure 1. It should be noted that monitoring points located in both sides of this tunnel. So the settlements of these monitoring points are termed as settlements of left line and right line respectively in this paper.

The varying rules of longitudinal settlement obtained from longitudinal settlement curve can be concluded as follows.

1. Most settlement magnitude of observation points is between -30.0 mm and 30.0 mm. The annual settlement curve is parallel approximately. It means that the increase of settlements of observation points is uniform and tunnel longitude deformation shows favorable integrity. The deformation difference is little between adjacent points, and they almost rise and fall together.

2. The settlement magnitudes are variable from time. Most settlement points oscillate up and down. For instance, the accumulative longitudinal settlement in 2004 is larger than that in 2002, but the settlement in 2006 is less than that in 2004. Some of these settlements increase continuously in the oscillating and some incline towards steady.

#### 2.2 *The curvature of longitudinal settlement curve*

The curvature change is the important characteristic of longitudinal settlement for shield tunnel, because it demonstrates the uneven settlement of tunnel.

The settlement data from tunnel longitude observation points is made to spline interpolation fitting, and then, based on the interpolation result, the curvature  $\kappa$  is elicited by numerical differential. Therefore, the curvature varying from longitudinal distance can be drawn, shown as Figure 2. The curvature varies from tunnel longitudinal distance and the increased range of curvature is large. This is in line with the rule of tunnel

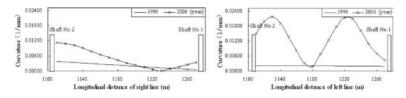


Figure 3. Longitudinal curvature of right and left line from Shaft No. 2 to No. 1

longitude settlement development. But it should be noted that the curvature change of left curve is different from right curve, so it will lead to lateral deformation and lateral additional inner force.

- The curvature of settlement curve for the points from No. 4 to No. 2 shaft is small. But the curvature where closed to shaft is larger than that between shafts.
- 2. The curvature is very large from No. 2 to No. 1 shaft, so another figure is drawn to describe it. The average value of curvature is 0.01 mm-1, even up to 0.021 mm-1, shown as Figure 3. So the structural safety and durability of this part tunnel should be calculated and evaluated in detail.

#### 3 SHIELD TUNNELING LONGITUDINAL STRUCTURE MODEL

At present, there are two main kind of theoretical research for tunnel longitude structure. One is numerical solving model based on the finite-element method. And the other is theoretical solving model.

Japanese scholars proposed two kinds of tunnel longitude structure theory based on the different simplification for tunnel segments joints and bolts. One is beam-spring model (Zhu, 1998); the other is equivalent continual rigidity model (Shiba & Kawashima, 1988). The study herein adopts the equivalent continual rigidity model proposed by Shiba and Kawashima (1988) to analyze tunnel longitude performance. Considering the tunnel analyzed in this case study has structural lateral deformation caused by long-term operation, so a relative correction factor is introduced to reduce the affection on tunnel longitude deformation and inner force.

#### 3.1 Equivalent rigidity continuous model

Equivalent rigidity continuous model is supposed to be continuous lining rings and homogeneous in the cross section of tunnel. The rings compose a tubular structure connected by longitudinal steel bolts termed as girth joint of tunnel segment. The girth joint bolt is simulated to spring with different characteristic in compression and tension. The simulate spring is

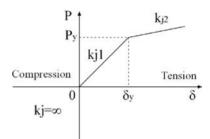


Figure 4. Relationship for  $P - \delta$ 

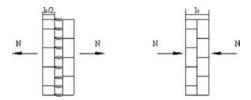


Figure 5. Stress distribution for segments in tension and compression.

bilinear material in tension, while is rigid absolutely in compression. The relationship of  $P - \delta$  is shown as Figure 4.

Herein,  $k_{j1}$  is elastic stiffness for individual bolt, and  $k_{j2}$  is plastic stiffness for individual bolt,  $P_y$  is the elastic ultimate pressure for bolt.

The assumptions in equivalent continual rigidity model are as follows.

- Circumferential inhomogeneity of lining ring is not taken into consideration. Additionally, the shear stress and deformation in longitudinal seam are ignored.
- Compressive pressure is resisted by concrete segment singly, and tensile pressure is resisted by concrete segment in share with steel bolts.
- 3. *Plane sections remain plane* and *small deformation* assumptions are applicable in this model.

The major results of equivalent rigidity continuous model referenced with the analysis of this paper are as follows.

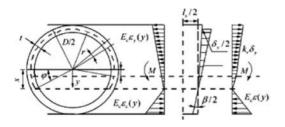


Figure 6. Stress distribution for segment in elastic state.

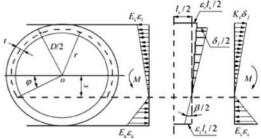


Figure 7. Stress distribution for segment in plastic state.

#### 3.2 Equivalent compressive and tensile rigidity

A segment with length  $l_s$  is taken as a calculate unit. Its stress distribution in tension or compression can be seen in Figure 5. The equivalent axial tensile and compressive rigidity is gained from deformation compatibility and force balance conditions, shown in Equation 1.

$$\begin{cases} (EA)_{eq}^{\ \ C} = E_c A_c & (N \le N_0) \\ \\ (EA)_{eq}^{\ \ T1} = \frac{E_c A_c}{1 + \frac{E_c A_c}{l_s K_{j1}}} & (N_0 < N \le N_y) \\ \\ (EA)_{eq}^{\ \ T2} = \frac{E_c A_c}{1 + \frac{E_c A_c}{l_s K_{j2}}} & (N > N_y) \end{cases}$$
(1)

 $(EA)_{eq}^{C}, (EA)_{eq}^{T1}, (EA)_{eq}^{T2}$  is equivalent longitude compression rigidity, the elastic and plastic equivalent tensile rigidity respectively.  $K_{j1}, K_{j2}$  is elastic and plastic spring coefficient.  $K_{j1} = nk_{j1}$ ;  $K_{j2} = nk_{j2}$ .  $N, N_0$  and  $N_y$  is longitude axial force for unit, pre-pressure for joints and the elastic ultimate tension for unit respectively.  $E_c, A_c$  is Young's modulus of concrete and area of segment.

#### 3.3 Equivalent bending rigidity

When the segment is in absolute elasticity state and the tensile stress of all the bolts is less than *Py*, the unit has the stress and strain shown in the Figure 6, in which, x,  $\varphi$  indicate the position of neutral axis, where,  $x = r \sin \varphi$ .

In terms of the deformation compatibility condition and the force balance equation, where,  $\varphi$  meets the Equation 2:

$$\varphi + ctg\varphi = \pi \left(\frac{1}{2} + \frac{K_{j1}}{E_c A_c / I_s}\right) \tag{2}$$

Then the equivalent elastic bending stiffness for tunnels can be expressed as follow:

$$(EI)_{eq}^{-1} = \frac{\cos^3 \varphi}{\cos \varphi + (\frac{\pi}{2} + \varphi) \sin \varphi} E_e I_e$$
(3)

Elastic ultimate flexural moment for priestesses segments, defined as:

$$M_{y} = \frac{(N_{y} - N_{0})(EI)_{eq}^{-1}}{r(1 + \sin \varphi)(EA)_{eq}^{-T1}}$$
(4)

Therefore, the tunnel radius of curvature corresponding to elastic ultimate flexural moment is:

$$\rho_0 = \frac{(EI)_{eq}^{-1}}{M_y} = \frac{r(1+\sin\varphi)(EA)_{eq}^{-T1}}{N_y - N_0}$$
(5)

When y = D/2, the maximum compression stress for concrete segment can be expressed as:

$$\sigma_c = \frac{M}{I_c} \cdot \frac{\cos \varphi + (\pi/2 + \varphi) \sin \varphi}{\cos^3 \varphi} \left( \frac{D}{2} - x \right)$$
(6)

When y = -D/2, the maximum tensile stress for concrete segments can be expressed as:

$$\sigma_{t} = \frac{M}{I_{c}} \cdot \frac{\cos \varphi - (\pi/2 - \varphi) \sin \varphi}{\cos^{3} \varphi} \left( \frac{D}{2} + x \right)$$
(7)

When y = -r, the maximum tension and deformation for joint bolts respectively expressed as:

$$f_{\tau} = k_{j1} \delta_{j} + P_{0} = \frac{MI_{s}}{E_{c} I_{c}} \cdot \frac{\pi \sin \varphi}{\cos^{3} \varphi} (r + x) k_{j1} + P_{0}$$
(8)

$$\delta_{j} = \frac{M I_{s}}{E_{c} I_{c}} \cdot \frac{\pi \sin \varphi}{\cos^{3} \varphi} \cdot (r+x)$$
(9)

When the flexural moment of tunnel segment is larger than elastic ultimate flexural moment  $M_y$ , the outermost joint bolts would demonstrate plastic

Table 1. Main structural parameters for shield tunnel.

Inner diameter (m)	Outer diameter (m)	Ring width (m)	Young's modulus of concrete (kPa)	Compression strength of concrete (kPa)	Bolt diameter /length/ quantity (mm/mm)	Young's modulus of bolt (kPa)	Yield strength of bolt (kPa)	Ultimate strength of bolt (kPa)	Ratio of elastic stiffness and plastic stiffness (α)
8.8	10.0	0.9	$3.35 \times 10^7$	$2.96 \times 10^4$	30/580/64	$2.06 \times 10^8$	$5.4 \times 10^5$	$8.35 \times 10^5$	0.01

characteristic. The stress and strain of segment in which some bolts reach the plastic state is shown in Figure 7.  $\eta$  and  $\phi$  demonstrate the critical position where the deformation of bolts is elastic ultimate deformation  $\delta_{\gamma}$ , and  $\eta = r \sin \phi$ .

In terms of the deformation compatibility condition and the force balance equation, an equation concerned with  $\eta$  and  $\phi$  can be obtained as follows.

$$(1 - R_1)(\cos\varphi + \varphi\sin\varphi) - \frac{\pi}{2}(1 + R_1)\sin\varphi + (R_1 - R_2)\left[\cos\varphi + \left(\frac{\pi}{2} + \varphi\right)\sin\varphi\right] = 0$$
(10)

$$(EI)_{cq}^{2} = \frac{2M}{(N_{y} - N_{0})r} \cdot R_{1} \cdot (\sin \varphi - \sin \phi) \cdot E_{c} I_{c}$$
(11)

When y = -r, the maximum deformation of joint bolts  $\delta_{\text{max}}$  expressed as:

$$\delta_{\max} = \left(1 + \frac{1 - R_2}{1 - R_1} \cdot \frac{1 + \sin\phi}{\sin\phi - \sin\phi}\right) \delta_y \tag{12}$$

Where,  $\delta_y$  is elastic ultimate deformation for joint bolts,

$$\delta_{y} = (\sigma_{y} - \sigma_{0}) I_{j} I E_{j}$$
<sup>(13)</sup>

$$\delta_{\max} = \left[ (\sigma_y - \sigma_0) / E_j + (\sigma_u - \sigma_y) / (E_j \alpha) \right] l_j$$
(14)

(Zheng, 2005)

Where  $\sigma_0$  is pre-pressure of bolts,  $\sigma_y$  is yield stress,  $\sigma_u$  is ultimate stress and  $E_j$  is Young's modulus.  $R_1$ ,  $R_2$  are coefficients in equation,  $R_1 = \frac{1}{1 + \frac{E_c A_c}{l_s K_1}}$ , and

$$R_2 = \frac{1}{1 + \frac{E_c A_c}{l_s K_{j2}}}.$$

The maximum compression stress of concrete segments

$$\sigma_{c} = \frac{M}{(EI)_{eq}^{2}} \cdot E_{c} = \frac{(N_{y} - N_{0})r}{2I_{c}R_{1}(\sin\varphi - \sin\phi)} \left(\frac{D}{2} - x\right)$$
(15)

The tunnel longitude deformation curvature can be defined as follows:

$$k = \frac{\theta}{l_s} = \frac{\delta_y}{(1 - R_1)r(\sin\varphi - \sin\varphi)l_s}$$
(16)

The relationships between tunnel longitude curvature and stress of concrete segment, joints crack, bolts inner force and tunnel ultimate flexural moment are established according to the conclusions drawn from analytic solution above. It can be used to evaluate structural performance of tunnels if the longitude uneven settlement has been known.

#### 4 LONGITUDINAL STRUCTURAL PERFORMANCE OF TUNNEL

#### 4.1 Critical state for evaluating performance

There are several critical conditions in the development of longitudinal stress and deformation of tunnels. In order to evaluate the safety status of tunnels, some primary critical states will be studied in the following contents.

- Crack of joints. The joints crack is limited for meeting the demands of waterproof and structure.
- Stress of bolts. There are two key critical states, namely, the stress of outermost bolts reaches to yield stress and ultimate stress.
- Compression strength of concrete. Concrete stress in compressive area reaches flexure compression strength of concrete segments.

## 4.2 *The evaluation for tunnel longitudinal structural performance*

The elastic and plastic spring stiffness coefficient may be calculated by parameters listed in Table 1.  $k_{j1} = E_j A_j / l_j = 250929.0 \text{ kN/m}, k_{j2} = 2509.3 \text{ kN/m}.$ 

When  $k_{j1}$  is introduced in Equation 2,  $\varphi$  will be 0.9895. The elastic equivalent flexural stiffness  $(EI)_{eq}^1 = 2.0951 \times 10^9 \text{ kN} \cdot \text{m}^2$ . Then the value of  $R_1$ and  $R_2$  are gained based on the upper parameters, as  $R_1 = 2.38 \times 10^{-2}$ ,  $R_2 = 2.44 \times 10^{-4}$ . When  $\delta_y = 1.82 \text{ mm}$ ,  $\varphi$  and  $\varphi$  are calculated as  $\varphi = 1.36368$ ,  $\phi = 1.20615$ . So the corresponding curvature can be calculated. The inner force and deformation corresponding to the curvature of longitudinal settlement curve for several important critical states are acquired by the analytical method stated above, listed in Table 2 in detail.

Critical state	Curvature (m <sup>-1</sup> )	Radius of curvature (m)	Bolt stress (kPa)	Crack of segment joint (mm)	Concrete stress (kPa)
Waterproof requirement (crack of segment joint $\leq 1 \text{ mm}$ )	$1.10 \times 10^{-4}$	27300	$2.97 \times 10^{5}$	1.00	$2.47 \times 10^{4}$
Concrete stress reaching Compression strength	$1.59\times 10^{-4}$	18818	$4.29 \times 10^5$	1.21	$2.96  imes 10^4$
Outermost bolt stress reaching yield strength	$2.00 \times 10^{-4}$	15000	$5.40 \times 10^5$	1.82	$3.72 \times 10^4$
Outermost bolt stress reaching ultimate strength	$9.94 \times 10^{-3}$	302	$8.35 \times 10^{5}$	83.06	$8.35 \times 10^{5}$

Table 2. Inner force and deformation of tunnel in critical states.

When the radius of curvature is greater than 27300 m, the stress of segments, tension of bolts and meet the demands, so the tunnel is safe relatively, but the leakage may occur. When the radius is less than 18818 m, the compression stress of concrete in compression area will exceed the compression strength of segments. When the radius is less than 15000 m, the outermost bolts in compression state may be into plastic state. When the radius is less than 302m, the end outboard bolts have reached at ultimate stress state, and may be tensile failure.

#### 4.3 Evaluation on structural performance

As for the part of tunnel between Shaft No. 3 and No. 4, settlements' absolute value of segment between is small relatively and floats in the range of 20.0 mm. The radius of curve is large. So the structural inner force and deformation are in a low level, while the longitudinal deformation curvature is large in the area near Shaft No.3 and the crack of segment joints can not meet the waterproof requirements.

As for the part of this tunnel between Shaft No. 3 and Shaft No. 2, the radius of curvature is less, about 20000 m. The field inspection results show that there is severe leakage of inner lining of the tunnel, especially near Shaft No. 3.

As for the part between Shaft No. 2 and Shaft No. 1, there is great tunnel longitude settlement, and the uneven settlement develops dramatically, especially nearby Shaft No. 2, which becomes the weak position in the whole tunnel.

#### 5 CONCLUSIONS

From the tunnel settlement curve and curvature data, the settlement of shield segment trends to be steady after many years' operation. But the uneven settlement of tunnel develops gradually.

The magnitudes of longitude deformation curvature corresponding to several significant crucial states are obtained. These magnitudes can be used to evaluate the structural performance according to the tunnel's longitude settlement.

When the radius of curvature is less than 27300 m, the waterproof requirement for crack of segment joint does not meet the demands, and the leakage may occur. When the radius is less than 18818 m, the compression stress of concrete will exceed the compression strength of segments. When the radius is less than 15000 m, the outermost bolts in compression state may yield. When the radius of curvature is less than 302 m, the outboard bolts would reach ultimate stress state, and tensile failure may occur.

#### ACKNOWLEDGMENTS

This work was supported by Shanghai Leading Academic Discipline Project(Project Number: B308).

#### REFERENCES

- Liu, J. H. & Hou, X.Y. 1991. Shield tunnel. China railway press:369–395
- Shiba, Y. & Kawashima, T. 1988. The evaluation of the duct's longitudinal rigidity in the seismic analysis of shield tunnel. *In: Proceedings of the Civil Academy* :319–327.
- Zheng, Y.L. 2005. Study on longitudinal crack of shield tunnel segment joint due to asymmetric settlement in soft soil. *Chinese journal of rock mechanics and engineering*, 24(24):4552–4558.
- Zhu, H.H. & Tao, L.B. 1998. Study on two beam-spring models for the numerical analysis of segments in shield tunnel. *Rock and Soil mechanics*, 19(2):26–32.

Theme 4: Safety issues, risk analysis hazard management and control

# Research on stochastic seismic analysis of underground pipeline based on physical earthquake model

#### X.Q. Ai

Shanghai Institute of Disaster Prevention and Relief, Tongji University, Shanghai, P.R. China

#### J. Li

Department of Building Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: In this paper, a new approach for the stochastic analysis of finite element modeled underground pipeline system under earthquake excitations is proposed, where a new developed physical random earthquake model and the probability density evolution method are adopted. Based on the physical process of seismic spread, the random earthquake model adopted not only indicates the physical relationship between the random earthquake ground motion and several key random parameters, but also presents the probability configuration of random seismic motions. Associating with the developed probability density evolution method for stochastic structures, the instantaneous probability density and the evolution process for the seismic response of underground pipeline can be analyzed numerically, and then the stochastic response can be obtained easily. With the finite element method, the stochastic response of underground pipeline under seismic excitation can be studied effectively. Using the proposed method, numerical example under different random conditions is investigated, showing that the proposed method is of high application in considering random seismic excitation with random soil parameter at the same time.

#### 1 GENERAL INSTRUCTIONS

As a basic and important problem in seismic reliability research for the underground lifelines system, the stochastic seismic response of underground pipelines has for a long time been extensively studied, coming up with a variety of theoretical and numerical methods applicable to engineering practice (Machida & Yoshimura 2002; Nedjara et al. 2007). However, on account of the calculation limitation in previous studies, the stochastic response analysis considering the randomness simultaneously caused by the seismic input and the structural parameters of soil-pipeline system is still an unsolved problem.

For the non-linear stochastic structures, it is difficult to capture the accurate probabilistic information of dynamic performance. The dynamic response of non-linear stochastic structures was analyzed usually by the random simulation, the random perturbation method or by the equivalent linearization technique. In 1986, Liu et al.(1986) started to investigate the problem with extension of the random perturbation technique. In 1980s, the Monte Carlo simulation is also employed (Deodatis & Shinozuka 1988), and some researchers studied the techniques to reduce the considerable computational work (Spanos & Zeldin, 1998). As the alternative approaches, the equivalent linearization technique (Klosner et al. 1992) and the extension methods used for linear stochastic structures such as the orthogonal polynomial expansion (Iwan & Huang 1996), are also investigated. However, for the dynamic response analysis of non-linear stochastic structures, the random perturbation method had great difficulty because of the secular terms problem and the requirement of small coefficient of variation of the random parameters. There are some arguments for the equivalent linearization method because of the misleading results in some occasions. The application of orthogonal polynomials expansion seems also unfeasible for multiple-degree-of-freedom system and non-polynomial-form non-linearity.

Evidently, more investigations are necessary for the dynamic response analysis of the underground pipelines. In resent years, a newly developed probability density evolution method for stochastic structures has been proposed. The instantaneous probability density function and the evolution against the time can be obtained precisely (Li & Chen 2005). For the dynamic response of stochastic structures, the solution can be derived through solving the probability density evolution equation with an initial value condition. On the other hand, different from the models from the power spectral density function, a new physics-based stochastic earthquake model to reflect the fundamental correlation between the critical factors and the random earthquake motions has been proposed by the authors (Li & Ai 2006). Several key random parameters are based on to realize the purpose, and a relational expression with physical background can be constructed considering the propagation process of earthquake motion in an engineering site. Associated with the probability density evolution method, the proposed random earthquake model can be of comparative advance to provide the basis for the stochastic seismic analysis.

In this paper, based on the probability density evolution method and the physics-based stochastic earthquake model, a new approach for the dynamic response analysis of the finite element modeled underground pipeline structures under random earthquake excitations and uncertain soil parameters is proposed and numerical example is investigated.

#### 2 PROBABILITY DENSITY EVOLUTION METHOD FOR STOCHASTIC STRUCTURES

#### 2.1 Principle of preservation of probability

The principle of probability conservation can be described as, during a conservative probability transformation process, within the state space, the increment of the probability within a unit volume equals to the inflow probability that gets across this unit.

If the ordinary differential of a state vector *Y* can be expressed as,

$$\dot{Y} = G(Y, t) \tag{1}$$

where  $Y = (y_1, y_2, \dots, y_n)^T$ ,  $G = (g_1, g_2, \dots, g_n)^T$ .

If  $p_Y(y, t)$  is supposed to be the probability density of Y(t), based on the principle of probability conservation, the probability density evolution equation can be expressed as followed,

$$\frac{\partial}{\partial t}p_{\gamma}(y,t) + \sum_{j=1}^{n} \frac{\partial}{\partial y_{j}} [p_{\gamma}(y,t)g_{j}(y,t)] = 0$$
(2)

## 2.2 Probability density evolution equation of the seismic response of stochastic structures

The dynamic equation of the nonlinear structure can be written as,

$$M(\zeta)\ddot{U} + C(\zeta)\dot{U} + f(\zeta,U) = -M(\zeta)\ddot{U}_g$$
(3)

where  $\zeta$  is the random parameter vector which represents the physical character of a stochastic structure, and its joint probability density function is  $p_{\zeta}(\mathbf{x})$ ; M, C are stochastic mass and damping matrices respectively, which include random parameters with the rank of  $n \times n$ , n is the dynamic freedom degree;  $U, \dot{U}, \ddot{U}$  are the displacement, the velocity, and the acceleration vectors, respectively;  $f(\zeta, U)$  is the nonlinear restoring force vector; and  $\ddot{U}_g$  is the acceleration vector of input earthquake.

Let a response vector  $X = (U^{T}, \dot{U}^{T})^{T}$ , the dynamic equation is then changed into a format of state equation including random parameters,

$$X = A(X, \zeta, t) \tag{4}$$

$$A = \begin{cases} \ddot{U}_{g} \\ -M^{-1}C\dot{U} - M^{-1}f - \ddot{U}_{g} \end{cases}$$
(5)

If  $\dot{X} = \frac{\partial}{\partial t} X(\zeta, t) = G(\zeta, t)$ , based on the above probability density evolution equation, the joint probability density function of X and  $\zeta$  will satisfy the following equation,

$$\frac{\partial}{\partial t}p_{X\zeta}(x,x_{\zeta},t) + \sum_{i=1}^{2n} \frac{\partial}{\partial x_i} [p_{X\zeta}(x,x_{\zeta},t)g_i(x,x_{\zeta},t)] = 0 \quad (6)$$

Since the velocity component  $\dot{X}_i$  is only the function of variable  $\zeta$ , the above equation can be re-written as

$$\frac{\partial p_{X\zeta}(x,x_{\zeta},t)}{\partial t} + \sum_{i=1}^{2n} \dot{X}_i(x_{\zeta},t) \frac{\partial p_{X\zeta}(x,x_{\zeta},t)}{\partial x_i} = 0$$
(7)

Having integral at both sides with  $x_1, \ldots, x_{l-1}, x_{l+1}, \ldots, x_{2n}$ , the probability density evolution equation will be decoupled,

$$\frac{\partial p_{X_{i,\zeta}}(x_{i}, \mathbf{x}_{\zeta}, t)}{\partial t} + \dot{X}_{i}(\mathbf{x}_{\zeta}, t) \frac{\partial p_{X_{i,\zeta}}(x_{i}, \mathbf{x}_{\zeta}, t)}{\partial x_{i}} = 0$$
(8)

where  $p_{X_l\zeta}(x_l, x_{\zeta}, t) = \int p_Y(x, t) dy_1 \dots dy_{l-1} dy_{l+1} \dots dy_{2n}$ , is the joint probability density function of  $(X_l, \zeta^{\mathrm{T}})^{\mathrm{T}}$ .

When the initial displacement and the velocity are independent of the physical parameters of the structure, the corresponding initial condition is described as,

$$p_{X,\zeta}(x_{l}, \mathbf{x}_{\zeta}, t)|_{t=0} = \delta(x_{l} - X_{l,0}) p_{\zeta}(\mathbf{x}_{\zeta})$$
(9)

where  $X_{l,0}$  is the determinate initial value of  $X_l$ , for a initial static structure, there is  $X_{l,0} = 0$ ;  $\delta(\cdot)$  is the Dirac function; and  $p_{\xi}(\mathbf{x}_{\xi})$  is the joint probability density function of the random vector  $\boldsymbol{\zeta}$ .

Solving the above partial differential equation with initial-boundary-values, which include the decoupled probability density evolution equation and the initial condition, the joint probability density function  $p_{X_l\xi}(x_l, x_{\xi}, t)$  can be calculated. After the integral with  $x_{\xi}$ , the probability density function of  $X_l(t)$  can also be solved,

$$p_{X_i}(x_i,t) = \int p_{X_i\zeta}(x_i, \mathbf{x}_{\zeta}, t) d\mathbf{x}_{\zeta}$$
(10)

The probability density evolution method of the stochastic structures is an effective approach to obtain the instantaneous probability density function and its evolution for the stochastic response.

## 2.3 Computational algorithm of probability density evolution equation

Step 1: Disperse the value region that corresponds to  $\boldsymbol{\zeta}$ , and disperse the initial condition simultaneously;

Step 2: After dispersing, for each determinate value of variable  $\zeta$ , a determinate dynamic analysis is carried out and the corresponding differential item with time of the target response  $\dot{X}_l(x_{\zeta}, t)$  can be deduced;

Step 3: Using the method of finite difference to solve the probability density evolution equation, then the joint probability density function  $p_{X_i\xi}(x_i, \mathbf{x}_{\xi}, t)$  and the probability density function of the target response can be calculated.

#### 3 RANDOM MODEL OF EARTHQUAKE GROUND MOTION FOR ENGINEERING SITE

Using a random Fourier function with adherent probability as the modelling form, the proposed model is to reflect the intrinsic relationship between the random seismic motion and the critical parameters.

#### 3.1 The physical relation and modeling

If searching from the mechanism of the seismic wave propagating through engineering site, earthquake ground motion can be regarded as a physical process, which includes being input from the bedrock base and filtered by the site. Accordingly, it is believed that because of the uncontrollability of the random factors including the afferent wave energy and the soil medium characteristics that the actual earthquake ground motions are observed with significant randomness as a result. The mentioned factors are supposed to be the energy factor of afferent wave, the periodic factor and the dissipative factor of seismic site. Correspondingly, these factors can be practically indicated by the following stochastic variables with operational physical meaning: the basal spectrum parameter, the free angular frequency and the damping ratio of engineering site. Consequently, a physical relation between random earthquake motion and critical factors can be constructed and modeled.

Without loss of generality, the soil layer can be simulated as an equivalent linear single-degree-offreedom system which is input by a seismic motion from the base, and then the absolute response of this system can be supposed to present the seismic ground motion process. Therefore, if this linear single-degree-of-freedom system is input with onedimension seismic motion, in frequency domain, the absolute acceleration response is expressed as

$$\ddot{Y}(\omega) = \frac{\omega_0^2 + i2\zeta_0\omega_0\omega}{\omega_0^2 - \omega^2 + i2\zeta_0\omega_0\omega} \cdot \ddot{U}(\omega)$$
(11)

where  $\omega$  is independent parameter, read as angularfrequency;  $\ddot{Y}(\omega)$  is the absolute output acceleration in frequency domain;  $\ddot{U}(\omega)$  is the absolute basal input acceleration in frequency domain;  $\omega_0$  is the natural angular-frequency of the site;  $\zeta_0$  is the damping ratio of the site.

According to the previous statement, the basal input motion and the medium characteristics of soil layer, which lead to the randomness of earthquake acceleration motion, are both of uncertainty. Therefore, the parameters of soil medium—the free frequency  $\omega_0$  and the damping ratio  $\zeta_0$  are apparently stochastic variables, noted as  $X_{\omega}$  and  $X_{\zeta}$  respectively. Assume random variable vector  $X_H = (X_{\omega}, X_{\zeta})^T$  to relate to the soil medium.

If the basal input Fourier spectrum  $\ddot{U}(\omega)$  is the function of stochastic variables  $X_1, \ldots, X_n, \ddot{U}(\omega)$  is defined with

$$\ddot{U}(\omega) = G(X_1, \cdots, X_n, \omega) \tag{12}$$

Assume random variable vector presenting the basal input Fourier spectrum to be  $\mathbf{X}_G = (X_1, \dots, X_n)^T$ , and assign random variable vector  $\mathbf{X} = (\mathbf{X}_H^T, \mathbf{X}_G^T)^T$  with the joint probability  $p_{\mathbf{X}}(x)$ . Then the random earthquake ground process described with acceleration can be expressed by a random Fourier spectrum function  $F(\mathbf{X}, \omega)$ , which is expressed as

$$\mathbf{F}(\mathbf{X},\omega) = \mathbf{H}(\mathbf{X}_{H},\omega) \cdot \mathbf{G}(\mathbf{X}_{G},\omega)$$
(13)

where 
$$H(\mathbf{X}_{H}, \omega) = \frac{X_{\omega}^{2} + i2X_{\zeta}X_{\omega}\omega}{X_{\omega}^{2} - \omega^{2} + i2X_{\zeta}X_{\omega}\omega}$$

#### 3.2 The basal input spectrum

If presented by power spectral density function, the bedrock seismic responses are generally assumed to be a band-limited white noise spectrum. However, a large amount of observed acceleration records have shown that the bedrock spectrum presents filtering and limited band character. Moreover, relevant analysis indicates that the frequency range greater than

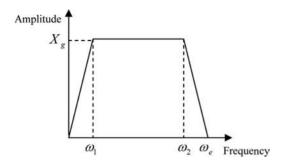
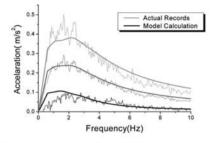
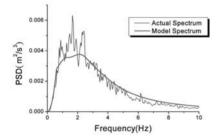


Figure 1. Basal Input Spectrum.



(a) Mean and Plus-Minus Single Standard Deviation of Fourier Amplitude Spectrum



(b) Power Spectral Density (PSD) Curves

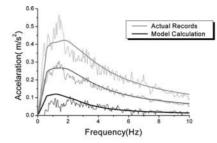
Figure 2. Contrast of the proposed random earthquake model and real records for Soil Type I.

94 Rad/s (15 Hz) can be neglected because of corresponding minute amplitude.

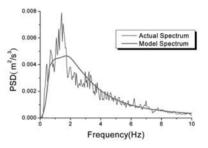
According to the preceding thoughts, a basal input spectrum is constructed (Li & Ai 2006), which is shown in Figure 1, where,  $\omega_1$ ,  $\omega_2$  are the control angular-frequency;  $\omega_e$  is the truncated angular-frequency;  $X_u$  is the random variable defining the amplitude of the basal input spectrum.

## 3.3 Random earthquake model basing on physical process

In order to determine the physical variables in the proposed random earthquake model, according to



(a) Mean and Plus-Minus Single Standard Deviation of Fourier Amplitude Spectrum



(b) Power Spectral Density (PSD) Curves

Figure 3. Contrast of the proposed random earthquake model and real records for Soil Type II.

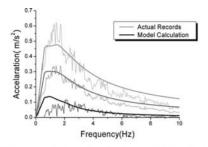
stochastic modeling theory, actual strong seismic records are regarded as a target sample aggregation and numerical methods are adopted to identify different distribution parameters of the variables corresponding to the proposed model (Li & Ai 2006). According to different site class, a collection of acceleration records, which chiefly come from western American strong seismic records, are collected and reorganized to establish different equivalent random earthquake model.

For different site class, the contrasts between actual seismic records and the proposed model are shown in Figures 2–5. The contrasts imply that the proposed model is shown of definite physical concept and propriety to reflect the variation of random seismic motion.

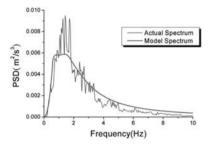
#### 4 STOCHASTIC SEISMIC ANALYSIS OF UNDERGROUND PIPELINE

#### 4.1 Computational example

The site is a  $50 \times 10$  m uniform saturated sandy soil space and the ground water is set at the ground surface. And the pipe made of cast iron is buried at the depth of 1m with the diameter is 0.4 m.

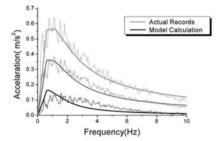


(a) Mean and Plus-Minus Single Standard Deviation of Fourier Amplitude Spectrum

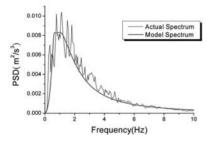


(b) Power Spectral Density (PSD) Curves

Figure 4. Contrast of the proposed random earthquake model and real records for Soil Type III.



(a) Mean and Plus-Minus Single Standard Deviation of Fourier Amplitude Spectrum



(b) Power Spectral Density (PSD) Curves

Figure 5. Contrast of the proposed random earthquake model and real records for Soil Type IV.

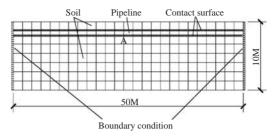


Figure 6. The Structural Mode.

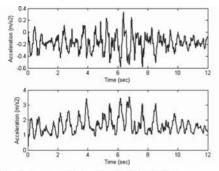
Figure 6 shows the finite element mode of the computational example. The structure is subjected to earthquake excitation in the shape of the conditional random earthquake function which is defined subsequently. The earthquake is input from the bottom horizontally with the propagation velocity 250 m/s and the duration is 20 s. The joint elements adopt the same form with the pipeline element, while different Young's module and density are defined.

For the deterministic analysis of the seismic response of buried pipelines, the finite element method is adopted to study the seismic response of buried pipelines and the surrounding soil. The soil that surrounds the pipeline is regarded as a solid-liquid two-phase medium. The effective stress method and nonlinear constitutive model of soil are used to study the increase and the dissipation of pore water pressure during the seismic process. At the same time, the contact interface between the pipeline and the surrounding soil is also included. Detailed techniques and the dynamic parameters of the sandy soil and the contact surface can be referred to the related paper of the authors (Ai & Li 2004).

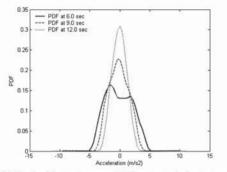
## 4.2 Stochastic seismic analysis of underground pipeline

In this paper, the random inputs and the uncertain soil parameter are both considered and defined according to the reference (Li and Ai 2006): associated with the seismic risk analysis, in the future definite time range, the conditional random earthquake function is defined as the case of the transcendental probability of the earthquake motion being p = 3% and the peak value of the seismic acceleration is 0.2 g; The internal friction angle  $\phi$ , which is defined as the random soil parameter, has a logarithmic normal distribution with the mean  $15^0$  and the standard deviation 0.30.

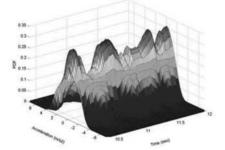
Under the proposed stochastic conditions, the probability density function (PDF) of the response for joint E is presented in Figure 7, including the mean and the standard deviation, typical instantaneous PDFs at certain instants of time, evolution of PDF against time and contour to the PDF surface. These indicate the



(a) The mean and the standard deviation



(b) Typical instantaneous PDFs at certain instants of time



(c) Evolution of PDF against time

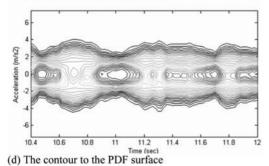


Figure 7. The PDF of the response for joint E.

stochastic fluctuation character of a nonlinear random response.

#### 5 CONCLUSIONS

A new approach is proposed for the stochastic seismic response analysis of finite element modeled underground pipeline structures under earthquake excitations and uncertain soil parameters. The approach is established based on the thoughts of the newly developed probability density evolution method. Associated with the physics-based stochastic earthquake model and the finite element method, the instantaneous probability density and the evolution process for the seismic response of underground pipeline can be studied effectively. A computational example is investigated with stochastic input and random soil parameters. Some features of the responses are observed and discussed. It is found that the proposed method is of high application in analyzing the stochastic response of underground pipeline and the stochastic seismic response for different structural forms requires further investigation. Furthermore, the seismic reliability problem can be investigated easily by imposing the failure criterion of the first passage problem reliability theory.

#### ACKNOWLEDGEMENTS

The support of the Natural Science Foundation of China for Innovative Research Groups (Grant No.50321803) is gratefully appreciated.

#### REFERENCES

- Ai, X.Q. & Li, J. 2004. Seismic response analysis of underground pipelines using effective stress method. Proceeding of 13th World Conference on Earthquake Engineering. Vancouver, Canada
- Deodatis, G. & Shinozuka, M. 1988. Stochastic FEM analysis of non-linear dynamic problems. In: Shinozuka M (ed). Stochastic Mechanics III, Department of Civil Engineering and Operations Research. Princeton University.
- Iwan, W.D. & Huang, C.T. 1996. On the dynamic response of non-linear systems with parameter uncertainty. *International Journal of Non-Linear Mechanics* 31(5): 631–45.
- Klosner, J.M., Haber, S.F. & Voltz, P. 1992. Response of nonlinear systems with parameter uncertainties. *International Journal of Non-Linear Mechanics* 27(4): 547–63.
- Li, J. & Ai, X.Q. 2006. Study on Random Model of Earthquake Ground Motion Based on Physical Process. *Earthquake Engineering and Engineering Vibration* (in Chinese) 26(5): 21–26.

- Li, J. & Chen, J.B. 2005. Dynamic response and reliability analysis of structures with uncertain parameters. *International Journal for Numerical Methods in Engineering* 62(2): 289–315
- Liu, W.K., Belytschko, T. & Mani, A. 1986. Probability finite elements for nonlinear structural dynamics. *Computer Methods in Applied Mechanics and Engineering* 56: 61–81.
- Machida, H. & Yoshimura, S. 2002. Probabilistic fracture mechanics analysis of nuclear piping considering

variation of seismic loading. International Journal of Pressure Vessels and Piping 79:193–202

- Nedjara, D., Hamanea, M., Bensafia, M., Elachachia, S.M. & Breysse, D. 2007. Seismic response analysis of pipes by a probabilistic approach. *Soil Dynamics and Earthquake Engineering* 27:111–115
- Spanos, P.D. & Zeldin, B.A. 1998. Monte Carlo treatment of random fields: a broad perspective. *Applied Mechanics Review* 51(3): 219–37.

#### Risk assessment for the safe grade of deep excavation

#### X.H. Bao & H.W. Huang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: This paper embarks from risk idea, obtains safety factors of excavation and their percentages from investigation results of excavation accidents, the safe grade of excavation is designed and analyzed. First, the present developing situation, possibly existed risks in excavation projects and the state of present excavation grade division in shanghai area is briefly introduced. Reasons of accidents are obtained from analyzing a large number of excavation accidents. Later, fuzzy synthetic evaluation method is introduced and two levels of judgment on excavation safety are carried on. Finally, the paper takes the example of shanghai international passenger transport centre, the results conform to the actual situation.

#### 1 INTRODUCTION

## 1.1 The present risk study of deep excavation engineering

In recent years, excavation engineering develops quickly, and excavations are becoming larger and larger. The emergence of these large excavations has brought new opportunities and challenges to engineering design, construction and management. On the other hand, a great many of engineering practices helped technical development, improved management level, also made the risk analysis apply in many aspects of engineering. Currently, most of the excavations are mainly designed according to the specifications, rules and accepted practice. However, excavations can be designed according to the risk idea that is, to study latent risk of excavation during the period of construction and operation, and reduce the unavoidable risk to the insignificant level by increasing construction cost. This method bases on the principles of reliability; it can make the probability of reaching limit state small enough, and reduce the excavation risk to acceptable level according to risk acceptance criteria. The research in this aspect at home and abroad is still at a beginning stage, specialized research of risk concerning deep excavation engineering is seldom, the results obtained are almost qualitative, quantificational research is seldom, applicable risk analysis and evaluation methods that are both qualitative and quantificational are needed (Huang and Bian, 2005), so it is difficult to construct probability models of risk factors.

## 1.2 The present grade division and risk factors in excavation

There are still some problems in present grade division both at home and abroad. For example, uncertainty of various factors which can influence the result, small applicability scope of calculation theories and so on. These will make construction cost be increased. Currently, the grade of excavation is divided according to deformation values (Specification for Excavation in Shanghai Metro Construction, SZ-08-2000). Every grade has its control values. When the grade division is not precise, if higher which means the safety warning value is smaller, it will forecast the safe condition as dangerous condition, then the excavation requests high rigidity retaining structures, also more bracings and bottom strengthening may be needed to resist deformation. Actually in this condition, investment has been increased. If lower which means the safety warning value is larger, the condition of excavation has already been dangerous before forecasting, it may be too later to remedy, so it can cause great loss and the work may be delayed.

The characteristics of excavation engineering mean that all owners have to face huge risk during construction. As the soil layer conditions and groundwater circumstances are uncertain, technologies are complex, man may make mistakes in techniques and managements, accidents may always happen in disadvantageous soil layers, so, there would not only exist environment risk, but also other risks such as overspending and extension for completion date, etc. The serious collapses appeared recently have revealed that

Table 1. Statistics of reasons for deep excavation accidents.

Reason	frequency	Percentage (%)
Investigation	16	4.65120
Design	125	36.3372
Construction	176	51.1628
Supervision	5	1.45350
Monitor	10	2.90700
Owner	12	3.48840
Total	344	100.000

Table 2. Retaining structures for the failed deep excavations.

Retaining method	frequency	Percentage (%)
Row piles	150	66.3717
Soil-nailing support	30	13.2743
Deep mixing pile	13	5.75220
Diaphragm	21	9.29200
Excavated slope	5	2.21240
Soil nailed wall	5	2.21240
Others	2	0.88500
Total	226	100.000

accidents can cause many big troubles in the excavation engineering. The excavation constructed in cities can cause life and property loss to the third. The social problems and public protest caused by excavation can extend construction period (International Tunneling Association, Second Unit, Guiding Rules of Tunnel Risk Processing, 2002), so special attention should be paid.

#### 2 RISK ASSESSMENT FOR THE SAFE GRADE OF DEEP EXCAVATION

#### 2.1 Reasons for deep excavation accidents

There are numerous reasons for deep excavation accidents, and a lot of scholars have papers in this aspect. The following statistics are gotten from about 300 deep excavation accidents (h > 6 m).

From table 1 it can be seen that construction and Design are the main reasons. On the other hand, consider 226 accidents of these failed excavations; divide them according to retaining method in table 2. Most of them are retained by row piles. Although there may be problems in this division, and it can not be said that row pile is more damageable than other kinds of retaining structures, but accidents happened frequently, so more attention should be paid on excavations retained by row piles.

#### 2.2 Fuzzy synthetic evaluation method

The fuzzy synthetic evaluation process (Liang and Bi, 2001) is as follows:

- U is influence-factor set,  $U = \{U_1, U_2, \dots, U_m\}$ ,  $U_m(i = 1, 2, \dots, m)$  is the *i* factor, in the secondary synthetic evaluation,  $U_i = \{U_{i1}, U_{i2}, \dots, U_{in}\}$ ,  $U_{in}$  is the sub-factor of  $U_i$ .
- V is comment set,  $V = \{V_1, V_2, ..., V_p\}, V_j (j = 1, 2, ..., p)$  is the j grade.
- Construct evaluation matrix R

$$R = (R_1, R_2, \dots, R_m)^T = \begin{bmatrix} r_{11} & r_{12} & \cdots & r_{1p} \\ r_{21} & r_{22} & \cdots & r_{2p} \\ \vdots & \vdots & & \\ r_{m1} & r_{m2} & \cdots & r_{mp} \end{bmatrix}_{mxp}$$
(1)

Where *R* is evaluation matrix of single factor,  $r_{ij}$  is the relative membership of  $U_i$  to  $V_j$ ,  $R_i$ (i = 1, 2, ..., m) is subordination vector.

- *A* is weight set of main factors,  $A = (a_1, a_2 \dots a_m)$ ,  $a_i$  is the importance degree of  $U_i$  compared to the other factors,  $0 \le a_i \le 1$ .
- Choose composite operators and get composite evaluation result *B* by multiplying *A* and *R*:

$$B = A \times R = (b_1, b_2, \dots b_p) \tag{2}$$

Where *B* is subordination vector of main factor to comment congregation, *R* is evaluation matrix of single factor,  $b_i$  subordination vector.

Analyze results

#### 2.3 Influence factors

Considering investigations and analysis of other researches (Yang and Ding, 1998), this paper considers five main factors that have influence on excavation safety: size of foundation pit  $U_1$ , hydrogeology  $U_2$ , design  $U_3$ , construction  $U_4$  and surrounding environment  $U_5$ . Size of foundation pit includes area  $U_{11}$ , shape  $U_{12}$ , depth  $U_{12}$ . Hydrogeology includes layer condition  $U_{21}$ , confined water  $U_{22}$ , drifting sand  $U_{23}$ . Design includes calculation method  $U_{31}$ , value of parameter  $U_{32}$ , value of load  $U_{33}$ , material selection  $U_{34}$  and retaining structure  $U_{35}$ . Construction includes if according to design and rules  $U_{41}$ , construction method  $U_{42}$ , construction experience  $U_{43}$ , dewatering  $U_{44}$ , supporting  $U_{45}$ . Surrounding environment includes dynamic load  $U_{51}$ , surcharge load  $U_{52}$ , adjacent construction  $U_{53}$ . In fuzzy synthetic evaluation, these factors are also called indexes; the membership between factors and their sub-factors (indexes system) is shown in figure 1.

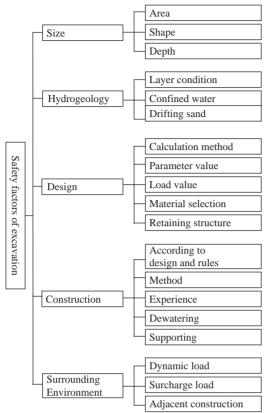


Figure 1. Factors influencing the safe grade of deep excavation.

Table 3. Weights of  $U_i$ .

Number	Influence factors	Weights
1	Size	0.038
2	Hydrogeology	0.046
3	Design	0.360
4	Construction	0.510
5	Surrounding environment	0.046

#### 2.4 Weights of factors

#### 2.4.1 Weights of the factors $U_i$

From the investigation of 344 excavation accidents (h > 6 m), the following results can be gotten as shown in table 3.

Use the Analytic Hierarchy Process. First, construct a judgment matrix to calculate the maximum Eigen value of the matrix and vector feature, and then test the consistency of judgment matrix to determine the weights of sub-factors (Wang and Huang, 2005).

Table 4. The meaning of 1~9 scale.

Scale	Meaning(compare two factors)
1	The important degree is equal
2	The former is slightly important
3	The former is obviously important
4	The former is mightily important
5	The former is extremely important

\* 2, 4, 6, 8 mean the middle value of two scales close together.

Table 5. Judgment matrix  $\overline{U_1}$ .

$U_1$	$U_{11}$	<i>U</i> <sub>12</sub>	U <sub>13</sub>
$U_{11}$	1	3	1/3
$U_{12}$	1/3	1	1/3 1/5
$U_{11} U_{12} U_{13}$	3	5	1

 $U_2, U_3, U_4, U_5$  can be gotten in the same way

#### 2.4.2 Construct judgment matrix

Judgment matrices can be constructed using Expert Grades method, according to T.L. Satty's 1~9 scale.

 $U_{ii}$  is subordinated to factor  $U_i$ , compare mutually the importance degree of every sub-factor  $U_{ii}$ , ratio scale  $u_{ii}$  can be gotten, it reflects the relative importance of two sub-factors, if the first sub-factor compares to the second sub-factor and the result is  $u_{ii}$ , then the second compares to the first and the result is  $u_{ij} = 1/u_{ij}$ , so judgment matrix  $\overline{U_1} = (u_{ij})_{n \times n}$  can be gotten as follows:

#### 2.4.3 Weights of sub-factors

As it does not request high accuracy, to be simple, calculate the maximum Eigen value  $\lambda_{max}$  of the matrix  $U_{\rm i}$  and the feature vector A, and A is also the weight set of sub-factors. The calculation process of A and  $\lambda_{max}$ is as follows:

Elements in the matrix multiply each other in rows and open a n (n=3) power to get  $\overline{A_{1i}}$ 

$$\overline{A}_{1i} = \sqrt[3]{\sum_{j=1}^{3} u_{ij}}$$
(3)

So,  $\overline{A_{11}} = 1$ Normalize  $\overline{A_{1i}}$  to get  $A_{1i}$ :

$$A_{1i} = \frac{\overline{A_{1i}}}{\sum_{i=1}^{3} \overline{A_{1i}}}$$
(4)

So,  $A_{11} = 0.296$ 

Using the same method,  $A_{12}$ ,  $A_{13}$  can be gotten, and  $A_1 = (0.296 \ 0.120 \ 0.584), A_2 = (0.120 \ 0.584 \ 0.296),$ 

Table 6. Average random consistency index R.I.

Order of matrix	1	2	3	4	5
<i>R.I</i> .	0	0	0.52	0.89	1.12

 $A_3 = (0.104 \ 0.246 \ 0.104 \ 0.045 \ 0.501), A_4 = (0.203 \ 0.086 \ 0.466 \ 0.203 \ 0.042), A_5 = (0.143 \ 0.714 \ 0.143).$ 

The largest Eigen value calculation:

$$\lambda_{1\max} = \sum_{i=1}^{3} \frac{(U_1 A_1)_i}{3A_{1i}}$$
(5)

So,  $\lambda_{1 \max} = 3.072$ ,  $\lambda_{2 \max} = 3.072$ ,  $\lambda_{3 \max} = 5.125$ ,  $\lambda_{4 \max} = 5.125$ ,  $\lambda_{5 \max} = 3.001$ 

#### 2.4.4 Test of consistency

On account of fuzziness of many factors and people's understanding is different because of subjectivity, so, to overcome these subjective errors, the consistency of judgment matrix should be tested. When stochastic *CR* satisfies: *C.R.*  $\leq$  0.1, through Saaty's *CR* standard, the direction and extent of the improvement would be controlled, and the optimal matrix with acceptable consistency could be available, that had preserved furthest the decision maker's primitive judgment information.

Consistency index C.I.

$$C.I. = \frac{\lambda_{\max} - n}{n - 1} \tag{6}$$

So,  $C.I_{.1} = 0.036$ ,  $C.I_{.2} = 0.036$ ,  $C.I_{.3} = 0.031$ ,  $C.I_{.4} = 0.031$ ,  $C.I_{.5} = 0.0005$ 

Check table 6 (Gong and Xu, 1986) to get the corresponded average random consistency index R.I.

The average random consistency indexes *R.I.* of every matrix are: 0.52 0.52 1.12 1.12 0.52

Consistency ratio C.R.

$$C.R. = \frac{C.I.}{R.I.} \tag{7}$$

So,  $C.R._1 = 0.069 < 0.1$ ,  $C.R._2 = 0.069 < 0.1$ ,  $C.R._3 = 0.028 < 0.1$ ,  $C.R._4 = 0.028 < 0.1$ ,  $C.R._5 = 0.001 < 0.1$ , the consistency of judgment matrices is acceptable.

#### 2.5 Comprehensive evaluation and results analysis

Fuzzy synthetic evaluation method is a kind of very valid method that can completely evaluate the things which are influenced by various factors, so, it can be used largely in many projects and other systems. This paper carries on two levels of judgment to the excavation safety, first primary fuzzy synthetic evaluation of sub-factors, and then secondary fuzzy synthetic evaluation of main factors.

#### - Comments set

Generally, the deep excavation safety can be divided into 4 grades: very safe, safe, less safe, unsafe, these can reflect the safety condition of deep excavation, so the comments set  $V = \{$ very safe, safe, less safe, unsafe $\}$ . To calculate simply, quantify the comments set, then  $V = \{4 \ 3 \ 2 \ 1\}$ .

#### - Construct evaluation matrix R

Obtain the relative membership degree of 19 factors to comment set V and evaluation matrix R by using Delphi Method. Invite 10 experts who know well about the excavation as an evaluation group, and then get marks of all indexes from experts. The steps are as follows:

Invite 10 experts to evaluate, the factor sets are  $U_1 = \{U_{11}, U_{12}, U_{13}\}; U_2 = \{U_{21}, U_{22}, U_{23}\}; U_3 = \{U_{31}, U_{32}, U_{33}, U_{34}, U_{35}\}; U_4 = \{U_{41}, U_{42}, U_{43}, U_{44}, U_{45}\}; U_5 = \{U_{51}, U_{52}, U_{53}\}.$  The relative membership degree is  $r_{ij}: 0 \le r_{ij} \le 1$ , the size of  $r_{ij}$  shows the influence degree of the factor on "safe grade of deep excavation", and if bigger, more close to the corresponding grade, conversely, has little influence on the corresponding grade. For the index  $U_{ij}$ , if from 10 experts,  $m_i$  experts consider the comment is  $v_1, n_i$ experts consider the comment is  $v_2, p_i$  experts consider the comment is  $v_3, q_i$  experts consider the comment is  $v_4$ , then the subordination vector  $R_{ij}$  of the index  $U_{ij}$  is:  $R_{ij} = (r_{i1} \quad r_{i2} \quad r_{i3} \quad r_{i4}) = (m_{i1}/10 \quad n_{i2}/10 \quad p_{i3}/10$  $q_{i4}/10), i = 1, 2, 3, 4, 5.$ 

Evaluation matrices for the five main factors can be constructed by using subordination vectors as rows. So,  $R_1 = (R_{11} \ R_{12} \ R_{13})^T$ ,  $R_2 = (R_{21} \ R_{22} \ R_{23})^T$ ,  $R_3 = (R_{31} \ R_{32} \ R_{33} \ R_{34} \ R_{35})^T$ ,  $R_4 = (R_{41} \ R_{42} \ R_{43} \ R_{44} \ R_{45})^T$ ,  $R_5 = (R_{51} \ R_{52} \ R_{53})^T$ .

#### - Primary and secondary fuzzy synthetic evaluation

The paper uses weighted averaging fuzzy synthetic evaluation model which can consider all factors and single-factor evaluation results.

For the primary fuzzy synthetic evaluation, the weighted averaging fuzzy synthetic model is:

$$R = A \times R \tag{8}$$

Where A' is the weight vectors of sub-factors, R' is single-factor evaluation matrix and R subordination vector of sub-factor to comment congregation.

So, for every single-factor:  $R_i = A_i \times R'_i$  (*i*=1,2,3,4,5) For the secondary fuzzy synthetic evaluation:

$$B = A \times R \tag{9}$$

Where *A* is weight set of main factors, *R* is the secondary evaluation matrix that is made up of  $R_j$ , *B* is subordination vector of main factor to comment congregation.

So, for each single-factor:  $B_i = A_i \times R_i$  (*i*=1,2,3,4,5)

- The final value of comprehensive evaluation

For the grade of deep excavation, we use the comments set  $V = \{very \text{ safe, safe, less safe, unsafe}\}$ , after qualification,  $V = \{4 \ 3 \ 2 \ 1\}$ . *W* is the evaluation value of deep excavation grade.

$$W = B \times V \tag{10}$$

So, for every single-factor:  $W_i = B_i \times V(_{i=1,2,3,4,5})$ If.

3.5< <i>W</i> <4.0,	very safe;
2.5 <w<3.5,< td=""><td>safe;</td></w<3.5,<>	safe;
1.5 < W < 2.5,	less safe;
0.0 < W < 1.5,	unsafe.

#### 3 THE EXAMPLE

#### 3.1 The general situation of the project

Shanghai International Passenger Transport Centre (west area) locates in south of east Daming Road, west of Liyang Road, east of Gaoyang Road, north of Huangpu River. The project covers an area of 128400 square meters, the shape of foundation pit is rectangular and the depth is 13.10 meters.

#### 3.2 Primary fuzzy synthetic evaluation

The membership of the factors and their sub-factors is known in fig. 1.

#### 3.2.1 Construct judgment matrix

Use Delphi Method. From 10 experts, for the index  $U_{11}$ : the area of this foundation pit, no expert considers it very safe for the project, 3 experts consider it safe, 6 experts consider it less safe, and 1 expert considers it unsafe. So, the comment score to different safe grades are: 0, 3, 6, 1, so the subordination vector  $R_{11} = (0/10 \ 3/10 \ 6/10 \ 1/10) = (0 \ 0.3 \ 0.6 \ 0.1)$ . After getting the other subordination vectors, the evaluation matrix for  $U_1$  is as follows:

$$R_{1} = (R_{11} \quad R_{12} \quad R_{13})^{T} = \begin{bmatrix} 0 & 0.3 & 0.6 & 0.1 \\ 0 & 0.3 & 0.6 & 0.1 \\ 0 & 0.6 & 0.3 & 0.1 \end{bmatrix}$$

#### 3.2.2 Primary evaluation

As is known, the weights vectors of the sub-factors to main factor  $U_1$  is:  $A_1 = (0.296 \ 0.120 \ 0.584)$ ,  $B_1 = A_1 \times R_1 = (0 \ 0.475 \ 0.425 \ 0.1)$ 

So, the evaluation value for single-factor  $U_1$  is:  $W_1 = B_1 \times V = 2.375$ , in the same way,  $W_2 = 1.978$ ,  $W_3 = 2.893$ ,  $W_4 = 2.533$ ,  $W_5 = 3.158$ .

Table 7. The safety grade of each main factor.

Factor	Value	Grade
Size Hydrogeology Design Construction Surrounding environment	$\begin{array}{c} 1.5 < W_1 = 2.375 < 2.5 \\ 1.5 < W_2 = 1.978 < 2.5 \\ 2.5 < W_3 = 2.893 < 3.5 \\ 2.5 < W_4 = 2.533 < 3.5 \\ 2.5 < W_5 = 3.158 < 3.5 \end{array}$	less safe less safe safe safe safe

#### 3.2.3 *Secondary evaluation* The comprehensive evaluation matrix is:

$$R = (B_1, B_2, B_3, B_4, B_5)^T = \begin{bmatrix} 0 & 0.475 & 0.425 & 0.1 \\ 0 & 0.312 & 0.354 & 0.334 \\ 0.204 & 0.485 & 0.311 & 0 \\ 0.025 & 0.558 & 0.342 & 0.075 \\ 0.343 & 0.486 & 0.157 & 0.014 \end{bmatrix}$$

As the weights for five main factors are known,

 $A = (0.038 \ 0.046 \ 0.360 \ 0.510 \ 0.046)$ , so, the subordination vector *B* with assessment target *U* to comment set *V* is:

$$B = A \times R = (0.102 \quad 0.514 \quad 0.326 \quad 0.058)$$
  
 $W = B \times V = 2.660$ 

#### 3.2.4 Evaluation results analysis

For the five main influence factors of the deep foundation pit, the safety grade division is in table 7:

Final comprehensive evaluation value:

2.5<W=2.660<3.5

Ì

So on the whole, the situation of this deep excavation is safe; the system safety can be basically accepted. But according to the evaluation values for single-factors, the size and hydrogeology are disadvantageous for the whole stability. To prevent this deep excavation from accident, some measurements should be taken during construction, for example proper retaining method, good dewatering measure, timely monitoring, etc.

The size and hydrogeology evaluation values can be good advice to managers when they make decision. As the size and hydrogeology are disadvantageous for this excavation safety, so they are considered to be two risk factors, then during the project management process, risk management can be carried on to decrease the probability of failure that may be caused by these two risk factors. Through the master and supervision of risk factors, managers can adopt active measures and scientific management.

#### 4 CONCLUSION

In deep excavation engineering, using fuzzy synthetic evaluation method can quantify qualitative analysis and make inaccurate expression become numeral, so the evaluation process is more scientific. The safety influence factors in deep excavation engineering are various; the paper obtains the safe factors and its weights from the investigation results of excavation accidents, so the safe grade evaluation is more scientific and rational.

In the primary evaluation, the results are advantageous for discovering the trouble and weakness that lurked in engineering; they can provide decision to managers in order to do prevention, so the whole safety level of deep excavation is raised.

The example demonstrates that the results conform to the project's actual situation; and they can help raising the overall safety level of the excavation and provide some reference values for the projects. Thus this paper has proved that the Fuzzy Synthetic Evaluation, Analytic Hierarchy Process and Delphi Method are favorable in deep excavation engineering.

#### ACKNOWLEDGEMENT

The author is very grateful of Doctor Y.H. Bian for the investigation results of deep excavation accidents.

#### REFERENCES

- Guo, Z.W. 1986. Risk Analysis and Decision. Beijing: China Machine Press.
- Huang, H.W. & Bian, Y.H. 2005. Risk management in deep excavation construction. *Chinese Journal of Underground Space and Engineering* 1(4): 611–614.
- Liang, S. & Bi, J.H. 2001. Fuzzy synthetic evaluation method of construction engineering quality grade. *Journal of Tianjin University* 34(5): 664–669.
- Shanghai Metro Construction Corporation LTD, SZ-08-2000. Specification for Excavation in Shanghai Metro Construction. Shanghai.
- Soren, D.E., Per, T. & Jorgen, K. 2004. Guidelines for tunneling risk management: *International* Tunneling Association, Working Group No. 2. *Tunnel and Underground Space Technology* 19(2004): 217–237.
- Wang, Y. & Huang, H.W. 2005. Hierarchy—fuzzy synthetic safety evaluation method of tunnel in subway. *Journal of Underground Space* 24(3): 301–305.
- Yang, Z.M. & Ding, X.G. 1998. Safety accident and reason analysis of excavation engineering. *Safety Regulations for Geological Prospecting Operation* 5(1): 10–13.

#### Multi-factors durability evaluation in subway concrete structure

C. Chen & L. Yang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

#### C. Han

School of Economics & Management, Tongji University, Shanghai, P.R. China

ABSTRACT: In this paper, an erosion model of subway concrete lining structure was proposed, then factors which the influence durability of subway structure were determined, and the framework of evaluating measure index was established. Finally, Fuzzy Multiple Attribute Decision Making was used to carry out durability prediction model on the basis of the single factor evaluation in the subway structure. Through case study, the durability of subway structure is evaluated by the methods in this article and get good or bad schemes.

#### 1 INTRODUCTION

Shanghai Subway belongs to the Fast Orbit Transportation System in the city. Because of such characteristics as being free from weather influence and being safe and fast, it becomes the key vehicle within network of the passenger transportation (Shen et al. 1998). The subway structure is a permanent concrete structure, so the investment is usually huge, and it always crosses key districts of city. The requirement of durability of subway is higher than that of other engineering (the base period for design is 100 years), and the problem of durability is of most urgency.

The factors influencing concrete structural durability are numerous and very complicated, and they are interrelated and influence on each other. Therefore, it has two kinds of manifestations: one is the uncertainty of the occurrence of an affair, and the other is the uncertainty of the behavior of the affair. The existing evaluation methods of concrete durability mostly evaluate the influence of a single factor by setting up a model. However, actually, the concrete structure durability is a comprehensive function containing many factors, and its evaluation is a compound decision process of multiple attributes, multiple factors and multiple indexes. That is to say, the evaluation of concrete durability is a FMADM (Fuzzy Multiple Attribute Decision Making) (Li 2002, Leung & Cao 2000) problem. In view of this problem, FMADM is to be used to carry out the multiple factors evaluation on the basis of the single factor evaluation in the subway structure (Heilpern 1997, Zimmermann 1987).

#### 2 ANALYSIS OF SUBWAY STRUCTURE DURABILITY

#### 2.1 Some concepts

The durability of concrete structure is the capability of the structure which keeps its safety, function and requirement of appearance under suitably maintaining condition during the base period of design where the effect of the environment must be considered. Here, the environment is a general concept, which contains any factor resulting in degradation and destruction of the performance of the structure.

Therefore, the durability of concrete structure is a integrated functional index which is concerned with not only raw materials and proportioning of concrete but also mechanics environment, natural environment, service environment and production technology of concrete (Tan 2003). When studying the durability of concrete structure, two states, which include the limit of bearing capacity and the limit of function and appearance requirement, should be considered. Namely, it is better to obtain economical and rational life-span on the premise of ensuring the structure safety.

#### 2.2 Influencing-factors of subway structure durability

There are a few factors which can modify the durability of concrete structure. When proportioning has been confirmed, the maximal modifying factor to durability is its environment. Here, the factors of environment mainly include carbonation, chloride ions ingress, sulfate attack and cycle of freezing and thawing (Du & Zhang 2003).

The erosion model of concrete lining structure is shown as Figure 1.

The concrete lining of subway structure is subjected to interactive action of multi-destroy factors that maybe come from interior environment of atmosphere or form exterior corrosion of geotechnical medium. Obviously, durability of subway structure should be analyzed on the basis of its circumstance as it is. Compared with other's concrete structure, the subway structure has two prominent characters. Firstly, the temperature of subway structure changes very little because it is long-term located under the surface of earth. Therefore, the probability of destruction of subway structure is very little owing to the action of cycle of freezing and thawing. Secondly, the subway

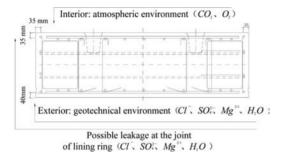


Figure 1. Erosion model of concrete lining structure.

is operated by the traction of electric power. As a result of this, it produces stray electrical current which plays a very important role in causing the corrosive destruction in subway structure.

Therefore, the influencing-factors of durability of subway structure mainly include stray current corrosion, chloride ions ingress, sulfate attack and carbonation. The life-span of durability of structure is a function of these influencing-factors.

#### 2.3 Evaluating measure index framework of subway structure durability

As described previously, the influencing-factors of durability of subway structure mainly include stray current corrosion, chloride ions ingress, sulfate attack and carbonation. Actually, each factor interact with many others relative factors. Due to lack of space, this paper analyzes only these four factors described previously.

Table 1 shows the evaluation measure index framework of durability of subway structure.

#### 2.4 Evaluation of subway structure durability

Durability of subway structure is a comprehensive function containing many factors, such as stray current corrosion, chloride ions ingress, sulfate attack and carbonation (Figure 1). At the same time, the single factor is always the function of other influential factors (Table 1).

$$L_{cr} = F(\varphi_{1i}, \varphi_{2i}, \varphi_{3i}, \varphi_{4i}) \quad i = (1, 2, ..., n)$$
(1)

Table 1. Evaluation measure index framework of durability of subway structure.

Objective level	Primary criteria level	Secondary criteria level	Operational level
Durability of Subway	Stray Current Corrosion	Coltage factor	Load current of train; Contact potential difference; Transformer substation spacing
Structure		Resistance factor	Electrical resistivity of concrete; Transition resistance; Longitudinal resistance of rail
	Chloride ions Ingress	Concrete characteristic	Dosage and variety of cement; Variety and graded of mixture; Admixture variety; Water cement radio; Air content; Curing time; Hydration degree
		Diffusivity Coefficient	Environmental temperature and humidity; Open-assembly time; Chloride ion concentration; Stress state; Crack
	Sulfate Attack	Exterior factors	Sulfate ion concentration; Magnesia ion; Chloride ion; PH value; Alternation of wetting and drying; Freezing and thawing cycle; Stress state; Crack
		Interior factors	Dosage and variety of cement; Variety and graded of mixture; Water cement radio; Admixture variety; Pore content and distribution
	Carbonation	Exterior factors	Light and temperature; Relative humidity; co2 concentration; Stress state; Crack
		Interior factors	Dosage and variety of cement; Variety and graded of mixture; Water cement radio; Compressive strength of concrete; Construction quality; Curing method

where  $L_{cr}$  is the durability of subway structure,  $\varphi_1$  is the influential factor of stray current corrosion,  $\varphi_2$  is the influential factor of chloride ions ingress,  $\varphi_3$  is the influential factor of sulfate attack,  $\varphi_4$  is the influential factor of carbonation, *i* is the influential factor index of the single factor.

The durability evaluation of a single factor is related to the same subject that influences the durability of subway structure, and it is the basis of the durability evaluation containing multiple factors. However, because of the different damage mechanism of each single factor, so, each factor's evaluation model and evaluation method are different, and the influential degree to durability of subway structure of each factor is of difference.

For example: the sulfate attack is the entrance of  $SO_4^{2-}$  from external corrosion media into the inner concrete through the surface of the concrete, and if chemical reaction happens between  $SO_4^{2-}$  and substance of the cement, such as Ca(OH)<sub>2</sub>, then the chemical reaction will generate Aft, which is of distensibility and of hazard to concrete by causing expansive cracks, and the concrete is in fragile and loose state. Therefore, the procedure of sulfate attack is from the external to the interior, and the damage degree deepens with the corrosion depth. However, chloride ions ingress through the concrete surface and the permeation is of little damage to concrete. But, the result of the permeation changes the alkali environment of the concrete, which makes the passivating film of concrete structure disappear to induce the rust-eaten of bars, thus, the concrete structure is damaged. Therefore, the damage procedure of chloride ions ingress is from the interior to the external. The damage degree is slight in initial state, and develops in geometric series with the time.

Durability of subway structure is the comprehensive interaction of multiple factors, and it is insufficient to analyze the issue from the action of the single factor or the simple perspective of single factors. Because of the fuzziness of many factors, it is not suitable to evaluate it with an accurate mathematic model. Therefore, to guarantee the objectivism of the evaluation of subway structure, it is of ultimate caution to choose a suitable evaluation method.

On the basis of the Fuzzy Multiple Attribute Decision Making theory, the article sets up fuzzy multi-factors evaluation model for durability of subway structure applying the method AHP and expert investigation to determine the weight of each factor.

#### 3 FUZZY MULTIPLE ATTRIBUTE DECISION MODEL (FMADM)

Fuzzy Multiple Attribute Decision Model (FAMDM) is a comprehensive analysis method which can solve

multi-factors and indefinite problems. In this paper, the author use fuzzy multiple attribute decision model to evaluate the durability of subway structure. The steps are described as follows:

- 1. Suppose the collection of evaluating scheme  $X = \{X_1, X_2, X_3, \dots, X_m\}$ , which X are m dimensions pending optimum schemes. Determine the index vector  $G_m = \{G_{1m}, G_{2m}, \dots, G_{nm}\}^T$  if each scheme has n dimensions evaluating index.
- Determine attribute value matrix of scheme collection *G<sub>ij</sub>(i = 1, 2, ..., n, j = 1, 2, ..., m)*, which is abbreviated as *r* = (*r<sub>ij</sub>*)<sub>*n*×m</sub>.
- 3. Transform attribute value matrix to relative grade of membership matrix.

Multi-targets decision-making is lack of commensurability. Namely, each target hasn't unified standard of measurement and so they can't be compared each other. For compared easily, the value of attribution should be quantified and changed them all into [0, 1]. Based the type of target, different quantified method should be chosen. The target can be divided the following types: the cost type (the smaller one is better one), the benefit type (the larger one is better one), the moderation type (appropriate one is better) and interval type (be located in stable zone is better).

Here the benefit type is used as quality matrix of target attribute of durability of subway structure. (The life-span of durability is older, the effect is better).

$$R = (R_{ij})_{m \times m} = \begin{cases} 1 & r_{ij} \ge M_k \\ \frac{r_{ij} - m_k}{M_k - m_k} & r_{ij} \in d_k \\ 0 & r_{ij} \le m_k \end{cases}$$
(2)

where  $m_k = \text{minimum}$  eigenvalue;  $M_k = \text{maximum}$  eigenvalue; and  $d_k = [m_k, M_k]$ 

The base period for subway design is 100 years. Hence, here one gets  $m_k = 100$ . What's more, considering economical efficiency and demand of sustainable development in urban construction synthetically, here one gets  $M_k = 150$ .

- 4. Ascertain the weight of each factor, the weight vector of attribution is  $(w_1, w_2, \dots, w_n)^T$ , here  $\sum_{j=1}^n w_j = 1$ , and  $w_j \ge 0$ . The weight vector is ascertained by the method of arrangement analysis, which can be seen in 4.4 in details.
- 5. Calculate the equation:

$$A_i = W \bullet R = (a_1, a_2, \dots a_m)$$
 (3)

 $A_i$  is the collection of evaluating result.  $a_j(j = 1, 2, \dots, m)$  is the fuzzy evaluating value of each

	Primary material content of concrete (kg/m <sup>3</sup> )						Attribute of influencing-factors of durability (years)				
Scheme Number	Cement	Water	Sand	Cobble $(5 \sim 25 \text{ mm})$	Fly ash	Mineral Powder	HLC	Stray Current Corrosion	Chloride ions ingress	Sulfate attack	Carbonation
D300	339	165	784	1148	0	0	45	114	110	163	112
D310	254	161	713	1138	108	0	45	126	159	144	100
D320	138	159	718	1051	0	228	45	134	134	119	150
D331	163	150	703	1080	110	110	45	143	150	122	113
D332	110	157	712	1062	110	148	45	180	173	135	118

Table 2. Alternative scheme of concrete proportioning and attribute of influencing-factors of durability.

\* D300: Primary Standard Concrete D310: Concrete Containing Fly Ash D320: Concrete Containing Mineral Powder D331 ~ D332: Concrete Containing Fly Ash and Mineral Powder

HLC: crack-resistance and seepage-proofing agent

scheme. The superior or inferior rank of schemes is estimated by the evaluating value (0-1), and the evaluating value is larger, the scheme is better.

#### 4 ILLUSTRATIVE EXAMPLE

Here are 5 Alternative schemes for researching high performance concrete. Through the researching to a series of experiments and correlative durability works, the life-span of durability of these schemes is obtained under the action of stray current corrosion, chloride ions ingress, sulfate attack and carbonation.

The Alternative schemes of concrete proportioning and the researching attribute of the life-span of durability under the action of each factor individually can be shown in Table 2.

## 4.1 *The collection of decision-making evaluation schemes and collection of attributions*

As analyzed previously, the collection of decisionmaking evaluation schemes can be described as:  $X = \{A, B, C, D, E\}$ ; the collection of attribution can be described as:

 $G = \{G_1, G_2, G_3, G_4\}^T = \{\text{Stray Current Corrosion, Chloride Ions Ingress, Sulfate Attack, Carbonate}\}^T$ 

## 4.2 *The evaluating attribution matrix of subway durability*

The value of attribution can be described as following matrix:

$$r = (r_{ij})_{m \times n} = \begin{vmatrix} 114 & 126 & 134 & 143 & 180 \\ 110 & 159 & 134 & 150 & 173 \\ 163 & 144 & 119 & 122 & 135 \\ 112 & 100 & 150 & 113 & 118 \end{vmatrix}$$

## 4.3 The relative quality matrix of evaluation system of subway durability

According to formula (2), the relative quality matrix of evaluation system of subway durability is written as:

$$R_{ij} = u(r_{ij}) = \begin{vmatrix} 0.28 & 0.52 & 0.68 & 0.86 & 1 \\ 0.2 & 1 & 0.68 & 1 & 1 \\ 1 & 0.88 & 0.38 & 0.44 & 0.7 \\ 0.24 & 0 & 1 & 0.26 & 0.36 \end{vmatrix}$$

## 4.4 *The weight of each factor in the evaluation scheme*

The ascertainment of weight adopts to the method AHP. According to the stray current corrosion and the concentration of  $CO_2$ , analyze the influence of single factor to the life-span of durability of concrete. And use Delphi method to analyze and get the comparing estimation matrix between any two ones in these factors. The standard of ascertained index is: when one compares with another one, the index get 1 if both is the same important; the index get 3 if the former one is only a bit more important than the latter one; the index get 5 if the former one is markedly more important than the latter one; the index get 7 if the former one is much more important than the latter one; the index get 9 if the former one is absolutely more important than the latter one. The calculation procedure in detail can be shown in reference (Wei 2002). Here get the solution of comparison:

X	$G_1$	$G_2$	$G_3$	$G_4$
$G_1$	1		5	5
$G_2$	1/7 1/5 1/5	1	1/3	1/3
$G_3$	1/5	3	1	1/3
$G_4$	1/5	1	1	3

Use the method of square root to make certain the weight of each index and get:

 $W = (W_1, W_2, W_3, W_4) = (0.33, 0.18, 0.23, 0.26)$ 

These weight values are available through proving.

#### 4.5 Fuzzy evaluation of the system of subway durability

Using formula (3), the fuzzy values are:

$$A_{j} = (0.33, 0.18, 0.23, 0.26) \cdot \begin{pmatrix} 0.28 & 0.52 & 0.68 & 0.86 & 1 \\ 0.2 & 1 & 0.68 & 1 & 1 \\ 1 & 0.88 & 0.38 & 0.44 & 0.7 \\ 0.24 & 0 & 1 & 0.26 & 0.36 \end{pmatrix} = (0.42, 0.55, 0.69, 0.63, 0.76)$$

Therefore, the alternatives  $A_i(i = 1, 2, 3, 4, 5)$  can be ranked as  $A_5 > A_3 > A_4 > A_2 > A_1$ , the relevant order of proportioning from good to bad is sorted as followed: E > C > D > B > A

Obviously, the scheme E is best one. The recommendation uses the best proportioning for the serial number E.

#### 5 CONCLUSION

Durability of subway structure is a problem of comprehensive function under many factors, and its evaluation is a compound decision process of multiple attributes, multiple factors and multiple indexes. In view of this problem, the paper determined influencing-factors of durability of subway structure for stray current corrosion, chloride ions ingress, sulfate attack and carbonation, established the evaluating measure index framework of durability of subway structure. Further, FMADM is to be used to carry out the multiple factors evaluation on the basis of single factor evaluations in the subway structure. Then, the durability fuzzy values of alternative scheme of concrete proportioning are obtained. Finally, these values are ordered by their size and make them clear which scheme is good or bad. The present results show that scheme E is the best proportioning scheme of subway concrete which obviously enhances durable life-span (the durable life-span is longer than 100 year) by mixing fly ash and mineral powder reasonable. The recommendation uses the best proportioning for the serial number E.

It should be mentioned that superimposed effect between various factors is not to be considered in the paper. For instance, carbonation affect chloride ions ingress, chloride ions ingress affect sulfate attack. Therefore, durability of subway structure considering superimposed effect between various factors needs further research.

#### ACKNOWLEDGEMENT

This work was supported by the National Natural Science Foundation of China under Grant Nos.50678135. The author would like to acknowledge Prof. Yang Linde for this research project.

#### REFERENCES

- Du, Y.J. & Zhang, H.H. 2003. Elementary study in evaluation of enduring span for HPC of subway engineering. *Northwest Water Resources & Water Engineering* (3): 49–53.
- Heilpern, S. 1997. Representation and application of fuzzy numbers. *Fuzzy Sets and System*: 259–268.
- Leung, L.C. & Cao, D. 2000. On consistency and ranking of alternatives in fuzzy AHP. *European Journal of Operational Research* 124:102–113.
- Li, T.J. 2002. Theory and application of Fuzzy Multiple Attribute Decision Making. Beijing: Science Press.
- Shen, G. 1998. The NO. 1 subway project. The series of design and construction of shanghai municipal works. Shanghai: Shanghai Science and Technology Press: 1–3.
- Tan, W.Z. 2003. Holistic view of durability of concrete structure. Architecture Technology (1): 19–22.
- Wei, R.Q. 2002. Operational research. Beijing: Tsinghua University Press,: 461–466.
- Zimmermann, H.J. 1987. Fuzzy Set Theory and its Application. Kluwer2Nijhoff Publication, USA,: 51–52.

# The use of artificial neural networks to predict ground movements caused by tunneling

I. Chissolucombe, A.P. Assis & M.M. Farias University of Brasilia, Brazil

ABSTRACT: During the design phase of a tunnel, one of the concerns of tunnel engineers is estimating ground movements induced by tunneling, in order to take steps to prevent or minimize possible damage caused by these movements, to adjacent structures. In this work, an estimate is made of displacements in the ground brought on by tunnel excavation by using an artificial intelligence technique called Artificial Neural Networks (ANN). The computer tool utilized was the MatLab program and the networks were of the feedforward type, with the Resilient Backpropagation (trainrp) learning algorithm.

#### 1 INTRODUCTION

Tunnelling provokes changes in the stress state of a ground and consequently, some displacements are generated that spread across the excavation's zone of influence, which may induce damage in the structures located therein. Several methodologies have been utilized to estimate the displacements generated by the excavation of a tunnel and the damage that such displacements can cause to the structures lying adjacent to the excavation area. These methodologies are distributed by the empirical, analytical and numerical approaches. The empirical approaches are easy to use, but have the disadvantage of not considering the ground resistance and deformability parameters (Chissolucombe et al., 2005a). The analytical approaches are also easy to use, but have the primary disadvantage of determining the various coefficients necessary for the use thereof. In using numerical approaches, one must have access to a test program that can supply the necessary parameters in a way as to be able to use constitutive models that faithfully represent the conditions of the ground and often have-as in the case of three-dimensional analyses-a high computational cost.

The aim of this study is to estimate the displacements of the ground induced by tunneling and the inflection point of the surface settlements basin utilizing one of the artificial intelligence (AI) techniques, called artificial neural networks. To verify which of the input variables had the greatest impact on the output variable of the neural network a sensitivity analysis was carried out utilizing the values of the weights between and among the network connections as proposed by Garson (1991). In the training of the neural network, two data sets were used, one of which was obtained by instrumentation during the excavation of the Metrô-DF (subway) in the city of Brasilia, and the other was obtained through numerical simulations by the Finite Element Method. In all of the situations, the training algorithm utilized was Resilient Backpropagation (trainrp), with sigmoid transfer functions.

The use of two sets of data obtained differently is due to the following factors: (a) the input variables of the set of data of Metrô-DF were mostly geometrical parameters of the tunnel or of the ground , having the SPT value as a single parameter of resistance; (b) need to know the behavior of the neural network upon the presentation of the input variables as cohesion and the angle of friction, values obtained through numerical simulations.

#### 2 ARTIFICIAL NEURAL NETWORKS

A neural network is a massively parallel distributed processor that has a natural propensity for storing experiential knowledge and makes it available for use. It resembles the human brain in two respects: (a) knowledge is acquired by the network from its environment through a process of learning, and (b) connection strengths among neurons known as synaptic weights are used for storing the acquired knowledge. (Haykin, 2001).

The most important property of Artificial Neural Networks is their ability to learn from their environment and to improve their performance with such learning. This learning occurs when the neural network reaches a generalized solution for a particular class

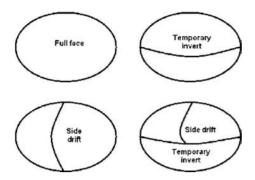


Figure 1. Types of partialization of the cross section.

of problems. In 1970, Mendel & McClaren (cited by Haykin, 2001) defined learning as being an interactive process whereby the free parameters of a neural network are adapted through a process of stimulation by the environment in which the network is inserted. This definition implies the following sequence of steps: (a) the neural network is stimulated by the environment; (b) the neural network undergoes modifications in its free parameters as a result of this stimulation; and (c) the neural network responds to the environment in a new way, due to the modifications that occurred in its internal structure.

The set of well-defined rules for the solution of a learning problem is called a learning algorithm. There are various kinds of learning algorithms, and the main difference among them is the way in which the weights are adjusted.

## 3 MODELING WITH ARTIFICIAL NEURAL NETWORKS

#### 3.1 Modeling 1 (surface marks)

In this modeling, the database utilized was comprised of data obtained from the instrumented sections during the excavation of the tunnel for Metrô-DF (subway) in the city of Brasilia.

The Brasilia Metro has an extension of 42 km, being 12 km underground. The South Wing section is totally underground, with 7.2 km of length, and encompasses nine stations. These stations were built by the cutand-cover method and the tunnel excavated by the sequential excavation method, following the NATM (New Austrian Tunnelling Method) principles. The geometry shape of the tunnel cross-section was an ovoid, with an equivalent diameter of 9.6 m. The types of partialization of the cross section of the Metrô DF tunnel are represented in Figure 1. The South Wing tunnel of the Brasilia Metro was mostly excavated inside a geological domain, called Paranoa group, which encompasses layers of porous clay and residual

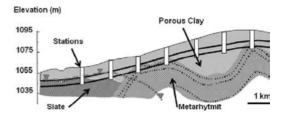


Figure 2. The geological profile of the tunnel from the South Wing.

Table 1. Input statistics for data sets.

Variables		Mean	Minimum	Maximum
H (m)	I (m) Training		6.70	14.30
	Testing	10.03	7.70	12.90
D (m)	Training	8.49	7.26	9.56
	Testing	8.61	7.26	9.46
SPT roof	Training	22.92	0.40	50.00
	Testing	22.70	0.90	50.00
SPT side	Training	3.75	1.50	8.50
	Testing	3.75	1.50	8.50
SPT floor	Training	8.72	3.50	18.00
	Testing	8.79	3.50	17.00

soil of slate (Chissolucombe et al., 2005b). The geological profile of the tunnel from the South Wing is represented in Figure 2.

The input variables in the network were as follows: ground cover above the tunnel roof (H), the type of partialization of the section (TS), the equivalent diameter of the section (D), the length of the balance (x), the level of the water table (N.A.), the SPT value of the tunnel roof, sides and floor. The output variables were the values of the stabilized surface settlement on the tunnel axis obtained from adjustments after taking readings of the surface marks and the inflection point (i). The medium, minimum and maximum values of some input variables are presented in Table 1.

The set of data was comprised of 74 pairs of examples, 62 being utilized in the training phase and 12 in the testing phase. The training was performed considering each output separately; this procedure was adopted because the performance of the network was better than when both outputs were considered simultaneously. The architecture that offered best generalization was 8-40-1 for the output variable "maximum surface settlement" and 8-10-1 for the output variable "inflection point of the settlement basin". Figures 3, 4, 5 and 6 show the results obtained during the training and testing phases of the neural network, and an average of 3,000 interactions was necessary to obtain the performance presented.

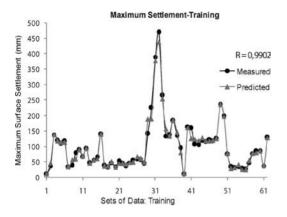


Figure 3. Comparison between the predicted and measured results for the maximum surface settlement during the training phase.

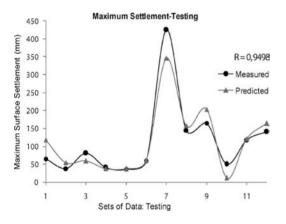


Figure 4. Comparison between the predicted and measured results for the maximum surface settlement during the testing phase.

#### 3.2 Modeling 2 (numerical simulation)

In this modeling, the data obtained from twodimensional numerical simulations under conditions of flat deformation was utilized, with the Plaxis finite element program (Brinkgreve & Vermeer, 1998) developed at Delft University of Technology. The section to be excavated was partialized and the numerical simulation obeyed the following steps: (a) generation of the initial stress based on coefficient of lateral earth pressure (Ko): (b) head excavation with application of 50% of the nodal forces in the area around the excavation; and (c) activation of the support with application of the remainder of the load. The excavation of the sides, nucleus and inverted arch followed the same sequence described above. The finite element mesh was comprised of 275 triangular elements with 6 nodes each, totaling 636 nodes and 825 Gauss points.

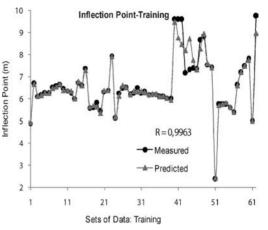


Figure 5. Comparison between the predicted and measured results for the inflection point of the settlement basin during the training phase.

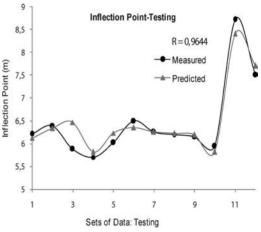
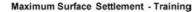


Figure 6. Comparison between the predicted and measured results for the inflection point of the settlement basin during the testing phase.

The model constituent adopted was the Mohr-Coulomb, the main parameters of which are cohesion (c), angle of friction ( $\phi$ ), Young module (E), and Poisson coefficient ( $\nu$ ). For the modeling, 432 numerical simulations were performed, varying the cohesion values (10; 20; 30; 40; 60; and 80 kPa), angle of friction (17°; 22°; 26° and 30°), Young module (3; 6; 10; 15; 20 and 30 MPa) and coefficient of lateral earth pressure (Ko – 0.35; 0.55 and 0.60). The value of the diameter (D) of the soil cover above the tunnel roof and of the Poisson coefficient were constant (4.5 m, 10 m and 0.33 respectively).



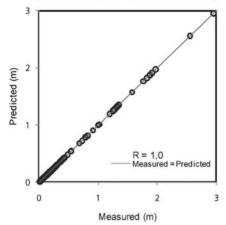


Figure 7. Performance of the neural network in the training phase for the output variable maximum surface settlement.

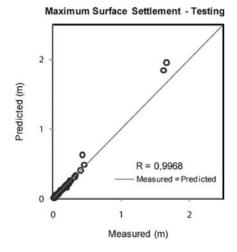


Figure 8. Performance of the neural network in the testing phase for the output variable maximum surface settlement.

The following variables: cohesion, angle of friction, Young module, and coefficient of lateral earth pressure, constituted the input variables in the neural network, while the maximum surface settlement and the settlement at tunnel roof constituted the output variables in the neural network. Out of the total of 432 pairs of examples available, 352 were utilized in the training phase and 80 were used in the testing phase. The best network architecture was 4-40-1, and each output variable was calculated separately. An average of 1,500 interactions was necessary for training the neural network in order to obtain a good generalization.

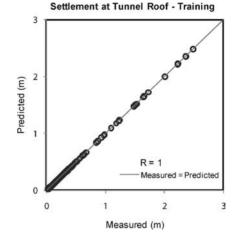


Figure 9. Performance of the neural network in the training phase for the output variable settlement at tunnel roof.

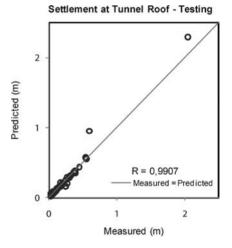


Figure 10. Performance of the neural network in the testing phase for the output variable settlement at tunnel roof.

Figures 7, 8, 9 and 10 show the results obtained in the training and testing phases for the output variables "maximum surface settlement" and "settlement at tunnel roof" with respective correlation coefficients (R) of the best linear adjustment. According to Figures 7, 8, 9 and 10, excellent performance was obtained for both variables in the training and testing phases.

#### 4 SENSITIVITY ANALYSIS

To verify the relative importance of each input variable in the estimate of the output variables, a sensitivity

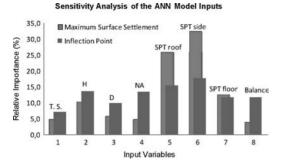


Figure 11. Relative importance of the input variables in the network to estimate maximum surface settlement and inflection point (Modeling 1).

analysis was conducted according to the that proposed by Garson (1991). Figure 11 shows the sensitivity analysis for Modeling 1, i.e., for the case where the network was trained using data obtained by instrumentation during the excavation of the tunnel for the Metrô-DF (subway) in Brasilia. For the output variable "maximum surface settlement," the SPT resistance parameter was the one that presented the greatest relative importance and its maximum value was 32.26% occurring for the SPT in the tunnel side; the input variable "balance" was the least significant, with a relative importance of 3.84%. In case of the output variable "inflection point of the settlement basin," the most significant input was also the SPT in the tunnel side, with a value of 17.52%, and the least significant input was balance, 7.13%.

In Modeling 2, where the data were obtained through the numerical simulations with the Plaxis finite element program and the output variables were "maximum surface settlement" and "settlement at tunnel roof," the relative importance of the input variables. In that case, the resistance parameters were the ones that presented greater relative importance, having a cohesion value of 48%, followed by a Young module of 28%, friction angle of 15%, and coefficient of lateral earth pressure with an average value of 9% (Figure 12).

#### 5 CONCLUSIONS

The following conclusions were reached after the training of the two sets of data utilizing the AI technique called artificial neural networks: (a) the computational tool utilized proved highly effective in the estimate of the displacements induced by tunneling and of the inflection point of the surface pressure basin, having obtained high correlation coefficient values and very minor errors when comparing the values measured with the values estimated by the network; (b) when the

Sensitivity Analysis of the ANN Model Inputs

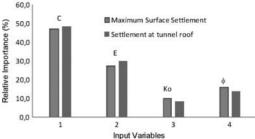


Figure 12. Relative importance of the input variables in the network to estimate the maximum surface settlement and the settlement at tunnel roof (Modeling 2).

objective is to excavate a tunnel in a region where a set of data on previous excavations is available, the use of neural networks will provoke a drastic reduction in the programs of geotechnical geological investigation and measurements of the construction work: (c) the network trained with the data obtained by instrumentation during the excavation of the Metrô-DF tunnel exhibited an inferior performance to the network trained with the data obtained through numerical simulations. This is due to the noise that the data obtained in the construction work presents. Nevertheless the performance of this network was reasonable with average correlation coefficients of 0.99 for training and 0.95 in the test phase; (d) the neural networks present an advantage in relation to traditional empirical methods as they permit the consideration of variables such as: constructive method, water level, geometry of the tunnel, and geometrical parameters and parameters of resistance of the ground; (e) compared with the numerical methods it has the advantage of estimating with high precision the displacements induced by the excavation, precision that is not always achieved in numerical analyses due to factors like: lack of an adequate constitutive model; difficulty in representation of the real geometry; consideration of the tridimensional effect in bidimensional analyses; (f) the sensitivity analysis showed in the two modelings that the parameters of resistance and deformability of the ground have the greatest impact on the estimate of displacements, with the SPT values on the sides, roof and floor of the tunnel in modeling 1 and soil cohesion and Young's module in modeling 2; (g) the main disadvantage of neural networks is the lack of a good set of data for training.

#### REFERENCES

Brinkgreve, R.B.J. & Vermeer, P.A. 1998. Finite Element Code for Soil and Rock Analyses. Plaxis Manual. Rotterdam, Netherlands.

- Chissolucombe, I., Assis, A.P. & Farias, M.M. 2005a. Métodos para previsão dos deslocamentos do maciço induzidos por escavações subterrâneas. 5° Simpósio Brasileiro de Aplicações de Informática Em Geotecnia (INFOGEO). Minas Gerais.
- Chissolucombe, I., Assis, A.P. & Farias, M.M. 2005b. Soilstructure interaction and its influence on displacements

*induced by tunnel excavations*. Fifth International Symposium – Geotechnical Aspects of Underground Construction in Soft Ground. Netherlands.

- Garson, G.D. 1991. Interpreting neural-network connection weigts. AI Expert, 6(7): 47–51.
- Haykin, S. 2001. Redes Neurais: Princípios e Prática. Bookman, 20 edição, RS, 900p.

# Research and application of road tunnel structural optimization

W.Q. Ding & Y. Xu

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotecnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: The optimization process of road tunnel is established in this paper with an engineering example. Through numerical computation, we can find a serious stress concentration occurring at the spring of arch. To solve this problem, a tunnel section shape optimization method is established at first. In this method, the dimensions of the tunnel section are taken as the design variables, the stress of spring is taken as the objective function. No augmentation of the excavated area and the satisfaction of construct boundary are taken as the constraint condition. A Complex Method is used as the optimization method and C++ language is used to program and implement the optimization. Using this optimization method, the stress condition of the tunnel lining can be greatly improved. Then, the optimization method for the thickness and reinforcement of the tunnel lining is also established and the construction cost is reduced.

# 1 INTRODUCTION

With the rapid development of highway in mountain area, the quantity of road tunnel becomes more and more. However, the cost of road tunnel is higher than that of road or bridge, so, how to reduce the total cost of the road tunnel, and how to construct road tunnel in the most economic way are the imperative problems which need to be solved by designers.

Optimization design of the road tunnel is an effective method to reduce the construction cost. But now, in the optimization design of the road tunnel, the traditional passive analysis method is used most. The designers often define several feasible cases, then, choose one from them through computation and analysis. The disadvantage of the traditional passive analysis method lies in that it can not contain all conditions, and it is time consuming. And the numerical optimization method is not used widely, more research is needed and imperative.

The numerical optimization method is a new branch of applied mathematics. In recent twenty years, it develops very quickly with the widespread application of computer. Applying the numerical optimization method in civil construction design makes the traditional passive analysis develop to the active search. This is a grate progress in the civil engineering design, and the numerical optimization method is a more scientific, more effective and more economic method in structural optimization design. But in the domain of optimization of tunnel and underground structure, there are many problems. Due to the complexity of tunnel and underground structure, a great many of parameters should be optimized (such as parameters of surrounding rock, parameters of support structure and decision options in constructing and so on). As to the optimization, there are problems such as multi-variable and highly non-linear, so the research and its application are still in the start stage, the references and examples are few.

With an engineering example, an optimization design method of automatic search is expounded in this paper. The section shape and the lining structure of the tunnel can be optimized. A Complex Method is used as the numerical optimization method, the C++ language is used to implement it. With the optimization analysis and design, the stress condition of the structure is greatly improved, and the cost of the tunnel is effectively reduced. This is of great practical significance to the construction of road tunnel.

# 2 THE THEORY OF COMPLEX METHOD

# 2.1 The mathematic model of complex method

The mathematic model of complex method usually can be expressed as follows:

Solve: design variable X Min f(x)

# S.T (Subject to constraint condition):

Inequation constraint condition:

$$g_j(x) \le 0 \quad (j = 1, 2, ...., m)$$
 (1)

Boundary constraint condition:

$$a_i \le x_i \le b_i \quad (i = 1, 2, ...., n)$$
 (2)

where  $a_i$  is the upper limit of the variable;  $b_i$  is the lower limit of the variable.

# 2.2 The iterative process of complex method

# 2.2.1 *The building of initial complex*

The complex is composed of k(k > n + 1) vertexes. There are two methods to define these vertexes.

a. Definitive method

Designers can define these vertexes by themselves according to the properties of the problem.

b. Random method

The vertexes are defined by the following formula:

$$x_{ij} = a_i + \gamma_{ij}(b_i - a_i) \quad (i = 1, 2, \dots, n; j = 1, 2, \dots, k)$$
(3)

where *i* is the number of variable; *j* is the number of vertex;  $\gamma_{ii}$  is a random value between 0 and 1.

The vertexes defined by random method conform to all boundary conditions, but do not always conform to all inequation of constraint conditions.

Supposing there are  $s(1 \le s \le k)$  vertexes conform to all constraint conditions, the center  $\overline{X}_s$  of the valid vertexes can be defined by following formula:

$$\overline{X}_{i} = \frac{1}{s} \sum_{j=1}^{s} X_{j}$$
(4)

The n-s vertexes that can not conform to all constraint conditions can be dealt with by following formula:

$$X_{i+1} = \bar{X}_i + 0.5(X_{i+1} - \bar{X}_i) \tag{5}$$

If the new vertexes still can not conform to all constraint conditions, they are dealt with formula (5) again until they can conform to all constraint conditions.

#### 2.2.2 Search for reflection point

Figure out all the values of objective function  $f(X_j)$ , and find out the worst vertex  $X_h$  which makes the value of  $f(X_j)$  maximum,  $X_c$  is the centre of the vertexes that do not include the worst vertex  $X_h$ .

$$Xc = \frac{1}{k-1} \left( \sum_{j=1}^{k} X_j - X_h \right)$$
(6)

Define a reflection coefficient  $\alpha(\alpha \ge 1)$ , the reflection point  $X_a$  can be defined by following formula:

$$X_a = X_c + \alpha (X_c - X_b) \tag{7}$$

Checking whether  $X_a$  is valid, if not, reducing the value of  $\alpha$  to its half, then deal  $X_a$  with formula (7) until it becomes valid.

2.2.3 Compare the value of  $f(X_j)$  at  $X_a$  with that at  $X_h$ 

There are two possible results:

If  $f(X_a) < f(X_h)$ , i.e.  $X_a$  is better than  $X_h$ , replace  $X_h$  with  $X_a$ , the new complex is formed, then, turn to step 2 and go on.

If  $f(X_a) \ge f(X_h)$ , i.e.  $X_a$  is not better than  $X_h$ , reduce the value of  $\alpha$  to its half again until  $X_a$  is better than  $X_h$ , then turn to step 2 and go on.

# 2.2.4 Criterion of convergence

There are many criteria of convergence, but the criterion used most widely is that the values of  $f(X_j)$  at all vertexes can conform to constraint condition as follows:

$$\{\frac{1}{k}\sum_{j=1}^{k} [f(\bar{X}_{j}) - f(X_{j})]^{2}\}^{1/2} < \varepsilon$$
(8)

where  $f(\overline{X}_s)$  is the value of objective function at the centre point;  $\varepsilon$  is a little positive number.

Find the vertex at which the value of objective function is least, the optimum vertex is found.

# 3 OPTIMIZATION OF SECTION SHAPE AND LINING OF ROAD TUNNEL

# 3.1 General situation of the road tunnel

The Bai Yang Chong No. 1 tunnel is a twin tunnel connected with arc mid-wall which is located in Jiangxi province, China. It is 215 m long. The longitudinal grade of this tunnel is 2.499%. The thickness of the overburden ranges from 10 m to 20 m. The stability of the surrounding rock is not good. The underground water condition is well. The width of tunnel is about 11m, and the height is about 5 m.

## 3.2 The optimization problem of road tunnel

Lining is the most important component of tunnel structure. The stress condition of lining is the key influence factor of the stability and the life-span of tunnel, it usually becomes the optimization object when tunnel is designed. When optimizing the tunnel section, the stress condition of lining should be analyzed at first. The most important influence factor on the stress condition of lining becomes the optimization object.

With the Tongji GeoFBA numerical analysis soft (a type of numerical analysis soft which can solve two-dimensional problem with finite element method, and it is developed by the Department of Geotecnical Engineering of Tongji University), we can get the inner force of the tunnel lining as follows:

According to Figure 1 and quantities of calculation results, it can be found that the maximum inner forces (including bending moment, shear force and

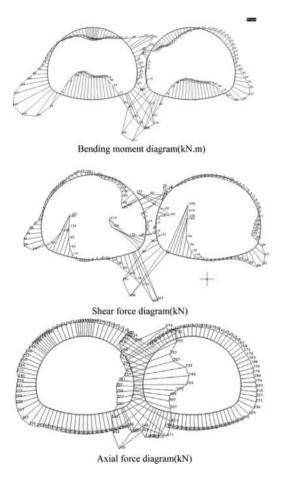


Figure 1. Inner force diagram of tunnel lining.

axial force) occur at the tunnel arch spring. The section shape of this tunnel is monocentric circle, and the invert arch is joined by arc, i.e. the spring is the linkage between the invert arch and the arch ring. It is the most critical part of the lining ring. Due to this, the safety factor of the lining is reduced. The reason is that the spring is the link of two arcs with different radius, and its radius is small, the stress concentration occurs easily. When optimizing the section form of the tunnel, the dimension parameter of this part is taken as the design variable, and the maximum inner force of lining becomes the control objective.

Tunnel lining takes about 50% of the total cost. After improving the stress condition by optimizing the section shape, the thickness and reinforcement of the tunnel lining can then be optimized to reduce the lining cost.

Generally, with an engineering example, the optimization of section shape and lining design of road tunnel is studied in this paper. By optimization design,

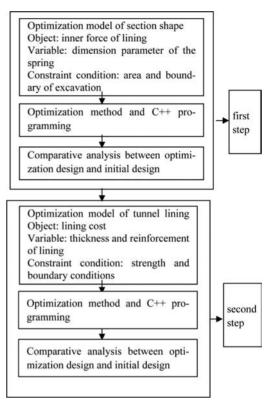


Figure 2. Flow chart of the optimization design.

the stress condition is greatly improved, and the cost is effectively reduced.

# 3.3 General process of the optimization design

There are two steps of the optimization design. First, optimize the section shape of the tunnel, then, optimize thickness and reinforcement of the tunnel lining.

# 3.4 Optimization of the tunnel section shape

# 3.4.1 The building of optimization model

3.4.1.1 Selection of the design variable As shown in Figure 3, to define the tunnel section, there are 4 independent variables:  $R_1$ ,  $R_2$ ,  $\theta_1$ ,  $\theta_2$ . According

to the geometrical relationship, 
$$R_3$$
,  $\theta_3$  can be defined as follows:

$$\theta_3 = 90^\circ - \theta_1 - \theta_2 \tag{9}$$

$$R_{3} = \frac{(R_{1} - R_{2})\cos\theta_{1}}{\cos(\theta_{1} + \theta_{2})} + R_{2}$$
(10)

The stress concentration occurs at the linkage part, the dimension parameters of this part  $R_2$ ,  $\theta_2$  is taken as the design variables,  $R_1$ ,  $\theta_1$  is taken as constant.

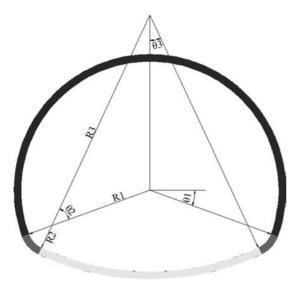


Figure 3. Sketch of tunnel section.

where  $R_1$  is the radius of roof;  $R_2$  is the radius of the linkage part;  $R_3$  is the radius of invert arch;  $\theta_1$  is the angle between  $R_1$  and springing line;  $\theta_2$  is the central angle of the linkage part;  $\theta_3$  is the angle between  $R_3$  and the center line of tunnel section.

3.4.1.2 The building of the objective function The radius of the linkage part is small, the curvature is big, this is the key factor of the stress concentration. So, the curvature of the linkage part is taken as the objective function, the optimization problem is getting its minimum value.

Objective function is shown as follows:

$$Z = 1/R_2$$
 (11)

3.4.1.3 The building of the constraint conditions  $R_1$ ,  $\theta_1$  are constants, the inner contour line of the tunnel section will not enter into the construction boundary. In order not to increase the excavation cost, the minimum excavation area should not be larger than that of the initial design.

The minimum excavation area can be defined as follows:

$$A = \pi R_1^2 \frac{180 + 2\theta_2}{360} + \pi R_2^2 \frac{2\theta_2}{360} + \pi R_3^2 \frac{2\theta_3}{360} + \pi R_3^2 \frac{2\theta_3}{360} - (R_3 - R_2)(R_1 - R_2)\sin\theta_2$$
(12)

Then the constraint condition can be gained as follows:

$$\pi R_1^2 \cdot \frac{180 + 2\theta_2}{360} + \pi R_2^2 \cdot \frac{2\theta_2}{360} + \pi R_3^2 \cdot \frac{2\theta_3}{360} - (R_3 - R_2) \cdot (R_1 - R_2) \sin \theta_2 \le A_{\text{ini}}$$
(13)

where  $R_1$ ,  $\theta_1$  are constants;  $R_2$ ,  $\theta_2$  are design variables;  $R_3$ ,  $\theta_3$  can be defined by formulas (9) and (10).

The upper limit and the lower limit can be defined according to the practical situation.

# 3.4.2 Implement of the optimization process

Complex method is used as the numerical optimization method, programmed with C++ language, and the optimum solution of the design variables can be gained by running the C++ program.

# 3.4.3 Optimization design and comparative analysis

Define the values of every design variable according to the optimum solution.

The initial design:

$$R_2 = 205 cm$$
,  $\theta_2 = 14.4775^\circ$ 

The optimization design:

 $R_2 = 266 cm$ ,  $\theta_2 = 69^{\circ}$ 

The optimization of road tunnel is realized by the Tongji GeoFBA numerical analysis soft, the inner forces of the tunnel lining can be gained after the optimization design as follows:

As shown in Figure 4, the maximum inner force also occurs at spring, but its value has been reduced a lot. The stress condition has been improved greatly by the optimization design.

From Table 1, we can find that the stress condition of tunnel lining has been improved, especially the bending moment which is the determining factor of the lining safety has been reduced greatly. It illustrated that decreasing curvature of the linkage part is an effective measure to improve the stress condition.

# 3.5 Optimization of the tunnel lining

#### 3.5.1 *The building of optimization model*

3.5.1.1 Defining of objective function and design variable

After the optimization design of the tunnel section shape, the stress condition has been improved greatly, then, we can optimize the tunnel lining, and take the lining cost as the objective function.

Cost of the steel concrete lining is composed of the cost of steel and the concrete. The cost of lining can be defined as follows:

$$Z = C_c + C_s \tag{14}$$

where  $C_c$  is the cost of concrete;  $C_s$  is the cost of steel; Z is the objective function.

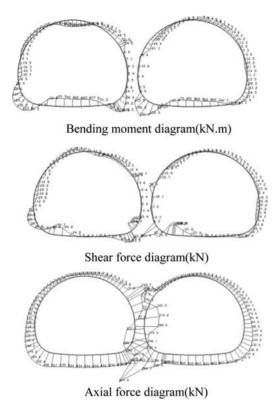


Figure 4. Inner force diagram of tunnel lining of the optimized section shape.

 Table 1.
 Inner force of tunnel lining comparing between the optimization design and the initial design.

	M <sub>max</sub> (kN.m)	N <sub>max</sub> (kN)	Q <sub>max</sub> (kN)
Initial design	311.1100	715.7400	285.8700
Optimization design	191.6200	578.9500	221.4300
Reduction ratio	38.41%	19.11%	22.54%

Multiplying the quantity of consumption by the unit price can get the cost of the materials. As the section shape is already determined,  $C_c$  is only related to h (thickness of the tunnel lining), as the thickness of the protective concrete layer is a constant,  $C_c$  is only related to  $h_o$  (effective section height). The longitudinal reinforcement is arranged as the constructional reinforcement, the hoop reinforcement is the main working reinforcement. So  $h_o$  and  $A_g$  (area of the hoop reinforcement of lining per meter) are taken as the design variables of the objective function. 3.5.1.2 Building of the constraint conditions

1) Constraint condition of shear resistance

Check the safety of the section where the shear force reaches the maximum value.

$$KQ \le 0.3R_a bh_o \tag{15}$$

where *K* is the safety factor, it can be defined according to the standard code; *Q* is the maximum shear force of the tunnel lining;  $R_a$  is compressive strength of concrete; b is the section width, which is equal to 1000 mm;  $h_a$  is the design variable.

2) Constraint condition of compressive and bending resistance

Check the safety of the sections where the axial force reaches the maximum value, or the bending moment reaches the maximum value. Then we can get the following equations.

In big eccentric compression condition:

$$KNe' \le R_g A_g (h_o - a') \tag{16}$$

In small eccentric compression condition:

$$KNe \le 0.5R_a bh_o^2 + R_g A_g'(h_o - a')$$
(17)

or

$$KNe' \le 0.5R_a bh_o^2 + R_g A_g (h_o - a)$$
 (18)

# 3) Constraint condition of boundary

The upper limit and the lower limit of the lining thickness are determined according to the criterion. The upper limit and the lower limit of the reinforcement area are determined according to the maximum reinforcement ratio and the minimum reinforcement ratio.

### 3.5.2 Implement of the optimization process

Complex Method is used as the numerical optimization method, C++ language is used to program and gain the optimum solution of the design variables.

# 3.5.3 Optimization design and comparative analysis

The values of every design variable can be determined according to the optimum solution, then the comparison can be done between initial and optimized design.

Initial design:  $h = 550 \text{ cm}, A_g = 1571 \text{ mm}^2$ 

Optimized design:  $h = 450 \text{ cm}, A_g = 1451 \text{ mm}^2$ 

where *h* is the thickness of the tunnel lining; for symmetric arrangement,  $A_g$  is the hoop reinforcement area of one side of the tunnel lining.

For this tunnel, the lining cost of the optimized design can be reduced by 12.1%, and 630,000RMB can be saved. It can conclude that optimization design is an effective measure to reduce the engineering cost, while the safety of the structure can be ensured simultaneously.

# 4 CONCLUSION

From optimization design of the road tunnel and the comparative analysis, conclusions can be gotten as follows:

- When defining the objective function and the design variables, many factors can be taken into account, one step optimizing not only makes the problem complex, but also get no effect results. By dividing the optimization problem into several steps, the optimization model and operation process can be simplified greatly when solved step by step, it can get good and practical results.
- 2. By numerical computation and analysis, it can be found that the maximum inner forces occur at the tunnel spring, the reason is that the curvature of this part is small, and the stress concentration occurs easily, reducing the curvature of this part is an effective measure to improve the lining stress condition, and optimized section shape can then be gained.
- 3. Introducing optimization method into structure design can reduce the construction cost effectively without lowering the safty grade, it should be used widespread.
- 4. To solve the non-linear optimization problem with inequation constraint condition, Complex Method is an effective optimization method, its theory is simple, and the operation is easy and effective.

# ACKNOWLEDGEMENTS

This paper are sponsored by The National High Technology Research and Development Program (863 Program) of China, No. 2006AA11Z118, Popularization Project of Key Research Technology of Twin Tunnel Construction and Shanghai Leading Academic Discipline Project, Project Number: B308.

# REFERENCES

- Ding, W.Q., Yue, Z.Q., Tham, L.G., Zhu, H.H., Lee C.F. & Hashimoto, T. 2004. Analysis of Shield Tunnel. *Interna*tional Journal for Numerical & Analytical Methods in Geomechanics. 28: 57–91.
- JTG D70-2004. Road tunnel design code.
- Liu, Y.J. & Gao, G.F. & Feng, W.X. 2004. Study on optimization design of cross section of the large-span highway tunnel. *Liao Ning communication science and techlonogy*. (2): 48–50.
- Qian, N. 1999. *C++ program design course*. Bei Jing: Qing Hua University Press.
- Zhang, B.H. 1998. *Civil structure optimization design*. Shang Hai: Tongji University Press.

# Floor heave behavior and control of roadway intersection in deep mine

B.H. Guo & T.K. Lu

School of Energy Science & Engineering, Henan Polytechnic University, Jiaozuo, Henan, P.R.China

ABSTRACT: For investigating floor heave behavior and controlling technique of roadway intersection in deep mine, creep deformation characteristics of floor around roadway intersection was studied by Flac<sup>3D</sup> and the effects of shear plastic critical value, reinforcement measurement on creep deformation of floor around roadway intersection were discussed. As a result, the creep deformation curve can be divided into two stages including initial creep deformation stage and softening deformation stage, which can explain the case that deformation of roadway intersection is little before a certain time, but then increases at a high speed; only when total shear plastic value exceeds the critical value can second creep stage take place; reinforcement measurement with bolting in roof and ribs has little effect on floor heave, yet exerting pressure against floor can reduce floor heave obviously.

# 1 INTRODUCTION

Rocks mass of roadway in deep mine show soft behavior because they are located in high stress environment. Due to its larger cross section and complex geometries deformation of floor around roadway intersection is usually greater than roadway doesn't intersect. Floor heave has a great influence on ventilation, transportation, and so on, thus a lot of literatures already exist about this issue, but there are only a few in which was based on alternating effects between softening and creeping (Yang et al. 2006).

Strata of North ventilation roadway at -990 m level in Tangkou colliery is silty mud rock in mainly green and cinereous, mingled with a little fine sand rock at some locality. Protodrakonov scale of hardness f is 2.2, pressive strength is 19 MPa, and tensile strength ranges from 1.67 MPa to 4.45 MPa. The burial depth of roadway is nearly from 1029 m to 1035 m, maximum main stress is about 31.6 MPa, and the vertical stress is between 25 MPa and 26 MPa (Liu et al. 2005), which is larger than its pressive strength, thus softening deformation and creeping deformation will occur simultaneously (Wang et al. 1994). A case has been observed that the deformation of roadway intersection was less before about the 45th day after it was formed, but then accelerated and convergence between roof and floor reached 565 mm at the 90th day, among which floor heave accounted for more than 70 percent. Therefore, behavior and controlling technique of roadway intersection in deep mine was investigated according to conditions of a roadway intersection at -990 m level in Tangkou colliery, with effects of softening/creeping of rocks mass interaction being considered by means of numerical simulation method.

# 2 NUMERICAL SIMULATIONS PROCEDURE

# 2.1 Failure criteria

Pwipp model (a visco-plastic model combining WIPP model (the rock creep visco-elastic model) and the Drucker-Prager model) is used in this study. Total strain includes deviate strain and average strain. Deviated strain contains three components of elastic strain, plastic strain and viscosity strain, while average strain contains two components mentioned above except for viscosity strain (Liu et al. 2005).

Strength of rocks mass decreases gradually along with development of deformation after rocks mass begin to failure (Wang et al. 2006), and the bearing capacity of rocks mass in plastic regions is lower than in elastic regions. Cohesion and friction angle lower at different degrees (Xiao et al. 2005, Zhang et al. 2005) along with development of plastic deformation, residual cohesion will lost totally and bearing capacity is provided by only friction force (You 2005). Therefore, strain-softening model can be used to study deformation of roadway intersection undoubtedly (Yang et al. 2002).

Change of strain especially plastic strain externalizes the loading path and history, and reflects softening process of material from initial condition to final failure (Zheng 2007). Referring to relationship between softening process and plastic deformation (see Fig.1) from You (2000), Diagrammatic sketch of full shear plastic strain-softening curve (see Fig. 2a) was got. Figure 2a shows that the softening coefficients of strength *k* (the ratio of stress corresponding to plastic strain to strength peak value) increases with shear plastic strain when shear plastic strain is below  $\varepsilon_{p1}$ ,

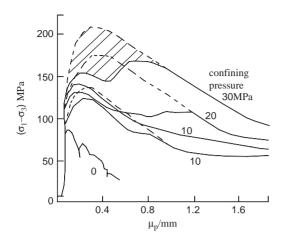


Figure 1. Relationship between softening process and plastic deformation (after You, 2000).

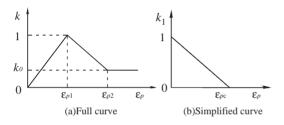


Figure 2. Diagrammatic sketch of softening coefficientshear plastic strain curve.

decrease when the shear plastic strain is between  $\varepsilon_{p1}$ and  $\varepsilon_{p2}$ , and keep unchanged as a constant value  $k_0$ when shear plastic strain is above  $\varepsilon_{p2}$ . Because  $\varepsilon_{p1}$  and  $k_0$  are usually very little, they are both assumed as 0 here, thus the simplified shear plastic strain-softening curve (see Fig. 2b) was obtained. Besides, when total strain of rocks mass exceeds the strain value at peak strength, deformation modulus lessen gradually with strain, but residual deformation modulus is never reach 0 (see Fig. 3).Other researchers (Zeng et al. 2005, Liang et al. 2005, Cheng et al. 2005, Chen et al. 2005, Li et al. 2006, Qiang et al. 2006, Wang et al. 2007) also reported the similar opinion.

If we define  $k_1$  as strength softening coefficient and  $k_2$  as deformation softening coefficient, they can be defined by following formulas:

$$\begin{cases} k_1 = 1 - \frac{\varepsilon_p}{\varepsilon_{pc}} & 0 < \varepsilon_p \le \varepsilon_{pc} \\ k_1 = 0 & \varepsilon_p > \varepsilon_{pc} \end{cases}$$
(1)

$$\begin{cases} k_2 = 0.8 & 0 < \varepsilon_p \le \varepsilon_{pc} \\ k_2 = 0.5 & \varepsilon_p > \varepsilon_{pc} \end{cases}$$

$$(2)$$

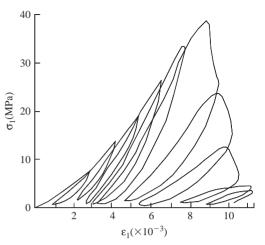


Figure 3. Stress-strain curves under cycle loading (after Zhu, 1985).

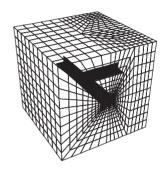


Figure 4. Simulated model.

where  $\varepsilon_p$  = shear plastic value; and  $\varepsilon_{pc}$  = shear plastic critical value, 2e-3 here.

## 2.2 Model development

Assumption of rocks mass being homogeneous and in hydrostatic-pressure state was made in this work. Simulated model containing a T shaped roadway intersection and bolt supporting sketch were plotted in Figure 4 and Figure 5 respectively. The sizes of calculated model were 40 m length, 40 m width and 40 m height. The bottom of the model was fixed in all directions, four sides were fixed in horizontal direction, and overburden weight was exerted on the top of the model. The section of roadway was rectangle with width of 4 m and height of 3 m. The length and diameter of 2 anchors bolted in the roof were 6300 mm and 17.8 mm respectively, the interval along the axial and circumferential direction of the roadway were 3000 mm and 2400 mm respectively; for cables, the corresponding values were 2300 mm, 18 mm, 1000 mm and 1200 mm

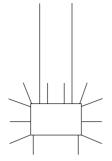


Figure 5. Bolt supporting sketch.

Table 1. Mechanical parameters of rocks mass.

Parameters	Bulk modulus <i>K</i>	Shear modulus G	Density D	Shear strength τ <sub>f</sub>	Tensile strength $\sigma_t$
Units	GPa	GPa	kg/m <sup>3</sup>	MPa	MPa
Values	8	4.8	2640	9.5	3.06

Table 2. Creep parameters of rocks mass.

Parameters	Activation energy, Q	Zone tempera T	ature,	Gas constant, <i>R</i>		WIPP- Model constant, D
Units Values	J · mol <sup>−1</sup> 50160	K 300		J · mol <sup>−1</sup> 1.987	$\cdot K^{-1}$	$Pa^{-n} \cdot s^{-1}$ 28.95e-36
Parameters	WIPP-m constant			P-model tant, <i>B</i>		PP-model onent, n
Values	22.8		25.4		4.9	
Parameters	Material parameter, $q_{\varphi}$		Material parameter, q <sub>k</sub>		Critical steady-state creep rate	
Values	0.55		0.5		1.078 e-8	

respectively. The cables near corner deviated  $20^{\circ}$  from normal direction of wall to corner. According to relative references (Liu, J.H et al. 2005, Liu, T.S et al. 2005), mechanical parameters and creep parameters of rocks mass were chosen and listed in table 1 and table 2; mechanical parameters of cables and anchors were listed in table 3. Following 4 reinforcement measurement methods were investigated by numerical simulation: method A was for naked roadway intersection, method B and C were for bolted roadway intersection without and with cables applied in floor at two corners, and the last method was method C companied with exerted pressure against floor, the pressure values ranged from 0.1 MPa to 1.0 MPa with interval of 0.1 MPa.

Table 3. Mechanical parameters of cables and anchors.

Parameters	Elastic modulus MPa	Grout cohesion strength MPa	Grout exposed perimeter m
Cable	45	0.2	1
Anchor	195	0.42	1
Parameters	Tensile	Grout	Cross-section
	strength	stiffness	area
	MPa	MPa	$m^2 \times 10^{-6}$
Cable	0.25	17.5	254
Anchor	1.85e3	5.35e3	249

# 3 RESULTS

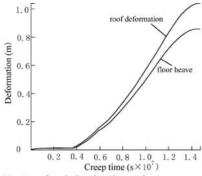
# 3.1 Analysis of creep stages

XU et al. (2007) discussed that deformation process of the roadway can be divided into three stages including adjustment deformation stage, stable defor mation stage and accelerated deformation stage by investigation in the field.

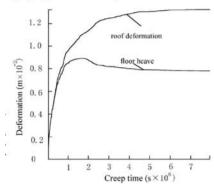
Figure 6 shows deformations of naked roadway intersection versus creep time curves under different critical values. When  $\varepsilon_{pt} > \varepsilon_{pc}$ , Figure 6a illustrates that creep curves can be divided into two creep stages named initial creep stage and softening creep stage respectively, and each stage include a decelerated creep stage and a stable creep stage. At initial creep stage where the shear plastic value is lower than the critical value, deformation increases rapidly and then comes to a nearly constant value until shear plastic value exceeds the critical value. At softening creep stage, deformation of the roadway intersection increases rapidly with creep time for a longer period, and comes to stable state finally. Therefore, we can explain the case mentioned in section 1 that the deformation is little before a certain time, but increases at a high speed later.

For naked roadway intersection, when  $\varepsilon_{pt} < \varepsilon_{pc}$ , only initial creep stage occurs (see Fig. 6b). Roof deformation raises rapidly with a gradual decreasing velocity, then approaches a relatively constant value while the deformation velocity come to about 0; floor heave raises rapidly also, but it reaches a peak value, then decreases a little, finally come to a lower constant value. There is a deformation rebound in the deformation process of floor heave.

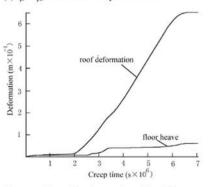
When  $\varepsilon_{pt} > \varepsilon_{pc}$ , and roadway intersection is reinforced by bolting and exerting pressure against floor, the relationships between deformation and creep time are shown in Figure 6c. Figure 6c illustrates that the



(a)  $\varepsilon_{pt} > \varepsilon_{pc}$ , for naked roadway intersection.



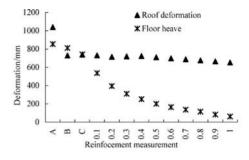
(b)  $\varepsilon_{pt} < \varepsilon_{pc}$ , for naked roadway intersection.



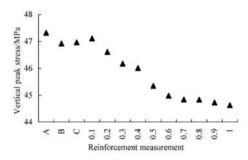
(c)  $\epsilon_{pt} > \!\! \epsilon_{pc},$  for roadway intersection with reinforcement measurement.

Figure 6. Deformation-creep time curves under three conditions,  $\varepsilon_{pt}$  represents total shear plastic train value.

floor heave is very little compared with roof deformation; the profile of roof deformation is similar with that shown in Figure 6a, yet floor heave curve is very different. The beginning time of softening creep stage takes place late compared with that of roof deformation, and its total deformation value is less greatly than that of roadway intersection without reinforcement shown in Figure 6a. Therefore, Reinforcement for floor can reduce floor heave effectively.



(a) Relationship between deformation and reinforcement methods.

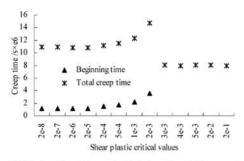


(b) Relationship between vertical peak stress and reinforcement methods.

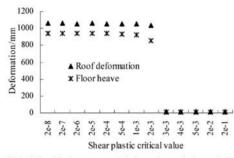
Figure 7. The relationships between deformations, vertical stress peak value and reinforcement measurement: A, B and C represent Method A, method B and method C described in section 2.2;  $0.1 \sim 1.0$  MPa refer to the pressure values exerted against floor based on method C.

# 3.2 Controlling technique

Figure 7 illustrates that roof deformation decreases abruptly when naked roadway intersection are reinforced by bolting in roof and ribs, yet other reinforcement measurements have little affection on it; floor heave decreases a little when naked roadway intersection are reinforced by bolting in roof and ribs, even when cables were applied in floor near two corners (see Fig.7a), but floor heave lessens obviously with increment of pressure value exerted on the floor, and vertical stress peak value around roadway intersection also decreases (see Fig.7b); vertical stress peak values around roadway intersection with reinforcement measurement are all lower than naked roadway intersection. Only when the pressure value against floor is 0.1 Mpa and with reinforcement measurement method C, the vertical stress peak value has a little rebound. Therefore, we can reduce floor heave amount by exerting pressure against floor together with bolting support. If it is not enough yet, measurements of vertical cutting in floor (Guo, unpubl.) and so on can be used additionally.



(a) Relationships between the beginning time of the second creep stage and total creep time and shear plastic critical value.



(b) Relationship between total deformation and shear plastic critical value.

Figure 8. Relationship between creep time, total deformation and shear plastic critical value.

# 4 DISCUSSION

#### 4.1 Affection of shear plastic critical value

It is very important to choose reasonable shear plastic critical value when numerical simulation is performed. Figure 8 illustrates that when the critical value is above 2e-3, the second creep stage doesn't occur, and the deformation of the roadway intersection is very little, and the total creep time that calculation last is shorter. Otherwise, the second creep stage takes place. At the second creep stage, the deformation of the roadway intersection is larger and the deformation value of each critical value seems to be similar. But along with increment of the critical value, the beginning and end time of the softening creep stage delay a little, and total deformation lower a little. In summary, the shear plastic critical value determines if the softening creep stage takes place, and has a key influence on the beginning time, end time and total deformation of the second creep stage. So creep deformation of roadway intersection can be reduced by improving the shear plastic value.

## 4.2 Generation mechanics of creep stages

Research workers (Zhu et al. 2002, Wang et al. 2004, Li et al. 2006, Fan 2007) considered that rocks samples will failure finally if shear plastic value is large

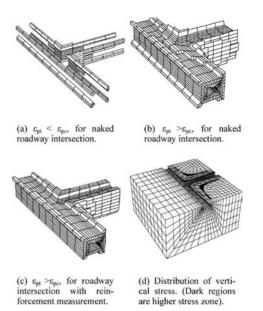


Figure 9. Distribution of higher vertical stress and softening zone distributions under three conditions.

enough. Why can roadway intersection come to stable state finally?

Distribution of shear plastic zones (i.e. softening zone (Wang et al. 2004)) and vertical stress filed are plotted in Figure 9. As the results shown in Figure 9, the final shear plastic zone is small when  $\varepsilon_{pt} < \varepsilon_{pc}$  (see Fig. 9a), greater when  $\varepsilon_{pt} > \varepsilon_{pc}$  (see Fig. 9b), and can be reduced by reinforcement measurements (see Fig. 9c). When rocks mass in the softening zone lost their bearing capacity to some extent, adjacent rocks mass must provide more bearing capacity, if they can't, they will be softened also, and overburden pressure will continue seeking another rocks mass until the rocks mass can bear it sufficiently and keep stable finally (see Fig. 9d). Along with softening zone's generation and enlargement, peak stress value in surrounding rocks mass increases and its location diverts to rocks mass that hasn't been softened. All in all, although softened rocks mass adjacent to excavated room lost bearing capacity at a different degree, but rocks mass adjacent to softened rocks mass provide higher bearing capacity, so that the construction of roadway intersection do not lost its stability completely.

# 5 CONCLUSIONS

The conclusions of this research are summarized below:

 Pwipp model of flac<sup>3D</sup> software can be used effectively to simulate creep deformation of roadway intersection in deep mine. Strain-softening used in this study includes softening of strength and deformation modulus.

- 2. Creep deformation-creep time curves can be divided into two stages including initial creep stage and softening creep stage, and only when total shear plastic value exceeds shear plastic critical value can softening creep stage occur. Determination of the critical value is very important in calculation, and creep deformation of roadway intersection can be reduced by means of improving the shear plastic value.
- 3. Bolting in surrounding rocks mass and exerting pressure against floor can reduce deformation of roadway intersection in deep mine. Method of exerting pressure against floor has more obvious effects to reduce floor heave than other measurements used in this research, if it is not enough, other measurement can be used additionally.

# ACKNOWLEDGEMENTS

This work was carried out under the Outstanding Talent Innovation Fund of Henan province (No. 0621000400).

# REFERENCES

- Chen, Z.J. & Yang, J.W. 2005. Measures for supporting deep High Stress crack-expansion creep rock mass in Jinchuan mining district. Metal Mine (1): 18–22.
- Cheng, R.H., Qian, M.G. & Miao, X.X. 2005. Numerical simulation in mining pressure control of thick and strong stratum caving by water-infusion softening method. Chinese Journal of Rock Mechanics and Engineering 24(13):2266–2271.
- Fan, Q.Z. 2007. Experimental study on creep and its disturbed effect of rocks. Chinese Journal of Rock Mechanics and Engineering 26(1):216–216.
- Li, Y.J., Pan, Y.S. & Zhang, M.T. 2006. Time effect on zonal disintegration process of deep rock mass. The Chinese Journal of Geological Hazard and control 17(4): 119–122.
- Liang, Z.Z., Yang, T.H. & Tang, C.A. et al. 2005. Threedimensional damage soften model for failure process of heterogeneous rocks and associated numerical simulation. Chinese Journal of Geotechnical Engineering 27(12): 1447–1452.
- Liu, J.H., Zhu, W.S. & Li, S.C. et al. 2005. Analysis of rheological characteristics and stability of surrounding rock masses of Xiaolangdi hydrojunction underground caverns by using Flac3D. Chinese Journal of Rock Mechanics and Engineering 24(14): 2484–2489.
- Liu, T.S., Zhao, J.Z. & Yang, J.E. et al. 2005. Support technology of bolt and anchor truss applied in large chamber of Tangkou mine. Coal Science and Technology 33(5):12–15.
- Qiang, H., Zhou, H.Q. & Chang, Q.L. 2006. Regress analysis of mechanics characteristic of the rock strength at the prepeak and post-peak. JiangXi Coal Science & Technology (2): 48–50.

- Wang, H.P., Gao, Y.F. & Li, S.C. 2007. Uniaxial experiment study on mechanical properties of reinforced broken rocks pre-and-post grouting. Chinese Journal of Underground Space and Engineering 3(1): 27–31, 39.
- Wang, J.A., Jiao, S.H. & Xie, G.X. 2006. Study on influence of mining rate on stress environment in surrounding rock of mechanized top caving mining face. Chinese Journal of Rock Mechanics and Engineering 25(6): 1118–1126.
- Wang, J.J., Lu, Z.Y. & Liu, X.F. 2005. Discussion on floor upheaval mechanism of mine soft rock roadway. Coal Engineering (9): 67–68.
- Wang, L.G., Wang Y.J. & Liu, X. 1994. The rheological instability theory for rock sample and its criteria. Journal of Fuxin Mining Institute (Natural Science) 13(3):93–97.
- Wang, X.Q., Yang, L.D. & Gao, W.H. 2004. Creep damage mechanism and back analysis of optimum support time for soften rock mass. Chinese Journal of Rock Mechanics and Engineering 23(5): 793–796.
- Xiao, H.F., He, X.Q. & Feng, T. et al. 2005. Research on coupling laws between EME and stress fields during deformation and fracture of mine tunnel excavation by Flac3D simulation. Chinese Journal of Rock Mechanics and Engineering 24(5):812–817.
- Xu, X.L., Zhang, N. & Xu, J.G. et al. 2007. Principle and practice of process control over soft broken roadway with high ground stress. Journal of Mining & Safety Engineering 24 (1):51–55.
- Yang, C., Chui, X.M. & Xu, S.P. 2002. Establishment and study of strain-softening numerical constitutive model for soft rock. Rock and Soil Mechanics 23 (6):695–697.
- Yang, C.H. & Li, J.G. 2006. Analysis of creeping mechanisms of non-uniformity soft rocks. Journal of Mining and Safety Engineering 23(4):476–479.
- You, M.Q. 2005. Study of deformation and failure of rock based on properties of cohesion and friction. Journal of Geomechanics 11(3):287–291.
- You, M.Q. 2000. Strength of rock specimens and process of deformation and failure. Beijing: Geology Press.
- Zeng, S., Yang, S.J. & Man, C. et al. 2005. Statistical constitutive model for limestone rock damage under uniaxial compression and its experimental study. Journal of Nanhua University (Science and Technology) 19(1): 69–72, 95.
- Zhang, F., Dong, Z.H. & Ding, X.L. 2005. Numerical analysis of excavation process for tailrace tunnels of Pengshui project. Journal of Yangtze River Scientific Research Institute 22(6):59–62.
- Zheng, Y.R. 2007. Discussion on yield and failure of geomaterials and stability analysis methods of slope/landslide – communion and discussion summary of special topic forum on geologic disasters in the three gorges project region. Chinese Journal of Rock Mechanics and Engineering 26(4):649–661.
- Zhu, H.H. & Ye, B. 2002. Experimental study on mechanical properties of rock creep in saturation. Chinese Journal of Rock Mechanics and Engineering 21(12):1791–1796.
- Zhu, Z. F. 1985. Stiff test machine. Beijing: coal industry press.

# Squeezing potential of tunnels in clays and clayshales from normalized undrained shear strength, unconfined compressive strength and seismic velocity

# M. Gutierrez

Colorado School of Mines, Golden, CO, USA

# C.C. Xia

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University Tongji University, Shanghai, P.R. China

ABSTRACT: The tunnel squeezing phenomenon was first described by Terzaghi (1946) who associated squeezing mainly with clay-rich rocks. Consistent with Terzaghi's original description, the focus of this paper is on tunnels in clays and clay-rich rocks such as clayshales. The paper develops a simple procedure to predict tunnel squeezing potential using normalized undrained shear strength, unconfined compressive strength and P-wave velocity. A collection of a large amount of undrained triaxial test data is used to show that the undrained shear strength of clay shales can be normalized with respect to the effective vertical stress. The normalized undrained shear strength can then be used to predict the squeezing potential since it can be directly related to Peck's (1969) stability factor N. Values of N provide estimates of the degree of squeezing potential, with values of N > 1indicating potential for squeezing. The normalized undrained shear strength of clay and shales is shown to be related to the apparent overconsolidation ratio OCR, which accounts for both mechanical and other diagenetic pre-consolidation, following the SHANSEP procedure for clays (Ladd and Foott, 1977). To facilitate the use of the proposed normalized undrained shear strength vs. OCR relationship, empirical equations are established to predict the apparent pre-consolidation stress from unconfined compressive strength  $\sigma_c$  and seismic P-wave velocity. Together with the in situ effective vertical stress prior to excavation at the tunnel location, the apparent pre-consolidation from  $\sigma_c$  or P-wave velocity can be used to estimate the apparent OCR and thence the squeezing potential. The proposed approach is compared with available field data and to existing methods to predict tunnel squeezing potential.

# 1 INTRODUCTION

Barla (2001) defined squeezing "as the large timedependent convergence during tunnel excavation. It takes place when a particular combination of induced stresses and material properties pushes some zones around the tunnel beyond the limiting shear stress at which creep starts. Deformation may terminate during construction or continue over a long period of time." Several other definitions of tunnel squeezing have been proposed including those by Gioda (1982), O'Rourke (1984), Kovari (1988), Singh (1988), and Aydan et al. (1993). A full review of the different definitions is given in Barla (2001). In general, squeezing cannot always be distinguished from swelling conditions (Steiner, 1993).

The tunnel squeezing phenomenon was first described by Terzaghi (1946) who associated

squeezing mainly with clay-rich rocks. Consistent with Terzaghi's original description, the focus of this paper is on clay-rich rocks such as clayshales, although squeezing can also occur in other rock types. Several procedures have been developed to predict the squeezing potential of tunnels. One of the first stability criteria to predict squeezing was developed by Peck (1969) for tunnels in clays based on Broms and Bennermark's (1967) stability criterion for open excavations. He proposed a stability number N which is expressed as the ratio between the total vertical stress  $\sigma_v$  at the tunnel location and the undrained shear of the clay  $S_u$ :

$$N = \frac{\sigma_v}{S_u} \tag{1}$$

Values of N have been correlated with observations of tunnel stability response, and these correlations are

Table 1.Stability criteria for tunnels in cohesive soils (Peck1969).

Ν	Problems encountered
1–5	Tunneling without unusual difficulties
5-6	Clay may squeeze rapidly into shield void
6–7	Shear failure ahead of tunnel causes ground movements into the face even in shield tunneling
>7	General shear failures and ground movements around tunnel heading cause shield contact to become difficult; shield tends to dive

summarized in Table 1. Cases where N > 5 are considered to have high degrees of rapid squeezing, and higher values of N indicate potential for general shear failure and large tunnel displacements close to the tunnel heading.

For tunnels in rocks, several tunnel squeezing criteria, which are mostly empirical, have also been proposed. Singh et al. (1992) developed a criterion based on the Q-system of rock mass classification (Barton et al. 1974) and the overburden height H(in m) which separates the squeezing cases from the non-squeezing cases:

$$H = 350 Q^{1/3}$$
(2)

Tunnels with overburden height greater than that given in Eq. (2) will experience squeezing.

Semi-empirical approaches have also been proposed by Goel et al. (1995, 2000), Jethwa et al. (1984), Aydan et al. (1993), and Hoek and Marinos (2000). The last three criteria use a stability number, which is a reciprocal of Peck's stability. Almost all these squeezing criteria summarized are empirically or semi-empirically based on direct observations of tunnel response. In the semi-empirical approaches of Jethwa et al., Aydan et al., and Hoek and Marinos, the ratio of capacity and load (or strength and stress) is analogous to the factor of safety FS (or the reciprocal if the load to capacity ratio is used). In addition to linking the degree of squeezing to the factor of safety, the advantage of using stress and strength is that these can be related to strain levels, provided the full stressstrain curve are known, which in turn can be related to degree of squeezing in the tunnel.

The main challenge in the use of these semiempirical approaches is in the determination of the rock mass strength. In Peck's approach, the strength is expressed in terms of the undrained shear strength, while in Jethwa et al., Aydan et al., and Hoek and Marinos, the strength is expressed in terms of the unconfined compressive strength of either the intact rock or the rock mass. It should also be noted that the proposed methodologies have been developed mainly for clays or hard rocks. Fewer studies have investigated the applicability of the above mentioned criteria to intermediate materials such as hard soils or soft rocks (e.g. Hoek and Marinos, 2000). Based on these observations, the main objective of this paper is to propose simple methods to estimate the shear strength of intermediate soil-rock materials, particularly cemented clays and shales, and to use the simple methods to estimate squeezing potential in tunnels. Estimates of the shear strength of shales are obtained from a database of triaxial test data on several clayshales. These estimates of shear strengths are then linked to field observations of squeezing to develop simple procedures for the preliminary investigation of squeezing in tunnels in clays and shales.

# 2 NORMALIZED UNDRAINED SHEAR BEHAVIOR OF CLAY AND SHALES

A procedure that is widely used to characterize the undrained shear strength of clays is the SHANSEP (Stress History and Normalized Soil Engineering Properties) procedure developed by Ladd and Foott (1977). According to this procedure, the undrained shear strength  $S_u$  of normally consolidated (NC) clays normalized with respect to the current effective vertical stress  $\sigma'_v$  is unique, and for overconsolidated (OC) clays, the following relationship adequately represents the normalized undrained shear strength:

$$\frac{S_u}{\sigma'_v} = a(OCR)^b \tag{3}$$

where *OCR* is the overconsolidation ratio defined as the ratio between the maximum past effective vertical stress  $\sigma'_p$  and the current effective vertical stress  $\sigma'_v$ , that is:

$$OCR = \frac{\sigma'_p}{\sigma'_v} \tag{4}$$

The parameter *b* is an empirical exponent, and  $a = (S_u/\sigma'_v)_{\rm NC}$  is the normalized undrained shear strength of NC clay, i.e., the value of  $S_u/\sigma'_v$  for OCR = 1.

Numerous studies in the literature have shown the applicability of SHANSEP in representing the undrained shear strength of many types of clays, including marine, residual and glacial soils. Although some studies have shown the successful use of SHANSEP for clays with some degree of cementation (e.g. Bo et al. 2003), the applicability of SHANSEP to lithified materials like shales has not been fully established.

SHANSEP has been successfully used in practice in geotechnical analysis and design, and it would

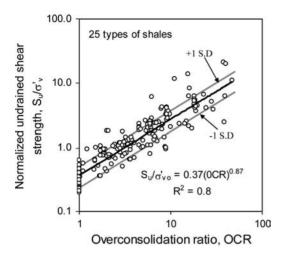


Figure 1. Normalized undrained shear strength as function of apparent OCR for 25 different types of shales.

be valuable to extend the procedure to shales and highly cemented cohesive materials. The main difference with clays is that overconsolidation and shear strength in shales are effected not only by mechanical loading-unloading, but also by the other diagenetic processes particularly cementation at the clay particle contacts. The increase in overconsolidation due to non-mechanical processes is called apparent or quasi preconsolidation (e.g. Bjerrum and Wu, 1960).

To investigate the applicability of SHANSEP to shales, a database of triaxial test results on 25 types of clayshales from different locations was assembled. Included in this database are consolidated undrained (CU) triaxial test results on shales with different consolidation stresses. The normalized undrained shear strength of 25 types of materials are plotted against apparent OCR in Fig. 1. Only clayshales, which are shales containing more than 50% clay particles by weight, and stiff cemented clays are included in the study. The apparent OCR is defined as ratio of the current effective vertical stress and the apparent preconsolidation stress, i.e.  $OCR = \sigma'_v / \sigma'_v$ , where  $\sigma'_v$  is simply the effective vertical stress at which yielding can be observed from the experimental consolidation stress-strain curve. In case of uncemented materials,  $\sigma'_{y} = \sigma'_{p}$ 

The results show a linear relationship between the  $\log(S_u/\sigma'_v)$  and  $\log(OCR)$ , which agrees with the power function given in Eq. (3). The reasonably good correlation between  $S_u/\sigma'_v$  and OCR for 25 types of materials is very promising, and indicates that SHANSEP can provide a reliable approach to predicting the undrained shear strength of shales. The average values of *a* and *b* for the 25 different materials are equal to 0.37 and 0.87, respectively. In comparison, Ladd and Foott (1977)

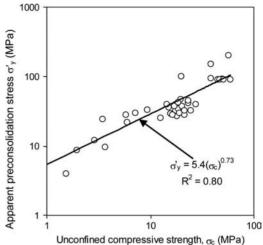


Figure 2. Correlation between apparent preconsolidation and unconfined compressive strength for different shales.

obtained values of a = 0.20 and b = 0.77 for several clays from CU triaxial tests. It is important to note that the data shown in Fig. 1 are for clayshales with different degrees of diagenesis and cementation. Although the normalized undrained shear strength of shale samples are similar, more lithified samples are actually stronger (i.e., have higher undrained shear strength  $S_u$ ) than younger uncemented samples because of higher values of the apparent preconsolidation stress.

# 3 DETERMINATION OF THE APPARENT PRECONSOLIDATION STRESS IN SHALES

The most widely used laboratory approach to determine  $\sigma'_{v}$  experimentally is Casagrande's (1936) procedure where  $\sigma'_{v}$  corresponds to the sharpest bend in consolidation plot. In addition to this approach, it is useful to develop procedures to estimate the apparent preconsolidation stress from simple index tests to facilitate the application of SHANSEP to shales. One parameter that can be obtained with relative ease in the field or in the laboratory is the unconfined compressive strength  $\sigma_c$ . In the following, it is assumed that the intact rock and rock mass  $\sigma_c$  are the same, as was done by Aydan et al. (1993), and Hoek and Marinos (2000). Figure 2 presents a plot of  $\sigma'_v$  vs.  $\sigma_c$  (both in MPa) for different clayshales, which shows a reasonable correlation between the two parameters of the following form:

$$\sigma'_{v} = 5.4 (\sigma_{c})^{0.73}$$
(5)

Although there is some scatter in the data, they are in the range of typical  $\sigma_c$  values of about 1 to

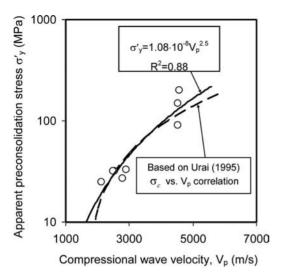


Figure 3. Correlation between apparent preconsolidation and compressional wave velocity for different shales.

70 MPa for shales. It should be reasonable to expect that a correlation exists between  $\sigma'_y$  and  $\sigma_c$  since both can be considered as material parameters, which depend on the degree of mechanical consolidation and cementation.

A more convenient procedure is to relate  $\sigma_c$  with data that can be directly measured in the field using in situ tests such as the compressional wave velocity  $V_p$ . One such correlation obtained by curve-fitting through experimental data, shown in Fig. 3 in terms of  $\sigma'_y$  (in MPa) and compressional wave velocity  $V_p$  (in m/s), also assumed the same for the intact rock and the rock mass, is obtained as

$$\sigma'_{y} = 1.08 \cdot 10^{-8} \left( V_{p} \right)^{2.5} \tag{6}$$

Again, data are limited but are within the typical range of  $V_p$  of 2 to 5 km/s for shales. To support the reliability of Eq. (6), an additional relationship is formulated via the following correlation for shales developed by Urai (1995) between  $\sigma_c$  (in MPa) and  $V_p$  (in m/s):

$$\log(\sigma_c) = -6.36 + 2.45 \log(0.86 V_p - 1172)$$
(7)

Substituting Eq. (7) in (5) provides the second  $\sigma'_y$  vs.  $\sigma_c$  curve shown in Fig. 3, which is in close agreement with Eq. (6). The similarity of the two curves shown in Fig. 3 appears to support the correlation between  $\sigma'_y$ and  $V_p$ . Obviously more data are needed, but Eqs. (5) to (7) can provide preliminary estimates of the apparent preconsolidation stress in the field.

# 4 SQUEEZING POTENTIAL OF TUNNELS IN CLAYS AND SHALES

Substituting Eq. (3) in Eq. (1) yields the following expression for the stability number:

$$N = \frac{\sigma_v}{S_u} = \frac{\sigma_v}{a\sigma'_v (OCR)^b}$$
(8)

The total and effective vertical stresses are related to the depth to the tunnel *H* and the total unit weight  $\gamma$ and the effective unit weight  $\gamma'$  of the material above the tunnel, i.e.  $\sigma_v = \gamma H$  and  $\sigma'_v = \gamma' H$ . Both these unit weights depend on the specific gravity of the solids  $G_s$ , the porosity *n* of the material, and the degree of saturation. Assuming fully saturated conditions, the ratio  $\sigma_v / \sigma'_v$  can be expressed as:

$$\frac{\sigma_{\nu}}{\sigma_{\nu}'} = \frac{\gamma}{\gamma'} = F(n) = \frac{G_s(1-n)+n}{(G_s-1)(1-n)}$$
(9)

Re-writing Eq. (8)

$$N = \frac{F(n)}{a(\sigma_y'/\sigma_y')^b}$$
(10)

and solving for the effective vertical stress gives a relationship between the effective vertical stress and the apparent yield stress:

$$\sigma'_{y} = \left(\frac{F(n)}{aN}\right)^{(-1/b)} \sigma'_{y} \tag{11}$$

In turn, substituting Eqs. (5) and (6) in the above equation results in the following relationships between the effective vertical stress, and the unconfined compressive strength and the P-wave velocity:

$$\sigma_{\nu}' = \left(\frac{F(n)}{aN}\right)^{(-1/b)} \left(5.4\sigma_{c}^{0.73}\right)$$
(12)

$$\sigma'_{v} = \left(\frac{F(n)}{aN}\right)^{(-1/b)} \left(1 \cdot 08 \cdot 10^{-8} V_{p}^{2.5}\right)$$
(13)

Equations (12) and (13) may be viewed as stability criteria which relate the effective vertical stress corresponding to the tunnel depth  $\sigma'_v$ , the unconfined compressive strength  $\sigma_c$  or P-wave velocity  $V_p$ , the stability number N, the porosity function F(n), and the empirical constants a and b. Tunnels with  $\sigma'_v$ , larger than those given in Eqs. (12) and (13) have high potential for squeezing.

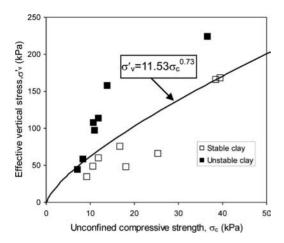


Figure 4. Comparison of the stability criterion given in Eq. (14) with observed cases of squeezing and non-squeezing in tunnels in clays (data from Broms and Bennermark, 1967).

# 5 COMPARISON WITH FIELD DATA AND OTHER EMPIRICAL TUNNEL SQUEEZING CRITERIA

To show their validity, the stability criteria given in Eqs. (12) and (13) are compared with field data on response of tunnels in clays and clayshales. The validity of Eq. (12) for tunnels in clays is shown in Fig. 4, which shows data taken from Broms and Bennermark (1967). The data have been re-plotted in terms of the effective vertical stress and unconfined compressive strength. A boundary curve between the non-squeezing and squeezing cases is established using Eq. (12) in conjunction with a value of  $F(n) \approx 2.5$  (based on typical values of  $n \approx 60\%$  and  $G_s = 2.7$  for clays), and a stability of number of N = 5.3. Also, values of a = 0.20 and b = 0.77 were used, which are representative values for clays given by Ladd and Foott (1977). Substituting these values in Eq. (12) gives the following criterion which demarcates cases of squeezing and non-squeezing for tunnels in clays:

$$\sigma'_{v} = 11.53\sigma_{c}^{0.73} \qquad (\sigma'_{v} \text{ and } \sigma_{c} \text{ in kPa}) \qquad (14)$$

It can be seen that Eq. (14) provides a clear boundary between cases of squeezing and non-squeezing in tunnels in clays.

For clayshales, the validity of Eq. (12) is validated by comparison with data on Himalayan tunnels collected by Bhasin (1991). The data of Bhasin (1991) have been re-plotted as for the data for clay and are shown in Fig. 5. A boundary curve between the nonsqueezing and squeezing cases is established using Eq. (12) with values of  $F(n) \approx 1.8$  (for typical values of  $n \approx 30\%$  and  $G_s = 2.7$  for shales), N = 2.5, and

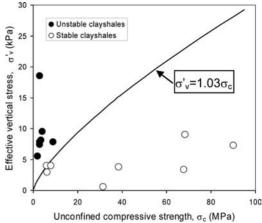


Figure 5. Comparison of the stability criterion given in Eq. (15) with observed cases of squeezing and non-squeezing in tunnels in clayshales (data from Bhasin, 1991).

a = 0.37 and b = 0.87 taken from Fig. 1. Substituting these values in Eq. (12) gives the following equation:

$$\sigma'_{v} = 1.05 \sigma_{c}^{0.73} \qquad (\sigma'_{v} \text{ and } \sigma_{c} \text{ in MPa}) \qquad (15)$$

It can be seen that Eq. (15) provides a clear boundary between cases of squeezing and non-squeezing in tunnels in shales.

Figure 6 shows the combined data from clays and clayshales. A boundary curve demarcating the case of squeezing from non-squeezing for tunnels in both clays and shales is established from the average of the stability criteria given in Eqs. (14) and (15). This curve is shown in Fig. (6), which shows that Eq. (15) can adequately separate the case histories of squeezing and non-squeezing for tunnel in both clays and clayshales.

$$\sigma'_{v} = 0.0145 \sigma_{c}^{0.6} \qquad (\sigma'_{v} \text{ and } \sigma_{c} \text{ in MPa}) \qquad (16)$$

Figure 6 also shows the stability criterion of Singh et al. (1992) combining Eq. (2) with the empirical relationship between  $\sigma_c$  and Q-value also from Singh et al. (1992). It can be seen from Fig. 6 that the Singh's criterion significantly underestimates the boundary between squeezing and no-squeezing for both clays and shales.

The final comparison with field data is shown in Fig. 7 to show the validity of the criterion given in Eq. (13). The unconfined compressive strength data of Bhasin (1991) were converted to  $V_p$  using Eq. (7). Using similar parameters used for Eq. (15), the following stability criterion is obtained in terms of  $\sigma'_v$  and  $V_p$ :

$$\sigma_v' = 1.22 \cdot 10^{-8} V_p^{2.5} \tag{17}$$

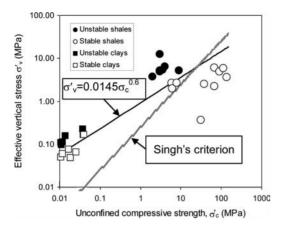


Figure 6. Comparison of the stability criterion given in Eqs. (16) and (18) with observed cases of squeezing and non-squeezing in tunnels in clays and shales (data from Broms and Bennermark, 1967; and Bhasin, 1991).

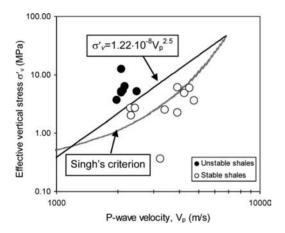


Figure 7. Comparison of the stability criterion given in Eq. (17) with observed cases of squeezing and non-squeezing in tunnels in shales (data from Bhasin, 1991).

As can be seen, Eq. (17) provides an adequate criterion for separating case histories of squeezing and non-squeezing in tunnels in clay and shales. Also shown is the criterion of Singh et al. (1992) re-plotted in terms of  $V_p$  by using the relationship between  $V_p$  and Q-value developed by Barton (2002). It can be seen that Singh's criterion expressed in terms of  $V_p$  underestimates the boundary between cases of squeezing and non-squeezing.

# 6 CONCLUSIONS

Squeezing criteria for materials that range from clays to clayshales were proposed. The criteria were based on normalized undrained shear strength as embodied in the SHANSEP procedure for clays. It was shown that the undrained shear strength of clayshales also exhibit normalized behavior similar to unlithified clays. In addition to relating stability to overconsolidation ratio, squeezing criteria in terms of unconfined compressive strength and P-wave velocity were proposed.

The validity of the proposed criteria was demonstrated by comparison with field observed squeezing in tunnels in clays and shales. It was shown that the proposed criteria are capable of delineating cases of squeezing and non-squeezing in tunnels in clays and clay shales. The proposed criteria provide better predictions of squeezing in clays and clayshales than other empirical criteria for hard rocks such as the one proposed by Singh et al. (1992).

The proposed criteria can be used for preliminary analysis of squeezing in tunnels in clays and clayshales where there are limited data. For tunnels with sufficient shear strength data, the paper has demonstrated the possibility of using SHANSEP procedure in conjunction with project specific soil parameters to estimate tunnel stability.

# ACKNOWLEDGEMENT

This paper are based upon works supported by the National Science Foundation under Grant No. 0324889 and Shanghai Leading Academic Discipline Project, Project Number: B308. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the National Science Foundation.

# REFERENCES

- Aydan, O., Akagi, T. & Kawamoto, T. 1993. The squeezing potential of rock around tunnels: theory and prediction. *Rock Mech. Rock Eng.* 2: 137–163.
- Barla, G. 2001. Tunneling under squeezing rock conditions. Lecture Notes, Eurosummer-School in Tunnel Mechanics, Innsbruck.
- Barton, N., Lien, R. & Lunde, I. 1974. Engineering classification of rock masses for the design of tunnel supports. *Rock Mech.* 6(4): 189–239.
- Barton, N. 2002. Some new Q-value correlations to assist in site characterisation and tunnel design. *Intl. J. Rock Mech. Mining Sci.* 39: 185–216.
- Bhasin, R. 1991. Evaluation of soft rock conditions in tunnels through the Lower Himalayan regions; A Contribution for updating of the Q-system. MSc Thesis, University of Oslo, 1991.
- Bjerrrum, L. & Wu, T.H. 1960. Fundamental shear strength properties of the Lilla Edet clay. *Géotechnique*, 10(3): 101–109.

- Bo, M.W., Choa, V. & Hong, K.H. 2003. Material characterization of Singapore Clay at Changi. J. Eng. Geol. Hydrogeol. 36: 305–319.
- Broms, B. & Bennermark, H. 1967. Stability of clays at vertical openings. Swedish Geotechnical Institute Publ. No. 16.
- Casagrande, A. 1936. The determination of the preconsolidation load and its practical significance. *Proc. 1st Intl. Conf. Soil Mech. Fnd. Eng.*, Cambridge, Mass. 60.
- Gioda, G. & Cividini, A. 1996. Numerical methods for the analysis of tunnel perform-ance in squeezing rocks. *Rock Mech. Rock Eng.* 29(4): 171–193.
- Goel, R.K., Jethwa, J.L. & Paithakan, A.G. 1995. Tunnelling through the young Himalayas – A case history of the Maneri-Uttarkashi power tunnel. *Engrg. Geol.* 39: 31–44.
- Hoek, E. & Marinos, P. 2000. Predicting tunnel squeezing problems in weak heterogeneous rock masses. *Tunnels Tunnel. Intl.*: pp. 45–51 (part one), 33–36 (part two).
- Jethwa, J.L., Singh, B. & Singh, B. 1984. Estimation of ultimate rock pressure for tunnel linings under squeezing rock conditions – a new approach. In E.T. Brown, J.A. Hudson (eds.), *Design and Performance of Underground Excavations, ISRM Symposium, Cambridge*: 231–238.

- Kovari, K. 1998. Tunnelbau in druckhaftem Gebirge Tunnelling in squeezing rock. *Tunnel* 5: 12–31.
- Ladd, C.C. & Foott, R. 1977. New design procedure for stability of soft clays. J. Geotech. Eng. Div., ASCE, 100(GT4): 763–779.
- O'Rourke, T.D. 1984. *Guidelines for tunnel lining design*. ASCE.
- Peck, R.B. 1969. Deep excavations and tunneling in soft ground. State of the art volume, 7th Intl. Conf. Soil Mech. Fnd. Eng., Mexico: 225–282.
- Singh, B., Jethwa, J.L., Dube, A.K. & Singh, B. 1992. Correlation between observed supportpressure and rock mass quality. *Tunnel. Undergr. Space Tech.* 7: 59–74.
- Steiner, W. 1993. Swelling rocks in tunnels: Rock characerization, effect of horizontal stress and construction procedures. *Intl. J. Rock Mech. Min. Sci. & Geomech. Abstr.* 30(4): 361–380.
- Terzaghi, K. 1946. Rock defects and loads in tunnel supports. Rock tunneling with steelsupports. In R.V. Proctor & T.L. White (eds.), *The Commercial Shearing and Stamping Co., Youngstown, Ohio*: 17–99.
- Urai, J.L. 1995. Brittle and ductile deformation of mudrocks. EOS Transactions, American Geophysical Union, Nov. 7, 1995, F656.

# Framework of performance-based fire protection design method for road tunnel

# X. Han

Shanghai Institute of Disaster Prevention and Relief, Tongji University, Shanghai, P. R. China

# G.Y. Ding

Shanghai Tunnel Engineering Co. Ltd, Shanghai, P. R. China

ABSTRACT: Under the background of performance-based code for fire protection design of building, and on the basis of the state-of-the-art of fire safety engineering as well as prescriptive-based code for fire protection design of road tunnel, this paper outlines the framework of performance-based fire protection analysis and design method for large cross-section road tunnel. The framework provide a preliminary guidance on the application of scientific and engineering principles to the protection of such road tunnels from the unwanted effects of fire.

# 1 INTRODUCTION

With rapid development of tunnel construction in China, the number of road tunnels has increased sharply to serve the perpetual growth of both freight and transport for the populace. The consequence of the growth in the number of road tunnels and the volume of traffic is an increasing occurrence of severe fire incidents in tunnels. Also, as tunnel construction technology has evolved, so the feasible length and cross-section of road tunnels have expanded. Subsequently, the control of such incidents has become more difficult for the emergency services of large cross-section road tunnel.

Currently, the fire protection design of most road tunnels are in accordance with prescriptivebased codes. Prescriptive-based codes provide specific requirements for broad classifications of tunnels that establish acceptable or tolerable levels of risk for a variety of health, safety, and public welfare issues. These requirements are generally stated in terms of fixed values, such as maximum travel distance, minimum fire resistance ratings, and minimum features of required systems (e.g. detection, alarm, suppression, and ventilation), and not in terms of overall tunnel performance. However, the construction of large cross-section road tunnel would challenge more complicated fire protection requirement and should be provided scientific and more cost-effective fire protection solution.

This paper established a framework of performancebased fire protection analysis and design method for large cross-section road tunnel. The framework provided preliminary guidance on the application of scientific and engineering principles to the protection of such road tunnels from the unwanted effects of fire. It also constructed a process for undertaking a performance-based fire safety engineering approach to road tunnel fire safety analysis and design. The method requires the use of a variety of tools in the analysis, bringing increased engineering rigor and often resulting in innovative design options. It allows the safety levels provided by alternative design options to be compared. In this way, it could result in a comprehensive fire protection strategy for large crosssection road tunnel in which all fire safety systems are integrated, rather than designed in isolation.

# 2 MAIN FRAMEWORK

The establishment of performance-based fire protection analysis and design method for large cross-section road tunnel is under the background of performancebased code for fire protection design of building and the prescriptive-based code for fire protection design of tunnel. The whole performance of fire protection for road tunnel should be comprehensively considered. By means of developing design fire scenarios, the fire protection goals, such as provide life safety, protect property and fireproof of tunnel structure, have to be anticipated. Thereby, the fire protection level of road tunnel could be synthetically assessed according to design criteria and the corresponding fire protection measures could be determined. The construction process of the main framework involves three steps: (1) Concept Establish; (2) System Analysis; (3) System Design. Among these steps, step of Concept Establish is to definite core need of the method, step of System Analysis is to set up total model of the framework and step of System Design is to create the implement system.

# 2.1 Concept establish

In order to enable the method be applied in the practical fire protection engineering of large cross-section road tunnel, the fundamental concept of the framework should be established on the basis of current fire protection codes.

During the evaluation process of fire protection feature for large cross-section road tunnel, the performance of corresponding fire protection facilities could be confirmed. The traffic volume in the road tunnel could be anticipated and the related fire protection management could be ascertained. In general situation, the assumed conditions tally with practical project. As for some special situation, the further research works have to be carried out.

# 2.2 System analysis

Based on fundamental concept of framework establishment, facing qualified designer, authority having jurisdiction, owner and management personnel involving fire protection of road tunnel, the basic framework of fire protection method for large cross-section road tunnel is constructed. The framework consists of three modules, including Comprehensive Evaluation, Information Integration as well as Performance Prediction respectively.

The framework of performance-based fire protection analysis and design method for large cross-section is shown in Figure 1.

The main function of modules is as follows.

Module 1: Comprehensive Evaluation

This module is the key part of the framework, including main functions such as:

- 1. Define object of fire protection design;
- 2. Establish design criteria of fire protection;
- 3. Evaluate comprehensively the fire protection features.

Module 1 is appropriate for stakeholders of road tunnel, including road tunnel owner, qualified designer, authority having jurisdiction and insurer, etc.

Module 2: Information Integration

This module is the basic part of the framework, including main functions such as:

1. Collect corresponding design materials according to current prescriptive-based code. Determine different kinds of parameters for fire protection design of road tunnel;

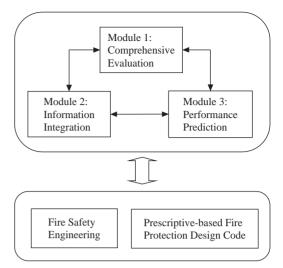


Figure 1. Framework of performance-based fire protection analysis and design method for large cross-section road tunnel.

- Collect basic data of fire protection design. The detailed design parameters included road tunnel feature, vehicle variety, fire characteristic and fire protection facility, etc.
- 3. Definite other comprehensive performance, including background information related with fire protection design.

Module 2 is applicable for qualified designers of road tunnel.

Module 3: Performance Prediction

This module is the quantitative analysis part of the framework, including main functions such as:

- 1. Define design fire scenario. According to statistical data of tunnel fire and design goal of fire protection, the design fire scenarios should be chosen.
- 2. Definite performance anticipation approach. The analyses may be deterministic or probabilistic, deterministic method would normally be used.
- 3. Predict system performance. Taking the whole fire process as object of study, all components or subsystems which possibly affect the fire protection feature of the road tunnel should be thoroughly analyzed. As for performance evaluations, the components or subsystems of the fire safety system as well as the interactions between the subsystems would need to be considered. The subsystems that may be concerned include: (1) fire initiation and development; (2) spread, control and management of smoke; (3) fire detection; (4) fire suppression; (5) personnel behavior and egress; (6) passive fire protection.

Module 3 is applied for professional assessment personnel and researcher.

# 2.3 System design

In accordance with the function specificities determined by the three modules for fire protection analysis and design method of large cross-section road tunnel, the detailed content of corresponding implementation structure constructed by System Design is presented in Figure 2.

# 2.3.1 Define scope of large cross-section road tunnel

The scope of large cross-section road tunnel is an identification of the boundaries related with the performance-based analysis or design, including: road tunnel constraints, design and construction team organization, project schedules, applicable regulations and other useful information to assist in comprehending the scope definition.

# 2.3.2 Determine design goal and objective of fire protection

Fire protection of large cross-section road tunnel generally has four interrelated fundamental fire safety goals: (a) Provide life safety for the public and fire fighters. Minimize fire-related injuries and prevent undue loss of life. (b) Protect road tunnel. Minimize damage to road tunnel from fire and fire protection measures. (c) Provide for continuity of road tunnel operations (i.e., protect the passage of vehicles). Minimize undue loss of operations due to fire-related damage. (d) Limit the environmental impact of fire and fire protection measures.

Once the fire protection goals have been established and agreed to, the design objectives to meet the fire protection goals must be defined. The design objective provides more detail than a fire protection goal, and is often stated in terms meeting the requirements of a specific code or standard provision (prescriptiveor performance-based), of a specific insurance-related requirement, or in addition to a specific code, standard, or insurance provision or requirement.

# 2.3.3 Collect related performance information

There are many types of information that may affect road tunnel design performance that should be considered and collected, including characteristics such as: vehicle and personnel, location of the tunnel, fire service, utilities, environmental considerations, tunnel management and security, economic and social value of the tunnel, the tunnel delivery process, applicable regulations, etc: It is quite important to identify the appropriate codes, regulations and insurance requirements for the performance-based analysis.

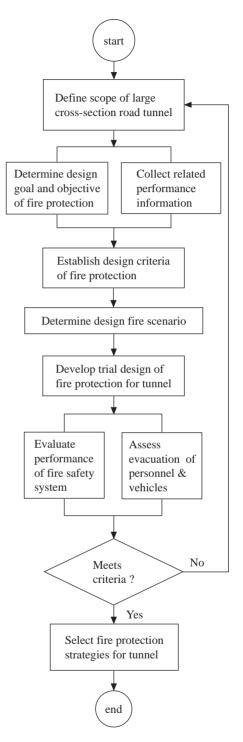


Figure 2. Fundamental content of performance-based fire protection analysis and design method for large cross-section road tunnel.

## 2.3.4 Establish design criteria of fire protection

The design criteria of fire protection for road tunnel in this paper are mainly concerned performance criteria. Performance criteria are usually threshold values, ranges of threshold values, or distributions that are used to develop and evaluate trial designs for a given design of large cross-section road tunnel. Performance criteria may include temperatures of materials, gas temperatures, smoke concentration or obscuration levels, radiant flux levels, and human response, decision, reaction, and movement times. For example, performance criteria may include values for thermal radiation exposure  $(kW/m^2)$ or gas (air) temperature. Other types of performance criteria include concentration of toxic gases (ppm), distance of the smoke layer above the floor (m), visibility(m), or other measurable or calculable parameters.

## 2.3.5 Determine design fire scenario

In case the performance criteria have been established, the qualified designer needs to focus on the development and analysis of design alternatives to meet these criteria. The possible fire scenarios should be considered are then filtered into selected design fire scenarios. As design fire scenarios constructed, then trial designs of road tunnel could be developed and evaluated to determine whether the performance criteria will be successfully met by the trial design for a given design fire scenario.

A fire scenario represents one of a set of fire conditions that are thought to be threatening to a certain road tunnel. For a given fire scenario of road tunnel, there are many factors that may affect fire development. These different factors may include: (a) form of ignition source; (b) different items first ignited; (c) ignition in different locations of a road tunnel; (d) effects of tunnel geometry; (e) ventilation, whether longitudinal ventilation or transverse ventilation, etc; (f) form of intervention (i.e. personnel, sprinklers, the fire department, etc.).

# 2.3.6 Develop trial design of fire protection for tunnel

After the performance criteria have been established and the design fire scenarios have been determined, the trial design of the road tunnel should be developed. Developing trial design may simply require selecting features similar to that of the prescriptive based design option, but with enhanced capabilities or features. The design features being developed should give consideration to the capabilities, reliability, costs and maintenance. However, it should be considered that the trial design would be evaluated. Trial design can be evaluated on a system performance that relies on an evaluation relative to established performance criteria.

# 2.3.7 Evaluate trial design of fire protection for tunnel

Evaluation is the process of determining if a trial design meets the performance criteria during the postulated design fires. The intent is to demonstrate that in the design fire scenario, performance criteria will not be exceeded.

If the trial design of the road tunnel is found successful, any remaining trial designs may be evaluated as necessary. If one of trial designs is not found to be successful, then it may be modified and re-tested, or it may be dismissed. After the selected trial designs have been tested, a final design should be selected from among those found successful. If there are no trial designs that are found successful, the qualified designer should ensure that the trial designs considered all possible mitigation strategies. If after considering all possible mitigation strategies, there still are not any trial designs that are found successful, the design goal and objective of fire protection as well as the performance criteria should be reexamined.

Many techniques can be used to evaluate the adequacy of a trial design for road tunnel in case of fires. These techniques typically fit into two principal categories, probabilistic and deterministic. A deterministic analysis examines the hazard posed by the potential design fire scenarios independently. A probabilistic evaluation uses risk analysis to identify consequences of specific events and their respective likelihood.

The evaluation of a performance-based fire protection design for a large cross-section road tunnel is a function of several factors. These factors include: (a) complexity of road tunnel geometry; (b) level of subsystem interaction; (c) type of performance criteria; (d) sensitivity of subsystem output to design objectives; (e) absolute or comparative evaluation; (f) knowledge level; (g) benefit versus cost; (h) expert judgment and experience. The evaluation should also account for known variations and uncertainties.

The main evaluation content of the trial design for road tunnel may concern evaluating performance of corresponding fire safety system as well as assessing evacuation of personnel and vehicles, etc. It is necessary to point out that timelines could be valuable tools in evaluation. Therefore, it may be essential to determine the time of key events such as: (a) ignition; (b) fire detected; (c) evacuation begins; (d) untenable conditions reached in road tunnel; (e) fire spreads scope; (f) suppression begins; (g) failure of structural elements; (h) fire extinguished.

# 2.3.8 Select fire protection strategies for tunnel

In term of performance-based fire protection design of a large cross-section road tunnel, while the acceptable trial designs are identified by the evaluation, they can be considered for the selection of fire protection strategies. The choice of which acceptable trial design is selected for the final design may be based on a variety of different factors, including financial considerations, timeliness of installation, system and material availability, ease of installation, maintenance and use, and other factors.

Once the fire protection strategies are identified, the relevant design documents need to be prepared. Proper documentation will ensure that all stakeholders understand what is necessary for the strategies implementation, maintenance and continuity of the fire protection design. The documentation should include the design brief, a performance design report, detailed specifications and drawings, and a road tunnel operations and maintenance manual.

# 3 CONCLUSION

It is evident that the framework of performancebased fire protection analysis and design method for a large cross-section road tunnel offers a number of advantages over prescriptive-based design. It provides a basis for development and selection of alternative fire protection options based on the road tunnel's needs and results in a composite fire protection strategy in which all fire safety systems are integrated, rather than designed in isolation. Hence, such a comprehensive engineering approach often provides more cost-effective fire protection solution for the road tunnel. Apparently the intensive study about the framework should be further conducted.

# ACKNOWLEDGMENTS

The support of the Natural Science Foundation of China (Grant No. 50678124) is gratefully appreciated.

# REFERENCES

- Eberl, G. 2001. The Tauern Tunnel Incident: What happened and What has to be Learned, *Proc. 4th Int. Conf. on Safety in Road and Rail Tunnels*, Madrid, Spain, 2–6th April 2001: 17–30.
- Han, X., Shen, Z.Y. et al. 2000. Study on a framework of performance-based assessment method for fire safety of large public building, *Proceeding of the Sixth International Symposium on Structural Engineering for Young Experts*, Yunnan Science and Technology Press, Kunming, China, August 2000: 18–20.
- Lacroix, H.D. 2001. The Mont Blanc Tunnel Fire: What Happened and What Has Been Learned, *Proc.4th Int. Conf.* on Safety in Road and Rail Tunnels, Madrid, Spain, 2–6th April 2001: 3–16.
- Turner, S. 2001. St. Gotthard Tunnel Fire, New Civil Engineer, 1st Nov. 2001: 5–7.

# Prediction of surface settlements induced by shield tunneling: An ANFIS model

# J. Hou, M.X. Zhang & M. Tu

Department of Civil Engineering, Shanghai University, Shanghai, P.R. China

ABSTRACT: A new method – fuzzy system combination neural network was used to estimate ground surface settlements. According to the measured data of Shanghai No.2 Subway, and considering various kinds of factors synthetically, an ANFIS fuzzy neural network prediction model was built. Comparing with the prediction results by other three kinds of methods, the validity of the ANFIS fuzzy neural network model was appraised. Confirmed by the instance, ANFIS fuzzy neural network is valuable in predicting ground settlements induced by shield tunneling.

# 1 INTRODUCTION

Shield tunneling has become one of the most popular methods used in the construction of urban tunnels, such as rapid transit systems and large diameter underground pipelines. However, shield tunneling construction inevitably disturbs the ground and the original stress field of soil, which in turn causes surface settlement that may yield damage of existing adjacent structures and underground facilities. Therefore, it is of significant importance for engineers to accurately predict the surface settlement during the design and construction stages.

Considering the complexity of the problem that involves intricate geological makeup of the ground composed of different materials with varying layer patterns plus different construction methods, it is apparent that using classical method makes it difficult to provide accurate predictions of ground settlement. In addition, it is very difficult to determine input parameters representative of the mixed geological compositions in a prediction model. Therefore, the analytical methods relying on observed data are widely used in investigation of surface settlement. A hybrid intelligent system called ANFIS(Jang 1993) (Adaptive-Network-Based Fuzzy Inference System) combining the ability of a neural network to fuzzy logic have the advantages of both neural networks (e.g. learning abilities, optimization abilities, and connectionist structure) and fuzzy systems (e.g. humanlike 'if-then' rules, and ease of incorporating expert knowledge and judgment available in linguistic terms). Such a hybrid intelligent system holds much potential in prediction. In this paper, a model based on ANFIS is proposed to predict the ground settlement induced by shield tunneling.

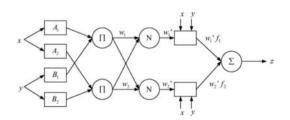


Figure 1. Architecture of ANFIS.

## 2 DEFINITION OF THE MODEL

# 2.1 Brief description on the Adaptive-Network-Based Fuzzy Inference System (ANFIS)

Both artificial neural network and fuzzy logic are used in ANFIS's architecture (Chang & Lee 2003). The ANFIS is one of the methods to organize the fuzzy inference system with given input-output data pairs. The ANFIS optimizes the parameters of consequent part using least square method and those of premise part using steepest descent method.

For simplicity, we assume the fuzzy inference system under consideration has two inputs (x and y) and one output (z) in the present study. The corresponding ANFIS architecture is shown in Figure 1.

Layer 1: In this layer x and y represent different types of input parameters of the adaptive node i of an adaptive function, and  $A_i$  is the linguistic label associated with this node function. Parameters in this layer are referred to as premise parameters.

Layer 2: Every node in this layer is a fixed node labeled  $\Pi$  which multiplies the incoming signals and

sends the product out. Each node output represents the firing strength of a rule.

Layer 3: Every node in this layer is a fixed node labeled N. The *i*th node calculates the ratio of the *i*th rule's firing strength to the sum of all rules' firing strengths.

Layer 4: Every node i in this layer is an adaptive node. Parameters in this layer will be referred to as consequent parameters.

Layer 5: The single node in this layer is a fixed node labeled  $\Sigma$  that computes the overall output as the summation of all incoming signals.

The steepest descent method can be applied to find the premise parameters and least square estimate can he applied to optimize the consequent parameters.

# 2.2 Input and output parameters

A relatively too large surface settlement may induce damage or even crash of existing structures. It is therefore very crucial to accurately predict the total surface settlement (the maximum surface settlement after construction) of a point before the shield arrives at this point. It is straightforward to set the total ground surface settlement to be an output variable in a prediction model. In this paper, similar to many other researchers, we choose the total surface settlement 5 meters ahead of the shield face as an output parameter. However, how to choose the input variables depends on the collection of monitored data. Systematic analyses of the measured data show that the disturbed area by shield tunneling of the original stress field of soil is approximately limited in a zone within 15 m from the working point, and almost has no effect on the area out of this zone. M. Karakus's measured data showed also that the maximum settlement is over the tunnel centerline. Considering these two aspects, we choose six parameters, i.e., five total ground surface settlements at the points over the tunnel centerline (i.e., 0, 5, 10, 20 and 30 meters behind the tunnel face) and one of the shield working parameters, the total number of the working cycles of the day, as the inputs.

The investigation in this paper is based on the measured data in the shielding tunnel from Zhongshan Park station to Longdong Road station in the Shanghai No. 2 Subway Tunnel Project. The total length of the tunnel is 1624 m. The earth pressure balance (EPB) shield was used in the project. The outer diameter of the EPB machine is 6340 mm and the length is 6540 mm. The tunnel shield tunneling was started in 18 July, 1997 and completed by 9 November, 1997, and the surface settlements were measured from 21 August. Large numbers of surface settlement markers were installed to measure surface settlements during excavation. The installation of surface settlement markers is described in detail by Sun and Yuan. The soil profile around the tunnel can also be found in the paper. The collected data have formed a database. The database not

Table 1. Samples used for network training.

Serial number	$X_1$	$X_2$	$X_3$	$X_4$	$X_5$	$X_6$	у
1 2	1.50	24.70 7.65	24.00	36.40	42.29	5	
3  81		52.10 14.85					

Table 2. Samples used for network validation testing.

Serial number	$X_1$	$X_2$	$X_3$	$X_4$	$X_5$	$X_6$	у
1	5.20	20.30	26.75	23.20	9.95	8	40.95
2	-0.05	-1.07	0.35	-2.45	4.95	12	62.45
3	13.07	54.42	82.62	96.60	25.45	11	89.87
4	10.00	27.25	63.00	81.10	91.60	11	53.50
5	14.45	51.30	57.85	74.85	90.52	10	54.50
6	4.75	12.65	28.55	38.11	46.20	4	57.00
7	6.80	52.10	72.57	89.67	98.75	8	83.15
8	3.20	9.75	2.60	1.42	-2.27	11	99.57
9	4.17	7.32	7.32	17.00	44.90	7	41.10
10	14.05	16.15	14.39	22.1	34.50	8	47.22
11	0.95	2.85	16.25	54.45	47.65	12	76.42
12	3.30	15.55	33.30	45.30	52.00	6	53.90
13	1.70	15.60	30.25	47.05	55.40	11	47.30
14	2.95	4.95	13.75	44.15	86.40	10	61.30
15	13.25	43.75	76.30	74.85	68.52	10	56.73
16	3.75	22.02	47.77	66.90	72.75	13	88.15
17	4.57	18.75	46.40	68.75	47.15	7	79.02
18	4.00	11.65	34.05	47.65	77.40	8	61.30
19	0.72	6.92	18.37	43.37	51.65	10	79.60
20	1.90	12.20	30.40	49.65	39.30	11	71.95

only allows one to study the behavior of ground movements occurring during excavation, but also becomes a useful source for developing predictive models for the ground settlement. We randomly chose 81 in the database experimental data (from August to October in 1997, see Table 1, detailed list can be found in Tu (2005)) as a training set and 20 data as a validating set (data in November, see Table 2).

Here,  $X_1$ ,  $X_2$ ,  $X_3$ ,  $X_4$ ,  $X_5$  is total ground surface settlements (unit: mm) at the points over the tunnel centerline (i.e., 0, 5, 10, 20 and 30 meters behind the tunnel face) separately.  $X_6$  is the total number of the working cycles of the day.

# 2.3 Data pretreatment

To speeding calculation, we standardize the original data to have a minimum of 0 and a maximum of 1 by

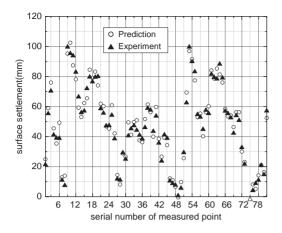


Figure 2. The fitting map of the ANFIS output and the measured data.

linear transformation. It is shown that such a transformation will make the calculation very efficient. The algorithm of the linear transformation is as follows.

The original data of six input variables can be expressed as:

$$S_{1} = (X_{1(1)}, X_{1(2)} \dots X_{1(P)}),$$
(1)

$$S_6 = (X_{6(1)}, X_{6(2)} \dots X_{6(P)}).$$

We note that i = 1, 2, ..., P is the number of samples and totally 81 samples are used in this paper to train the model, i.e., P = 81.

We normalize each input datum as

$$X_{1(i)}^{0} = \frac{X_{1(i)} - \min S_{1}}{\max S_{1} - \min S_{1}},$$

$$X_{6(i)}^{0} = \frac{X_{6(i)} - \min S_{6}}{\max S_{6} - \min S_{6}}.$$

Then the normalized input data can be expressed as

$$IN_{(i)} = (X_{1(i)}^0, X_{2(i)}^0, \dots, X_{6(i)}^0).$$

# 3 RESULTS AND DISCUSSION

# 3.1 Text and indenting

The present model is realized via a Matlab package of ANFIS. The fitting map of the ANFIS output and the measured data was obtained (Fig. 2).

Table 3. The comparison of predicted surface settlement and measured settlement.

Serial number	Measured settlement (mm)	Predicted Settlement (mm)	Absolute error (mm)	Relative error %
1	40.95	39.24	1.71	0.042
2	62.45	63.3	-1.15	0.018
3	89.87	92.34	-2.47	0.027
4	53.5	52.46	1.04	0.019
5	54.5	52.59	1.91	0.035
6	57	58.38	-1.38	0.027
7	83.15	81.55	1.6	0.019
8	99.57	103.28	-3.71	0.037
9	41.1	38.25	2.85	0.069
10	47.22	45.39	1.83	0.039
11	76.42	74.23	2.19	0.029
12	53.9	54.96	-1.06	0.02
13	47.3	48.71	-1.41	0.03
14	61.3	62.37	1.07	0.017
15	56.73	54.42	2.31	0.041
16	88.15	86.29	1.86	0.021
17	79.02	80.16	-1.14	0.014
18	61.3	63.91	-2.61	0.043
19	79.6	78.22	1.36	0.017
20	71.95	73.28	-1.33	0.018

The comparison of predicted surface settlement and measured settlement was also shown in Table 3.

They all show that all the predicted results from the ANFIS model are in good agreement with measured data.

Peck (1969) presented the first available method for estimating the ground settlement due to tunneling and excavation. In his method, charts were developed based on the field data obtained from subway constructions at different places worldwide. The data points, though scattered, revealed the shape of a Gaussian distribution curve. The charts have been widely used for estimating the transverse ground settlement profile caused by tunneling and excavation. Theoretical models based on the combined pi-sigma approach (Gupta & Rao 1994) and BP (Back Propagation) neural network are also used to predict the surface settlement. Due to the limitation of the paper length, we do not discuss them here.

Comparison between the above methods and the ANFIS method is shown in Fig. 3.

It is found that the current prediction is in good agreement with measurement. The maximum error between the model and the testing data is not more than 7%. For comparison, we proposed also the predicted results from two theoretical models based on the BP (Back Propagation) neural network and the combined pi-sigma approach (which are frequently used in predicting the surface settlement), and those from Peck's equation (Fig. 4). We can see from the figure that

(2)

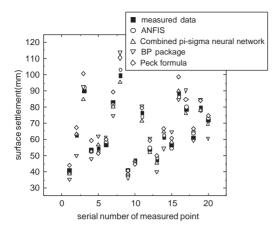


Figure 3. Comparison of predicted results with measured ones.

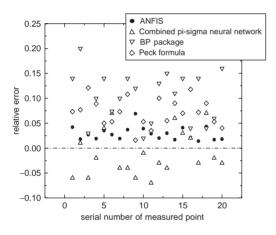


Figure 4. Comparison relative error from ANFIS' to other methods.

the BP neural network has the maximum error among these methods; that the combined pi-sigma approach underestimates but the Peck's formula overestimates the surface settlement.

# 4 CONCLUSION

This paper has shown the potential for applying fuzzy neural networks to ground surface settlement analysis. five total ground surface settlements at the points over the tunnel centerline (i.e., 0, 5, 10, 20 and 30 meters behind the tunnel face) and one of the shield working parameters, the total number of the working cycles of the day, as input parameters to predict the surface settlement induced by tunnel shield tunneling. The predicted results from the proposed model are in good agreement with field observations and the maximum error between the model and the testing data is not more than 7% in our example. Comparison with some other predicting approach shows that the present model is accurate, steady and efficient.

# REFERENCES

- Chang, K.H. & Lee, J.J. 2003. Adaptive Network-based Fuzzy Inference System with Pruning. SICE Annual Conference in Fukui, 4–6 August 2003. Japan: Fukui University.
- Gupta, M.M. & Rao, D.H. 1994. On the principles of fuzzy neural networks. *Fuzzy Sets and Systems*, 61(1):1–8.
- Jang, J.S. 1993. ANFIS: Adaptive-Network-based Fuzzy Inference Systems. *IEEE Transactions on Systems, Man* and Cybernetics, 23(3): 665–685.
- Karakus, M. & Fowell, R.J. Effects of different tunnel face advance excavation on the settlement by FEM. *Tunnelling and Underground Space Technology*, 2003, 18(5): 513–523.
- Peck, R.B. 1969. Deep excavation and tunneling in soft ground. Proceedings of 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico.
- Sun, J & Yuan, J.R. Soil disturbance and ground movement under shield tunnelling and it's intelligent prediction by using ANN technology. *Chinese Journal of Geotechnical Engineering*, 2001, 23(3):261–267. (in Chinese)
- Tu M. The prediction of the ground settlements induced by based on the Fuzzy Neural Network. *Shanghai university master's thesis.* 2005. (in Chinese)

# Experimental studies of a geological measuring system for tunnel with ultrasonic transducer

D.H. Kim, U.Y. Kim, S.P. Lee & H.Y. Lee

GS Engineering & Construction, Seoul, Korea

J.S. Lee Korea University, Seoul, Korea

ABSTRACT: Predicting ground conditions ahead of the tunnel face has been one of the most important requirements of tunnel construction. This study investigated the development and application of a high resolution ultrasonic wave imaging system, which captures the multiple reflections of ultrasonic waves at the interface, to detect discontinuities at laboratory scale rock mass model. Ultrasonic wave reflection imaging based on A- and B-modes was obtained through stacking, signal compensation, demodulation, and display. Experiments were carried out by using horizontal scanning and rotational scanning. Experimental studies showed that the rotational scanning method was able to identify horizontal and inclined discontinuities and the cavity on the plaster block at a fixed location. Furthermore, two discontinuities including horizontal and inclined discontinuity planes were detected. The rotating scanning technique produced images similar to those obtained by the typical horizontal scanning technique. This paper contains basic theories about the ultrasonic transducer and several experimental application results. The full-scale field application and other application will be scheduled in the future.

# 1 INTRODUCTION

With the rapid growth of the world's population and economics, increasing number of tunnels have been constructed to provide better transport links. Safe and economic tunneling has always been a challenging topic in tunnel construction because of complex ground conditions like faults, fractures, caverns, and wet layers. Therefore, the prediction of ground conditions ahead of tunnel face is considered one of the important requirements.

Many techniques to predict the ground condition ahead of the advancing tunnel face have been developed, improved and widely applied in tunnel construction projects. Three types of techniques are representives for tunnel ground condition predicting: coring, displacement monitoring and analysis, and wave-based non-destructive evaluations(Figure 1(a)).

In this study, a new high resolution seismic technique based on rotating scanning equipment is introduced to detect geological discontinuities, as shown in Figure 1(b). This technique uses the received time series which captures multiple reflections due to the impedance mismatch at a geological interface. The source and receiver transducers are located on the tunnel face at one fixed point. As a preliminary study of this technique, this paper describes experimental studies carried out on lab-scaled models. This paper

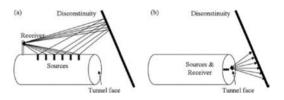


Figure 1. Seismic method for predicting tunnel face: (a) TSP method; (b) Rotating scanning method.

presents basic theories about the ultrasonic transducer including transducer frequency characteristics, transducer beam characteristics, and coupling layer characteristics, experimental setup and signal processing, application examples, discussion, and conclusions.

# 2 ULTRASONIC TRANSDUCER

Ultrasonic wave reflection imaging system may be an economical and effective tool for the forecast of the ground condition ahead of the tunnel face. A transducer refers to any device that can convert an electrical signal into a mechanical energy and vice versa. In ultrasonic reflection imaging, a piezoelectric leadzirconate-titanate (PZT) type transducer is generally used to as a source and detector (Wells, 1977). The selection of the transducer is the most important factor in ultrasonic reflection imaging because the transducer determines the image resolution and skin depth.

# 2.1 Transducer frequency characteristics

The choice of transducer frequency in ultrasonic reflection imaging is the result of a compromise between the resolution (lateral and axial) requirement and the acquirement of satisfactory beam penetration for the imaging of the part of interest. Lateral resolution refers to the ability to distinguish two closely spaced reflectors, which are positioned perpendicular to the axis of the ultrasound beam. Lateral resolution is most closely related to the transducer beamwidth. Axial resolution refers to the minimum reflector spacing along the axis of an ultrasonic beam that results in separate, distinguishable echoes on the display.

For a given frequency, the shorter the pulse duration, the wider the frequency bandwidth. If a shorter pulse duration is used, high resolution can be attained, but it lowers the sensitivity and skin depth. But sensitivity can be improved by either increasing the energy of the transmitter or by amplifying the captured signals at the receiver. Note, skin depth is the ability to detect an anomaly at a given depth and it depends on the amplitude of the reflected signal. When the resolution increases with increasing frequency, the skin depth decreases (Lee and Santamarina 2005).

# 2.2 Transducer beam characteristics

Near Field and Far Field. The transducer beam is characterized as near field and far field, which are sketched in Figure 2. In the near field, which is called the Fresnel zone, wave amplitude fluctuates. Note the beams are almost parallel rather than divergent in the near field. The near field length NFL or Fresnel zone length is dependent on the transducer radius r and wavelength  $\lambda$  (Krautkramer and Krautkramer, 1990; Rose, 1999).

$$NFL \approx \frac{\gamma^2}{\lambda} = \frac{r^2 f}{V}$$
 (1)

where f is the frequency, and V is the ultrasonic wave velocity of the medium. Therefore, the near field length NFL increases with the increase of the transducer radius r and/or the increase of the frequency f. The zone beyond the near field is the far field, which is also called the Fraunhofer zone. The divergence angle depends on the directivity.

*Directivity*. The far field divergence angle depends on the wavelength and radius r (or diameter d) of the transducer (Zagzebski, 1996), as shown in Figure 2.

$$\sin \theta = \frac{1.2\lambda}{d} = 1.2 \frac{V}{fd}$$
(2)

The divergence angle should be made small to increase lateral resolution. Higher frequency and larger diameter render higher lateral resolution (far field is assumed).

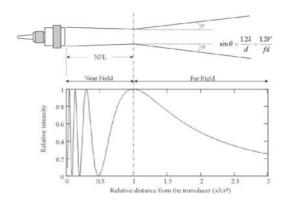


Figure 2. Beam characteristics of transducer.

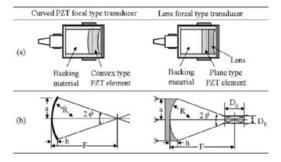


Figure 3. Transducer type and focal length: (a) Schematic drawing of transducer; (b) Focal length.

The directivity of the transducer is affected by the velocity of the test medium, and the frequency and size of the transducer. High directivity yields higher resolution. The directivity of an ultrasonic beam may be altered by focusing the beam: 1) a curved PZT element instead of a plane PZT element is used and 2) a lens is attached in front of the plane PZT element (Schmerr, 1998), as shown in Figure 3. Because the wave velocity in lens is greater than that in water, the shape of the lens will be concave. The focal length F is defined as the distance from the point on the curved surface on the central axis to the midpoint of the region of convergence. Note, if the lens is used, the focal zone instead of the focal point appears, as shown in Figure 3(b). The focal length is dependent on the radius of curvature R, the radius of the lens a, and the aperture angle  $2\psi$  (see Figure 3).

For the investigation of the directivity, two types of transducers are used: focal type transducer (Panametrics A3441) and non-focal transducer (Panametrics V318). Both are high damping immersion transducers having 500 kHz in resonant frequency and 19 mm in diameter. Directivities were investigated by measuring the amplitude of the signal at fixed axial distances (50 mm, 100 mm and 150 mm) and the lateral offsets of the transducers, as shown in Figure 4(a). A focal

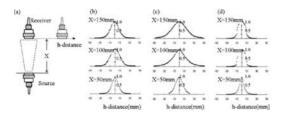


Figure 4. Directivity: (a) Test procedures; (b) Focal type transducer; (c) Non-focal transducer; (d) Lens focal type transducer.

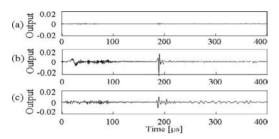


Figure 5. Coupling layer effects: (a) Vacuum grease-not adhered tightly; (b) Vacuum grease-adhered tightly; (c) Water.

type transducer has good directivity, as shown in Figure 4(b). Whereas a non-focal type transducer diverges much of the energy, as shown in Figure 4(c), a nonfocal type transducer with a concave lens (lens focal type transducer) improves the directivity dramatically, as shown in Figure 4(d).

Coupling layer characteristics. The coupling layer, which is an agent between transducer and medium, should minimize the reflection from the surface of the medium tested to maximize the energy transferred to the medium. Vacuum grease and water were examined as agents in this study. The test medium was a plaster block of 300 mm in height, 300 mm in width and 150 mm in thickness. The minor energy was reflected with vacuum grease when the transducers were lightly placed on the top of the vacuum grease (it is not adhered tightly), as shown in Figure 5(a). However, when the transducers were tightly adhered, the amplitude of the reflected signal increased, as shown in Figure 5(b). Note, the amplitude of the directly transmitted wave, which propagates from the source to the receiver through the tested medium, also increases. Furthermore, it is very difficult to maintain the constant contact between the transducer and the vacuum grease during the scanning. When water was used as a coupling layer, it produced a relatively high amplitude reflection as shown in Figure 5(c). In addition, the directly transmitted wave can be effectively shielded by using several layers of aluminum foil. Therefore, water was selected as the coupling layer for scanning in this study.

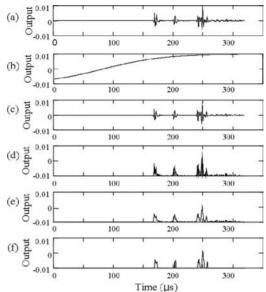


Figure 6. Signal processing: (a) Measured signal after stacking; (b) logsig function(window function) for time gain compensation; (c) Compensated signal; (d) Rectification; (e) Moving averaging; (f) Rejection.

# 3 EXPERIMENTAL SETUP AND SIGNAL PROCESSING

# 3.1 Experimental setup

A Pulser (JSR, DPR300) was used to generate the ultrasonic waves through the transducer (Panametrics A3441). Two transducers were used as the source and receiver, respectively. The reflected signals due to the impedance mismatch at the interface were detected and converted into electrical signal by the receiver transducer. The electrical signal was fed through an amplifier (Krohn-Hite 3945: frequency range from 170 Hz to 26.5 MHz) because the amplitude of the signals measured by the receiver was generally too low to identify the meaningful reflection. The amplified signal was digitized in the oscilloscope (Agilent 54624A or National Instruments PXI-5112).

# 3.2 Signal processing

Signal processing for ultrasonic imaging consists of stacking, signal compensation, demodulation including rectification and smoothing, rejection, and display (Zagzebski 1996).

*Stacking*. Signals were captured after stacking, which is the most effective signal processing technique for removing the high frequency, uncorrelated noise. The stacking means averaging multiple signals. The 1024 signals were averaged to obtain the signal trace, as shown Figure 6(a).

Signal compensation. The amplitude of the reflected signals at the receiver generally decreases as the distance of the interfaces, which the waves are reflected from, increases due to the geometrical spreading under identical impedance mismatch. This attenuation may be compensated by using time gain compensation (TGC), which increases the amplitude with time. Thus, the amplification factor is higher at a longer distance than at a shorter distance from the receiver. In this study, the adopted TGC is a log sigmoid (logsig) transfer function, as shown in Figure 6(b). The compensated signal is obtained through point-by-point multiplication between the original signal and logsig function. Figure 6(c) shows the compensated signal from the original signal. The amplitude of the first and second reflections decreases.

*Demodulation*. Demodulation includes rectification and smoothing. Rectification is an inversion of negative components. Thus, the signal has only positive values, as shown in Figure 6(d). Smoothing (moving average) was carried out by using the kernel of [1/20, 2/20, 4/20, 6/20, 4/20, 2/20, 1/20] in this study. The signal after the smoothing process is shown in Figure 6(e).

*Rejection.* Rejection is an elimination process, which removes the signal whose amplitude is less than threshold value. Thus, rejection removes noises and low amplitude signals. In this study, the threshold value was about 20% of the maximum amplitude. The signal after rejection is shown in Figure 6(f).

*Display.* Two modes were used to display the ultrasonic reflection imaging in this study. First, the amplitude mode (A-mode), which represents the amplitude of the signal after rejection versus time, was used. In the A-mode, the first arrival time of the reflected signal can be easily determined. Note the height of the trace in the A-mode is the amplitude. Second, the amplitude of each signal was converted to brightness, which is proportional to the amplitude of the reflected signal. The brightness versus time is the brightness mode (B-mode).

# 4 APPLICATION EXAMPLES

Several unique applications of ultrasonic wave monitoring and imaging were explored by using several small scale plaster blocks and one large scale plaster block.

# 4.1 Rotating scanning test for the small scale plaster blocks

Three kinds of specimens were used for the rotating scanning tests: an intact plaster block, a plaster block with an inclined crack, and a plaster block with a cavity. The dimensions of the plaster block were 300 mm in height, 300 mm in width and 150 mm in thickness. The middle of the top surface of the plaster block was dug

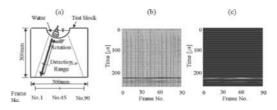


Figure 7. Rotating scanning test for an intact plaster block: (a) Test setup; (b) A-mode image; (c) B-mode image.

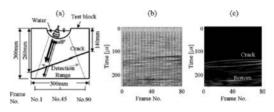


Figure 8. Rotating scanning test for a plaster with crack: (a) Test setup; (b) A-mode image; (c) B-mode image.

to form the concave shape for the rotational scanning, as shown in Figures 7–9. Water, which was filled in the concave part of the plaster block, was used as the coupling layer.

*Intact Plaster Block.* The rotational scanning tests on the intact block were carried out, as shown in Figure 8(a). The rotational scanning test on an intact plaster block clearly shows the interface at the bottom of the plaster block as shown in Figures 7(b) and (c). The brightest part in the B-mode occurred in the middle of the bottom. Because the reflected signal at the bottom of the plaster block was trapped in the water coupling layer, the signals with the intermediate amplitude were also detected below the bottom of the plaster.

Plaster Block with an Inclined Crack. After the intact plaster block was cut into two parts in the inclined direction as shown in Figure 8(a), the two parts were filled with thin vacuum grease to maximize the energy transmitted through the crack. The result of the rotational scanning test is shown in Figures 8(b) and (c). Two strong reflected signals were detected from the inclined crack and from the bottom of the block, respectively. The first strong reflection occurred at the inclined crack. Note the inclination of the crack was observed in the B-mode. The amplitude or the brightness of the reflected signals increases as the angle of incidence of the transducer beam becomes zero degree with the normal to the crack (right side). Note the brightest part in the B-mode corresponds to the strongest reflection from the discontinuity, which is perpendicular or normal to the transducer. Therefore, the brightest section can be used to determine the angle of the inclined discontinuity. In addition, because the reflected signal at the inclined crack was also trapped in the water coupling layer, the multiple reflections were observed. Note the inclination in the

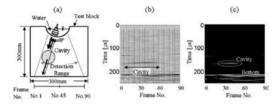


Figure 9. Rotating scanning test for a plaster cavity: (a) Test setup; (b) A-mode image; (c) B-mode image.

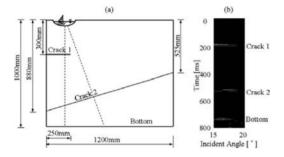


Figure 10. Rotating scanning test for a large scale plaster block: (a) Test setup; (b) B-mode image.

multiple reflections was identical with that of the first reflection, which came from the inclined crack. The second strong reflection was measured at the middle of the bottom of the plaster block because the angle of the incident wave became zero with the normal to the bottom of the plaster block.

Plaster Block with a Cavity. A hole (Diameter = 30 mm) was drilled on the intact block, as shown in Figure 9(a). The results of the rotational scanning test are plotted in Figures 9(b) and (c). Strong signals were also reflected from the bottom. The estimated diameter of the cavity from the B-mode was about 43 mm. Note, the difference between the real size (30 mm) and the estimated size (43 mm) results from the divergence of the beam, Fresnell's ellipse, and the transducer size. Although the inclined crack produced a continuous reflection line as shown in Figure 8(c), the cavity only yielded the reflections in a limited range. Furthermore, the amplitude of the incident waves diverted from the normal to the cavity.

# 4.2 Rotating scanning test for a large scale plaster block

The dimensions of the large scale plaster block were 1000 mm in height, 1200 mm in width and 150 mm in thickness. For the rotational scanning test, the left part of the top surface of the plaster block was dug to form the concave shape, and water was filled, as shown in Figure 10. The rotational interval was 0.5 degree, which corresponded to 8.7 mm horizontal displacement at the bottom of the large scale plaster

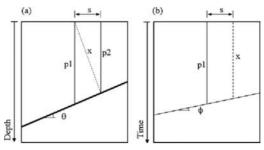


Figure 11. Effect of inclination angle: (a) Real discontinuity; (b) Ultrasonic reflection image.

block. To simulate the multiple discontinuities, two discontinuities were prefabricated: one was horizontal, and the other was inclined. The first horizontal crack whose gap and length were about 10 mm and 250 mm was filled with very weak and low viscous plaster paste for partial transmission and reflection of the ultrasonic waves at this discontinuity. The second long and inclined crack was filled with vacuum grease. B-mode is represented in Figure 10(b), that shows clear reflections from the first horizontal crack and the second inclined crack. In addition, the bottom of the plaster block could be identified, even though it was not clear.

# 5 DISCUSSION

# 5.1 Inclination angle of images

The inclination angle of the reflected image from the crack was flatter than that from a real crack, as shown in Figure 8. The relationship of the inclination angle between the reflected image  $\phi$  and the real discontinuity  $\theta$  is

$$\tan \varphi = \frac{p_1 - \sqrt{s^2 + p_2^2}}{p_1 - p_2} \tan \theta$$
(4)

where,  $p_1$  and  $p_2$  are the depths of the discontinuity at two points separated by the horizontal distances (see Figure 11), and note,  $p_1$  in the reflected image is the travel distance based on the travel time and velocity of the ultrasonic wave. The inclination angle of the discontinuity estimated by Equation (4) may be confirmed by the strongest brightness in the B-mode (see Figure 8).

# 5.2 Horizontal scanning versus rotating scanning

After a paraffin block was installed underwater, typical horizontal scanning (see details in Lee and Santamarina 2005) and rotating scanning tests were carried out. The experimental setups are shown in Figure 12(a). The thickness of the paraffin was 35 mm. For the typical horizontal scanning test, the ultrasonic waves

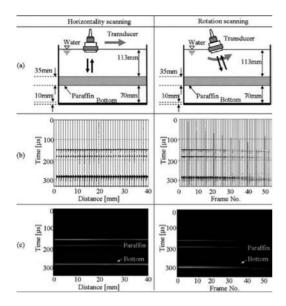


Figure 12. Comparison between rotating scanning and horizontal scanning tests: (a) Test setup; (b) A-mode image; (c) B-mode image.

reached perpendicularly to the interface. That is, the angle of incidence of the transducer beam was always zero degree with the normal to the interface. Thus, only reflection and transmission occurred at the interface. The amplitudes of the reflected and transmitted waves depend on the impedance mismatch. The scanning interval was set to 1 mm to avoid spatial aliasing (wavelength in water is  $\lambda = 3 \text{ mm}$  because the velocity of the ultrasonic waves in water is about 1500 m/s and the frequency of the transducer is 500 kHz). Note no migration processing was required because high directivity transducers were used (Figure 4). Horizontal scanning could clearly detect the horizontal paraffin wax layer in water. Furthermore, the bottom of the water box was also clearly seen, as shown in Figures 12(b) and (c).

For the rotating scanning test, the ultrasonic wave may reach non-perpendicularly to the interface, and therefore, a mode conversion may occur (see details in Richart et al. 1970; Aki and Richards 1980). As the angle of incidence of the transducer beam increases from zero degree as shown in Figure 12(a), the arrival time of the reflected wave increases, the amplitude of the reflected waves decreases and finally diminishes, as shown in Figures 12(b) and (c), for the rotating scanning test. However, B-mode obtained by the horizontal scanning and by rotation scanning is almost identical due to the size of the transducer (19 mm in diameter), divergence (see Figure 4), and Fresnell's ellipse. Note images obtained by horizontal scanning and rotating scanning techniques are related to the lateral resolution. Thus, the reflected waves are still measured even when the incident waves diverges from the normal to the interface.

#### 6 CONCLUSIONS

The design and application of ultrasonic wave reflection imaging were documented in this study for the detection of the discontinuity planes or cavities in laboratory scale rock models. The signal processing, which includes stacking, signal compensation, demodulation, rectification, smoothing, and rejection, were carried out to produce the amplitude and brightness modes (As and B-modes). The main observations of this study follow:

Although vacuum grease transmits more energy to the medium tested, water is recommended as the coupling layer for the horizontal and rotating scanning techniques because constant contact area is maintained in water and the directly transmitted wave between transducers may be effectively removed.

The discontinuities of the plaster block, including horizontal and inclined cracks, and the cavity were clearly detected by using the new rotating scanning technique. While the horizontal and inclined cracks yielded continuous reflections, the cavity produced reflections at a limited zone. B-mode is more appropriate for the detection of discontinuities.

The paraffin wax underwater was clearly detected by the typical horizontal scanning and rotating scanning techniques. Furthermore, the two techniques produced almost identical images. However, cautions are required for the analysis of results obtained by the rotating scanning technique.

The angle of the inclination obtained by the rotating scanning technique may give the angle of the original inclined crack through the comparisons of brightness and through a simple calculation based on geometry.

#### REFERENCES

- Aki, K. & Richards, P.G. 1980. Quantitative Seismology Theory and Methods Vol. 1 and 2: 932. Freeman Company, San Francisco.
- Gomm, T.J. & Mauseth, J.A. 1999. State of the Technology: Ultrasonic Tomography, Materials Evaluation Vol. 57: 737–755.
- Krautkramer, J. & Krautkramer, H. 1990. Ultrasonic Testing of Materials: 677. Springer-verlag, London.
- Lee, J.S. & Santanmarina, J.C. 2005. P-Wave Reflection Imaging. *Geotechnical Testing Journal* Vol. 28: 197–206.
- Richart, F.E., Hall, J.R. & Woods, R.D. 1970. Vibrations of Soils and Foundations: 414. Prentice-Hall, USA.
- Schmmer, L.W. Jr. 1998. Fundamentals of Ultrasonic Nondestructive Evaluation – A Modeling Approach: 559 Plenum Press.
- Wells, P.N.T. 1977. Biomedical Ultrasonics: 635. Academic Press, London.
- Zagzebski, J.A. 1996. Essentials of Ultrasound Physics: 220. Mosby, Inc., Missouri.

# Performance review of a pipe jacking project in Hong Kong

## T.S.K. Lam

Geotechnical Engineering Office, Civil Engineering and Development Department, Government of the Hong Kong Special Administrative Region

ABSTRACT: The pipe jacking method is commonly used in Hong Kong for construction of underground cable duct crossings and stormwater drains. The method minimizes the disturbance to or interference with the activities and facilities on the ground surface. In this paper, details of a pipe jacking project completed recently in Hong Kong, involving use of a pressurized slurry tunnel boring machine to form a 222 m long, 1.95 m diameter cable tunnel, are described. Results of the performance review carried out on completion of the project are also presented.

#### 1 INTRODUCTION

A cable duct crossing was constructed in an urban area at Kowloon West of Hong Kong. The cable duct crossing had to traverse a highway and some railway tracks. Conventional open cut excavation method was not used because of the disturbance that could be caused to the facilities on the ground surface. Pipe jacking method was used instead. The cable duct crossing was constructed in fill comprising loose to medium dense, silty coarse sand, and a pressurized slurry tunnel boring machine (TBM) was selected for the project.

In this paper, details of the project are described. A performance review was carried out on completion of the project. The key construction aspects, monitoring data obtained at the site during construction, impact on sensitive features in and surrounding the site and a summary of the observations and decisions made during construction are presented. The information given in this paper is obtained during auditing of the project by the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department (CEDD) of the Government of the Hong Kong Special Administrative Region. The auditing was carried out to exercise geotechnical control in the interest of public safety.

#### 2 PROJECT DESCRIPTION

### 2.1 Details

The site is located near West Kowloon Highway at Kowloon West (see Figure 1), Kowloon.

The project involves construction of a 222 m long 1.95 m diameter tunnel to serve as cable duct crossing

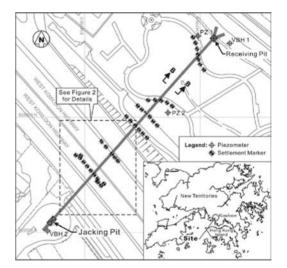


Figure 1. Location plan.

across a highway, the MTR Tung Chung line, the Airport Express Line and a public park. Pipe jacking method was used to form the 2 m diameter pipe opening at 8 to 9 m depth in fill, the properties of which are shown in Figure 8. The groundwater level was measured at about 2.0 m below ground. Two working pits of 14 m long  $\times$  4 m wide  $\times$  10 m deep were constructed at both ends of the cable duct crossing for the tunneling operation.

#### 2.2 Geotechnical aspects of the tunnel works

At the design stage, plans and supporting documentation of the geotechnical design of the pipe jacking

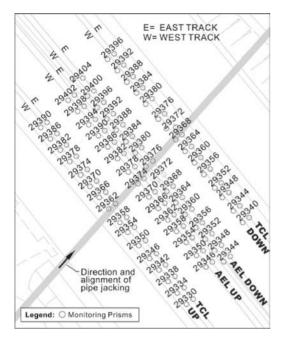


Figure 2. Locations of monitoring prisms on the railway tracks.

works, a geotechnical risk assessment and a risk mitigation plan were prepared by the designer.

The most sensitive features affected by the works were the railway tracks and the underground utilities, including drainage pipes and sewers, close to the alignment of the pipe jacking.

Prior to the commencement of works, a condition survey of the existing road and the structures within 25 m of the alignment of the pipe jacking and a CCTV survey of the existing drainage pipes and sewers within 20 m of the works were carried out. Another CCTV survey was also carried out on completion of the works.

An instrumentation scheme consisting of 168 settlement markers for the road surface, 123 settlement monitoring prisms for the railway tracks and four piezometers for groundwater level was adopted. Locations of the settlement markers, settlement monitoring prisms and piezometers are shown in Figures 1 and 2.

The settlement markers were installed adjacent to the jacking pit and receiving pit and on the road surface along the alignment of the pipe jacking. Out of the 123 settlement monitoring prisms installed, 32 were installed on the Tung Chung Line (up) (TCL up) (on both rails), 30 on the Airport Express Line (up) (AEL up), 31 on the Tung Chung Line (down) (TCL down) and 30 on the Airport Express Line (down) (AEL down) (see Figure 2).

One piezometer each was installed close to the jacking pit (VBH1) and receiving pit (VBH2) and two



Figure 3. Close-up view of settlement monitoring prism.

piezometers were installed along the alignment of the pipe jacking (PZ1 and PZ2) (see Figure 1).

A monitoring plan including monitoring of settlement, groundwater level and vibration, and a site supervision plan for the works were prepared. The method statement for the works and details of the pressurized slurry machine selected were also submitted to the relevant departments including the GEO for review.

The stakeholders affected by the project were also notified and consulted by the designer. For the railway tracks, trigger levels of "alert", "action" and "alarm" (or AAA) of 12 mm, 16 mm and 20 mm respectively for settlement and 1 in 1500, 1 in 1250 and 1 in 1000 respectively for angular distortion were set and agreed by the MTR Corporation Limited. At alert level, readings would be reviewed and plans for remedial measures and contingency actions would be prepared. At action level, the planned remedial measures would be implemented and the works could only continue if the remedial measures taken were effective. The alert and action levels would also be revised if necessary. At alarm level, all works would be stopped or contingency actions taken and the design, construction method and the planned remedial measures would be reviewed. A maximum tolerable track settlement of 25 mm was allowed.

An Automatic Deformation Monitoring System was used to take readings for monitoring the railway track settlement. Three CYCLOPS theodolites were positioned on opposite sides along the tracks, and the track settlement was monitored on a daily basis in real time. The data taken were transferred to a computer in the site office which were then processed and posted onto the internet for real time monitoring by relevant parties (Figures 3 to 5).

#### 2.3 Construction

A RASA DHL1650 pressurized slurry TBM was used (Figure 6). The diameter of the shield body



Figure 4. Positions of settlement monitoring prisms on the track.



Figure 5. A CYCLOPS theodolite.

and the equipment tube of this TBM is 2,000 mm and 1,990 mm respectively. The excavated diameter is 2,040 mm. The pipe installed has an outside diameter of 1,950 mm and an internal diameter of 1,650 mm (Figure 7).

The construction method involved use of a pressurized slurry system. The slurry support pressure at the excavation face was set to balance the ground and groundwater pressure as indicated in the control panel of the TBM. The TBM operator checked the pressure gauge to control the pressure at the excavation face and the slurry-discharge pressure gauge to control the circulation pressure. The typical groundwater pressure at the excavation face was 50 to 70 kPa with an average value of 60 kPa at the tunnel axis level. To balance the ground and groundwater pressure, a slurry pressure of 10 to 20 kPa above the water pressure, which is 60 to 90 kPa, was applied at the excavation face for the operation (Figure 8). The slurry pressure applied to the

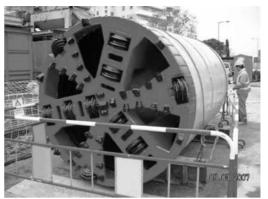


Figure 6. Pressurized slurry TBM.

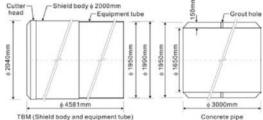


Figure 7. Dimensions of TBM and concrete pipe.

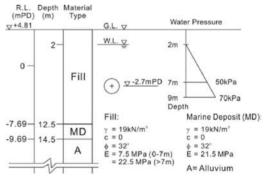


Figure 8. Typical groundwater pressure.

excavation face was controlled by the pressure control valve of a by-pass unit placed in the jacking pit.

During operation, a lubricant consisting of water, bentonite and mineral oil was injected into the annulus around the pipes to fill the voids around the pipes to reduce soil movement. On completion of the pipe jacking, the cavities around the pipes were grouted. A cement grout with a minimum compressive strength of 20 MPa was used.

#### 3 PERFORMANCE REVIEW

#### 3.1 Control of amount of materials excavated

In a pressurized slurry system, the ground was mechanically excavated while the excavated face was stabilized and supported by slurry. The excavated materials were removed by slurry transport. To check if voids had been formed in the soil above the tunnel during excavation, the amount of materials excavated was monitored during the tunneling operation. A desander was used to separate the solids from the slurry. The solids were mainly fine sand which was later used for backfilling the pits. The weight of the soil excavated from the jacking pit, receiving pit and the tunnel was worked out to be 2,262 tonnes before the works commenced. During the works, 593 tonnes of sand and 1.461 tonnes of finer soil were disposed of. The total amount of soil disposed was therefore 2,054 tonnes which was less than the calculated value. Although limited by the accuracy of measurement, this provided a rough indication that no significant voids had been formed in the ground above the tunnel. No boulders were encountered and no cutter had to be replaced for this project.

#### 3.2 Settlement of railway tracks

In this project, the most sensitive features are the railway tracks.

The excavation sequence of the pipe jacking works in terms of the distance from the railway tracks is shown in Figure 9. Pipe jacking commenced on 29 March 2007. Fourteen days after commencement of the pipe jacking (12 April 2007), the TBM reached the TCL Up track and 5 more days later (17 April 2007), it went past through all the tracks. Included in the figure is also the track settlement in response to the pipe jacking.

The tracks settled as soon as the pipe was jacked past and away from the section. At the TCL Up track, the track settled by 12 mm when the pipe was installed at a distance of 23.5 m away from the section. Similar rate of settlement was noted at tracks AEL Up, AEL Down and TCL Down except in those cases, the track settlement was less. On completion of the pipe jacking, maximum settlement of tracks TCL Up, AEL Up, AEL Down and AEL Down were 22 mm, 15 mm, 10 mm and 3 mm respectively which are less than the maximum tolerable track settlement of 25 mm.

In pipe jacking, the whole length of the pipe is jacked and moved forward, resulting in disturbance to the soil around the whole length of the pipe. With the cutter head about 90 mm larger than the pipe to be jacked (see Figure 7), it is possible that voids would form around the pipe which would result in ground movement, and hence settlement along the length of the pipe even after excavation

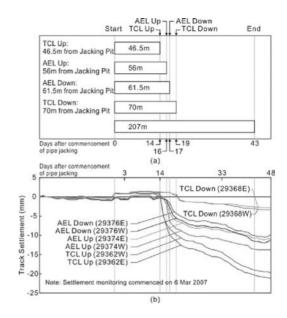


Figure 9. Excavation sequence of pipe jacking (a) and settlement profiles at tracks TCL Up, AEL Up, AEL Down and TCL Down (b).

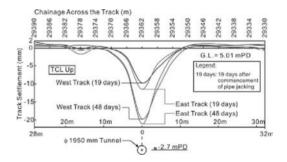


Figure 10. Settlement trough at track TCL Up.

has been carried out to some distance away from the tracks.

The settlement troughs at tracks TCL Up, AEL Up, AEL Down and TCL Down are shown in Figures 10 to 13 respectively. The settlement troughs are almost symmetrical in these cases.

The maximum settlement on the pavement was 15 mm which was recorded by settlement point SC19 adjacent to TCL Up track (Chainage 29362).

Given the ground conditions and the method of construction as described above, a maximum volume loss of 3.6% was calculated (see Figure 14).

The maximum groundwater drawdown recorded by the piezometers at the jacking pit and receiving pit (VBH2 and VBH1) was 1.1 m, which was only slightly above the permissible value of 1 m. At PZ1 location,

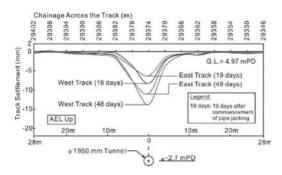


Figure 11. Settlement trough at track AEL Up.

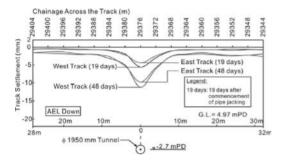


Figure 12. Settlement trough at track AEL Down.

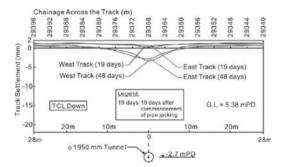


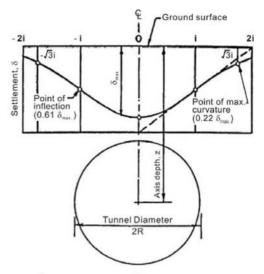
Figure 13. Settlement trough at track TCL Down.

the maximum drawdown recorded was 0.7 m, and at PZ2 location, no significant amount of groundwater movement was observed.

No undue settlement was recorded at the drainage pipes, sewers and the structures nearby.

# 3.3 Actions taken at action level during construction

Twelve one days after commencement of the pipe jacking (19 April 2007), a settlement of 15 mm was recorded at the TCL Up track which was close to the action level. An urgent meeting was held among the client, designer and contractor on that night. After



Ratio  $\frac{i}{R}$  is a function of  $\frac{z}{2R}$  and the soil conditions Volume of trough  $\approx 2.5$  i  $\delta_{max}$  per metre run

Volume loss (%) = 
$$\frac{250 \text{ i } \delta_{\text{max}}}{A}$$

where A = cross-sectional area of tunnel (m<sup>2</sup>)

i = distance to the point of inflection (m)

 $\delta_{max}$  = assessed immediate settlement over the centreline of the tunnel (m)

Figure 14. Properties of error function to represent the settlement trough above a tunnel (after Peck, 1969).

the meeting, it was decided that for the remaining works, an alternative type of lubricant, in lieu of a mixture of water, bentonite and mineral oil, was used to fill the gap between the pipe and the TBM over-break. This lubricant formed a solid substance once injected into the gap and it helped to reduce settlement of the overburden fill caused by closure of the overbreak voids around the pipe. The density of the bentonite slurry was also increased. The advance speed of the TBM was reduced from 9 m per night shift to about 6 m per night shift. With the actions taken, the rate of increase of track settlement was reduced and the final settlement was contained to within the permissible value.

#### 4 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations can be made from the project:

1. The use of pipe jacking method was effective in forming a 222 m long, 1.95 m diameter cable tunnel

at 8 to 9 m depth. The pressurized slurry TBM used is appropriate for the type of ground conditions encountered.

- 2. It is necessary to apply appropriate geotechnical control measures to this type of project affecting sensitive features in the interest of public safety. With the geotechnical control measures applied, the geotechnical risk is assessed and the potential hazards are identified early. Any undue settlement or irregularities observed during construction can be detected promptly and appropriate remedial measures, such as use of a different annulus filler to suit the actual ground conditions encountered, change of the advance speed of TBM, etc. can be taken to prevent catastrophic failure from happening.
- 3. It is important to control the amount of materials excavated from the tunnel opening to prevent significant ground loss. This could be achieved by comparing the excavated volume with the theoretical excavation volume to check if significant voids have been formed in the soil above the tunnel and checking the amount of filler/grout used to fill the voids around the pipes. In this project, only the total amount of materials excavated from the tunnel opening is obtained. A better method should be devised for future similar operation measuring the amount of materials excavated for each section of the tunnel excavation and checking the amount of filler/grout used to fill the voids around the pipes.

#### ACKNOWLEDGEMENT

The author would like to thank the Director of the Civil Engineering and Development Department and the Head of the Geotechnical Engineering Office for the permission to publish this paper. Comments on the paper from Mr Joe B.N. Leung and Dr Richard Pang are gratefully acknowledged.

The author would also like to thank the Employer of the Project – CLP Power Hong Kong Limited, the Designer – Black & Veatch Hong Kong Limited and the Main Contractor – Kum Shing (K.F.) Construction Co. Limited for the co-operation to exercise geotechnical control in the interest of public safety in the project and the permission to publish the technical data/details given in the paper. The views given in the paper reflect only those of the author and not of the above companies.

### REFERENCES

Peck, R.B. 1969. Deep excavation and tunneling in soft ground. Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering, State-of-the-Art Volume: 225–290.

# Geotechnical control of a major railway project involving tunnel works in Hong Kong

# W. Lee, S.S. Chung, K.J. Roberts & P.L.R. Pang

Geotechnical Engineering Office, Civil Engineering and Development Department, Hong Kong Special Administration Region

ABSTRACT: This paper describes the role of the Geotechnical Engineering Office, Civil Engineering and Development Department of the Hong Kong Special Administration Region (HKSAR) Government in exercising geotechnical control for a major railway project in Hong Kong. It highlights the geotechnical aspects of the tunnel works and how the geotechnical control process protects public safety and adds value by ensuring an adequate standard of design, site supervision and risk management is applied. The successful implementation of best geotechnical risk management practice in the project is strongly influenced by the commitment of the project client to follow the core element of the Joint Code of Practice for Risk Management of Tunnelling Works and to follow up on the results of the independent auditing under the geotechnical control process.

# 1 INTRODUCTION

# 1.1 Project description

The Kowloon Canton Railway Corporation's (KCRC) Kowloon Southern Link (KSL) Project links West Rail's Nam Cheong Station and East Rail's East Tsim Sha Tsui (ETST) Station in the HKSAR. The 3.8 km long railway will have one new station, West Kowloon (WKN) Station.

The civil construction works of the project were packaged into three Design and Build contracts. The total project cost, including railway systems, is expected to be about HK\$8.3 billion. The project is scheduled for completion in late 2009. The details of the three contracts are described below.

# 1.2 Contract KDB200

Contract KDB200 involves the section between ETST Station and Jordan Road via the former Marine Police Headquarters. WKN Station, two railway tunnels and two emergency vertical shafts, namely Canton Road Emergency Access Point (CREAP) and Peking Road Emergency Egress Point (PREEP), were included in this contract (see Figure 1 below). The contract was awarded to Link 200 JV (formally known as the Leighton – Balfour Beatty – Kumagai – John Holland Joint Venture).



Figure 1. Aerial photograph showing the alignment of Contract KDB200.



Figure 2. Slurry-type tunnel boring machine.

The geological sequence along the alignment generally comprises the following principal geotechnical units:

- fill (generally a few metres thick but up to 20 m thick at the West Kowloon Reclamation site);
- marine deposits and alluvium (described as clayey/silty sand and sandy silt/clay with some gravel, mainly found along Salisbury Road but also present locally near Haiphong Road and undredged pockets at the WKN Station);
- residual soil and saprolite, mainly completely to highly decomposed granite; and
- moderately decomposed to fresh medium-grained granite.

At the preliminary design stage, a cut-and-cover option was planned for the tunnels. However, KCRC decided to employ a Tunnel Boring Machine (TBM) for the twin railway tunnels between the launching shaft at the southern tip of WKN Station and the retrieval shaft at Salisbury Road to minimize potential disruption to road users, pedestrians, business operations and residents in the area. The twin railway tunnels are approximately 8 m in diameter and 1.1 km in length. The depth to tunnel crown varies from 8 m to 24 m.

The slurry-type TBM (see Figure 2) uses 3800 kilowatts of electricity, which is equivalent to over

5000 horsepower. It was design in Germany and its components were manufactured in Germany and China and finally assembled in Guangzhou. The 50 steel cutting discs enable the TBM to cut through 1.5 m of rock in about 40 minutes.

Notwithstanding the use of TBM, some works such as ground investigation and grouting were required under the contract on the already very busy road surface. Such works are to provide additional data for geotechnical design and to mitigate the impacts before construction of the relevant tunnel works sections.

The excavation for WKN Station was supported by diaphragm walls, while that for the remaining cut and cover tunnels was supported by temporary walls comprising sheet piles, pipe piles and struts.

The most challenging part of these works from a geotechnical perspective is the provision of a retrieval shaft outside the Sheraton Hotel in Salisbury Road serving as a works portal for the retrieval operation of the TBM. Traffic diversions were unavoidable. This part of the works was constructed using the cutand-cover method. Temporary road decking has been erected to facilitate the underground works and to shorten the construction period.

#### 1.3 Contracts KDB300 and KDB400

Contracts KDB300 and KDB400 involve the section between Jordan Road and Nam Cheong Station of the KCRC's existing West Rail (WR). The two contracts are divided at the Yau Ma Tei Ventilation Building (YMTVB) (see Figure 3). The tunnel lengths are 0.85 km and 1.06 km respectively. These contracts were awarded to China State Construction Engineering (Hong Kong) Limited.

In view of the ground conditions (mainly fill, marine deposits, alluvium and completely decomposed granite), the depth of the tunnel (about 20 m to soffit) and the lack of constraints on the ground surface, the cut-and-cover tunnel method has been adopted as the most suitable method in terms of time and cost for the majority of the tunnel length. The exception is the tunnel beneath Cherry Street, which would be constructed using a mined tunnelling method. The cut and cover excavation was supported by temporary walls comprising sheet piles, pipe piles, diaphragm walls, bored pile walls and struts.

The tunnel alignment is very close to many existing buildings, structures and utility services. Some are very sensitive to construction-induced ground movements such as the operating Mass Transit Railway tracks and buildings, highway bridge structures, Drainage Services Department structures (e.g. box culverts) and nearby buildings (e.g. the HSBC Centre). In order to monitor the effects of the construction on the surrounding buildings, structures and utility services, extensive geotechnical instrumentation



Figure 3. Aerial photograph showing the alignment of Contracts KDB300 and KDB400.

has been installed, including ground/ structure/utility settlement checkpoints, inclinometers, extensometers, tiltmeters, vibration monitoring checkpoints, standpipes and piezometers. Instrument readings are monitored against alert, action and alarm values as defined individually for particular instruments and sensitive receivers.

#### 2 THE ROLE OF THE GEOTECHNICAL ENGINEERING OFFICE

#### 2.1 Buildings ordinance and regulations

In the HKSAR, the regulatory control of building works, in the interest of protecting public safety, is by application of the Buildings Ordinance and Regulations (BOR). Under the BOR the definition of a "building" includes, inter alia, "...any underground space adapted or constructed for occupation or use for any purpose including its associated access tunnels and shafts". Therefore, the development of private underground space such as tunnels or caverns including their planning, design and construction all fall under the BOR.

A KCRC project is considered a private project in which the tunnel works may be exempted from the administrative procedures of approval and consent under the Buildings Ordinance. Under certain conditions, an Instrument of Exemption (IoE) would be prepared and issued by the Building Authority (BA) of the HKSAR Government. For the KSL Project, because of the significant risk to life and property, the design, risk management, construction and site supervision would need to be implemented to a good standard to protect public safety. Auditing of the standard of geotechnical design, site supervision and risk management is carried out by the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department, as a technical adviser to the Buildings Department (BD) of the HKSAR Government.

# 2.2 Technical standards

GEO has issued a technical guidance note TGN24 (GEO 2005a), on specific aspects of site investigation for tunnel works in the HKSAR. It supplements guidance on site investigation given in Geoguide 2 (GEO 1987) and Geoguide 4 (GEO 1992). The technical guidance note was prepared with the benefit of the experience gained from the Harbour Area Treatment Scheme Stage I (Pang et al 2006, Massey et al 2007).

GEO has also issued TGN25 (GEO 2005b), on the implementation of geotechnical risk management in relation to tunnel works. Tunnel works are defined as tunnels, shafts, caverns and associated underground facilities, however constructed. Construction of tunnel works may involve use of drill and blast methods, tunnel-boring machines, cut and cover methods, techniques that incorporate insitu ground treatment, groundwater control, installation of temporary and permanent supports, etc.

The relevant Government departments and the profession were consulted in the preparation of these guidance notes.

# 3 GEOTECHNICAL CONTROL PROCESS

#### 3.1 Instrument of exemption

Within GEO a review panel has been set up to agree on the geotechnical auditing requirements to be included into the IoE, examine the key geotechnical aspects of major submissions related to tunnel construction for the project, and to oversee the standard of auditing. The input started right from the planning stage of the project in which KCRC demonstrated to GEO that they had adequately identified and assessed the geotechnical risks, and had taken suitable risk mitigation and control actions to manage the risks. In addition, KCRC committed to adopt the Joint Code of Practice (ABI & BTS 2004). The risks identified for the TBM tunnels related to plant procurement, manufacture of segmental linings, delay in launch shaft availability, break-in to retrieval shaft, TBM assembly/removal and operation including encountering adverse ground conditions, unforeseen underground obstructions, causing excessive settlements or even damage to nearby buildings/structures/utilities during TBM operation and interventions, compressed air blow-out along areas of low soil cover and crossing over the operating MTR tunnels.

In early 2005, KCRC's consultants submitted preliminary scheme designs for auditing by BD/GEO, including a Geotechnical Basis of Design Report, Ground Movement Prediction Report, Geotechnical Instrumentation Report and an Existing Buildings and Structures Report, supported by a Geotechnical Data Report.

The BA issued the IoE to the KSL construction works on 30 July 2005 pursuant to the Kowloon Canton Railway Corporation Ordinance. The exemption is confined to only those procedures and requirements relating to approval of plans, consent to commencement and resumption of works and occupation of buildings provided under the BOR, such that the BA's duties and sanctioning powers to ensure standards of health and safety are not undermined.

Guidelines which KCRC were required to follow for making submissions to GEO for geotechnical works, including tunnel works, under the KSL project were agreed and included into the IoE and summarized in a Management Plan (KCRC 2005).

Under the agreement, KCRC had submitted the design statements and various method statements for the TBM tunnels, the cut and cover tunnels, CREAP, PREEP, the launching shaft and the retrieval shaft to BD/GEO for consultation.

KCRC had also submitted the excavation and lateral support (ELS) plans for the TBM tunnels where there is excavation, loading/unloading of the ground or changes to the groundwater regime. The critical parameters shown in the TBM ELS plans include the operating slurry pressures, the planned interventions (use of free air or compressed air depends on the anticipated ground conditions) and the air pressures where compressed air interventions are planned.

#### 3.2 Other requirements for KCRC under the IoE

Under the IoE, KCRC is required to appoint Authorized Persons (AP), Registered Structural Engineers (RSE) and Registered Geotechnical Engineers (RGE) to co-ordinate the works and to certify the plans and documents as well as completion of the works; and to appoint Registered General Building Contractors (RGBC), and Registered Specialist Contractors (RSC) in the case of specialized railway construction works, to supervise and carry out the relevant works.

KCRC is also required to instigate an assurance system and control scheme to ensure that management of the construction works is at a standard not inferior to that required under the BOR. KCRC also employed a team of Resident Site Staff (RSS), led by a Construction Manager to act as the Engineer's Representative, to supervise the works and the implementation of the Project Risk Management Plan (KCRC 2006). In addition, AP, RSE, RGE, RGBC and RSC have jointly prepared Site Supervision Plans (SSP) in accordance with the Codes of Practice for Site Supervision (BD 2005a, b).

The Contractors were also required to appoint Independent Checking Engineers (ICE) to provide certification of consultation documents and verify geotechnical design submissions for both permanent and temporary works prior to forwarding to KCRC for review under the contracts.

KCRC ensure that submissions are made at all stages to BD, and to GEO where geotechnical aspects are involved, in a timely manner prior to the commencement of elements of the construction works through the AP/RSE/RGE. KCRC and the AP/RSE/RGE are required to ensure that all comments given by BD and other relevant parties, in connection with their submitted consultation documents, have been resolved to the satisfaction of BD and the party concerned prior to commencement of construction of the relevant part of the works.

#### 3.3 During construction

During the course of works, the AP/RSE/RGE are required to keep on site copies of certified working plans, inspection and test records and other relevant reports for regular audit inspections by BD and GEO. When significant changes in design or method of working are necessary, then KCRC through the AP/RSE/RGE are required to report this to BD and GEO and ensure that all comments given by BD and GEO are resolved to the satisfaction of BD.

KCRC through the AP/RSE/RGE are required to report the following to BD and GEO immediately when the following circumstances arise:

- Construction accidents causing nuisance to the public.
- Irregularities causing inconvenience to the public and/or damage to nearby property.
- Construction non-conformities.

Since award of the first contract for the KSL Project in August 2005, monthly Buildings Ordinance Management Committee (BOMC) Meetings have been held by KCRC with representatives from KCRC Management, the AP/RSE/RGE and from Government including BD and GEO. One of the items of discussion is the scope, standard and timing of the geotechnical submissions from the RGE to GEO. Geotechnical auditing of the KSL Project is currently in progress.

### ACKNOWLEGEMENTS

This paper is published with the approval of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development.

The authors acknowledge with gratitude the information and materials provided by the Kowloon-Canton Railway Corporation and Link 200 JV for production of this paper.

#### REFERENCES

- ABI/BTS 2004. A Joint Code of Practice for the Procurement, Design and Construction of Tunnels and Associated Underground Structures [In the United Kingdom]. London: The Association of British Insurers/The British Tunnelling Society.
- BD 2005a. Code of Practice for Site Supervision 2005. Hong Kong: Buildings Department.
- BD 2005b. Technical Memorandum for Supervision Plans 2005. Hong Kong: Buildings Department.

- GEO 1987. *Guide to Site Investigation (Geoguide 2)*. Hong Kong: Geotechnical Engineering Office.
- GEO 1992. Guide to Cavern Engineering (Geoguide 4). Hong Kong: Geotechnical Engineering Office.
- GEO 2005a. Site Investigation for Tunnel Works (TGN24). Hong Kong: Geotechnical Engineering Office.
- GEO 2005b. Geotechnical Risk Management for Tunnel Works (TGN25). Hong Kong: Geotechnical Engineering Office.
- KCRC 2005. Kowloon Southern Link Management Plan for Compliance under the Buildings Ordinance Revision D. Hong Kong: Kowloon-Canton Railway Corporation.
- KCRC 2006. Kowloon Southern Link Project Risk Management Plan. Hong Kong: Kowloon-Canton Railway Corporation.
- Massey, J.B., Pang, P.L.R., Lo, J.Y.C. & Salisbury, D. 2007. Developments in Tunnel Engineering in Hong Kong. *Proceedings of the HKIE Geotechnical Division Annual Seminar 2007*. Hong Kong: 137–155.
- Pang, P.L.R., Woodrow, L.K.R. & Massey, J.B. 2006. Development of geotechnical control and risk management for tunnel works in Hong Kong. *Proceedings of the HKIE Geotechnical Division 26th Annual Seminar*. Hong Kong: 75–88.

# Research on structural status of operating tunnel of metro in Shanghai and treatment ideas

# J.P. Li, R.L. Wang & J.Y. Yan

Shanghai Metro Operation Co., Ltd., Shanghai, P.R.China

ABSTRACT: Recently, with network operating and servicing time going on, the safety of operating tunnel in Shanghai is becoming a focus in the circle of civil engineering. It is well known that safety of operating tunnel is greatly influenced by the structural status of the tunnel. This article firstly give an introduction of main diseases of operating tunnel based on the large amount of information and date collected in the past decades about Shanghai Metro. Then causes for the disease are analyzed and related suggestions to prevent the diseases deteriorate are given. It is shown that the main problem of operating tunnel include leakage, crack, longitudinal settlement and constringency.

## 1 INTRODUCTION

Recently, with network operating and servicing time going on, the safety of operating tunnel in Shanghai is becoming a focus in the circle of civil engineering. However, only decades after shield tunnel was introduced to China, few successful experiences can be used for reference from domestic and abroad, especially for Shanghai metro where tunnel was constructed in thick soft clay soil. This article firstly gives an introduction of main disease of operating tunnel based on the large amount of information and data collected in the past decades about Shanghai Metro. Then causes for the disease are analyzed and the related suggestions for prevention are given. It is expected that this article can be useful for those who is interested in the health of tunnel of Shanghai metro.

# 2 MAIN DISEASES

#### 2.1 Leakage

Based on the large amount of information collected recently about Shanghai tunnel of metro, more than ten seepage spots can be found at each section between adjacent stations. Seepage spots in several sections could up to fifty places. Figure 1 indicates that leakage mainly occurs at both sides of tunnel. joints of station and tunnel and the by-pass between up and down line. Minority leakage places were founded at the hole of bolt and the hole of grouting. According to technical specification for water-proof of shield-driven tunnel (DBJ08-50-96, 1996), Leakage of the operating tunnel of Shanghai metro belongs to the second or the third level.

After decades of operating, the function of waterproof of sealing rod has already reduced to a low grade for the non-uniform settlement of tunnel caused by adjacent construction, pumping of ground-water, etc. For the special stress state, most segment rings exhibit an ellipse shape with horizontal radius enlarged and vertical radius reduced. Correspondingly, the compressive stress at the hance of tunnel segment ring decrease near outer surface and increase near interior surface. Gaps at the joint of segment rings maybe formed. Since the sealing rod was set at the outer side of segment ring, so the capability of water-proof was reduced at the hance of the tunnel, and this lead to that most leakage take place at the side of tunnel. In addition, for the great difference of structural style among the shield tunnel, the station and the by-pass, differential settlement occurs at these positions and this also leads to serious seepage.

# 2.2 Segment crack

Few cracks are formed in the segment itself, but the phenomenon that joint filler has pull-out can be found for non-uniform settlement of operating tunnel which makes the joints uncoupled. Most cracks located at the corner of segment or unfilled corner take place during the construction phase, such as production, maintenance, handling and consolidation. Additional, cracks can be found between segment and track bed at the position where large differential settlement take place and where turning radius is small.



(a) Leakage at circumferential seam.



(c) Leakage at linkage of tunnel and station.

Figure 1. Leakage of the segment ring.

#### 2.3 Longitudinal settlement

Large settlement take place to the tunnel after decades of operation, and the longitudinal settlement represents with regional characteristic. Figure 2 indicates that the longitudinal settlement curve of Line 1 of Shanghai metro contains two huge settle pits. One of them is about 1400 m long, located at the Hengshan road station and the maximum settlement is about 20 cm. The other one is located at the interval from the South Huangpi road station to Shanghai Railway station with the maximum settlement is nearly 30 cm.

#### 2.4 Rate of settlement

Figure 3 is duration curves of settlement of tunnel near the People Square. Although the tunnel have been constructed for more than ten years and the rate of settlement has being slowed down, it can not leads to that the tunnel has already been stabilized for the creep properties of Shanghai soft clay. How to control the settlement is still a difficult problem faced by civil engineering.



(b) Leakage at by-pass.



(d) Leakage at the bolt hole.

# 2.5 Convergence

Horizontal diameter of most segment ring enlarge from 2 cm to 4 cm, few of them even up to 7 cm, This already exceed the design safety limit  $(1 \sim 5D\%)$ . According to statistics, segment ring with horizontal convergence deformation greater than 3 cm occupy 69.98% and those of greater than 6 cm occupy 6.63%. The maximum convergence is about 15 cm and the gap along the longitudinal joint is 11 mm which means that circumferential bolt have already reach yield limit.

#### 3 GENETIC ANALYSIS OF DISEASE

According to decade's subway monitoring results, the main factors related to the deformation of metro structure are listed as following:

#### (1) Local Ground Subsidence

Based on the ground subsidence database of Shanghai city, it shows that the settlements of subway station and tunnels are large if they located at the center

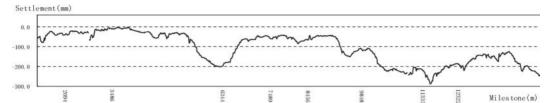


Figure 2. Longitudinal Settlement Curve of Line 1 of Shanghai Metro (1995.5-2007.12).

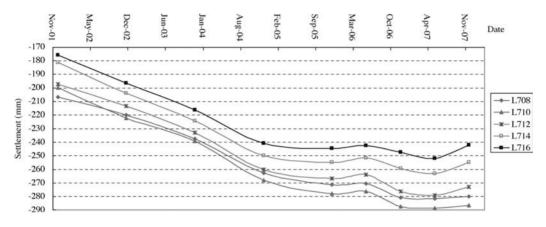


Figure 3. Duration curve of tunnel adjacent to People Square.

of subsidence region. Comparison of time-settlement curves between ground surface and tunnel structure measurement in People Square region gives great agreement. Figure 4 shows the time-settlement curves from 1999 to 2008. But the exacted ratio of tunnel settlement to ground subsidence is not able to be obtained.

#### (2) Geology Conditions

In Shanghai area, the soft soil stratum is about 30 m deep from ground surface. The subway station and tunnel mainly located in soft soil. The soil is basically saturated flowing-plastic or soft-plastic clay with low shear strength ( $0.005 \sim 0.01$  MPa), high water content (above 40%), high compressibility ( $0.5 \sim 1.0$  MPa-1), sensitivity varying from 4 to 5, and rheological behavior. In this very soft ground, the influence of excavation and tunnel drive to environment could not be ignored either for construction period or for long-term operating period.

#### (3) Quality of Construction

It would lead to large deformation if there were some accidents occurring during the construction of station and tunnel driving. The differential settlement would develop along the operation life and overstep the operating safety standard eventually. There are some typical cases presenting this phenomena, such as the tunnel deformation near to West Ninghai Rd. pumping station of metro Line.1, leakage of water and sand in pumping station of metro Line.2 crossing rive part, Shilong Rd. station of metro Line 3, have been found and treated against to the excessive settlement.

#### (4) Maintenance and Operation Work

Vibrations of running trains would lead to tunnel settlement, and then water leakage will takes place due to great tunnel settlement. In case the water leakage becomes serious, it will induce more water/soil loss and behave as larger settlement. The vicious circle is formed in this way. The in-time and frequent maintenance work is a good way to prevent this vicious circle. The research on long-term soil mechanics under vibration of high consistercy and low frequency is still undergoing. But it could be ensured that disadvantage of the complicated soil behavior is the main factor of tunnel operation safe.

#### (5) Loading and Unloading activities near to tunnel

Due to the shortage of landing resource in urban environment like Shanghai city, the projects of foundation pit located in the area of subway protection area tend to

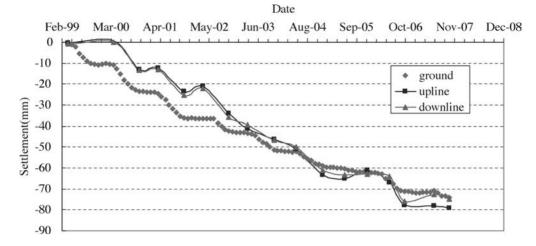


Figure 4. Time-settlement curve of ground surface and tunnel.

be deeper, bigger, closer, more difficult and more risk. As a consequence, it is necessary to pay more attention to the influence of deep foundation pit excavation and high-rise building to the subway station and tunnel.

(6) Groundwater exploitation, dewatering and pore pressure reduction

Soil consolidation induced by the groundwater exploitation would also influence the subway structures.

#### 4 CONTROL MEASURES

The safety guarantee system for subway structure should take many aspects into account, including the regulation, standard, monitoring management and pre-alarm management.

(1) Regulation and Standard for Subway Control and Protection

Supervised regulation should be established for subway line surveys, inspection and projects construction. Process management and responsibility definition should be enhanced to make sure that the whole subway lines are under control.

(2) Standard of the Monitoring Procedure

It is the responsibility of monitoring engineer to monitor, inspect and analyze subway structure, which aims to find the problem of structure in time and to guarantee the safety of subway. There are at least twice settlement measurements and one convergence measurement for operating line;  $3 \sim 4$  times settlement measurements and once convergence measurement for new line; several times inspection for key positions. More attention should be paid to the monitoring and inspection of projects located in the subway protection area. Up to now, 30000 settlement monitoring points and 1 convergence monitoring point every 5 segment rings were set for metro Line1 to Line 4 which are all operating metro lines in Shanghai.

(3) Disease Record Card System for Subway Structure

Based on decade's observation of longitudinal settlement curves of subway tunnel, positions with large longitudinal settlement curvature are found. Disease record card of subway structure were established to make sure that inspection and monitoring could focus on these positions. Then quick response and actions could be taken according to the results of inspection and monitoring in case of any abnormal situation.

(4) Digital Information System

An overall digital scan for running subway and a big GIS system are planning and constructing respectively. The GIS system would control the safety of subway structure and operation risk using information technology including inspects of geological conditions, tunnel structure, waterproof system, settlement and convergence.

(5) New technology and Equipment

The monitoring system with long distance, large range, high precision and automatization characteristic should be developed continuously. Remote monitoring system and equipment configuration would be set on the key position. The data, graphs and information are able to be transferred and analyzed instantly.

(6) Project located in the Subway Protection Area

Seting protection area aims to reduce the influence of loading and unloading effect from building and excavation to tunnel by enhancing control standard to the projects located in the subway protection area.

(7) Scientific Research Project located in the Subway Protection Area

The investigation to structural problem of metro, including durability of tunnel, seepage and leakage, settlement and convergence, has been preformed cooperating with research institute. Evaluation system and index will be established to protect subway structure.

#### REFERENCES

Cui Z.D., Tang Y.Q., Lu C., et al. 2007. Prediction of ground settlement induced due to changes in engineering environment in Shanghai. *Journal of Engineering Geology* 15(2):233–236.

- Duan J.L., Tan Y.L., Wang Y., et al. 2006. Case study of foundation pits crossing subway tunnel. *Chinese Journal of Geotechnical Engineering* 28(Z):1877–1879.
- Li X.Z., Huang M.S. & Wang L.M. 2007. Experimental study on creep and cyclic creep characteristics of saturated soft clay. *Journal of Chongqing Jianzhu University* 29(2):56– 59.
- Ye Y.D., Zhu H.H. & Wang R.L. 2007. Analysis on the current status of metro operating tunnel damage in soft ground and its causes. *Chinese Journal of Underground Space* and Engineering 3(1):157–161.
- Zhu Z.F., Tao X.M. & Xie H.S. 2006. The influence and control of deep excavation on deformation of operating metro tunnel. *Chinese Journal of Underground Space and Engineering* 2(1):128–131.

# Maximising the potential of strain gauges: A Singapore perspective

# N.H. Osborne, C.C. Ng & D.C. Chen

Deputy Project Manager, Land Transport Authority, Singapore

# G.H. Tan

Managing Director, SysEng Pte Ltd, Singapore

# J. Rudi

Instrumentation Engineer, Kiso Jiban Consultants Co. Ltd, Singapore Branch

# K.M. Latt

Instrumentation Engineer, Soil & Foundation Pte Ltd, Singapore

ABSTRACT: Load monitoring of support struts in temporary excavations plays a crucial role in confirming the stability and safety of the excavation. Data gained provides valuable feedback for the design engineer to facilitate refinement of future designs. Much of this monitoring is undertaken by strain gauges, sensitive instruments attached to the temporary supports, which are then linked to automated alarms through real time systems. The success of the monitoring is directly linked to the performance and interpretation of the data derived from the strain gauges, and the reliability of the real time system. There are a number of factors which can interfere with the performance of the monitoring system, to successfully and usefully interpret the data; the influence of these factors needs to be understood. The emphasis must be on the production of high quality data that can be reliably processed and rapidly given to the end-user, such that erroneous readings are minimized and genuine load changes are identified for interpretation. Through a number of case studies of deep excavation projects in Singapore, influences on the monitoring system are reviewed, their potential impacts discussed, and recommendations given to produce a high quality and reliable monitoring system, thus maximizing the potential of strain gauges to be used for monitoring of performance.

# 1 INTRODUCTION

# 1.1 Importance of instrumentation

Instrumentation and monitoring has always played a crucial role within the construction industry and recent worldwide high profile construction failures have further raised its profile and importance. This rings very true for Singapore. Technological advancements have allowed the instrumentation industry to become more sophisticated in how data is monitored, collected and presented to the end-users. However the emphasis remains on the production of quality data for useful interpretation. Strain gauges are one very important component of a fully integrated and comprehensive monitoring system used to control the movements and loads generated during excavations. In Singapore they are used extensively for strut monitoring in deep excavations and provide invaluable data ensuring that construction control is maintained during excavation. The build-up of load in the strut is monitored in real time during construction and compared with the design predictions at the various excavation stages. If significant discrepancies are observed, reanalysis is required, and a review of the design assumptions undertaken. However strain gauges are notoriously sensitive instruments and their readings can be influenced by a number of factors, leading to misinterpretation when the data is reviewed.

There are a number of wide-ranging potential influences impacting the accuracy of strain gauge results, encompassing a number of related construction areas: These include installation, environmental effects, construction activities and their position relative to structural members, all of which can result in erratic changes in the strain gauge readings, some of which are genuine load changes, some not. With the development of real time systems, the results can be automatically transferred to the internet for interpretation or sent to a mobile phone, but without any filtering of the erroneous readings. If received by inexperienced

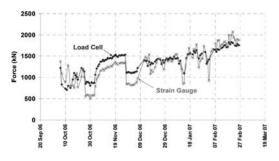


Figure 1. Comparison of strain gauge and load cell results.

personnel, commonly, all readings are treated as genuine, with the potential to cause unnecessary false panic and evacuation of the excavation, or erroneous and ignoring genuine changes in load. Either scenario results in a loss of confidence in the monitoring system. It is important therefore, that these very sensitive instruments and their performance be fully understood during interpretation and every effort taken to maximize the quality of the data.

# 1.2 Reliability of strain gauge – a comparison with load cell

Generally, load cell is known to provide reliable load measurement as it measures the full load across the full section of the strut. Strain gauge, on the other hand, measure less then 1% of the strut cross sectional area and then attribute that load to the whole strut. Figure 1 shows the data that gives an indication of the relative accuracies of strain gauges and load cells. The graph shows a general variance between 5% and 10%. Much of this difference can be accounted for by their different responses to the various construction influences, external factors such as temperature and EMI, and the different ways they monitor load. A detailed discussion of this topic is beyond the scope of this paper and needs to be part of a separate study. The correlation between the two instruments, however, is sufficient to indicate that strain gauge, is capable of monitoring the struts accurately.

With this fact established, strain gauge has other advantages over load cell. One fundamental benefit is the cost effectiveness of strain gauge in Singapore; with a strain gauge costing approximately U.S. \$200 and a load cell considerably more at U.S. \$4000. With cost, normally to the industry's detriment, significantly influencing the selection process, strain gauges are the instrument of choice. Cost aside from a technical perspective, introduction of load cell in the strut can create non typical loading conditions, but strain gauge does not change the loading condition on the strut. In addition, strain gauge can also be replaced easily if damaged.

#### 1.3 Choice of strain gauge type

Once a strain gauge is chosen to monitor a strut, there are two commonly available types for strut monitoring; spot weldable strain gauges (SWSG) or arc weldable (AWSG), also referred to as surface mount strain gauges. Both strain gauges are widely used in Singapore, with slightly different attributes and advantages. SWSG have the advantage of being attached low to the strut, therefore minimizing the impact caused by bending error. However the AWSG is more robust, its greater area fixed to the strut resulting in a better area to area connection, making it in theory less sensitive to fluctuation from vibration caused by accidental impact to the strut. By far the most important factor in ensuring the performance of the strain gauges is that they are installed correctly and not damaged during installation.

For SWSG the gauge flanges are spot welded to the strut, this requires exposing the thin 1.45mm gauge, increasing the possibility of damage. After the welding, the vibrating wire plucking coil housing is mounted onto the gauge for measurement and as a form of protective cover. It is crucial that the coil does not touch the gauge, which can occur with some brands, otherwise subsequent readings are affected. AWSG are arc welded to the flange, however a dummy gauge should always be used during welding to avoid damage, then replaced with the actual gauge, followed by mounting the vibrating wire plucking coil and further protection as required. The only potential problem with this installation is the introduction of residual stress into the structural member by the welding process. On overall comparison of the respective merits of the two gauge types the authors believe AWSG will provide more reliable results - a view shared by Broone & Crawford (2000).

#### 2 FACTORS INFLUENCING THE PERFORMANCE OF STRAIN GAUGE

#### 2.1 Location of strain gauge

When bending within the prop is likely to be significant, strain gauge reading will be affected by its location in the prop. Strain gauges placed at the same section may give different reading as they experience different stresses in the prop subject to bending. Connections between the prop and waling result in non-uniform stresses, as do connections to kingposts, cross-bracing or runner beams. Thus strain gauges installed near to these locations are likely to be affected by the non-uniform stresses and will not give a representation of the loads in the props.

#### 2.2 Impact of electromagnetic interference

As vibrating wire strain gauges operate at a frequency between 600 to 1500 Hz, they are subject

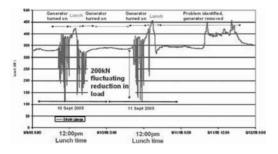


Figure 2. Impact of EMI noise on strain gauge readings.

to electromagnetic interference (EMI). This compromises the accuracy of the readings by introducing noise into the raw data, which can be very difficult to separate from genuine data, and therefore can be processed and calculated as load. There are numerous potential sources of EMI noise on construction sites including: arc welding, machinery ignition, power generators and power cables on the site. The noise takes one of two forms, either as a general underlying trend impacting the overall accuracy of data by increasing its spread. Or as a high voltage surge, causing a spike in the load readings when, for example, a machine ignition is started, electronic noise tends to lower readings, whereas magnetic noise increases them. With the advent of real time monitoring and data processing at 10 minute intervals, the impact of this interference becomes more significant. The first case leads to a general questioning the accuracy of the strain gauge readings as the accuracy range appears wider. The data spikes can result in an alarm being breached, with the potential for work to stop unnecessarily.

The impact of EMI noise can be clearly seen in Figure 2. During the working day on 9th & 10th September, electronic noise interference from a generator and power cable caused a 200 kN fluctuating reduction in load. The lunch hour can also be clearly seen when the generator was turned off. By the 11th September the noise had been identified and the generator removed, hence more stable readings.

#### 2.3 Temperature effect

The impact of temperature on the performance of strain gauges installed on struts has long been recognized as a significant factor affecting their performance, with papers first published on the issue in the early 1960s in Norway and Japan (NGI 1962 & Endo & Kawasaki 1963). Despite the advent of thermally matched strain gauges, thermal influence persists. In the United Kingdom for an un-decked excavation subject to the thermal effects due to sunlight, an annual temperature range of 50°C was measured leading to a variation of 2750 kN in strut loads, after the base of excavation was reached (Batten M et al. 1996). This equates to a change of load of 65 kN per 1°C. Similarly in the United States significant variations were measured and reported. Boone and Crawford 2000, recorded an 18.75 kN change per 1°C, with an annual temperature variation of 45°C on their site resulting in an annual fluctuation of 844 kN. The difference in these variations is due to the strut area, in combination with the stiffness of the retaining system, the ground it supports and the end restraint, with a greater stiffness resulting in a greater impact due to temperature variation.

Singapore lying  $1.5^{\circ}$  North of the equator experiences minimal seasonal variation in temperature, but a significant diurnal range, with temperatures fluctuating from a low of 20°C to a high of 36°C. This poses a different set of problems. The problem of temperature variation was first published by Niu et al (2005) discussing fluctuations due to a deep excavation in Singapore for the North East Line metro, in 1999. A load change of 37 kN per 1°C was recorded, against a theoretical change of 48 kN per 1°C. This being calculated by:

$$\Delta P = A_s E_s \alpha \Delta T \tag{1}$$

Where  $\Delta P$  is change in load due to temperature change ( $\Delta T$ ),  $A_s$  is cross sectional area of the strut,  $E_s$  is Young's modulus of steel and  $\alpha$  is the thermal coefficient of expansion for steel.

For this excavation, 23% of the theoretical increase in load was not observed in the instrumentation. The absence of this monitored load was explained and demonstrated by an outward movement of 2 mm of the retaining system during the higher temperatures. This phenomenon has been observed in the U.S. where a potential 13 mm movement into the ground was recorded in glacial till, (D. Druss 2000) and on other projects in Singapore, with a 2 mm movement into the ground of a 1.5 m thick diaphragm wall in soft Marine Clay.

The same temperature phenomenon is seen consistently across deep excavations in Singapore. For a 25 m deep excavation in soft Marine Clay on the Circle Line project an increase of 30 kN per 1°C was measured, Figure 3, across three different struts, over four days of non excavation. This equates to only 56% of the theoretical increase being transferred to the strut load. In this case the critical factor in mobilizing the full effect of the temperature lies in the ground. The retaining system was very stiff, with 1.5 m thick diaphragm walls, compared to the NELP example where a soldier pile system in considerably stiffer ground was used. Further to the general trend in the figure, there is some scatter in the data, this is due to readings taken at between 8:00 and 9:00 am and attributed to the very localized effect of plant start-up up causing EMI.

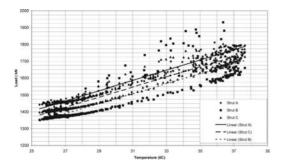


Figure 3. Impact of temperature on strut load.

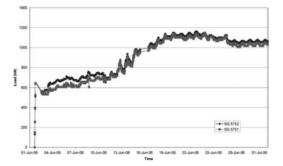


Figure 4. Loss of preload after strut installation.

#### 2.4 Preloading

Preloading has an important bearing on the strain gauge performance in early stages. The fundamental action is to install the strain gauge, take base readings and check it is working correctly prior to the onset of preloading. Base readings should ideally be taken prior to the strut being connected to its end supports.

During pre loading the load registered in the strain gauge and load cell should be checked against the jack load; however loads cannot be expected to match perfectly. The jack registers the highest load, followed by the strain gauge and then the load cell; this sequence tends to be the case observed in Singapore.

This can be attributed to a number of factors: the relative positions of the instrument, relatively low loads being used; construction difficulties in placing the strut truly perpendicular to the retaining system; temperature effects; the introduction of load cell in the struts affecting the overall load transfer pattern.

#### 2.5 Disturbance by construction activities

As with all excavations, the accuracy of the instrumentation readings can be accidentally influenced by a number of different construction activities, leading to potential misinterpretation of the readings. One of the obvious is the accidental damage of strain gauge

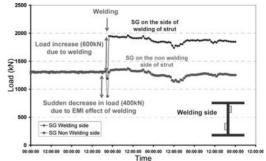


Figure 5. Impact of welding on adjacent strain gauges on the same strut.

by worker, which can either permanently damage the gauge or partially damage the gauge or connection.

#### 2.6 Welding

One of the construction effects resulting in erratic and significantly high loads registered by the strain gauge is welding. High heat from welding of horizontal ties or lacings to the strut member, which usually commences after preloading, can result in a high and sudden increase in the strut loads. As shown in Figure 5, the welding of lacing to a strut has caused the strut load measured by a pair of strain gauges located close to the lacing, to rise suddenly.

The impact of this welding on the strut load is clearly evident.

As shown in Figure 5, the readings on both sides of the strut web show a sudden drop, probably associated with EMI noise, followed by a sharp increase of load on the welding side. An increase from 1300 kN to 1900 kN was recorded. On the non-welding side a minimal rise in load was registered. On completion of welding the impacted gauge did not recover to its original load but remained at its elevated level, which is not representative of the overall load in the strut. This residual stress, recognized since 1964, is not representative of the actual load of the strut, and if clearly identified from the readings and construction activity, the reading can be adjusted to account for this effect.

#### 2.7 Casting of permanent slab

Another construction impact on temporary supports, and their strain gauges, is the effect due to casting of permanent components of a top-down excavation. During the casting of a 1.5 m thick roof slab, the impact of the curing and expansion of the slab can clearly be seen in the two layers of struts above the roof slab, Figure 6. A significant drop of 500 kN was observed across the full excavation, followed by an increase several days later and a return to the ongoing trend of the

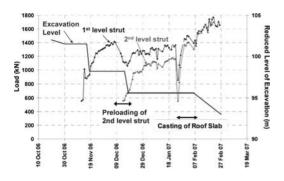


Figure 6. Impact of casting of permanent roof slab on strut loads above.

load. Once clearly identified, this phenomenon can be easily linked to construction activity, and not used to cast doubt on the accuracy of the strain gauge results.

#### 2.8 Negative values

Another phenomenon seen in strain gauge in deep excavation, particularly in soft clay, is that of negative loads in the top strut layers, indicating that the struts are in tension. This is frequently blamed on the instruments themselves and regarded as error readings. However investigations into a number of these cases have identified that the strain gauges are functioning well and recording genuine loads. Independent checks through cut off tests and inserting jacks have demonstrated that these struts are in tension. This is due to a combination of factors: the loads in the struts tend to be originally low at higher levels; loss of preload; and the deflection profile of the retaining wall. With soft clays, significant retaining wall movements are recorded, with deep seated movements occurring below excavation levels. As the excavation progresses, stiffer struts with greater preloads are used. Combined with these large movements below the struts, the retaining wall can rotate about the strut, resulting in a small backward movement into the soil at higher level. This is also reflected in the inclinometer readings.

#### 2.9 Problem related to real time system

To make the most effective use of strain gauge data, for deep excavations, it is prudent to link the instruments via data logger to the office computer and mobile phones in a real time system. However by implementing such a system two noteworthy problems need to be considered.

First the potential high number of alert alarms generated. The erroneous alerts can lead to a loss of confidence in the system and potentially a genuine alert being 'lost' amongst the false alarms.

The second and potentially more serious problem lies with the robustness of the real time system itself. The simile, a chain is only as strong as its weakest link, rings very true when applied to any real time system. Any failure of any component within the system compromises the whole monitoring scheme leading to an absence of results to the end user. Apart from the strain gauges, the potential numbers of points that can fail within the system are numerous. These include the cabling, the data logger itself, the phone system, the power and the server. Failures of all of these components have been experienced.

#### 3 HOW TO MAXIMIZE THE POTENTIAL OF STRAIN GAUGE

#### 3.1 Right location of strain gauge on strut

There are two important considerations when locating a strain gauge on a strut. First it's location on the strut, and then its position relative to other structural members. If bending within the prop is likely to be significant, strain gauges should be located to account for it, four gauges for a circular prop or proportionately spaced along the web for an I-beam. Connections between the prop and waling result in non-uniform stresses, as do connections to kingposts, cross-bracing or runner beams. Therefore the gauge locations should be at maximum distance from these areas, to be fully representative of the loads passing through the strut.

#### 3.2 Cross referencing with load cell

British CIRIA C517's guide on temporary design (Twine & Roscoe 1999) recommends strain gauges over load cells. However as cross referencing of data from different instrument types is critical to gaining a full picture of the excavation induced movement, it is recommended that at least some load cells be included in the instrumentation design. This concept is fully recognized in Singapore.

The Building Control Authority of Singapore (BCA) states that deep excavations must be monitored by a combination of strain gauges and loads cells. Land Transport Authority of Singapore (LTA), the client for the majority of deep excavations in Singapore which are for the continually growing underground rail network, specifies that as a minimum requirement, 25% of all struts for deep excavations shall be monitored for load by strain gauge and/or load cell, and of those monitored by strain gauge, 15% shall also be monitored by load cell.

#### 3.3 Minimizing EMI effect

EMI noise can be avoided by a few simple measures on site. It is recommended that cabling lengths be kept to a minimum; the cables and datalogger be located at least 5 m away from any potential source of EMI. Figure 7 shows a typical layout of data logger in an excavation site. Regular checks should be undertaken

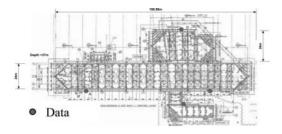


Figure 7. Typical layout of data logger on an excavation site.

by a portable EMI meter to ensure that cable and datalogger zones remain EMI free. Although cables can be protected from electronic noise, cable joints and the inevitable on site cutting and re-splicing of cables are weak points that can be easily corrupted, therefore particular attention should be paid to these locations.

#### 3.4 Account for temperature effect

The temperature range is less in Singapore, and therefore the impact of load less than elsewhere in the world. However strut loads are very closely linked with monitoring control of excavations. The maximum design load of a strut, using moderately conservative soil parameters, is used as the work suspension level, with an alert set at 70% of this capacity. There can be significant implications on the work, including stoppage of works if the temperature effect is not properly accounted for in the strut loads. A number of different solutions include painting struts white and daily spraying to reduce temperature impact. It is suggested that the most appropriate solution is to account for the theoretical temperature effects during design and to add them to the monitoring control values to ensure that work is not impacted unnecessarily.

# 3.5 *Protection against disturbance by construction activities*

Damage to strain gauge and disturbance on the data due to construction activities can be avoided through a combination of adequate protection, education of site staff through tools box talks and clear well marked signs on site. If damage occurs on a frequent basis, the more extreme measure of fining individuals or companies can be considered.

# 3.6 Maximizes strain gauge reading with real time system

To maximize strain gauge reading for monitoring of deep excavations, it is prudent to link the instruments via data logger to the office computer and mobile phones in a seamless fully automated machine to machine (M2M) system where no human intervention is required for onward transmission of results, resulting in a fully real time system (Tan et al 2004). It is strongly recommended that the capacity for data transfer of any such system is in minutes and that a wireless system be utilized.

An understanding of the potential problems can reduce false alerts, combined with alerts going only to knowledgeable personnel who are fully cognizant of the construction works being undertaken.

Potential failure of the cabling, the data logger, the phone system, the power and the server within the system can lead to loss of data or delay in transmission of data. To ensure that the system is fully automated and seamless, all these areas need to be rigorously checked and fail safes written into the systems to inform the system manager if any of these components fail, rather than assuming that all are functioning smoothly.

## 4 CONCLUSIONS

It is clearly evident that strain gauges are essential in monitoring and controlling deep excavations. With the increasing sophistication of the real time systems producing vast quantities of data, combined with M2M capabilities allowing automated alerts, and strict alarm limits on the monitored loads within the temporary structural systems, the results from strain gauges are under very close scrutiny. Therefore quality data and a clear understanding of both strain gauges and how the construction activities impact that data are crucial to the interpretation of strain gauge results. Without this, confidence in the performance of the system is lost, resulting in results being ignored as errors, very dangerous in the monitoring environment, or numerous unnecessary alarms received impacting the construction progress.

To maximize the potential of strain gauges, their locations and that of the cabling must be planned, installation carried out by skilled personnel aware of the problems and the potential to compromise data. Data interpretation should also be by skilled personnel fully aware of the design predictions for the excavation, the excavation progress and the potential impact of the excavation on the strain gauge results. Finally the processing system taking data from the strain gauge to the end user must be seamless and robust such that this component does not fail leading to a complete breakdown of the whole system.

#### REFERENCES

Batten, M., Powrie, W., Boorman, R. & Yu, H. 1996. Measurement of Prop Loads in a large excavation during construction of the JLE station at Canada Water, East London. Proceedings of *Geotechnical Aspects of Underground Construction in Soft Ground, London 1996 pp 7–12* 

- Boone, S.J. & Crawford, A.M. 2000. The effects of temperature and Use of Vibrating Wire Strain Gauges for Braced Excavations. *Geotechnical News September 2000.*
- Druss, D.L. 2000. Discussion The effects of temperature and Use of Vibrating Wire Strain Gauges for Braced Excavations, *Geotechnical News Vol. 18 No. 4 pp 24.*
- Endo, M. & Kawasaki, T. 1963. Study of Thermal Stresses Acting on Struts. Transactions of the Architectural Institute of Japan No. 63 pp 689–692
- NGI. 1962. Vibrating Wire measuring Devices Used at Strutted Excavations, Technical Report No. 9. Norwegian Geotechnical Institute, Oslo.
- Niu, J.X., Wong, I.H. & Makino, M. 2005. Temperature Effects on Strut Loads and Ground Movements for a 31m deep excavation in Singapore. *Proceedings of Under*ground Singapore 2005.
- Tan, G.H., Ng, T.G. & Brownjohn, J. 2004. Real Time monitoring and Alert Systems for Civil engineering Applications using Machine to machine Technologies. *Int. Conference on Structural and Foundation Failures, Singapore* 2004.
- Twine, D. & Roscoe, H. 1999. Temporary Propping of Deep Excavations, CIRIA Guide C517.

# Discussion on design method for retaining structures of metro station deep excavations in Shanghai

# R. Wang, G.B. Liu & D.P. Liu

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering of Tongji University, Shanghai, P.R. China

### Z.Z. Ma

Shanghai Shentong Holding Co., Ltd, Shanghai, P.R. China

ABSTRACT: By investigating many projects in Shanghai metro construction, notable differences are found to exist in present design schemes, especially in steel bar content of diaphragm wall and struts load. In this paper, some design schemes of retaining structures for metro station deep excavations which have the similar geological and structural situations are compared. Their rationality is evaluated and tested according to the results of field monitoring (e.g. strut load, bending moment of diaphragm wall deduced from lateral deformation curves). By adopting these measurements, a series of software, calculating models, methods and parameters can be summarized. Some key factors which affect the correctness of the design are emphasized. Finally, suggestions that satisfy demands of safety and economy are given. These suggestions help improve the design of retaining structures for deep excavation so that the design standard in Shanghai can be unified gradually.

# 1 GENERAL INSTRUCTIONS

In order to prepare the 2010 EXPO, construction activities of civil infrastructures and buildings in the municipality of Shanghai have been increased significantly. To meet the demands of high efficiency city traffic, extensive underground metro system in Shanghai has been rapidly constructed. How to accomplish the target of completing 400 km network in both quick and satisfactory way under the situation of large scale and high risk becomes a question to the constructors in Shanghai. Many problems during construction have been researched by many researchers and engineers over the years (Liu & Hou 1997; Liu et al. 1999; Liu et al. 2000, 2001). But few reports about the design of retaining structures of excavations are analyzed. Some retaining structure design schemes of normal metro station excavations which have the similar geological and structural situations are selected. According to the results of field monitoring (e.g. strut load, bending moment in diaphragm wall deduced from lateral deformation curves), the rationality of designs are analyzed. A series of software, calculating models, methods and parameters can be summarized, and key factors affecting the correctness of the design are emphasized. The intention of this paper is optimizing design methods satisfy demands of safety and economy. These suggestions help improve design of retaining structures for deep excavations so that the design standard in Shanghai can be unified gradually.

## 2 DESIGN OF RETAINING STRUCTURES OF DEEP EXCAVATION

# 2.1 Background of the design

# 2.1.1 Shape of the excavation

Compared with the diversification of common excavations, almost all metro station deep excavations are similar. The main excavation of metro station looks like a dumbbell on the plane, which divided into three parts—standard segment in the middle and end wells at two edges of the excavation.

# 2.1.2 Geological condition

Under the ground of Shanghai, there is soft soil about dozens meter deep, whose water content, degree of sensitivity, compressibility and rheology are high, and the unit weight, strength and permeability are low.

Protection grade	The requirement of maximum ground settlement and maximum retaining wall horizontal displacement	The requirement of surrounding protection
1	Maximum settlement of ground $\leq 0.1\%$ H*; Maximum horizontal displacement of retaining wall $\leq 0.14\%$ H; Ks** $\geq 1.8$	In the area not more than 0.7H away from excavation, there are metro, municipal pipes, gas pipes, important water pipes and so on. Those important building and utilities must be secured.
2	Maximum settlement of ground $\leq 0.2\%$ H; Maximum horizontal displacement of retaining wall $\leq 0.3\%$ H; Ks $\geq 1.6$	In the area $1 \sim 2H$ away from excavation, there are important mainlines, water pipes, important in-used structures and buildings.
3	Maximum settlement of ground $\leq 0.5\%$ H; Maximum horizontal displacement of retaining wall $\leq 0.7\%$ H; Ks $\geq 1.4$	In the area not more than 2H away from excavation, there are less important branch lines and general buildings and installations.

Table 1. The control criterion of environment protection in excavation.

\* H is the depth of excavation; \*\* Ks is the safety coefficient against basal heave.

# 2.2 Design method of the retaining structure in Shanghai

#### 2.2.1 Excavation protective grades

According to the engineering experience of Shanghai metro deep excavation engineering and the requirement of the surrounding environmental protection (Shanghai standard foundation design code DGJ08-11-1999), the deformation control criterion of excavation is classified into three protective grades (Table 1).

#### 2.2.2 Model

Finite element method (FEM) of two-dimensional vertical elastic subgrade beam was adopted for analyzing the deflection of diaphragm wall. In the FEM modeling, lateral earth pressure was taken as triangular and rectangular. The distribution of horizontal load is slope gradient on the excavation plane and rectangular under the excavation plane. The distribution of horizontal spring stiffness coefficient of the passive zone soil was also considered as two distributions. They have triangular distribution with slope gradient within effective depth under the excavation level and rectangular distribution below the depth (Figure 1).

### 3 DIVERSITY OF THE DESIGN RESULTS

Considering the similarity of excavation form and geological condition of metro stations in Shanghai, the retaining structures design of metro station deep excavations should be unified. But it is to find notable differences existing in the present design schemes by investigating many projects in Shanghai metro construction, especially in steel bar content of diaphragm wall and struts parameters. Results of retaining system design of fifteen deep excavations of metro stations in Shanghai are listed in Table 2. Regard two layers

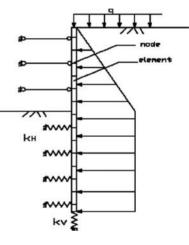


Figure 1. Vertical elastic subgrade beam model.

underground stations as the objects of analysis, the differences of them are as follows:

#### 3.1 Steel bar content of diaphragm wall

The excavation depth is at the range of about  $15 \sim 17$  m in standard segment of two layers underground excavation, but the unit weight of longitudinal bars plus hoops of diaphragm wall is quite distinction, from the minimum 127.05 kg/m<sup>3</sup> to the maximum 232.63 kg/m<sup>3</sup>, the value discrepancy is about 83%.

Station 1 and 4 are similar in excavation depth, but about 22% discrepancy in steel bar content of diaphragm wall.

The struts number of cross section of station 8 is one more than that of station 2, but still 44% higher steel bar content of diaphragm wall than that of station 2.

		Standard segment of the excavation						
Serial number of station	Structure form	Excavation depth/m	Length of the diaphragm wall/m	Thickness of the diaphragm wall/m	Embedded ratio of the diaphragm wall	Steel bar content of the diaphragm wall/kg $\cdot$ m <sup>-3</sup>	Vertical struts number	
1	Tow layers	17.28	34.0	0.8	0.97	127.05	5	
2	underground	16.80	30.0	0.8	0.79	128.86	4	
3	Ū.	16.60	32.0	0.8	0.93	145.82	5	
4		17.29	31.0	0.8	0.79	155.66	5	
5		16.30	30.0	0.8	0.84	155.85	5	
6		16.30	28.5	0.8	0.75	156.43	5	
7		15.66	30.0	0.8	0.92	169.47	4	
8		16.44	30.0	0.8	0.82	185.58	5	
9		15.57	28.5	0.6	0.83	186.44	4	
10		15.59	28.0	0.7	0.80	192.56	4	
11		14.56	28.6	0.6	0.96	196.97	4	
12		14.92	28.3	0.6	0.90	232.63	4	
13	Three layers	18.37	32.0	0.8	0.74	139.80	5	
14	underground	24.24	43.0	1.0	0.77	145.21	7	
15		21.56	37.0	1.0	0.72	153.10	6	

Table 2. Design results of bracing structural system of some stations of metro line M1.

\*Note: Steel bar content of the diaphragm wall in table 1 refers the unit weight of longitudinal bars plus hoops.

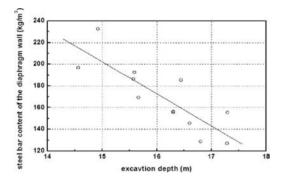


Figure 2. Variation of steel bar content of diaphragm wall with excavation depth.

The thickness of diaphragm wall of station 10 is 10 cm thicker than that of station 9, but almost the same in steel bar content.

All above shows that the regularity of steel bar content of diaphragm wall is not obvious now.

In addition, the decreasing tendency of steel bar content of diaphragm wall along with the excavation depth increasing is shown in Figure 2.

The steel bar content of diaphragm wall equals to Ag divided by V, which Ag is the total weight of longitudinal bars plus hoops of a diaphragm wall; V is the volume of a diaphragm wall. According to the design schemes, the diameter and spacing of the longitudinal bars and hoops are similar. But the spacing of both bars increases at the range of approaching wall toe; it causes the increment of weight of steel bars less than that of the volume when the excavation depth and length of the diaphragm wall increase. So the steel bar content of diaphragm wall decreased.

#### 3.2 Design parameters of steel struts

# 3.2.1 Comparison of design parameters of different stations

Strut design parameters of 3 stations which have the closely excavation depth and stratum distribution are compared (Table 3).

Spacing of the struts often ascertained according to experience, so there are less differential in it. But the design struts load calculating by model is discrepancy obviously.

# 3.2.2 Comparison of struts load by designing and by field monitoring

Field monitoring is an important means of providing immediate feedback to designers during excavation and of documenting the actual performance as a case history for future references.

Testing by the monitoring data of station 9 and 13 in Table 2 after excavation, real struts load are far less than the designing.

Station 9: The percentages occupied by real struts load to design values are 67% (level 2); 58% (level 3) and 67% (level 4).

Station 13: The percentages occupied by real struts load to design values are 72% (level 1); 69% (level 2); 46% (level 3); 37% (level 4) and 36% (level 5).

Considerable bearing capacity is wasted especially in the lower struts. So the struts design should be optimized.

Station		Ι	Π	III
Excavation depth/m		15.57	14.92	15.59
Spacing on plane/m		about 3.0	about 3.0	about 3.0
Vertical spacing/m	Level 1~ground level	0.5	0.4	1.0
	Level 2~Level 1	3.9	4.34	4.24
	Level 3~Level 2	3.9	3.7	4.1
	Level 4~Level 3	3.1	3.42	3.4
	Final level~Level 4	3.07	2.6	2.85
Struts load of design/kN	Level 1	/	800	800
	Level 2	1500	1660	1850
	Level 3	2000	2300	2000
	Level 4	1300	1975	1350

Table 3. Comparison of struts parameters of three station excavations (Standard segment).

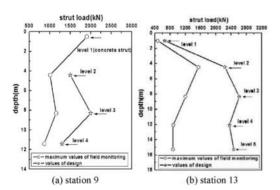


Figure 3. Struts load comparison of designing with field monitoring.

### 4 FACTORS ANALYSIS

By investigating eight design institutes which take on most design work of metro projects in Shanghai, there are many factors causing the disperse of design results. The main reasons are as follows:

#### 4.1 Diversification of calculating tools

The software calculating the internal force and lateral deflection of diaphragm wall is not unified. SUPER-SAP, ANSYS and MIDAS are used in calculating the main body of the excavation; FRWS, LIZHENG, YAO1, YAO2 etc. are used in checking the results or designing the affiliating structures.

#### 4.2 Values of the parameters

Effected by the designer's subjective understanding of the code, parameters like overload, lateral pressure, restrictions and so on are adopted differently. The difference of these initial parameters will lead to the different calculating results directly.

In addition, the phenomena of the designers concentrating on safe while ignoring the economy made the design schemes are quite conservative.

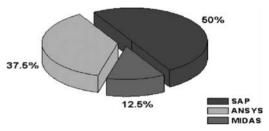


Figure 4. Usage percent of software in calculating the main body excavations.

#### 4.3 Relationship between design and construction

Construction is the means to realize the design schemes. It can test the rationality of the design. Therefore, mutual cooperation can help both of them getting better. But little contact between them makes the design theoretical; say nothing of optimizing the design according to the feedback of the field construction.

#### 5 DESIGN SUGGESTION

According to the design code and practical experience of Shanghai, some key points about the design of the retaining structure of normal metro station deep excavation are suggested or emphasized:

#### 5.1 Stricter design demands

The load coefficient has already increased from 1.25 to 1.35; The structure significance coefficient 1.1 should be considered; The concrete protection layer is thicker than before.

#### 5.2 Value of kv

The restriction under the toe of the diaphragm wall is simulated by vertical spring which kv often adopt  $10000 \text{ kN/m}^3$  in Shanghai.

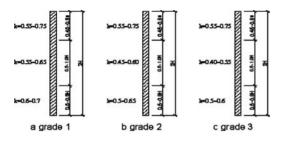


Figure 5. Empirical value of *k* in Shanghai.

#### 5.3 Calculation of lateral loads

Results of lateral loads don't have much difference no matter the earth and water pressure calculated together or not in Shanghai. But it had better to calculate them separately.

#### 5.4 Earth pressure

Earth pressure is a key parameter of excavation design. In the practices of soft clay's excavation engineering in Shanghai, by means of a lot of field measurements, indoors tests and centrifugal mode tests, the relation of earth pressure with different factors such as retaining wall's displacement, time, and construction parameters is erected. A practical formula of calculating earth pressure has already been put forward. The active earth pressure coefficient k can be ascertained by Figure 5 after excavating to the final level.

#### 5.5 Equivalent subgrade coefficient K<sub>h</sub>

 $K_h$  is a main calculation parameter in the calculation method of displacement and internal force of retaining wall used by design institutes. An equivalent level resistance coefficient  $K_h$  is used to reflect the capacity that soil resists deformation. It is a coefficient that involves viscidity, elasticity, plasticity and other diversified construction factors.

For purpose of facilitating design and engineering application, according to practices in Shanghai, usually the following simplified formula can be adopt:

when the subgrade without improvement:

$$K_{he} = 635 \cdot \frac{\gamma_i(h_i - h_{i-1})\tan^2\left(\frac{\pi}{4} + \frac{\varphi_i}{2}\right) + 4c_i \tan\left(\frac{\pi}{4} + \frac{\varphi_i}{2}\right)}{1.42\gamma_i(h_i - h_{i-1}) + 4.76} \cdot \exp\left(\frac{12.0 - T_j}{T_j}\right) \left[\frac{1.6(h_i - h_{i-1})}{B} + 0.1\right] \left[1.0 + 0.08\left(\frac{h_i + h_{i-1}}{2} - H_j\right)\right]$$
(1)

when the subgrade with improvement:

$$K_{hi} = (29.34 + 1431.9P_{i})\exp(\frac{12.0 - T_{j}}{T_{j}}) \times \left(\frac{1.6(h_{i} - h_{i-1})}{B} + 0.1\right) [1.0 + 0.08(\frac{h_{i} + h_{i-1}}{2} - H_{j})]$$
(2)

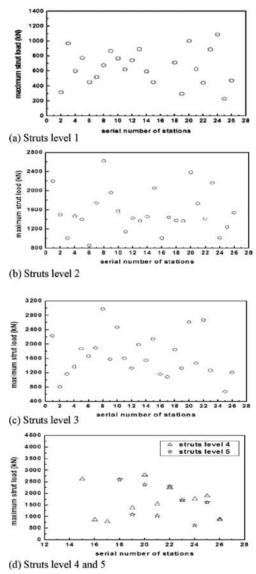


Figure 6. Maximum struts load by field monitoring.

Where  $\gamma_i$ ,  $c_i$ ,  $\varphi_i$  is specific gravity, cohesion, angle of internal friction of layer *i* respectively;  $h_i$  and  $h_{i-1}$  are bottom depth of layer *i* and layer i - 1 respectively;  $B_j$ ,  $T_j$ ,  $H_j$  is excavation width, excavation time, excavation depth of progress *j* respectively.

#### 5.6 Struts load

By survey of 26 station excavations, the maximum monitoring loads of each level struts are suggested. According to Figure 6, most values of struts load of excavation in Shanghai are 500~1000 kN (level 1),

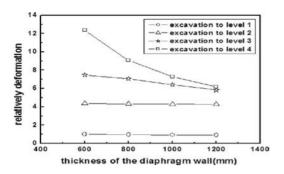


Figure 7. Deflection ratio of the wall in different cases and wall thickness.

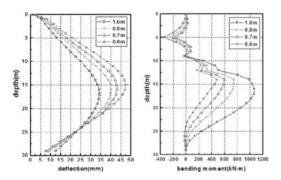


Figure 8. Comparison of deflection, bending moment in different wall thickness (excavate to final level).

 $1200 \sim 1800 \text{ kN}$  (level 2),  $1200 \sim 2200 \text{ kN}$  (level 3),  $1000 \sim 2500 \text{ kN}$  (level 4 and 5) respectively. The maximum values are about 1300 kN (level 1), 2800 kN (level 2), 3000 kN (level 3), 3000 kN (level 4 and 5) by deleting some special factors.

As shown in Figure 7, although the deflection of the diaphragm wall gradually increasing along with the excavation deeper, the increment of the deflection become lower with the wall getting thicker. But the method of confining the deflection by simply increasing the wall thickness is neither economical nor effective. It is not only playing the invalid function to limit the deflection but causing the bending moment of the wall increasing obviously when the thickness increase to specific value. (Figure 8) So the thinner wall should be better when the demand of safety can be satisfied. The thickness of the diaphragm wall in Shanghai often adopted 600 mm, 800 mm, 1000 mm and 1200 mm which increment is 200 mm. As the construction machine developed, thickness increment by 100 mm as modulus can be realized. Nowadays the attempt to apply the 700 mm diaphragm walls has succeeded in metro line m1 in Shanghai.



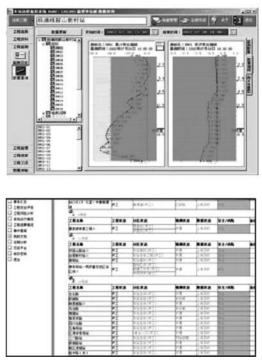


Figure 9. Video, data collection and net page platform of the remote monitoring system.

# 5.7 *Remote monitoring, management and consulting system by use of internet technology*

Remote monitoring and management system is intelligentized monitor, measurement and management system based on network technology and it is built on local network and Internet. Such a system can deal with the disperse projects, make the information, manpower and other resource of relative companies shared each other, make it convenient to follow the project and know what is going on.

People taking on monitoring of the construction field inputs the monitoring data into the system immediately after monitoring. The project managers and designers can consult the data through a specific net page platform; if there is any risk existing in the project, some construction methods or design adjustment can be done to prevent the accident happen. It will bring utmost profit to companies by providing intelligent and effective way of project management. At the same time, it can be also convenient to build and manage database and provide remote technology consultation.

#### 6 CONCLUSION

Based on the survey of large quantity of monitoring data of the retaining structure of metro station deep excavations in Shanghai, the following conclusions can be drawn:

Differences in the design schemes of the retaining structure of normal metro station deep excavation is depicted and analyzed. It mainly focuses on the steel bar content and steel strut load, which disaccordance with the character of similarity and regularity of metro station excavation.

A lot of factors affect the design results. Difference of calculating software, initial inputting parameters, designer objective understanding and so on becomes the main reasons. Suggestions are given according to the code and practical experience to help unify the design.

## REFERENCES

- Liu, G.B., Huang, Y.X. & Hou, X.Y. 2001. Study on Equivalent Lateral Resistance Factor of Soils in Excavation, *China Civil Engineering Journal.*
- Liu, J.H., Liu, G.B. & Fan, Y.Q. 1999. Theory of Time and Space Effect and its Practice in Soft Soil, *Journal of* Underground Engineering and Tunneling.
- Liu, J.H. & Hou, X.Y. 1997. Manual of Excavation engineering, Chinese Construction Industry Press, Beijing.
- Series of New Technologies of Environment Protection in the Construction of Subway; *from Shanghai Subway Corporation (General).*
- Shanghai Construction Commission. 1999. Shanghai standard foundation design code (DGJ08-11-1999), Shanghai Construction Commission (SCC), Shanghai, China. [In Chinese].
- Sun, G.S. & Zheng, D.T. 1987. Soft foundations and underground engineering, *China Architecture & Building Press*, Beijing, China.

# Risk analysis for cutterhead failure of composite EPB shield based on fuzzy fault tree

# Y.R. Yan & H.W. Huang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

# Q.F. Hu

Shanghai Institute of Disaster Prevention and Relief, Shanghai, P.R. China

ABSTRACT: All factors and related hazards in the cutterhead of composite EPB shield were systematically analyzed, and then fuzzy Fault Tree Analysis (FTA) model of cutterhead failure was proposed. Main basic events affecting the occurrence probability of the top event were verified by a quantitative analysis, which could be applied in the risk analysis of EPB shield machine's cutterhead. Compared with the traditional fault tree analysis, the fuzzy fault tree method can get the cutterhead failure possibility distribution of composite EPB shield. At last, the measures that help reducing the cutterhead failure occurrence were presented.

### 1 GENERAL INSTRUCTIONS

# 1.1 The actuality of composite EPB shield cutterhead failure

With the manufacture technique development of shield tunnelling, the scope of strata in which shields works becomes more and more widely. Shield technology is not only used in relatively uniformity or single ground, but also used in mixed ground that changes from hard rock to mixed face and soft ground (and vice versa) at the tunnel level. During the excavation, the highly abrasive and frequently changing mixed face ground causes high cutter wear, especially flat cutter wear; the accident rate of cutter disc to the total shield is highly to one half. The main forms of accidents are cutter abrasion, cutter disc abrasion, cutter disc distortion and so on (Lei Guo 2006). These accidents made the advance rate and cutterhead service life largely depressed. According to statistics, in domestic composite EPB (Earth Pressure Balance) shield construction, the failure of cutter abrasion came forth at differently degree (Weibin Zhu et al. 2006).

# 1.2 The introduction of fuzzy FTA

In the traditional fault tree, a failure event system (top event) is divided into many sub-events with a combination of series and parallel. Its failure probability can be back-calculated according to the logical relationship of the fault tree when the failure probability of each basic event is known. Fault tree analysis is based on Boolean algebra. A quantitative analysis needs probabilities of all basic events or the minimal cut set, which are mostly obtained by statistical data or subjective judgmental data based on experts' experiences. These data have uncertainty because of various influence factors during statistical procedures and limitation of experts' experiences. It is necessary to define a fuzzy value in the probabilistic space to represent a single probability.

Basic concepts and methods of the fuzzy fault tree were proposed in 1980s (Tanaka et al.1983, Furuta 1984). At present, the study of the fuzzy fault tree is almost focused on algorithm and the integrated theory systems have not been established and verified in practice (Singer 1999). A fuzzy set is introduced in this paper and the failure probability of basic events is replaced by the fuzzy failure probability. A triangular fuzzy number is introduced to represent the failure probability of a basic event and the fuzzy failure probability of a top event is obtained by fuzzy number operation. Measures to reduce the cutterhead failure possibility of the EPB shield are verified by analyzing the importance of basic events.

### 2 FUZZY SET THEORY AND ITS OPERATIONS

#### 2.1 Fuzzy set

Fuzzy set theory was introduced by Zadeh (1965) to deal with the problem in which the phenomena are

imprecise and vague. Let X be a collection of objects, called the universe, whose elements are denoted by x. A fuzzy subset A inX is characterized by a membership function  $f_A(X)$  which associates with each element x in X a real number in the interval [0, 1]. The function value  $f_A(X)$  represents the grade of membership of x in A. The larger the  $f_A(X)$  is, the stronger the degree of belongingness for x in A.

#### 2.2 Fuzzy numbers and its operations

Switch between the fuzzy numbers are used to handle imprecise information such as 'close to 5', 'high reliability', 'low failure rate', etc. There are many forms of fuzzy numbers to represent the linguistic values. In here, triangular fuzzy numbers are applied. Let *x*, *a*, *m*, *b*  $\in$  *R* (real line). A triangular fuzzy number is a fuzzy number *A* in *R*, if its membership function *f<sub>A</sub>*: *R* ~ [0, 1] is

$$\mu_{A}(x) = \begin{cases} 0 & \text{for } x < a \\ (x-a)/(m-a) & \text{for } a \le x \le m \\ (b-x)/(b-m) & \text{for } m \le x \le b \\ 0 & \text{for } x > b \end{cases}$$
(1)

In this study, triangular fuzzy numbers are employed. A triangular fuzzy number can be defined by a triplet  $\tilde{A} = (a, m, b)$ . The membership function is with  $a \le m \le b$ . The triangular fuzzy number can be denoted by a triplet  $\tilde{A} = (a, m, b)$ . The parameter mgives the maximal grade of  $f_A(X)$ , i.e.  $f_A(m) = 1$ , it is the most probable value of the evaluation data. a and m are the lower and upper bounds of the available area for the evaluation data.

For a given  $\lambda$  in the the interval [0, 1], the arithmetic operations of fuzzy numbers can be defined by means of  $\lambda$ -cut operations according to the extension principle(Zadeh 1965):

$$A_{\lambda} = \{x \mid x \in R, \mu_{\bar{\lambda}} \ge \lambda\} = [a_1^{\lambda}, b_1^{\lambda}]$$
$$B_{\lambda} = \{x \mid x \in R, \mu_{\bar{\mu}} \ge \lambda\} = [a_2^{\lambda}, b_2^{\lambda}]$$

Then

$$\widetilde{A}(+)\widetilde{B} = A_{\lambda} + B_{\lambda} = [a_1^{\lambda} + a_2^{\lambda}, b_1^{\lambda} + b_2^{\lambda}]$$
(2)

$$\tilde{A}(-)\tilde{B} = A_{\lambda} - B_{\lambda} = [a_{1}^{\lambda} - a_{2}^{\lambda}, b_{1}^{\lambda} - b_{2}^{\lambda}]$$
(3)

$$\widetilde{A}(\cdot)\widetilde{B} = A_{\lambda} \cdot B_{\lambda} = [a_{1}^{\lambda} \cdot a_{2}^{\lambda}, b_{1}^{\lambda} \cdot b_{2}^{\lambda}]$$

$$(a_{1}^{\lambda} \ge 0, a_{2}^{\lambda} \ge 0)$$
(4)

$$\tilde{A}(\div)\tilde{B} = A_{\lambda} \div B_{\lambda} = [a_1^{\lambda} / b_2^{\lambda}, b_1^{\lambda} / a_2^{\lambda}]$$
$$(a_1^{\lambda} \ge 0, a_2^{\lambda} > 0)$$

#### 3 FTA OF CUTTERHEAD FAILURE

## 3.1 Fault tree model for the risk of composite EPB shield cutterhead failure

The main purpose of the fault tree analysis is to find out all failure modes of the system and the event with a rather large failure probability. After weak sections have been enhanced, occurrence probabilities of these accidents are reduced so that the system reliability is improved. For researching the cutterhead failure risk of composite EPB shield, there were some supposition in the analysis as follows:

- It didn't take account of the disadvantageous influence that the engineering construction brought to the surroundings building, road surface, underground pipeline etc.
- Drag bits and scrapers would often produce normal wear during the shield advance, so their abrasion was not considered.

According to the above investigation accidents data, the cutterhead failure risk of composite EPB shield in mixed face ground was set by means of the fuzzy FTA method, shown in Figure 1.

The systemic risk probability of cutterhead is analyzed based on the established fault tree model. Firstly, the minimum basic event sets which cause the main event occurrence is solved, i.e. the minimum cut set (MCS for short) of fault tree. Each MCS corresponds to one accident type, and there are several MCSs with different occurrence probability of one fault tree. The MCS with the maximum occurrence probability is the most probably potential factor which may cause accident. Boolean algorithm is relatively simple in solving the MCS. The fault tree in Figure 1 is obtained from the Boolean algorithm as it shown in Eq. (6).

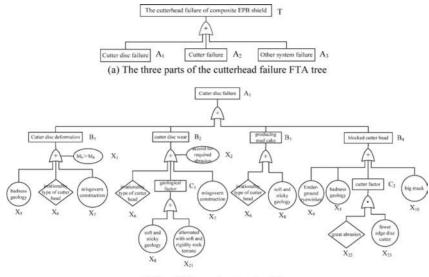
From the result obtained above, the top event *T* is the union of 28 sets which are the MCS of the fault tree, i.e.  $\{X_1X_7\}, \{X_2X_7\}, \{X_3X_7\}, \{X_3X_{11}\}, \{X_3X_{16}\}, \{X_3X_{18}\}, \{X_3X_{24}\}, \{X_3X_{25}\}, \{X_4X_{11}\}, \{X_4X_{12}\}, \{X_4X_{13}\}, \{X_4X_{14}\}, \{X_4X_{18}\}, \{X_4X_{24}\}, \{X_4X_{26}\}, \{X_{22}X_{23}\}, \{X_5\}, \{X_6\}, \{X_8\}, \{X_9\}, \{X_{10}\}, \{X_{15}\}, \{X_{17}\}, \{X_{19}\}, \{X_{20}\}, \{X_{21}\}, \{X_{27}\}, \{X_{28}\}$ , which correspond to 28 accident modes.

#### 3.2 Quantitative analysis

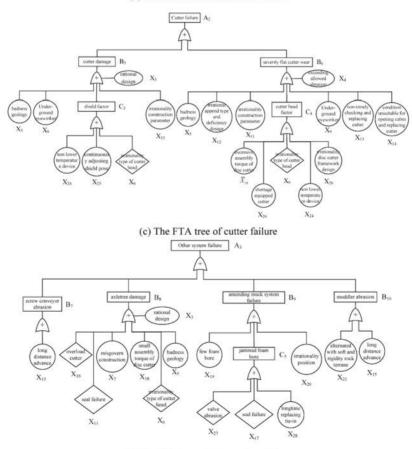
Quantitative analysis includes evaluation of failure probability of the top event and important analysis of the basic events. In practice, the occurrence probability of a top event (PT) is obtained using approximate probability formula of independent events as shown in Eq.(7)

$$P_{\rm T} = 1 - \prod_{i=1}^{m} [1 - P(M_i)]$$
(7)

(5) Where  $P(M_i)$  is the occurrence probability of the ith MCS. For example,  $P(M_i)$  is the occurrence



(b) The FTA tree of cutter disc failure



(d) The FTA tree of other system failure

Figure 1. Fault tree of the cutterhead failure risk of composite EPB shield.

$$T = A_{1} + A_{2} + A_{3}$$

$$= (B_{1} + B_{2} + B_{3} + B_{4}) + (B_{5} + B_{6}) + (B_{7} + B_{8} + B_{9} + B_{10})$$

$$= [X_{1}(X_{5} + X_{6} + X_{7}) + X_{2}(X_{6} + C_{1} + X_{7}) + (X_{6} + X_{8}) + (X_{9} + X_{5} + C_{2} + X_{10})]$$

$$+ [X_{3}(X_{5} + X_{9} + C_{3} + X_{11}) + X_{4}(X_{5} + X_{12} + X_{11} + C_{4} + X_{9} + X_{13} + X_{14})]$$

$$+ [X_{15} + X_{3}(X_{16} + X_{11} + X_{7} + X_{18} + X_{6} + X_{5}) + (X_{19} + C_{5} + X_{20}) + X_{21} + X_{15}]$$

$$= [X_{1}(X_{5} + X_{6} + X_{7}) + X_{2}(X_{6} + X_{8} + X_{21} + X_{7}) + (X_{6} + X_{8}) + (X_{9} + X_{5} + X_{22}X_{23} + X_{10})]$$

$$+ [X_{3}(X_{5} + X_{9} + X_{24} + X_{25} + X_{6} + X_{11}) + X_{4} \begin{pmatrix} X_{5} + X_{12} + X_{11} + \overline{X}_{18} + X_{26} + X_{6} + X_{24} \\ + X_{26} + X_{9} + X_{13} + X_{14} \end{pmatrix} ]$$

$$+ [X_{15} + X_{3}(X_{16} + X_{11} + X_{7} + X_{18} + X_{6} + X_{5}) + (X_{19} + X_{27} + X_{17} + X_{28} + X_{20}) + X_{21} + X_{15}]$$

$$= X_{1}X_{7} + X_{2}X_{7} + X_{3}X_{7} + X_{3}X_{11} + X_{3}X_{16} + X_{3}X_{18} + X_{3}X_{24} + X_{3}X_{25} + X_{4}X_{11} + X_{4}X_{12} + X_{4}X_{13} + X_{4}X_{14} + X_{4}\overline{X}_{18} + X_{4}X_{24} + X_{4}X_{26} + X_{22}X_{23} + X_{5} + X_{6} + X_{8} + X_{9} + X_{10} + X_{15} + X_{17} + X_{19} + X_{20} + X_{21} + X_{27} + X_{28}$$

probability of the first MCS  $\{X_1X_7\}$ , which depends on the probability multiplication of basic events  $X_1$ and  $X_7$ .

Probabilities of the basic events must be known in advance, in order to evaluate failure probability of the top event and important analysis of the basic events. Expert elicitation and fuzzy set theory will be used to get the probabilities of the basic events in this paper. Because the experts cannot exactly evaluate the probability of events, and sometimes some of the events are vague, the experts tend to apply natural linguistic expression, such as 'Impossible, Infrequent, Occasional, Possible and Frequent', to describe the probability of events. According to the 'Guidelines of Risk Management for Metro Tunnelling and Underground Engineering Works' (2007), it ranks the occurrence probability of risk into 5 class, shown in Table 1. Conventional mathematical ways cannot handle natural linguistic expression efficiently because of its vagueness. Therefore, fuzzy set theory is used to cope with it. According to the triangular fuzzy number discussed above, it is assumed herein

$$a_i = 0.95m_i$$
  $i = 1, 2, \cdots, n$  (8)

$$b_i = 1.05m_i$$
  $i = 1, 2, \cdots, n$  (9)

The occurrence probabilities of random basic events using Eq.(8) and Eq.(9), and their fuzzy probabilities are shown in Table 2.

Figure 2 shows that the fuzzy probability of the top event can be expressed as triangular fuzzy numbers and the parameters are (0.9925, 0.9960, 0.9982). The corresponding occurrence probability is  $0.9925 \sim 0.9982$ , however, the most possible probability is equal to 0.9960 with a membership grade equal to 1.

#### 3.3 Sensitivity analysis

The main basic events affecting the occurrence probability of the top event can be determined and some



Figure 2. Possibility distribution of cutterhead failure probability of composite EPB shield.

Table 1. The classification of risk occurring probability.

Rank	Accident description	Interval probability
One Two Three Four Five	Impossible Infrequent Occasional Possible Frequent	$\begin{array}{c} P < 0.01\% \\ 0.01\% \leq P < 0.1\% \\ 0.1\% \leq P < 1\% \\ 1\% \leq P < 10\% \\ P \geq 10\% \end{array}$

effective measures are verified by sensitivity analysis to reduce occurrence probability of the basic events and the top event. According to the fuzzy number model defined in this paper, sensitivity evaluation index  $V_i$  is simply defined in Eq.(10) (Chen and Zhang 2002)

$$V_{i} = \frac{\partial g(x)}{\partial x_{i}} \bigg|_{x=\mu_{si}} \frac{\mu_{xi}}{\mu_{g}}$$
(10)

Where  $\mu_g$  is the occurrence probability of the top event,  $\mu_{xi}$  is the average occurrence probability of the basic event *x*.

If  $V_i \ge V_j$ , it is more effective to minimize the occurrence probability of the top event by reducing the occurrence probability of the event *i* rather than the event *j*.

л <sup>.</sup>	Fuzzy prob	Fuzzy probability value				
Basic event	a	m	b	Sensitivity index		
$X_1$	0.000095	0.01%	0.000105	1.0043e-6		
$X_2$	0.000475	0.05%	0.000525	5.0216e-6		
$X_3$	0.095	10%	0.105	0.0083		
$X_4$	0.8075	85%	0.8925	0.9532		
$X_5$	0.0855	9%	0.0945	0.0803		
$X_6$	0.0475	5%	0.0525	0.0502		
$X_7$	0.0095	1%	0.0105	0.0010		
$X_8$	0.76	80%	0.84	0.8035		
$X_9$	0.000665	0.07%	0.000735	7.0302e-4		
$X_{10}$	0.000475	0.05%	0.000525	5.0216e-4		
$X_{11}$	0.0057	0.6%	0.0063	0.0057		
$X_{12}$	0.00076	0.08%	0.00084	6.8294e-4		
$X_{13}$	0.095	10%	0.105	0.0854		
$X_{14}$	0.0038	0.4%	0.0042	0.0034		
$X_{15}$	0.076	8%	0.084	0.0803		
$X_{16}$	0.057	6%	0.063	0.006		
$X_{17}$	0.00855	0.9%	0.00945	0.009		
$X_{18}$	0.00076	0.08%	0.00084	8.0345e-5		
$X_{19}$	0.000665	0.07%	0.000735	7.0302e-4		
$X_{20}$	0.000855	0.09%	0.000945	9.0389e-4		
$X_{21}$	0.76	80%	0.84	0.8035		
$X_{22}$	0.0855	9%	0.0945	6.3272e-4		
$X_{23}$	0.00665	0.7%	0.00735	6.3272e-4		
$X_{24}$	0.0057	0.6%	0.0063	0.0051		
X <sub>25</sub>	0.0038	0.4%	0.0042	4.0173e-4		
$X_{26}$	0.00057	0.06%	0.00063	5.1e-4		
$X_{27}$	0.00038	0.04%	0.00042	4.0173e-4		
$X_{28}$	0.057	6%	0.063	0.0603		

Table 2. Fuzzy probability of the basic event in the fault tree.

Sensitivity indices of all basic events are obtained by sensitivity analysis of the fault tree of the cutterhead failure risk of composite EPB shield as shown in Fig.1.

The basic event in which the sensitivity index is greater than 5% is chosen and arranged as follows:  $V_4 = 95.32\%$ ,  $V_8 = V_{21} = 80.35\%$ ,  $V_{13} = 8.54\%$ ,  $V_5 = 8.03\%$ ,  $V_{28} = 6.03\%$ ,  $V_{24} = 5.1\%$ ,  $V_{13} = 5.02\%$ .

#### 4 MAINLY INFLUENCE FACTOR AND IMPROVENMENT MEASURE

According to the order result, single minimum cut set  $X_5$ ,  $X_6$ ,  $X_5$ ,  $X_8$ ,  $X_{21}$  and so on easily cause the failure of cutterhead; Secondly, the basic events  $X_4$ ,  $X_7$ ,  $X_{24}$  which appear more times also easily cause the top event failure. They are the weakness parts of the system and the main risk factors arousing the failure of the shield cutterhead in the mixed face ground. Therefore, during the tunnel construction, it aims at surveying and managing the basic events which greatly influents the top event occurrence to lower the risk accidents of cutterhead failure.

- 1. For lowering the risks  $X_5$  (badness geology),  $X_8$  (soft and sticky geology),  $X_{21}$  (alternated with soft and rigidity rock terrane) influences on the cutterhead failure, it should strengthen to run the geology forecast, accurately certain the position of the badness geology and its distribute, and adopt corresponding measures in advance.
- 2. For lowering the risk  $X_6$  (irrationality type of cutterhead) influence on the cutterhead failure, it should accord to the geology and hydrology condition, structural design, construction advance request etc. factor, choose much adaptability cutterhead, and make an adequacy adjustment of shield installation under concrete conditions during construction.
- 3. For lowering the risks  $X_4$  (exceeding allowed abrasion),  $X_7$  (misgovern construction) influences on the cutterhead failure, it should choose reasonable advance model and parameters, continuously accumulate the experience, and reduce man-made breakage; installing the cutter wear monitor system, it can accurately obtain the information of the cutter wear, then adopt corresponding measures; enhancing cutter's replacing rate and quality.

#### 5 CONCLUSIONS

From risk analysis of composite EPB shield cutterhead failure in the mixed face ground based on fuzzy FTA, the conclusions are as follows:

- Having a directly view and simple character, the FTA is a valid method for analyzing the failure risk of composite EPB shield cutterhead in mixed face ground.
- 2. It totally considered 28 basic events for the failure risk fault tree of composite EPB shield cutterhead. Through the fuzzy fault tree calculation, it can definite the weakness parts of cutterhead failure, confirm the key factors of risk occurrence, and make an order for the importance of various influence factors.
- 3. Through fuzzy FTA of the cutterhead failure, it made sure the mostly reason and mechanism of leading to the failure risk of composite EPB shield cutterhead, brought forward the improved measures and suggestion. So it gave some useful conference for preventing or reducing the failure risk of composite EPB shield cutterhead during construction.

#### REFERENCES

- Cao, H.L., Lv, C.T. & Li, J.X. 2004. Failure analysis for cutters and its prevention measures. *Tunnel Construction* 24(6): 9–10.
- Evans, J.R. 2001. Itroduction to Simulation and Risk Analysis. 2/e, Prentice Hall Business Publishing.

- Furuta, H. & Shiraishi, N. 1984. Fuzzy importance in Fault tree analysis. J.Fuzzy sets and systems 12: 205–213.
- Guo, L. 2006. Fault elimination of mixed face shield during construction. *Modern Tunneling Technology* (supplement): 436–439.
- Meng, X.J. 2004. Causation analysis and resolve measures for the ordinary fault (damage) of the mixed EPB shield cutterhead. *Tunnel Construction* 24(2): 61–66, 73.
- Singer, D. 1999. Fuzzy set approach to fault tree and reliability analysis. J.Fuzzy sets and systems 34: 145–155.
- Tanaka, H., Fan, L.T. & Taguchi, T. 1983. Fault tree analysis by fuzzy probability. J. IEEE Trans on Reliability 32: 453–457.
- Tongji University 2007. Guidelines of Risk Management for Metro. Tunnelling and Underground Engineering Works. 2007.

- Zadeh, L.A. 1965. Fuzzy sets Information and Control 8: 338–353.
- Zhang, G.D. & Lu, Y.X. 1990. Analysis and design of reliability and maintenance of system. Beijing: Beijing Aeronautics and Astronautics University Press: 120–125.
- Zhao, J., Gong, Q.M. & Eisensten, Z. 2007. Tunnelling through a frequently changing and mixed ground: A case history in Singapore. *Tunnelling and Underground Space Technology* 22: 388–400.
- Zhu, W.B., Ju, S.J. etc. 2006. Shield tunnelling technology in mixed face ground conditions. Beijing: China Science and Technology Press.

### Risk assessment on environmental impact in Xizang Road Tunnel

C.P. Yao, H.W. Huang & Q.F. Hu

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: Nowadays, with the increase in the demands for environmental protection, the assessment of construction on the environmental impact has become a new topic. Based on the Risk Management Software (TRM 1.0) which is developed by Tongji University, the authors investigate the surrounding environment condition and construction process, calculate the probability of hazard risks and its loss, and provide the ranks of hazard risks and the corresponding treatment on risks. It may give some advice and suggestions on underground construction.

#### 1 INTRODUCTION

Because of its unique unseen character, isolation from nature and complexity, the construction of underground works is more difficult than the process of general buildings. In the age of high demands of environmental protection, the environmental impact caused by underground works construction is attracting increasing attention. The paper is for the risk assessment on environmental impact caused by the construction of Shanghai Xizang South Road Tunnel from the point view of risk. The authors make quantitative analysis on the impact of construction on surrounding buildings, the impact on surrounding roads and the impact on surrounding pipelines. At last, the ranks of hazard risks are provides as well as the risk reduction measures. It may give some advice and suggestions on underground construction.

#### 2 PROJECT PROFILE

Xizang South Road Tunnel is a planning river-crossing tunnel in Shanghai Expo. The main line of the tunnel is from the intersection of Puxi Xizang South Road and Zhongshan South Road to the intersection of Pudong Binzhou Road and Yunlian Road, including the intersection of Binzhou Road and Pudong South Road. The total project is 2673 meters long. Figure 1 shows the road tunnel plan.

The tunnel is advanced by Slurry Balance Shield machine of 11.36-meter diameter. Project consists

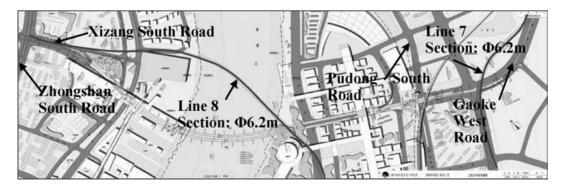


Figure 1. Xizang south road tunnel plan.

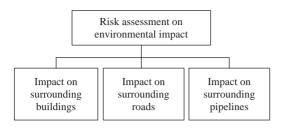


Figure 2. Risk identification map of environmental impact.

of the approach of Puxi, Puxi rectangular tunnel (including wells), the circular tunnel, Pudong rectangular tunnel (including wells) and Pudong approach. Along the setting location, there are great population, many buildings, complex pipelines and complex traffic systems. Also, the tunnel will up-cross Line 7 and down-cross Line 8, both of which are components of Shanghai Railway Transit System. Moreover, the tunnel will intersect with magnetic train line which will be constructed soon. In the later part, the authors will assess the risk of environmental impact caused by construction, including impact on surrounding buildings, on surrounding roads and on surrounding pipelines. It is shown in Figure 2.

#### 3 RISK ASSESSMENT ON ENVIRONMENTAL IMPACT

#### 3.1 Risk assessment process

Risk assessment is usually divided into three steps (Guo, 1986 & Chen, 2004):

- 1 Risk identification: Analyze all potential risk factors which may influence the environment during
- Table 1. Risk investigation form (part of it).

the construction process, and then classify, collating those parameters affecting greatly.

- 2 Risk analysis: Calculate the probability and consequence of risk factors.
- 3 Risk evaluation: According to some certain criteria, evaluate the risk factors.

Where, risk evaluation is to use Experts Investigation Method (EIM) and Confidence Index Method (CIM), to analyze the probability and consequence of risk factors, and to gain the ranks of risks. EIM is a kind of gathering information method. We send investigation forms concerning with the risk factors in engineering to experts, professors, senior consulters and so on. The CIM is just an index to show their confidence when doing the judgment. The data can be put into the database of TRM 1.0, which is developed by Tongji University (Huang, et al 2006b). The detailed data-input procedure can refer to the reference Huang, et al 2006b. Table 1 shows part of the risk investigation form.

In this paper, the applied risk evaluation criteria is "Guidelines for Tunnelling Risk Management" enacted by the International Tunnel Association in 2002 and "Guidelines of Risk Management for Metro Tunnelling and Underground Engineering Works" enacted by Tongji University (International Tunnel Association, 2002 & Huang, et al 2006a). They are shown in Tables 2, 3, 4 and 5.

## 3.2 Risk assessment on environmental impact on surrounding buildings

#### 3.2.1 Risk identification

The possible risk hazards of surrounding buildings caused by construction can be illustrated in Figure 3 (Yao, et al 2006).

Environmental assessment		Occurrence probab	oility	Loss	
Risk hazard	Risk hazard Risk-inducing factor		Confidence index	Loss rank	Confidence index
Table 2. Risk	occurrence probability	anks.			
Ranks	А	В	С	D	Е
Occurrence probability	Impossible P < 0.01%	$\begin{array}{l} \text{Seldom} \\ 0.01\% \leq P < 0.1\% \end{array}$	Occasional $0.1\% \le P < 1\%$	Possible $1\% \le P < 10\%$	Frequent $P \ge 10\%$
Table 3. Loss	s ranks of risk hazards*.				
Ranks	1	2	3	4	5
Details	Ignored	Considered	Serious	Very serious	Disastrous

\*Here, the criterion on rank of loss differs from country to country. In China, the loss is disastrous when 5 lives lost.

#### 3.2.2 Risk analysis

The overall sinking of buildings is caused by the even settlement of ground. Uneven settlement can induce the tilting and cracking of buildings. During the construction process, many factors can cause ground settlement (Yao & Huang, 2007). For example, the seepage of shield and pit, pit collapse, the improper earth pressure of shield and support damage. Such factors may also be the cause of building damage. If this happens, it will cause great social influence and economic loss.

The passing buildings of the tunnel can be shown in Table 6, Appendix 1 & 2, respectively.

We may see, the buildings are very dense, therefore the risk is high. It is very easy to tilt or crack for the buildings during the construction.

#### 3.2.3 Risk evaluation

By sending out investigation forms, and using the TRM 1.0, the authors get the risk ranks of these risk hazards. The results are shown in Table 7, from which we may see building's crack is more risky and is highly probable to happen.

		Loss					
Risk		1. Ignored	2. Considered	3. Serious	4. Very serious	5. Disastrous	
Occurrence	A: P < 0.01%	1A	2A	3A	4A	5A	
probability	B: $0.01\% \le P < 0.1\%$	1B	2B	3B	4B	5B	
	C: $0.1\% \le P < 1\%$	1C	2C	3C	4C	5C	
	D: $1\% \le P < 10\%$	1D	2D	3D	4D	5D	
	E: $P \ge 10\%$	1E	2E	3E	4E	5E	

#### Table 5. Risk acceptance criteria.

Ranks	Risk	Acceptance criteria	Measures
I	1A,2A,1B,1C	Ignored	No need for management and supervision
II	3A,2B,3B,2C,1D,1E	Allowable	Call for attention, need regular management and supervision
III	4A,5A,4B,3C,2D,2E	Accepted	Call for great attention, need prevention, and monitoring measures
IV	5B,4C,5C,3D,4D,3E	Unaccepted	Need the decision of policy-maker, need controlling measures
V	5D,4E,5E	Cannot be accepted	Immediately stop, need alternative plans

Table 6. Surrounding buildings in Puxi and Pudong District.

District		Buildings
Puxi	Manufacturing	6 buildings with $5 \sim 6$ floors, 2 buildings with 4 floors,
district	bureau	2 buildings with 1 floor, 2 buildings with 3 floors, 1 building with 10 floors, 1 building with 7 floors
	Xizang south road	Many buildings with $3 \sim 7$ floors, 2 buildings with 15 floors, 1 building with 12 floors, 1 building with 10 floors, several buildings with 1 floor, 1 building with 19 floors, 1 building with 28 floors (reserved for Shanghai Expo)
	South station road	1 building with 16 floors, 1 building with 5 floors, 2 buildings with 6 floors, 1 building with 7 floors
	Jiangnan shipyard	Three-floor defense project, 1 building with 6 floors, distribution substation with 5 floors, 3 docks
Pudong district	Shanghai hangbiao factory	Floating pier, floodwall, 3 steel pipe piles, several buildings with $3 \sim 6$ floors
	Nanchuan factory Shangnan road	Plant with $1 \sim 3$ floors, 1 building with 4 floors, 2 buildings with 6 floors Several buildings with $1 \sim 3$ floors, 1 building with 15 floors, 1 building with 13 floors, a library with 4 floor
	Pudong south road-Yaohua road	Several buildings with high floors, buildings with 6 floors
	Binzhou road	10 buildings with $4 \sim 6$ floors

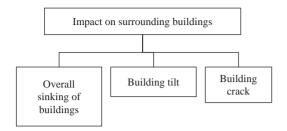


Figure 3. Risk identification of impact on surrounding buildings.

Table 7. Risk assessment of impact on surrounding buildings.

No.	Risk hazard	Probability	Loss	Ranks
1	Overall sinking of buildings	В	2	II
2	Building tilt	В	3	II
3	Building crack	С	3	III

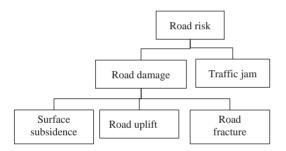


Figure 4. Risk identification of impact on surrounding roads.

#### 3.3 Risk assessment on environmental impact on surrounding roads

#### 3.3.1 Risk identification

The possible risk hazards of surrounding roads caused by construction can be illustrated in Figure 4.

#### 3.3.2 Risk analysis

The traffic situation along the works is extremely complex. The tunnel is the link between Puxi and Pudong district, along with Nanpu Bridge, Lupu Bridge and Da-pu road tunnel. Once the road is damaged, causing traffic jam, the consequences will be serious.

There are many roads along, but because of the limit of the pages, the figure of roads is omitted. The analysis is just the same as above.

#### 3.3.3 Risk evaluation

By sending out investigation forms, and using the TRM 1.0, the authors get the risk ranks of these risk hazards.

Table 8. Risk assessment of impact on surrounding roads.

No.	Risk haza	ard	Probability	Loss	Rank
1	Road damage	Surface subsidence	С	2	Π
	U	Road uplift	С	2	II
		Road fracture	С	3	III
2	Traffic ja	m	D	2	III

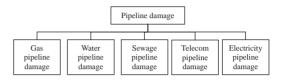


Figure 5. Risk identification of impact on surrounding pipelines.

Table 9. Risk assessment of impact on surrounding pipelines.

No. Ri	isk hazard	Probability	Loss	Rank
2 W 3 Se 4 Te 5 El	as pipeline damage (ater pipeline damage ewage pipeline damage elecom pipeline damage lectricity pipeline umage	B B C B C	3 4 2 3 3	II III II II

The results are shown in Table 8, from which we may see traffic jam and road fracture are more risky and are highly probable to happen.

#### 3.4 Risk assessment on environmental impact on surrounding pipelines

#### 3.4.1 Risk identification

The possible risk hazards of surrounding pipes caused by construction can be illustrated in Figure 5.

#### 3.4.2 Risk analysis

There are many pipelines along, but because of the limit of the pages, the figure of pipelines is omitted. The analysis is just the same as above.

#### 3.4.3 Risk evaluation

By sending out investigation forms, and using the TRM 1.0, the authors get the risk ranks of these risk hazards. The results are shown in Table 9, from which we may see water pipeline damage and electricity line damage are more risky.

Table 10. Risk assessment of environmental impact.

No.	Risk hazard	Rank
1	Impact on surrounding buildings	III
2	Impact on surrounding road	III
3	Impact on surrounding pipelines	III

#### 3.5 Summary

The whole risk rank for environmental impact is III. The result is shown in Table 10. It should be paid more attention.

Road tunnel construction on surrounding buildings impact was mainly due to surface subsidence during the process, causing buildings sinking, tilting or cracking. The main treatments are:

- 1 Grout timely, and make sure the shield is well sealed.
- 2 Control the earth pressure during the digging to prevent the face instability, a surface subsidence and uplift.
- 3 Protect the important buildings. If it is necessary, consolidate the foundation.
- 4 All kinds of anti-leakage measures are needed. Pay attention to foundation strengthening process, otherwise it will most prone to landslides.
- 5 Focus on the steps of construction during the foundation engineering. Control every step of the excavation depth and slope, including good support and the timely installation;

The measures about road and traffic involve with following instructions.

- Strictly control the quality of temporary road construction, to ensure road safety.
- 2 Control the load of vehicles travelling on the road to prevent overweight and crushing road.
- 3 Detailedly understand the traffic flow near the project before starting the construction. Disperse traffic flow to avoid jam.

There are many large diameter pipelines near the project, and once the pipelines are destroyed, it will not only have huge economic losses, but also resulted in a very bad social influence. Therefore we should take protective measures against dangerous pipelines.

- 1 Since there may be some errors occurring when planting the pipelines, which may induce incorrect position of pipelines. So before construction, positions of important pipelines must be verified.
- 2 Protect important pipelines by consolidating earth or isolation method.

- 3 Remove some load pushed on the pipelines.
- 4 Take informational construction. Monitor the pipelines frequently.

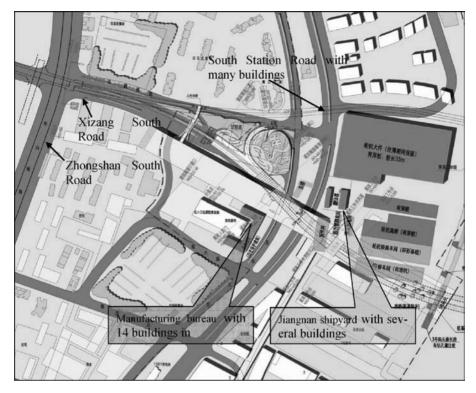
#### 4 CONCLUSION

From the study and research on this project, we may get some useful conclusions:

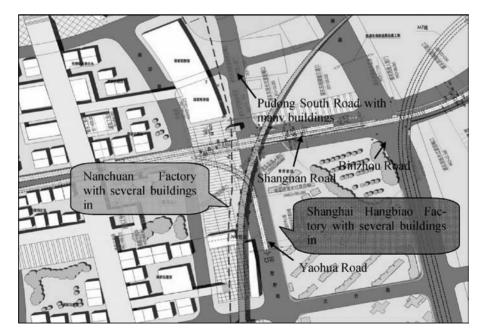
- 1 The whole risk rank for Xizang South Road Tunnel's environmental impact is III. It should be paid more attention. When the tunnel is constructed, the workers may strictly control ground settlement. Information construction is recommended.
- 2 There are plenty of methods to evaluate risk. It needs to discuss deeply which method is adopted. In this paper, Experts Investigation Method (EIM) and Confidence Index Method (CIM) are applied, and they are applicable.
- 3 The risk rank of environmental impact is obtained, which may give advice on project decision, bidding and insurance.
- 4 Risk management is a dynamic progress. With the development of the project, the risk rank may be changed. The authors should follow the project, and realize the dynamic risk management.

#### REFERENCES

- Chen, L. 2004. Risk Analysis and Assessment During Construction of Soft Soil Shield Tunnel in Urban Area. PhD Thesis of Tongji University.
- Guo, Z. W. 1986. *Risk analysis and decision*. Beijing: Machinery Industry Press.
- Huang, H. W. et al 2006a. Guidelines of Risk Management for Metro Tunnelling and Underground Engineering Works. Tongji University.
- Huang, H. W. et al. 2006b. Risk Management Software (TRM1.0) Based on Risk Databas for Shield Tunnelling. *Chinese Journal of Underground Space and Engineering*. Vol. 2: 36–41.
- International Tunnel Association. 2002. Working Group No. 2. Guidelines for tunnelling risk management. Balkema.
- Yao, C. P., Huang, H. W. & Hu, Q. F. 2006. The Study on Choosing Construction Methods of Underground Works Based on The Risk Analysis. *Modern Tunnelling Technol*ogy. Vol. 43: 581–585.
- Yao, C. P. & Huang, H. W. 2007. Risk analysis on the construction of connection tunnels in Shanghai Yangtze River Tunnel. Underground Construction and Risk Prevention – Proceedings of the 3rd International Symposium on Tunnelling-Shanghai 2007: 99–104. Tongji University Press.



Appendix 1. Surrounding buildings in Puxi district.



Appendix 2. Surrounding buildings in Pudong district.

### Risk analysis and fuzzy comprehensive assessment on construction of shield tunnel in Shanghai metro line

#### H.B. Zhou, H. Yao & W.J. Gao

Shanghai Jianke Project Management Co., Ltd., Shanghai, P.R. China

ABSTRACT: Based on risk management of metro line in Shanghai the risk analysis and fuzzy comprehensive assessment model was introduced to the very long metro line project risk assessment of shield tunnel construction. Firstly, Work Breakdown Structure method was brought forward to break down the shield tunnel construction works according to the features of the engineering geology and surrounding building and underground public pipes, the fault tree method was used to identify the shield tunnel construction's risk events and risk factors and the risk list can be established. Secondly, the fuzzy comprehensive assessment model was presented to assess the construction risk in view of the complexity of underground space and difficulty of quantifying the evaluation parameters, and the sub-project's risk level and the project' overall risk level can be calculated . Finally, a case about very long metro line project in Shanghai was studied, the assessment results is consistent with reality.

#### 1 INTRODUCTION

Eleven metro lines projects with the total length of more than 400 kilometers will be constructed in Shanghai before 2010, the construction of metro systems in Shanghai is in Large-scale construction now. Metro systems can effectively decrease traffic congestion, at the same time, due to the construction of metro mainly taken underground with many uncertainties as well as limited time, many serious accidents has happened such as the construction accident happened in Shanghai Rail Transit Line 4 in July 2003 and the collapse in Guangzhou Metro Line in 2004. These accidents not only brought enormous loss to the country, but also exerted a negative influence on the public. So, the risk assessment on the shield construction has vital significance to insure the project proceed smoothly (Mao 2004, Su 2004).

In this paper, a new risk analysis and assessment method during very long shield tunnel construction was put forward. Firstly the risk analysis method was introduced, which analyzed metro construction risk combining the WBS method with fault tree method because that the very long metro construction's risk was very complex, in the risk analysis process, the very long metro line project was classified according to the features of the engineering geology and surrounding building. Then the Work Breakdown Structure (WBS) method was brought forward to break down the shield tunnel construction' works. The fault tree method was used to identify the shield tunnel construction's risk events and risk factors, so the risk list can be established. Secondly, a new fuzzy comprehensive assessment model was presented to assess the construction risk in view of the complexity of underground space and difficulty of quantifying the evaluation parameters, which was proposed based on the fuzzy mathematical theory and risk matrix method proposed by International Tunneling Association in 2004. At last a case about one very long metro line construction risk assessment was introduced to demonstrate the risk analysis and risk assessment method. The assessment results were consistent with reality. All the risk analysis and assessment ideas and method given about very long metro line project are of great practical value for similar mega engineering construction risk assessment in the future.

#### 2 RISK ANALYSIS METHOD RESEARCH

Now, there were many risk analysis methods such as WBS-RBS method, fault tree method and so on. If the WBS-RBS method was used to analyze the risk factors of construction of very long metro line project, one very huge matrix would be established to identify the risk and the method could not distinguish relationship between the risk factors. The fault tree method also could not solve the problems. So a new risk analysis method that combined the WBS method with fault tree method was proposed. Firstly, the very long metro line project was classified according to the features of the engineering geology and surrounding building. Then the Work Breakdown Structure method was brought forward to break down the shield tunnel construction' works. The fault tree method was used to identify the shield tunnel construction's risk events and risk factors, so the risk list can be established.

#### 2.1 Work breakdown structure method

In order to analyze the risk, The work breakdown structure (WBS) method was brought forward to break down the metro construction works and to analysis the risk events and risk factors.

#### 2.1.1 The first grade structure breakdown

The metro line was divided into several sections to analyze risk according to the features of the engineering geology, surrounding building, underground public pipes and traffic conditions.

#### 2.1.2 The second grade structure breakdown

In order to analyze the risk of every section comprehensively during the construction, the activities of each section are divided further into portal-out, shield driving, segment works, grouting construction, shield machine, portal in and contact channel.

#### 2.2 Risk identification

The fault-tree method was used to identify the risk events and risk factors based on breaking down the works (Loosemore, Raftery, Reilly etal 2006), then the risk event list can be established.

#### 3 FUZZY ASSESSMENT METHOD RESEARCH

There have been limited attempts to exploit fuzzy logic within the construction risk management domain. Previous approaches to the use of fuzzy logic within construction risk management have proved to be either too simplistic for use in the real world, or have been very specific in their approach, targeting a particular area of construction on which to act or concentrating on specific types of risks.

In this paper, a new fuzzy comprehensive evaluation model used to the risk assessment was proposed, which attempted to solve previous problem. The fuzzy comprehensive evaluation model was established based on the fuzzy mathematical theory and risk matrix method proposed by International Tunneling Association in 2004. It can calculate the evaluating indicators' degree of membership and the weights, and the model can make sure the computed result is more objective.

#### 3.1 Calculating the indicators' weights

The indicators' weights were determined by the Analytic Hierarchy Process (AHP) method by comparing

Table 1. Estimation method of the risk effect consequence.

Degree	Estimate value	Explanation
Slight	1	The loss is not obvious
Medium	2	The loss is few (minimum in 100,000 RMB)
Serious	3	The loss is compensable (minimum in 1,000,000 RMB)
Significant	4	The loss is great but compensable (minimum in 10,000,000 RMB)
Disastrous	5	The loss is too enormous to compensate (above 10,000,000 RMB)

Table 2. Estimation of the risk probability.

Probability	Estimate value	Frequency
Rare	1	<0.0003
occasional	2	0.0003~0.003
Possible	3	0.003~0.03
anticipated	4	0.03~0.3
frequent	5	>0.3

the indicators' importance (Fang & Wang 1997, Shao 2004).

## 3.2 Calculating the risk event's degree of membership

Firstly, the expert scoring method was used to estimate each risk event's values of effect consequence C and possibility P, then the product of P multiply C was led to the risk rank's membership function and the risk level degree of membership to the every risk event are determined.

## 3.2.1 Determination of the risk event effect consequence C

The effect consequence C of risk event is determined according to the shield tunneling construction's actual situation (Mao 2004). The estimate method is shown on table 1.

3.2.2 *Determination of the risk event's possibility* The risk event's occurrence probability is ranked into five levels; estimate method is shown on table 2.

#### 3.2.3 Calculating the degree of membership

3.2.3.1 Determination of membership function In 2004, the International Tunnel Association put forward the risk matrix theory used to assessment risk, in this theory; the risk was classified four rank. In

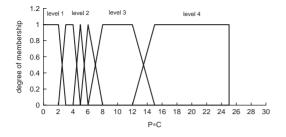


Figure 1. Membership function for risk.

Table 3.	The membership function.	
----------	--------------------------	--

Risk level	Membership function		
level 1	$r_{i1} = \begin{cases} 1 & 0 < x \le 2\\ 3 - x & 2 < x \le 3\\ 0 & x > 2 \end{cases}$		
level 2	$r_{i2} = \begin{cases} x-2 & 2 \le x < 3\\ 1 & 3 \le x \le 4\\ 5-x & 4 < x \le 5\\ x-5 & 5 < x \le 6\\ \frac{8-x}{2} & 6 < x \le 8\\ 0 & x > 2 \text{ or } x > 8 \end{cases}$		
level 3	$r_{i3} = \begin{cases} x - 4 & 4 \le x < 5 \\ \frac{6 - x}{2} & 5 \le x < 6 \\ \frac{x - 6}{2} & 6 \le x < 8 \\ 1 & 8 \le x \le 12 \\ \frac{15 - x}{3} & 12 < x \le 5 \\ 0 & x > 4 \text{ or } x > 15 \end{cases}$		
level 4	$r_{i4} = \begin{cases} \frac{x - 12}{3} & 12 \le x < 15 \\ 1 & 15 \le x \le 25 \\ 0 & x < 12 \end{cases}$		

this paper, the risk rank also was classified into four levels. The membership function was the L-R fuzzy function which used in project very widely, as shown on Equation 1, to be calculated easily, and according to the feature of shield tunnel construction risk assessment, the function was designed to trapezoidal function, as shown on figure 1 and table 3. (Carr & Tah 2001, Wang & Li 1996, Yao & zhou 2007).

$$r_{ij}(x) = \begin{cases} 0 & x \le a \gg \hat{\mathbf{o}} x \ge b \\ L(x) & a < x < m \\ R(x) & m \le x < b \end{cases}$$
(1)

## 3.2.3.2 Determination of membership degree value of risk events

Take the product of occurrence probability estimated value P and multiply consequence effect estimated

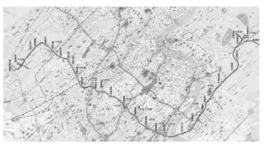


Figure 2. The chart of the metro route map.

value C into the risk rank membership function, then the corresponding membership degree of the risk events are determined.

#### 4 CASE STUDY OF RISK ANALYSIS AND ASSESSMENT

One length of the metro line is about 34.243 kilometers in all in Shanghai area, which passes through five district of Shanghai including Baoshan district, Putuo district, Jin'an district, Xuhui district and Pudong new district from north-west to south-east. The metro line's route map is shown on figure 2.

#### 4.1 Geological conditions

#### 4.1.1 Engineering geological conditions

The on-site soils within 65 m depth are the Quaternary sediments, which are mainly composed of saturated soft clay, silty clay and sand. The main soil profiles of the construction of shield tunnel are saturated soft clay and silty clay.

#### 4.1.2 The hydrogeological conditions

There are three kinds of groundwater in the soils, including the upper phreatic water mean embedded depth is  $0.5 \sim 0.7$  m under the ground surface, the middle feeble confined water mean embedded depth is 3-6 m under the ground surface in the 2-2 sub-layer of fifth layer soils and lower confined water mean embedded depth is 4-14 m in the seventh layer soil. The water elevation of groundwater was changed with the change of season, climate and tide.

#### 4.2 Risk analysis

The metro line was divided into 12 sections to analyze risk according to the features of the engineering geology, surrounding building, underground public pipes and traffic condition, as shown on table 4 then the shield tunneling construction's work structure was broke down, so the risk list was established

Category	surrounding environments	Mainly traversing soils of shiled
The 1st section	sub-center of the city and the complex of surrounding environments is common	soft clay and silty clay
the 2nd section	sub-center of the city and surrounding environment is relatively complex	soft clay and silty clay
the 3rd section	center of the city and traverse under the North Zhongshan road	soft clay and silty clay
the 4th section	center of the city and surrounding environment is complex	soft clay and silty clay
the 5th Section	center of the city and traverses through Suzhou River	soft clay and silty clay
the 6th Section	center of the city and traverses metro No. 1 line and metro No. 2 line closely	soft clay and silty clay
the 7th Section	center of the city and there are piles foundations intruding in the metro tunnel	soft clay and silty clay
the 8th Section	cross the Huangpu river and construction environment is very complex	soft clay and silty clay
the 9th Section	Traverse under or beside piles foundation and surrounding environment is complex	gray soft clay, clay and sandy silt
The 10th section	the complex of surrounding environment is common	gray soft clay, clay and sandy silt
The 11th section	traverses the river below and the complex of surrounding environments is common	gray soft clay, clay and sandy silt
The 12th section	traverses under metro No. 2 line	gray soft clay and clay

Table 4. The list of divided evaluating sections of metro line.

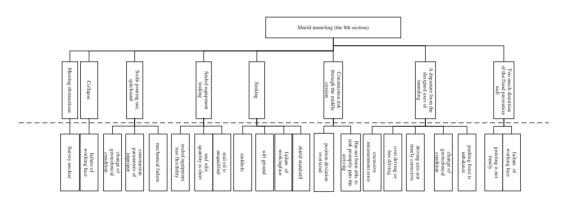


Figure 3. Shield tunneling construction's risk events and risk factors.

by using fault tree method to analyze shield tunneling construction risk. As an example, the result of risk identification about shield driving construction of the 8th kind of sector project (under Huangpu river section) was shown on figure 3.

#### 4.2.1 Weights calculation

The AHP method was used to determine the weight of each risk event. The estimation matrix was formatted based on the mutual comparison of important degree among the risk events.

Operating on the estimation matrix, the vector corresponding valued W of shield tunneling was as follows:

$$W = [0.0460.2070.2700.0260.0460.0780.2070.12]$$

#### 4.2.2 Membership function determining

Taking the product of possibility P of the risk event and consequences C into the membership function, each risk event's degree of membership was determined. The judging matrix R was also formatted and the value was as follows:

	-				-
	0	1	0	0	0
	0	0	1	0	0
	0	0	1	0	0
	0	1	0	0	0
( =	0	0	1	0	0
	0		1	0	0
	0	0	0	1	0
	0	0	1	0	0_

#### 4.2.3 Fuzzy comprehensive assessments

Taking weight vector W and judging matrix R into Fuzzy Comprehensive Assessment, the whole

1

Table 5. The point of each evaluated index.

	risk events	Р	С
Shield	meeting obstructions	2	3
tunneling	collapse	1	5
construction	soil's pouring out, quicksand	3	3
	sealed equipment leaking	3	2
	sinking of the shield	3	3
	a departure from the designed axes of tunneling	4	2
	construction risk through the middle caisson	4	4
	too much distortion of the flood prevention wall	2	4

Table 6. the shield tunneling construction risk events' risk level of the 8th section.

construction work of the 8th section	risk level	risk events	risk level
shield tunneling	level 3	meeting obstructions collapse soil's pouring out, quicksand sealed equipment leaking sinking construction risk through the middle cession a departure from the designed axes of tunneling too much distortion of the flood prevention wall	level 2 level 3 level 3 level 2 level 3 level 3 level 4 level 4

evaluation vector B about the tunneling construction was determined. The value was as follows:

 $B = W \bullet R$ 

		T	-				-					
	0.046	r i	0	1	0	0	0					
	0.207		0	0	1	0	0					
	0.270		0	0	1	0	0					
	0.026	*	0	1	0	0	0	_[_	0.072	0.721	0.207	0]
-	0.046	ਿੰ	0	0	1	0	0	= [0	0.072	0.721	0.207	٥J
	0.078		0	0	1	0	0					
	0.207		0	0	0	1	0					
	0.121		0	0	1	0	0					

Based on the maximal membership degree principle, the tunneling construction risk rank about shield tunneling of the 8th section is level 3, in which the highest risk is the construction through the middle caisson and the lowest risk is construction of meeting underground obstructions and sealed equipment

 Table 7.
 Risk level calculated of each section of the metro line.

category	risk level
the 1st section	Level 2
the 2nd section	Level 3
the 3rd section	Level 4
the 4th section	Level 4
the 5th section	Level 3
the 6th section	Level 3
the 7th section	Level 4
the 8th section	Level 4
the 9th section	Level 3
the 10th section	Level 2
the 11th section	Level 2
the 12th section	Level 4
total risk level	Level 3

leaking. In the meanwhile, risks of other construction activity during shield tunnel construction in 8th section were calculated; the result of risk assessment about the 8th section is as shown on table 6. In the end, risk level of the 8th construction section could be determined by the calculating upwards.

The risk level of other 11 construction section could be determined by the same method and the result is shown on table 7. From the table 7, it can be concluded the whole risk level of this ultra-length metro line project is 3. So certain measures must be taken to reduce the risk based on the risk acceptance principle.

#### 5 CONCLUSIONS

In accordance with the study on the risk analysis and assessment method of shield tunneling construction of very long metro line project and case study, conclusions were given as follows:

- 1 A new risk analysis method that combined the WBS method with fault tree method was proposed, in which classified the very long metro line project according to the features of the engineering geology and surrounding building, then broke down the shield tunnel construction' works, and used fault tree method to identify the shield tunnel construction's risk events and risk factors. The method not only can identify the risk events and risk factors more comprehensively, but also distinguish the relationship between each risk events or risk factors.
- 2 The fuzzy comprehensive risk assessment model could be used to solve the problem of non-accuracy, quantify the subjective concept, convenient mathematical processing and reduce the difference caused by subjective judgment.

3 The fuzzy comprehensive risk assessment model proposed in this paper could be operated upward from the bottom layers of risk structure up to obtain the overall evaluation results of the whole shield tunnel construction project. The result of risk assessment is very consistent with the practical construction conditions. Therefore, the method above mentioned is very practical and credible.

#### REFERENCES

- Carr, V. & Tah, J.H.M. 2001. A fuzzy approach to construction project risk assessment and analysis: construction project risk management system. Advances in Engineering Software (32): 847–857.
- Fang, S.C. & Wang, D.W. 1997. Fuzzy mathematics and fuzzy optimization. Beijing : Science Publishing Company.

- Loosemore, M., Raftery, J., Reilly, C. & Higgon, D. 2006. *Risk* Management in Projects. London :Tayor and Francis.
- Mao, R. 2004. Risk management on underground engineering. International Proseminar Memoir about Engineering Managemengt(11): 94–98.
- Shao, R.Q. 2004. A multi-level fuzzy synthetic evaluation on investment programs in shipping. *Transportation Engineering* 144(93): 497–502.
- Su,Y. 2004. Seismic risk management and insurance analysis of underground structures in soft soil. *Doctor Thesis of TongJi University*: 85–105.
- Wang, Z.P. & Li, H.X. 1996. Fuzzy system theory and fuzzy computer. Beijing : Science Publishing Company.
- Yao, H. & Zhou, H.B. 2007. Fuzzy synthetic evaluation on the constructing risk of EPBS during tunnelling in soft soil area. *Rock and Soil Mechanics*. Vol. 28(8):1753–1756.

Theme 5: Physical and numerical modelling

### Tunnel behaviour under seismic loads: Analysis by means of uncoupled and coupled approaches

#### D. Boldini

Department of Chemical, Mining and Environmental Engineering, University of Bologna, Bologna, Italy

#### A. Amorosi

Department of Civil and Environmental Engineering, Technical University of Bari, Bari, Italy

ABSTRACT: In this paper different approaches to investigate the behaviour of tunnel under seismic loads are presented. They include one-dimensional (1D) numerical analyses performed modelling the soil as a single phase non-linear visco-elastic medium, the results of which are then used to evaluate the input data for selected analytical solutions proposed in the literature (uncoupled approach), and 2D fully coupled Finite Element (FEM) simulations adopting a visco-elastic effective stress model for the soil (coupled approach).

#### 1 INTRODUCTION

The dynamic response of tunnels to seismic actions can be assessed by means of uncoupled or coupled approaches, depending on whether the evaluation of the seismic wave propagation and of the corresponding actions on the structure is undertaken in two separated steps or in one single analysis, respectively.

In this paper the transverse seismic behaviour of an idealised shallow tunnel in soft clay is analysed by means of uncoupled and coupled approaches. They include one-dimensional (1D) numerical analyses performed modelling the soil as a single phase non-linear visco-elastic medium and 2D fully coupled Finite Element (FEM) simulations adopting a visco-elastic effective stress model for the soil.

In the uncoupled approach 1D visco-elastic analyses, performed using the equivalent linear scheme implemented in the code EERA (Bardet et al. 2000), are aimed at establishing the role of stiffness and damping non-linearity on the free-field site response. The results of the analyses at the tunnel depth are then used to evaluate the input data for selected analytical solutions proposed in the literature to predict the transverse response of the structure both for fullslip and no-slip conditions (e.g., Wang 1993). This approach, widely used in the engineering practice, is based on the assumptions that the free-field site response is representative of the problem under study and that the seismic event at the tunnel depth can be satisfactorily represented by means of the maximum shear strain induced in the soil and the corresponding mobilised stiffness.

To overcome some of the limitations of the approach described above a fully coupled Finite Element analysis can be adopted, simulating in the time domain the soil-structure dynamic interaction during the seismic event. This latter is in this case realistically described by an accelerogram and the soil by an appropriate effective stress formulation.

The constitutive assumption for the soil is a key element of this class of analyses. A linear visco-elastic model is often adopted at the scope, for its apparent simplicity and limited number of parameters. In fact, a number of commercial codes nowadays allow the user to perform coupled dynamic analyses based on linear elasticity and viscous damping, this latter accounted for by the Rayleigh formulation. In this case a limitation of the coupled approach is in the non obvious selection of adequate elastic and viscous soil parameters, which sensibly influence the results of the analysis.

In this work a strategy to calibrate the parameters for the visco-elastic model adopted in the FEM analyses is proposed based on the free field soil response results obtained in the context of the uncoupled approach.

A critical comparison between coupled and uncoupled approaches is outlined in the paper with reference to a shallow tunnel excavated by a TBM. A realistic geotechnical characterisation is assumed for the idealised soft clay deposit under study.

#### 2 CASE STUDY

In the present study the acceleration time history recorded at Kalamata (Greece) during the 13.XI.1986

earthquake is considered. The original seismic signal is characterised by a duration of 29.74 s and a maximum acceleration of 0.24 g.

The input signal adopted in this work was scaled at 0.35 g and was filtered to prevent frequency levels higher than 7 Hz. A picture of the selected acceleration time history after manipulation is given in Figure 1 while the resulting Fourier spectrum is shown in Figure 2.

In all the analyses the seismic signal is assumed to be applied at the outcrop of the deposit. The corresponding bedrock motion was calculated by means of a deconvolution analysis performed by the code EERA, as described in the next paragraph.

A 60-m thick ideal deposit of soft clay is assumed as the reference soil profile, characterised by the following physical and mechanical parameters: plasticity index IP = 44%, unit weight of volume  $\gamma = 17 \text{ kN/m}^3$ , overconsolidation ratio in terms of mean effective stress R = 1.5, small strain shear stiffness  $G_O =$ variable with depth, Poisson's ratio  $\nu' = 0.25$ , damping ratio D = variable with depth, coefficient at rest

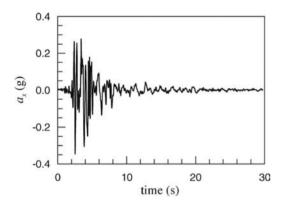


Figure 1. Modified acceleration time history scaled at 0.35 g.

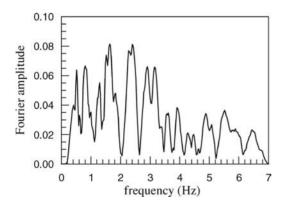


Figure 2. Frequency-filtered Fourier spectrum.

 $K_0 = 0.6$ . The water table is assumed at the ground surface.

A circular tunnel, located at a depth of 15 m and characterised by a 10.10 m diameter is selected as the reference underground structure for the present study. The lining is assumed to be composed by 0.50 m thick precast concrete segments characterised by the following parameters: Young's modulus  $E_l = 38$  GPa, Poisson's ratio  $v_l = 0.25$ , damping ratio  $D_l = 5\%$ .

#### 3 UNCOUPLED APPROACH

The 1D ground response analysis was performed with the code EERA (Bardet et al. 2000) that analyses the vertical propagation of shear waves in a onedimensional layered system based on an equivalent-linear visco-elastic scheme. It assumes that the shear modulus *G* and damping ratio *D* are function of shear strain amplitude  $\gamma$ .

The adopted profile of the small-strain shear stiffness  $G_O$  with depth (Figure 3) was evaluated by the relationship proposed by Viggiani (1992) as a function of the in situ mean stress, R and IP.

Figure 4 shows the curves of the variation of the normalised shear stiffness and damping ratio with shear strain  $\gamma$ , defined according to Vucetic & Dobry (1991).

A total number of 31 layers was adopted to discretise the soil stratum.

Figure 5 shows the results of the analysis in terms of maximum shear strain  $\gamma_{max}$ , normalised shear stiffness  $G/G_O$ , damping ratio D and maximum acceleration  $a_{max}$  with depth.

Values of  $\gamma_{max}$  and G obtained at the depth of 15 m, i.e. at the tunnel depth, were subsequently used

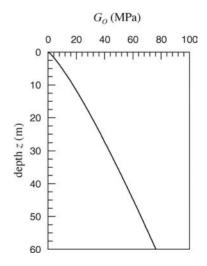


Figure 3. Profile of the small-strain shear stiffness  $G_O$ .

to evaluate the increment of hoop forces and bending moments acting on the tunnel lining during the earthquake, according to selected analytical solutions proposed in the literature.

Here, the solutions proposed by Wang (1993) to predict the response of the structure are taken into consideration for both full-slip and no-slip conditions.

The maximum increment of hoop force and bending moment in the transverse direction of the tunnel are given by:

$$\Delta N_{max} = \pm \frac{1}{6} K_1 \frac{E_u}{\left(1 + \nu_u\right)} r \gamma_{max} \tag{1}$$

$$\Delta M_{max} = \pm \frac{1}{6} K_1 \frac{E_u}{(1+\nu_u)} r^2 \gamma_{max}$$
(2)

for full-slip conditions and by:

$$\Delta N_{max} = \pm K_2 \frac{E_u}{2(1+V_u)} r \gamma_{max}$$
<sup>(3)</sup>

for no-slip conditions, the bending moment being the same for the two cases.  $E_u$  and  $v_u$  indicate the

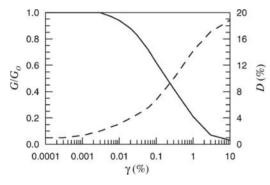


Figure 4. Modulus reduction curve  $G/G_O$  and variation of damping ratio *D* with shear strain  $\gamma$ .

mobilised soil Young's modulus (evaluated with reference to the previously calculated mobilised shear modulus *G*) and the Poisson's ratio (assumed equal to 0.5) in undrained conditions, respectively. *r* is the tunnel radius.  $K_1$  and  $K_2$  are given by the following expressions:

$$K_{1} = \frac{12(1-\nu_{u})}{2F+5-6\nu_{u}}$$
(4)

$$K_{2} = 1 + \frac{F\left[\left(1 - 2\nu_{u}\right) - \left(1 - 2\nu_{u}\right)C\right] - \frac{1}{2}\left(1 - \nu_{u}\right)^{2} + 2}{F\left[\left(3 - 2\nu_{u}\right) + \left(1 - 2\nu_{u}\right)C\right] + C\left[\frac{5}{2} - 8\nu_{u} + 6\nu_{u}^{2}\right] + 6 - 8\nu_{u}}$$
(5)

where:

$$C = \frac{E_u (1 - v_l^2) r}{E_l t (1 + v_u) (1 - 2v_u)}$$
(6)

$$F = \frac{E_u \left(1 - v_i^2\right) r^3}{6E_l I \left(1 + v_u\right)}$$
(7)

are the compressibility and flexibility ratios and *t* and *I* the thickness and the moment of inertia of the tunnel lining, respectively.

Table 1 summarises the computed increments of hoop force and bending moment for both full-slip and no-slip conditions.

#### 4 COUPLED APPROACH

The coupled analyses were performed by the Finite Element commercial code PLAXIS, which describes

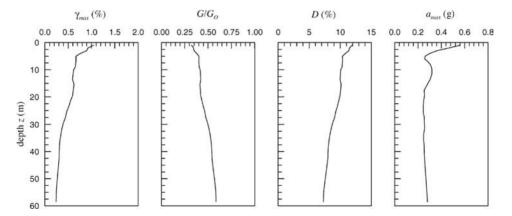


Figure 5. Results of the 1D ground response analysis performed with EERA.

Table 1. Increments of hoop force and bending moment according to Wang (1993).

Full-slip co	nditions	No-slip conditions			
$\Delta N_{max}$ (kN/m)	$\Delta M_{max}$ (kNm/m)	$\Delta N_{max}$ (kN/m)	$\Delta M_{max}$ (kNm/m)		
±159	±802	±473	±802		

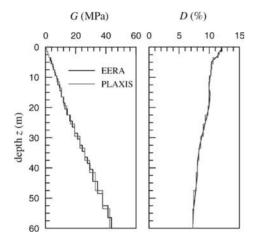


Figure 6. Calibration of the G and D profiles assumed in the FEM analyses on the basis of EERA results.

the mechanical behaviour of the soil in terms of effective stress.

In order to perform a comparative analysis with the EERA results a linear visco-elastic constitutive model was selected. In the constitutive model viscous damping was accounted for by the Rayleigh formulation.

#### 4.1 Calibration of the visco-elastic parameters

The constitutive model employed in the FEM analyses makes use of constant-value elastic and viscous parameters for each sub-layer of the discretised deposit. In this respect, it is of paramount importance to define appropriate values of these parameters as a function of the strain level attained in the soil deposit during the earthquake.

In this paper a recently developed calibration procedure of the visco-elastic parameters to be assumed in dynamic FEM analyses is adopted (Amorosi et al. 2007). *G* and *D* profiles are set in such a way to match the corresponding profiles obtained by the free-field EERA analysis. To this end the numerical model is divided into a relatively large number of sub-layers in order to obtain an as close as possible correspondence (Figure 6).

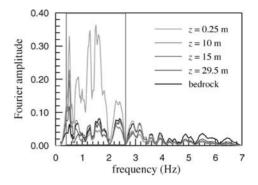


Figure 7. Fourier spectra computer by EERA at different depths and high-energy frequency interval.

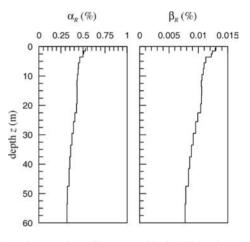


Figure 8.  $\alpha_R \in \beta_R$  profiles assumed in the FEM analyses on the basis of EERA results.

The profiles of the Rayleigh coefficients  $\alpha_R e \beta_R$  are obtained correspondingly, according to the following relationship with the damping ratio *D* (e.g., Clough & Penzien 2003):

$$\begin{cases} \alpha_{R} \\ \beta_{R} \end{cases} = \frac{2D}{\omega_{n} + \omega_{m}} \begin{cases} \omega_{n} \omega_{m} \\ 1 \end{cases}$$
(8)

where  $\omega_n e \omega_m$  are the circular frequencies related to the cyclic frequency interval  $f_n \div f_m$ . The frequency range was determined by selecting the interval where the Fourier spectra computed by EERA at different depths were characterised by the highest energy content (indicated in the box of Figure 7). The resulting  $\alpha_R e \beta_R$  profiles are shown in Figure 8.

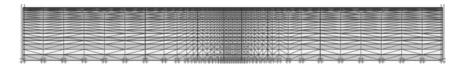


Figure 9. Mesh employed in the FEM analyses.

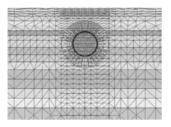


Figure 10. Detail of the mesh around the tunnel section.

#### 4.2 Numerical model

The dimension and boundary conditions of the 2D FEM numerical model were set up after a considerable number of preliminary analyses performed to minimise the influence of boundary conditions on the computed results.

The mesh employed in the present study is reported in Figure 9: it is characterised by a width equal to 8 times its height. The base is assumed to be rigid and at the lateral sides the viscous boundaries proposed by Lysmer & Kuhlmeyer (1969) were used.

The domain was discretised in 2431 15-node plane strain triangular elements. In the central part of the mesh, where the tunnel is located, the element dimension h always satisfies the condition:

$$h \le h_{\max} = V_s /(6 \div 7) f_{\max} \tag{9}$$

where  $V_S$  is the shear wave velocity and  $f_{max}$  is the maximum frequency of the seismic signal.

A detail of the mesh around the tunnel is shown in Figure 10.

The following stages were simulated in the numerical analysis:

- first, the tunnel was excavated in undrained conditions imposing a volume loss of 0.4%;
- next, the lining was installed and the post-excavation consolidation phase was studied;
- the select seismic signal was then applied at the bottom of the model in undrained conditions.

In all the static stages of the analysis the soil stiffness for each sub-layer was selected scaling down the value of the very small strain shear stiffness  $G_O$  by a factor of 0.45 to account for an average shear strain level involved in the excavation stages of  $\overline{\gamma} = 0.1\%$ .

An interface was activated between the soil and the lining, characterised by a normal and tangent stiffness

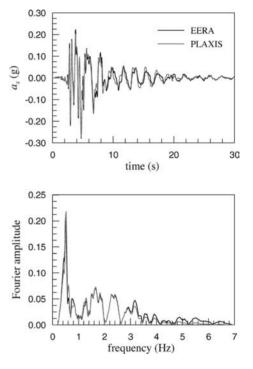


Figure 11. Comparison between EERA and PLAXIS free-field soil response analyses at 15 m depth.

corresponding to that of the adjacent soil: such a condition can be considered similar to the so-called no-slip conditions of the Wang's solutions.

#### 4.3 Free-field soil response results

In this paragraph a preliminary comparison between free-field soil response results at the tunnel depth is provided to check the consistency between the 1D and 2D proposed approaches. In this case the 2D model does not incorporate the tunnel and, as such, the result can be directly compared to that of the corresponding 1D free-field analysis.

Figure 11 shows the acceleration time histories and the Fourier spectra computed by EERA and PLAXIS. A satisfactory agreement is obtained between the two solutions, demonstrating the effectiveness of the proposed calibration strategy.

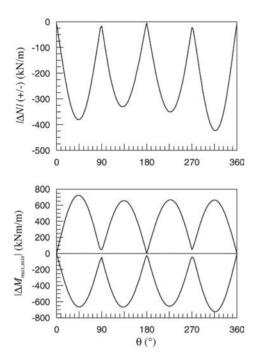


Figure 12. Increments of hoop force and bending moment during the seismic event.

#### 4.4 Results of the coupled analyses

The maximum increment of the seismic-induced absolute hoop forces and positive and negative bending moments in the tunnel lining were computed by the 2D FEM coupled analyses and are represented in Figure 12. The results are reported as a function of the angle  $\theta$  indicated in Figure 12 and defined positive in counter-wise direction.

Results indicate a satisfactory match between the maximum increments of hoop force and bending moments predicted by the visco-elastic FEM solution ( $\Delta N_{max} = \pm 423 \text{ kN/m e } \Delta M_{max} = \pm 724 \text{ kNm/m}$ ) and those resulting from the Wang's solutions for the no-slip case.

#### 5 CONCLUSIONS

In this paper the transverse dynamic response of a shallow tunnel subjected to seismic actions is investigated by means of uncoupled and coupled approaches.

The former approach, traditionally used in the engineering practice, combines a free-field site response analysis with a set of analytical solutions to evaluate the maximum increment of hoop force and bending moment in the tunnel lining. This approach is based on a limited numbers of parameters to describe the earthquake, the soil behaviour and thawt of the tunnel lining. The reliability of the uncoupled approach is supported by the satisfactory performance of a number of underground structures designed based on it.

Nowadays the availability of relatively sophisticated commercial FEM codes and the continuous increase in computational power allows to approach the same problem by means of a single analysis which, at the same time, accounts for the seismic site response, the soil-structure interaction and the actions in the tunnel lining. This coupled approach is characterised by a far more detailed description of the seismic actions at the tunnel depth as compared to the uncoupled one and allows to describe the soil behaviour by means of effective stress based constitutive models. Concerning this last feature in this paper a simple assumption was made: linear visco-elasticity was assumed for the clayey deposit under study, where viscous damping was accounted for by the Rayleigh formulation. Although very simple this constitutive assumption already poses some problems in the calibration of the stiffness and Rayleigh damping parameters. In fact, it is a well established fact that the profiles of these parameters with depth crucially influence the results of any FEM dynamic analysis (e.g.: Woodward & Griffiths 1996). Related to this, a strategy for the calibration of the stiffness and Rayleigh damping parameters is proposed in the paper. The results of the coupled analysis based on this calibration satisfactory match those obtained by the uncoupled approach in terms of hoop force and bending moments in the lining.

In the Authors' opinion this encouraging result represents a necessary condition to extend the use of the proposed calibration strategy to the case of more complex constitutive assumptions including plasticity.

#### REFERENCES

- Amorosi, A., Boldini, D., Elia, G., Lollino P. & Sasso M. 2007. Sull'analisi della risposta sismica locale mediante codici di calcolo numerici. *IARG 2007*, Salerno.
- Bardet, J.P., Ichii, K. & Lin C.H. 2000. EERA-A computer program for Equivalent-linear Earthquake site Response Analyses of layered soils deposits. User Manual.
- Clough, R. & Penzien, J. 2003. Dynamics of Structures. Computers and Structures Inc.
- Lysmer, J.& Kuhlemeyer, R.L. 1969. Finite dynamic model for infinite media. *ASCE EM* 90: 859–877.
- PLAXIS. 2003. Reference Manual, version 8.
- Viggiani, G.M.B. 1992. Small strain stiffness of fine grained soils. PhD thesis. City University. London.
- Vucetic, M. & Dobry, R. 1991. Effects of the soil plasticity on cyclic response. *Journal of Geotech. Eng. Div., ASCE* 117(1): 89–107.
- Wang, J.N. 1993. Seismic design of tunnels: a state-of-the-art approach. Monograph 7, Parsons, Brinckerhoff, Quade & Diuglas Inc., New York.
- Woodward, P.K. & Griffiths, D.V. 1996. Influence of viscous damping in the dynamic analysis of an earth dam using simple constitutive models. *Computers and Geotechnics* 19(3): 245–263.

# Investigating the influence of tunnel volume loss on piles using photoelastic techniques

#### W. Broere

Delft University of Technology, Delft, The Netherlands A. Broere BV, Amsterdam, The Netherlands

#### J. Dijkstra

Delft University of Technology, Delft, The Netherlands

ABSTRACT: This paper presents the results of plane strain model tests on tunnel-pile interaction in a photoelastic material. The effects of volume loss are simulated by contracting a tunnel. The soil in this test is represented by crushed glass. This allows for the determination of stresses in the model by the photoelastic method. The influence of the stress change in the soil due to volume loss is shown, as well as the effect on three rows of piles at varying distance from the tunnel. From the tests it is clear that significant stress changes occur close to the pile tips.

#### 1 INTRODUCTION

The upcoming boring of the North-South metro line in Amsterdam (Netzel & Kaalberg 2000, Kaalberg et al. 2005) will involve TBM excavation close to the pile tips of the wooden piles on which much of the historic inner city is founded. The influence of a volume loss due to tunnelling on the bearing capacity of these piles is relatively uncertain and hard to quantify even by numerical means (Broere & van Tol 2006). This prompted a different approach, to visualise the influence of volume loss on the stresses around the pile tips using the photo-elastic method. Photoelasticity has been used extensively to quantify stresses in homogeneous materials (e.g. Frocht 1948). And is extended to particle assemblies in index matching liquids by Drescher (1976).

A small research effort was started in 2002 to attempt to quantify the influence of a volume loss on the stresses close to pile tips in a photoelastic scale model. As the interpretation of the measurements proved more complicated than originally anticipated, it took considerably more effort to finalise the analysis and derive the complete stress state. An overview of these complexities is given in Petrucci & Restivo (2007). Standing & Leung (2005) had similar considerations as outlined above when they did comparable photoelastic scale model tests, but they decided to invert the problem and only qualify the influence of pile installation on the stress in existing tunnel linings.

#### 2 STRESS MEASUREMENTS

It is possible to obtain detailed information on the stress distribution of a granular material using the photoelastic measuring method. The soil is replaced by grains of a photoelastic material, in this case glass particles (Wakabayashi 1957, Drescher 1976). Crushed glass behaves similar to sand particles, although, the grains are more angular. Therefore it is a reasonable substitute to investigate sand behaviour. However, other authors, most noticably Lesniewska & Sklodowski (2005) argue that when uniform glass beads are used instead of crushed glass the stress trajectories a long failure planes can be better visualized.

When compared to the photoelastic measurement methods for continuous materials the quantification of the stress paths for a granular material is hampered. First, in order to eliminate light scatter the pores have to be filled with a liquid which has a similar refraction index as the glass particles. Secondly the analysis of the fringe patterns is impossible because of the stacked nature of the sample. Each layer of grains produces its own fringe pattern resulting in a mix of fringes which cannot be interpreted. The first condition leads to the qualitative visualization of these stress paths by a circular polariscope, see i.e. Wakabayashi (1957). For the second problem, quantification of the stress paths, Allersma (1982) developed an automated polariscope.

In general, granular material does not behave elastically, as is often assumed when formulating a

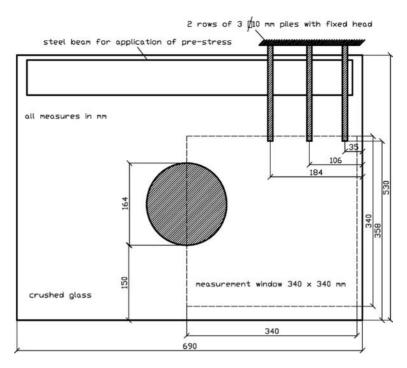


Figure 1. Front view of model setup, all measures in mm.

stressstrain relationship. Therefore it is not possible to derive the stress increments from strain measurements. The stresses have to be measured independently of the strains.

The method developed by Allersma (1987) and Allersma & Broere (2002) for measurement of the stress in a photoelastic material is followed. The method already published by Allersma in 1982, is most comparable to phase-stepping photoelasticity as described in e.g. Ajovalasit et al. (1998). The direction of the principal stress and the principal stress difference along the light path are measured with an automated circular polariscope. Therefore averaging of the stress in the direction normal to the plane is taken into account.

In a photoelastic material the refraction in each direction is dependent on the normal stress in that direction. The stress difference between the principal stress directions, i.e. twice the maximum shear stress,  $2 * \tau_{max}$ , leads to a relative light velocity difference and subsequently a change in polarisation. This change in retardation can be measured to obtain the shear stress in a point.

The maximum shear stress in a point can be derived from the elliptical polarisation of the light. The light changes from circular polarised light into elliptical polarised light when travelling through the sample, caused by the stress in the material. If at one or more points the complete stress state is known, the absolute values of the stress tensor can be determined by integrating the equilibrium equations. Unfortunate, this scheme is very sensitive for experimental noise, which hampers the results. In the present tests the vertical stress is calculated from the measured surcharge at the top of the sample.

#### 3 TEST SETUP AND TEST PROCEDURE

#### 3.1 Test setup

The test setup consists of a model container with inner dimensions of height × width of 690 mm × 530 mm and a depth of 70 mm. In Figure 1 a front view of the test setup is given. The front and rear sides are made of glass. The rectangular piles are made of stainless steel with a cross sectional area of  $10 \times 10$  mm. Two rows, with a c.t.c. distance of about 25 mm, of three piles are installed at respectively  $0.5D_{tunnel}$ ,  $1D_{tunnel}$  and  $1.5D_{tunnel}$ , simulating a piled foundation. Also a tunnel is embedded at 345 mm from the side and 232 mm from the bottom of the container.

The tunnel consists of an upper and a lower part milled of stainless steel which can be pushed out vertically by an inner stepper motor. Therefore, the volume change of the tunnel is primarely caused by vertical contraction. The mechanical realisation of this tunnel



Figure 2. Mechanical realisation of the tunnel.

can be seen in Figure 2. An additional surcharge is applied on a steel beam to increase the stress level in the sample. This surcharge is applied by spring loading the beam, i.e. between a fixed boundary and the beam a mechancial spring is applied, such that small displacements in the sample do not totally unload the boundary. Between the spring and the fixed boundary a load cell is monitoring the applied load.

The box is filled with crushed glass particles and the index matching liquid. The particle size of the glass particles is between 2 and 3 mm. The material behaves similar to sand of the same particle size as shown in triaxial tests performed by Allersma. According to Allersma, the angle of internal friction at constant volume,  $\phi_{cv}$ , for angular silica sand of this particle size is ca. 33° and for crushed glass ca. 39°.

#### 3.2 Test procedure

Medium dense conditions are obtained by first pouring the glass into the strongbox, followed by densification of the sample, and finally pumping the liquid through the sample from bottom to top. The tunnel is undisturbedly embedded during preparation, while keeping the diameter at its maximum position. The piles are pushed in afterwards before applying the surcharge. A surcharge of 75 kPa is applied at the steel beam after preparation of the sample is finished. Before starting the test first an equilibrium stage has to be reached in which no additional creep is apparent. During this phase the surcharge is kept constant at 75 kPa.

The objective of this test is to investigate the effect of volume loss on existing pile foundations, therefore at the beginning of the test the tunnel is in its opened position. Also the piles are already embedded. Subsequently a photoelastic measurement is made by means of a mechanical polariscope. In the next phase,

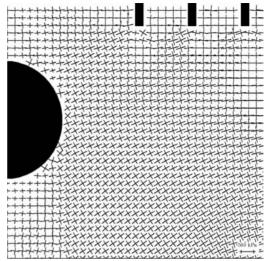


Figure 3. Principal stress direction and magnitude before tunnel contraction.

without altering the surcharge, the tunnel is contracted by decreasing the distance between the upper and lower tunnel parts by 2 mm or 1.2%. This results in a volume loss of only 0.6%, as the tunnel contracts vertically only and not uniformly. This will result in an unloading of the soil around the tunnel. Again a photoelastic measurement is made.

The mechanical polariscope is mounted on a computer controlled x-y scanner and scans approximately 1200 stress points spaced at 10 mm interval in an area of ca.  $340 \text{ mm} \times 340 \text{ mm}$  covering half of the tunnel and the lower part of the embedded piles. The measurement zone is also shown in Fig. 1 (dotted lines).

#### 4 TEST RESULTS

The results of the performed tests are summarized in three figures. The first Figure, Fig. 3, is showing the principal stress directions and the magnitude of the principal stresses measured before tunnel contraction, the second Figure, Fig. 4, is showing the same data, but now for the situation after tunnel contraction. The principal stress crosses are all plotted with the same scale, i.e. the length determines the magnitude. Finally, the last figure, Fig. 5, plots the contour plot of the difference in the measured maximum shear stress. The "before" measurement is taken as reference, resulting in a positive value for the difference when the maximum shear stress in a point before contraction was higher than after contraction and logically a negative value when the value afterwards is higher than before. The measured surcharge

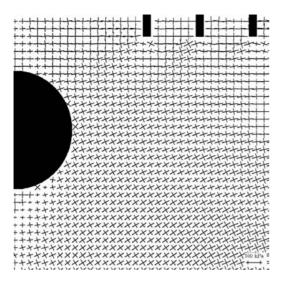


Figure 4. Principal stress direction and magnitude after tunnel contraction.

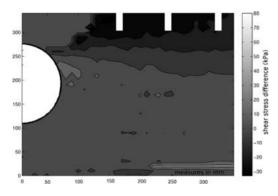


Figure 5. Maximum shear stress difference (kPa) "before-after".

after the first measurement is 74.8 kPa, while after the second measurement the reading for the surcharge is 74.3 kPa. Therefore, the changes found between the two measurements are mainly caused by the tunnel contraction and not by additional creep in the sample. As already elaborated, the angle of the principal stress and the maximum shear stress are direct measurements, while the principal stress magnitude is processed from the stress integration algorithm. All geometries, like piles and tunnels shown in the figures are plotted to scale at the proper position.

#### 4.1 Before tunnel contraction

When studying the results of the measurements taken before tunnel extraction, given in Figure 3 several things are noticed, for sense of scale the measurement points are horizontally and vertically spaced at 10 mm. Firstly the principal stress rotations right on top and below the tunnel are all negligible, the tunnel is loading the soil in compression, while to the side large stress rotations occur. The behaviour is therefore ver-tical symmetrical.

Another observation is that the spacing between the piles is such that arching seem to occur, spots without any rotation and with a lower stress magnitude can be observed. Clearly, a spacing of about seven times the pile diameter is still sufficient to cause this effect, however scale effects cannot be excluded as the grains are relatively large compared,  $d_{grains} = 2-3$  mm, to the pile diameter which is 10 mm. The most right pair of piles (remember two rows of piles are intstalled behind eachother when seen from the front side), or the pile row next to the boundary of the model container is distributing its load immediately in the side wall. As directly below a large undisturbed zone is seen. This is clearly a boundary effect. Lastly the pile closest to the tunnel is influenced by the tunnel, as between the pile and the tunnel a direct path of parallel stress directions can be seen. This pile row is at a tip-tunnel distance of about 12 cm or  $12D_{pile}$  or  $0.75D_{tunnel}$ .

#### 4.2 After tunnel contraction

Again the principal stress rotation and magnitude are plotted in Figure 4, this time for the situation after tunnel contraction. All former observations still apply, pile arching seems to still occuring, the closest pile row is still influenced by the tunnel and is unexpectedly in a higher stress regime. However, closer inspection reveals that the band of stress between the closest pile row and the tunnel seems smaller in width i.e. the stress intensity is lower. Also, the stresses directly below the tunnel seem changed also, the undisturbed zone below is smaller. From an experimental point of view this can be expected, because as the tunnel is decreased in height, the largest changes are expected in the vertical stress distribution. Not only unloading on the top side of the tunnel occurs but also an elastic rebound on the bottom.

#### 4.3 Shear stress difference

It is rather difficult so observe differences in magnitude in principal stress from the already shown figures, therefore one additional figure is presented. In Figure 5 the difference in measured maximum shear stress between the situation before and after tunnel contraction is plotted as the difference is more clear from this data than from the processed data. The numbers plotted next to the x-axis and y-axis are the distance from the origin of the measurement in mm. The values given in the colourbar of the contourplot are in kPa, positive values indicate a lower stress than before tunnel contraction, negative values point towards a higher stress than before contraction. It is immediately clear from this data that an increase in stress around the pile tips of the three pile rows is found. Another observation is that the first two pile rows are most effected by the contraction. The zones with the highest stress concentrations, in this case around the pile tips and at the point at which the stress arch reaches the tunnel are taking all the stress change, while the general field does hardly change at all. The last finding further supports the observation that creep in the sample during the tests is kept at a minimum. The increase in maximum shear stress can be explained by the fact that in the model setup due to volume loss the soil immediately begins to create movements of soil below the pile rows resulting in negative friction on the piles.

#### 5 DISCUSSION

It is our opinion that for the case considered, a scaled plane strain situation with rather large grain size and low stress situation, the results are still convincing. The effects of nearby tunnelling are seen in the measured stress field. Increase of maximum shear stress can be explained by negative friction on the piles. However, the change of the horizontal or vertical stress is more difficult to acquire, both due to difficulties from noise in experimental data, as measuring properties of light is always very sensitive to experimental errors, and due to the integration of the measured stress field. Therefore, no conclusive answer can be given at this stage.

All pile rows in the presented setup suffer from the tunnel contraction effects, but the first row is clearly most influenced, not only by the tunneling but also in the reference situation interaction is seen between the two structural elements. The influence of the tunneling on the last pile row is minor, but this pile row is already located at  $1.5D_{tunnel}$  to the center line of the tunnel.

When compared to the results of Standing & Leung (2005) a bit more information is derived from the measurements. In the current setup the model scale, especially when considering the dimensions of the pile, the model container and tunnel, are more realistic. Similar to Standing & Leung (2005) in our measurement nearby piles attract more stress lines, which reduce in intensity after the tunnel contraction. This indicates a stress release near the pile, which is consistent with the increase in shear along the pile as the pile starts to settle.

#### 6 CONCLUSIONS

The influence of a contracting tunnel on the stresses near the tips of displacement piles has been studied using photoelastic model tests. From these tests, the changes in principal stresses and shear stresses near the tunnel and piles have been quantified. Given the limited size of the scale model, and the relatively large grains used to improve the visibility of the photoelastic effect, some limitations of the model have come to light and these have been discussed. Especially as there is some evidence of stress arches between the different piles, the results of these tests cannot be translated directly to a field situation but should be interpreted with care.

Nonetheless, the tests show a clear influence of a volume loss of the tunnel of 0.6% on the stresses near the pile tips up to  $1D_{tunnel}$ . Here stress bands are decreasing in width, indicating a lowering of the principal stresses, and at the same time an increase of the shear stresses along the pile shafts is seen for these piles. These observation corroborate those of Standing & Leung (2005), but also indicate that for displacement piles close to failure the influence zone of a tunnel volume loss is wider than suggested there. Contrary to their findings not only piles with their toes inside the influence zone defined by Jacobsz et al. (2003) are affected, but also those just outside of that zone.

This suggests that the influence zone to be taken into account for displacement piles, which depend on both end bearing and skin friction, might be slightly larger than for bored piles, which are mostly founded on end bearing alone. To determine this would require a combination of further field observations, model testing and numerical modelling of the problem.

#### REFERENCES

- Ajovalasit, A., Barone, S., & Petrucci, G. 1998. A review of automated methods for the collection and analysis of photoelastic data. *Journal of Strain Analysis* 33(2), 75–91.
- Allersma, H. 1982. Determination of the stress distribution in assemblies of photoelastic particles. *Experimental Mechanics* 22(9), 336–341.
- Allersma, H. 1987. Optical analysis of stress and strain in photoelastic particle assemblies. Ph. D. thesis, Delft University of Technology.
- Allersma, H. & Broere, W. 2002. Optical analysis of stress around a penetrating probe in granular material. In *Physical Modelling in Geotechnics: Proceedings of the international Conference on Physical Modelling in Geotechnics*, pp. 149–154.
- Broere, W. & van Tol, A. 2006. Modelling the bearing capacity of displacement piles in sand. Proceedings of the Institution of Civil Engineers, Geotechnical Engineering 159(3), 195–206.
- Drescher, A. 1976. An experimental investigation of flow rules for granular materials using optically sensitive glass particles. *Géotechnique* 26(4), 591–601.
- Frocht, M. 1941 and 1948. *Photoelasticity volumes 1 and 2*. New York: John Wiley & Sons.
- Jacobsz, S., Standing, J., Mair, R., Soga, K., Hagiwara, T., & Sugiyama, T. 2003. Tunnelling effects on driven piles. proc. int. conf. on response of buildings to excavation-induced ground movements. In F. Jardine

(Ed.), CIRIA Special Publn 201, pp. 337–348. CIRIA, London.

- Kaalberg, F., Teunissen, E., Van Tol, A., & Bosch, J. 2005. Dutch research on the impact of shield tunnelling on pile foundations. In *Proceedings of the 5th international Symposium on Geotechnical Aspects of Underground Construction in Soft Ground*, pp. 123–131.
- Lesniewska, D. & Skłodowski, M. 2005. Photoelastic investigation of localization phenomena in granular materials. In *Powders and Grains*, pp. 69–72.
- Netzel, H. & Kaalberg, F. 2000. Numerical damage risk assessment studies on masonry structures due to TBMtunneling in Amsterdam. In O. Kusakabe, K. Fujita, and Y. Miyazaki (Eds.), *Geotechnical Aspects of Under-*

ground Construction in Soft Ground. Balkema. preprints pp. 235–244.

- Petrucci, G. & Restivo, G. 2007. Automated stress separation along stress trajectories. *Experimental Mechanics published online*.
- Standing, J. & Leung, W. 2005. Investigating stresses around tunnels and piles using photo-elasticity techniques. In Proceedings of the 5th international Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, pp. 171–177.
- Wakabayashi, T. 1957. Photoelastic method for determining of stress in powdered mass. In *Proceedings of the seventh Japanese National Conference on Applied Mechanics*, pp. 153–158.

### Assessment of tunnel stability in layered ground

#### P. Caporaletti

Atkins Geotechnics, London, UK (formerly University of Rome "La Sapienza", Rome, Italy)

A. Burghignoli University of Rome "La Sapienza", Rome, Italy

#### G. Scarpelli

Università Politecnica delle Marche, Ancona, Italy

R.N. Taylor

City University, London, UK

ABSTRACT: The stability of a circular tunnel in layered ground, with both fine-grained and coarse grained soils below the water table, is investigated experimentally and theoretically. Centrifuge tests have been carried out at City University, London, investigating in detail tunnelling effects in layered ground, in terms of both soil displacements and strains and in terms of the kinematics at failure. Also, analytical upper bound solutions for layered ground which closely reflect the observed failure mechanism of the tunnel have been derived independently. The results from all different approaches have been compared by emphasising the effects of the different hypotheses on the assessment of tunnel stability.

#### 1 INTRODUCTION

The assessment of tunnel stability is an important issue considering the catastrophic effects induced by the tunnel collapse, especially when it concerns urban areas. Surface surcharge loadings may occur in practice where tunnels are excavated below pre-existing structures through fine-grained soil which is overlain by granular material: typical conditions of excavation in urban areas. Although the weights of the overlying coarse-grained materials are easily taken into account when assessing the tunnel stability, the contribution of their strength and stiffness on tunnel stability is often ignored or too difficult to be quantified.

The problem will be approached through the upper bound plasticity theory together with results of centrifuge tests on model tunnels. The case of a tunnel excavated within an over consolidated clay deposit overlain by a sandy layer is considered. Tests included a masonry wall setting in the coarse grained upper layer of soil and thus also gave additional information on the interaction of the tunnel with pre-existing buildings.

This study began from literature references, following two different approaches. In the first case the sandy layer has been taken into consideration as surcharge acting on the top of the clay layer where the tunnel is excavated. In the second case, the sandy layer is properly considered by assigning its appropriate unit weight, thickness, and strength in order to simulate the real initial stress state of soil around the cavity and to emphasise the different behaviour in terms of both undrained and drained conditions.

#### 2 STABILITY OF A SHALLOW TUNNEL

The excavation of a shallow tunnel is clearly a threedimensional problem. Neglecting the effect due to the volume loss at the front face of excavation, this problem might be analyzed under plane conditions. Figure 1 shows an idealization of shield tunnelling, where a circular tunnel of diameter D is constructed with a depth of cover C. The tunnel lining is regarded

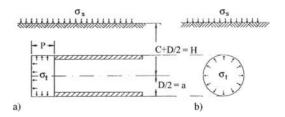


Figure 1. An idealization of shield tunnelling.

as rigid and in front of it the tunnel heading is represented by a cylindrical cavity of length P in which a uniform fluid pressure  $\sigma_t$  acts, and a uniform pressure  $\sigma_s$  acts on the soil surface (Figure 1a). For relatively big value of the distance from the tunnel lining to the tunnel face, the problem can be idealized under plane conditions (Figure 1b).

The collapse of tunnel heading is usually a sudden event caused, for example, by a sudden loss of tunnel support pressure. Then, the stability of tunnels in fine-grained soils can be evaluated by referring to undrained conditions, while the stability of tunnels in coarse-grained soils can be analyzed under drained conditions.

#### 2.1 Stability of a tunnel in undrained conditions

The stability of a tunnel excavated in undrained soils may be assessed using the stability ratio, N:

$$N = \frac{\sigma_s + \gamma H - \sigma_t}{S_u} \tag{1}$$

where  $\gamma$  is the soil unit weight; H is the tunnel axis depth from the ground surface;  $\sigma_s$  and  $\sigma_t$  are respectively the surface surcharge pressure and the tunnel support pressure (according to Figure 1); and S<sub>u</sub> is the soil undrained shear strength at the tunnel axis level.

Davis et al. (1980) estimated the safety of a shallow tunnel excavated without internal support by considering three different shapes of shallow underground opening, for which upper and lower bound stability solutions were derived. The soil strength  $S_u$  was assumed to be constant with depth. Sloan & Assadi (1992) presented a rigorous study taking into account for the variation of the undrained shear strength with depth. The theoretical approach of upper and lower bound solutions were compared to experimental results from a comprehensive study in clays conducted at Cambridge University over the last decades – e.g. Kimura & Mair (1981) who gave a range of design lines to estimate the stability ratio for different tunnel geometries, mainly focusing on undrained conditions.

Generally speaking, the bounding solutions gave good estimates of the collapse load and supported the use of classical limit analyses for undrained conditions. It should be noted that the stability of a plane strain unlined circular tunnel ( $L_R/D \rightarrow \infty$ ) is more critical than that for the lined circular tunnel heading with  $L_R/D = 0$  and the analysis is therefore conservative.

Previous literature proposed different charts where the tunnel pressure at collapse is related to soil mechanical properties and tunnel geometry: the term  $(\sigma_s - \sigma_t)/S_u$  or the stability ratio N is given as a function of C/D for different ratios of  $\gamma D/S_u < 4$ . If  $S_u$  is constant with depth, then for values of C/D greater than 3 the upper and lower bounds of N do not change significantly with  $\gamma D/S_u$ . Below C/D equal to 3, there is a larger spread but the lower bound for  $\gamma D/S_u = 0$  can be adopted as a safe criterion to calculate the required tunnel support pressure: see Figure 4, by referring to the upper and lower bound curves after Davis et al. (1980). If the ratio  $\gamma D/S_u$  is sufficiently large then the collapse will take place for any value of uniform tunnel pressure. Sloan & Assadi (1992) also concluded that for tunnels with C/D>3, these solutions are not fully reliable since the stability of deep tunnels is usually related to a complicated local collapse, involving both elastic and plastic deformation, with only small settlements taking place at the ground surface.

#### 2.2 Stability of a tunnel in drained conditions

Atkinson and Potts (1977) derived kinematic upper bound, and statically admissible lower bound plasticity solutions for the two-dimensional idealization in Figure 1. Only dry sand was considered, but the theoretical solutions, based on effective stress, may be applied to saturated sands provided that the pore water is stationary and the pore water pressures around the tunnel are known. The collapse of a tunnel in saturated sand is then simply the sum of the pressures predicted by either the upper or lower bound solutions and the pore water pressure. The situation is rather more complicated if there is a steady state, or transient, seepage. The authors gave equations to obtain the dimensionless ratio  $\sigma_t/\gamma D$  as function of C/D: the upper bound solution (being inherently unsafe) gives lower value than the lower bound solution which is inherently safe. The authors concluded that the support pressure is independent of the ratio C/D. They also obtained good comparisons between these theoretical solutions and experimental data from centrifuge tests.

#### 2.3 Stability of a tunnel in layered ground

As previously discussed, as far as the authors are aware there is currently no standard procedure to account for the contribution to the tunnel stability of overlying layers of different materials.

Grant & Taylor (2000) studied the stability of tunnels excavated in clay with overlying layers of coarse grained sands and gravels, referring to the design line for clay only given by Kimura & Mair (1981) for  $L_R/D \rightarrow \infty$ . Experimental data from centrifuge tests with an upper layer of loose sand fitted the design line for clay only, indicating that it may act as surcharge loading and not contribute to the stability of tunnel except in terms of weight. In contrast, the presence of a significant thick layer (at least 1D) of relatively dense coarse grained material, combined with little cover above the tunnel crown (C<2D), will aid the

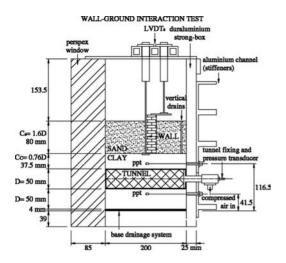


Figure 2. Picture of the experimental model: wall-ground interaction configuration.

stability of a tunnel, and the reasonable value of N equal to 4 might be assumed.

#### 3 CENTRIFUGE MODEL TESTS

Centrifuge model testing has proved a very useful tool to examine the behaviour of shallow tunnels in layered ground, in terms of soil displacements and strains, such as in terms of kinematic failure. A number of tests have been performed at City University, by modelling both the greenfield conditions and the interaction problem between tunnels and pre-existing masonry structures. Details of the apparatus and procedures of tests have been given elsewhere (Caporaletti 2005; Caporaletti et al. 2006) and only the essentials features will be described in this paper.

All centrifuge tests were performed under plane strain conditions: Figure 2 shows a schematic of the centrifuge model. A layer of pre-consolidated kaolin is overlain by a layer of medium dense sand. The kaolin slurry was pre-consolidated by applying a onedimensional load in a press to a vertical effective stress of 500 kPa, before allowing the clay to swell back to 250 kPa. The overconsolidation ratio ranged between 1.4 to 2.8 with depth. The stratum of medium dense sand was made by manual pluviation and then frozen. A 50 mm cavity was cut through the clay layer using a thin walled-cutter and lined with a thin rubber membrane of negligible stiffness and strength. All tests were carried out at the same scale factor  $N_g = 160$  in order to model a real tunnel of diameter equal to 8 m, excavated in a clay layer of 22 m depth overlain by a sand layer of 12.8 m depth, with the tunnel axis at about 23 m from the ground surface. The effects of tunnelling on pre-existing structures were studied by modelling a

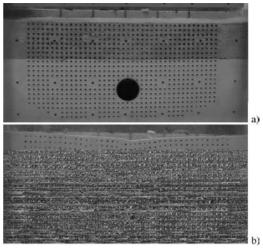


Figure 3. Front view of the model during the test from image processing: a) before reduction of  $\sigma_t$ ;  $\sigma_{t0} \cong 386 \text{ kPa}$ ; b) at tunnel collapse with  $\sigma_t \cong 25 \text{ kPa}$ .

completely buried thin masonry wall perpendicular to the tunnel axis, with foundations just at the sand-clay interface. It represents a stiffer and heavier inclusion in the sand layer.

During the centrifuge spin-up the tunnel air pressure  $\sigma_t$  applied a uniform radial total stress into the cavity to balance the total stress at tunnel axis level:

$$\sigma_t = \gamma_s C_s + \gamma_c (C_c + D/2) \tag{2}$$

where  $\gamma_{\rm S}$  is the unit weight of sand;  $\gamma_{\rm C}$  is the unit weight of clay;  $C_s$  is the thickness of the sand layer;  $C_c$  is the cover above the tunnel in the clay layer; and R is the radius of tunnel as shown in Figure 2. This tunnel pressure represents the compressed air, bentonite slurry or a shield used in practice to achieve the tunnel stability during the excavation process. Water was supplied to the model through a stand-pipe to maintain a predetermined water-table throughout the model: the water level was set up at different depths from the ground surface in order to evaluate the influence of the different effective stress distribution on the tunnel stability. Equilibrium pore pressure was measured by miniature pore pressure transducers around the cavity. The tunnel excavation has been performed by reducing the air tunnel pressure until the cavity collapse, in a period of 3-4 minutes at a rate of approximately 85 kPa/min.

Digital images were taken every second by using a video camera fixed on the swinging platform to view the front face of the strongbox during tests and to follow the movements of black targets pushed both in clay and in sand. Figure 3 shows images of the model in flight.

In spite of the relatively high value of the kaolin permeability, the stratum of clay performs general

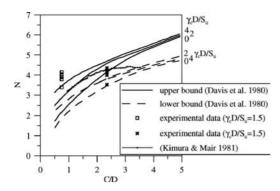


Figure 4. Stability ratio, N: experimental data against theoretical solutions for clays under plane conditions.

undrained conditions during the reduction of tunnel pressure. However, considering that the upper layer of sand represents a significant boundary of drainage for the lower kaolin layer, and considering the small thickness of the clay cover above the tunnel cavity, local drained conditions in this upper part of the kaolin layer were likely in those tests.

#### 3.1 Centrifuge test results

The value of the tunnel pressure,  $\sigma_t$ , has been evaluated for each test, in order to compare experimental results to the guidelines from literature references.

The cavity was excavated in clay, and so design lines for homogenous undrained soils have been considered first. Figure 4 and Figure 5 present the comparison in terms of the stability ratio, N, and of  $\sigma_t/S_u$  depending on the soil mechanical property and tunnel geometry. The profile of the undrained shear strength with depth has been calculated following the equation (Koutsoftas & Ladd 1985):

$$\left(\frac{S_u}{\sigma_v}\right)_{OC} = 0.22 \cdot OCR^{0.8} \tag{3}$$

were  $\sigma'_{\nu}$  is the vertical effective stress, and OCR is the overconsolidation ratio. Considering the relatively small thickness of the clay cover, a constant value of  $S_u$  has been chosen in order to compare test results with theoretical solutions by Davis et al. (1980) for the corresponding value of the term  $\gamma_c D/S_u = 2$ . In the interpretations analyzed afterwards, the constant value of  $S_u$  refers to a characteristic depth respectively equal to 5 m and 1.8 m from the sand/layer interface (Ribacchi et al. 1993).

Experimental data represented by open squares have been analyzed by simply assuming the sand layer as surcharge acting on the top of the clay layer,  $\sigma_s$ , and it is equal to its own weight. Then, the tunnel cover, C, is only equal to C<sub>c</sub>. On the contrary, full symbols

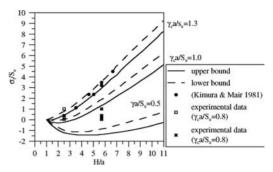


Figure 5. Tunnel pressure,  $\sigma_t$ : experimental data against theoretical solutions for clays under plane conditions.

represent the experimental results analyzed by properly including the sand layer in the evaluation of the tunnel cover. The sand layer is allowed to make a contribution to the tunnel stability as a soil with similar strength to the clay. The surface surcharge pressure,  $\sigma_{\rm s}$ , is therefore equal to zero, and the tunnel cover  $C = C_s + C_c$ . The comparison between the two different data interpretations emphasizes that the ratio of the cover to the diameter of the tunnel is clearly a significant aspect on the assessment of the pressure at collapse. A good fitting of experimental data to previous plasticity solutions from literature references was obtained only when the tunnel cover is properly considered (full symbols). While, the open squares are always out of the range given by theoretical lower and upper bounds. However, this agreement definitely worsens if the real value of S<sub>u</sub> at tunnel axis is assumed instead of the value at the characteristic depth as previously explained.

Afterwards, experimental data have been compared to theoretical solutions for cavity excavated in sands. Design lines after Atkinson and Potts (1977) are shown in Figure 6, by distinguish the original lower and upper bound approaches for dry sands, and the lines obtained for saturated soils. Once again, the physical results are clearly out of the two ranges given by theoretical solutions.

The load factor, which is the ratio between the stability ratio at working conditions and at collapse (the subscript  $_0$  refers to the beginning of the test before the reduction of the tunnel support pressure):

$$LF = \frac{\sigma_{t0} - \sigma_t}{\sigma_{t0} - \sigma_{tc}} \tag{4}$$

has been calculated at different values of the volume loss,  $V_L$ , defined as the volume of settlement trough at the tunnel axis, and plotted in Figure 7.

In order to have a clear representation of experimental data, only results from few tests have been shown. Test PC2 and test PC8 had greenfield conditions but with two different hydraulic boundaries: the

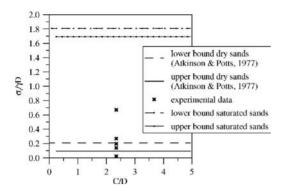


Figure 6. Tunnel pressure,  $\sigma_t$ : experimental data against theoretical solutions for sands under plane conditions.

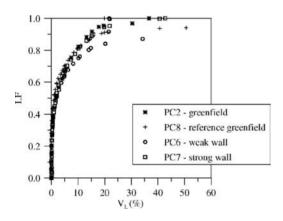


Figure 7. Curves of load factors against volume loss.

water level is very close to the ground surface (PC2), and very close to the sand/clay interface (PC8). Soilstructure interaction conditions existed in Tests PC6 and PC7, in which were modelled respectively the weakest and the strongest masonry structure, with the same hydraulic conditions fixed during test PC8. As expected, focusing at first on the greenfield tests, higher values of the tunnel support pressure were measured during test PC2, which means a more unstable configuration due to the different effective stress distribution experienced as the consequence of differences in the hydraulic boundary conditions. At the same way, analyzing the results of soil-structure interaction tests, the masonry wall built in the sand layer represented a stiff inclusion within the soil and made the tunnel less stable. The tunnel collapse occurred for higher values of the cavity pressure, and the worst condition was reached when the strong wall was modelled. Looking at the point of intersection between the load factor curves related to the soil-structure interaction tests, and the reference greenfield curve, it is evident that the weakest wall always stayed in contact with

the upper face of kaolin, which did not occur with the strong wall that lost this contact at value of  $V_L \approx 15\%$ . In spite of this, the failure mechanism observed in the soil around the tunnel seemed to be very similar. It is particularly interesting to emphasise that for all tests the experimental curves approximately show a horizontal asymptote after volume losses of around 20%. Therefore, the condition corresponding to the tunnel collapse has always been taken as that corresponding to the stage when  $V_L > 20\%$ .

#### 3.2 Tunnel mechanism of failure

Experimental data have been analyzed in terms of the effect due to the layered configuration on the field of soil stress and strain (Caporaletti, 2005; Caporaletti et al., 2006). The two fine-grained and coarse-grained materials, sand and clay, have clearly shown different mechanical behaviours. In particular, the sand tendency to dilate was constrained by the lateral walls of the strongbox, and resulted in vertical settlement decreasing with depth. At the base of the sand layer, the pattern of soil strains indicates expansion near the tunnel axis. In contrast, vertical settlements within the clay layer always increased with depth, and nonzero volumetric compressive strains were measured above the cavity close to the tunnel axis, maybe due to there not being perfectly undrained conditions locally maintained during the tests. According to literature references, the mechanism of failure for tunnel in clays propagates upwards and outwards from the cavity invert becoming significantly wider than the tunnel diameter. In sands, failure tends to involve a narrow "chimney", propagating almost vertically from the cavity up to the ground surface (Mair & Taylor (1997)). However, in these centrifuge tests the mechanism at failure for a layered configuration involves a wide area of soil both in sand and in clay, with pseudo-vertical settlements at the sand-clay interface. The kinematic mechanism is clearly characterized by soil displacements pointing towards the cavity on the top of the sand layer and everywhere within the clay layer, and by a rigid vertical block translation at the base of the sand layer, for a thickness of about D/2 (see Figure 8). The field of ground displacements in sand is strictly related to soil movements induced at the top of the clay layer more than to the reduction of the tunnel support pressure itself.

#### 4 THEORETICAL ANALYSIS

The comparison of experimental data to published literature guidelines has clearly shown discrepancies in terms of evaluation of the soil behaviour at collapse, when both fine and coarse grained material strata are involved in the problem. The effect due to the layered

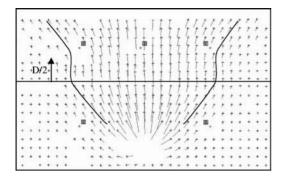


Figure 8. Mechanism of failure from centrifuge tests (VL  $\cong$  20%).

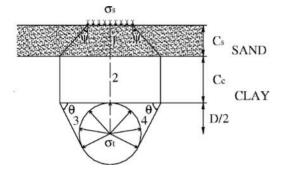


Figure 9. Mechanism for layered ground.

configuration need to be better understood: kinematic analyses could explain the observed behaviour.

A mechanism related to the problem of stability of a plane strain unlined circular tunnel excavated in layered ground has been evaluated. The calculation relies upon the assumption that failure happens under drained conditions in sand and undrained conditions in clay. The soil is idealized as a perfectly plastic material with an associated flow rule: unit weight,  $\gamma_s$  for sand, and  $\gamma_c$  for clay, undrained shear strength of clay, S<sub>u</sub>, and angle of dilancy of sand equal to the maximum angle of shear resistance,  $\Psi = \varphi'$ . The mechanism is shown in Figure 9. It starts from that given by Davis et al. (1980) for clays, but it assigns the appropriate unit weight, thickness, and strength to the coarse-grained stratum. Due to the assumption of associated flow, the mechanism shows vertical movements within the sand layer, according to the experimental data (see Figure 8). Vertical movements are also assumed to be generated in clay above the cavity, by simplifying the real kinematic observed from centrifuge tests (see Figure 8). The work calculation is performed by changing the angle  $\theta$  in order to minimize the value of the tunnel pressure and to achieve an upper bound solution, with an unsafe estimation of the pressure at collapse.

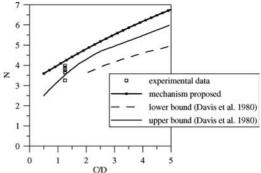


Figure 10. Theoretical solutions and experimental data.

The theoretical solution for this mechanism is plotted in Figure 10 together with centrifuge results and literature solutions for homogenous undrained soils: the stability ratio, N, refers to the actual undrained strength of clay at the tunnel axis. The cover in sand has been assumed equal to D/2, following the kinematic mechanism observed during centrifuge tests, while part of the thickness of the sand layer is evaluated as an external surcharge. The stabilizing contribution due to the friction acting in sand is clearly evident, and its influence on the evaluation of the stability ratio should not be neglected. Experimental data are clearly out of the range given by lower and upper bounds for clays, whereas a satisfactory upper bound is assessed by adopting the theoretical solution by following the mechanism herewith proposed.

#### 5 CONCLUSIONS

Physical modelling and theoretical solutions have been studied in this research and compared to one another, to highlight the effect on the tunnel stability due to a layered configuration that involves both fine-grained and coarse-grained materials, since previous works only refer to homogenous soils.

In the case of tunnels excavated in clay overlain by sand, the contribution to the stability due to the friction acting within the upper layer represents a significant contribution, which should not be neglected. Coarse-grained material cannot satisfactory be considered as an external surcharge equal to its own weight only. A significant over-estimate of the tunnel support pressure to prevent collapse might result if simple reference to the literature theoretical solutions obtained for homogenous clays is made and the sand layer treated only as a surcharge. The proposed new mechanism provides a good upper bound to the experimental data though some simplifications of the proposed kinematic mechanism could be removed, and though it is recognised that a new lower bound is needed to complete the analysis.

### REFERENCES

- Atkinson, J.H. & Potts, D.M. 1977. Stability of a shallow circular tunnel in cohesionless soil. *Géotechnique* 27(2): 203–215.
- Caporaletti, P. 2005. *Tunnelling in Layered Ground and its Effects on Pre-existing Masonry Structures*. PhD Thesis, University of Rome "La Sapienza", Italy.
- Caporaletti, P., Burghignoli, A. & Taylor, R.N. 2006. Centrifuge Study of Tunnel Movements and their Interaction with Structures. Proceedings 5th Int. Conference on Geotechnical Aspects of Underground Construction in Soft Ground. Amsterdam, The Netherlands: 99–105.
- Davis, E.H., Gunn, M.J., Mair, R.J. & Seneviratne, H.N. 1980. The stability of shallow tunnels and underground openings in cohesive material. *Géotechnique* 30(4): 397–416.

- Grant, R.J. & Taylor, R.N. 2000. Stability of tunnels in clay with overlying layers of coarse grained soil. *Proceedings GeoEng2000*. Melbourne, Australia.
- Kimura, T. & Mair, R.J: 1981. Centrifugal Testing of Model Tunnels in Soft Clay. Proceedings 10th International Conference on Soil Mechanics and Foundations Engineering. Stockholm, Vol. 1: 319–322.
- Koutsoftas, D.C. & Ladd, C.C. 1985. Design strength of an offshore clay. *Journal of the Geotechnical Engineering Division, ASCE* 111(3): 337–355.
- Mair, R.J. & Taylor, R.N. 1997. Theme lecture: Bored tunnelling in the urban environment. *Proceedings 19th International Conference on Soil Mechanics and Foundations Engineering*. Hamburg: 2353–2384.
- Ribacchi, R. 1993. Recenti orientamenti nella progettazione statica delle gallerie. XVIII Convegno Nazionale di Geotecnica. Proceedings 18th National Conference in Geotechnics. Rimini, pp. 37.
- Sloan, S.W. & Assadi, A. 1992. Stability of shallow tunnels in soft ground. *Predictive soil mechanics*, Thomas Telford, London, 1993: 644–662.

# Reinforcing effects of forepoling and facebolts in tunnelling

K. Date

Kajima Technical Research Institute, Kajima Corporation, Tokyo, Japan

R.J. Mair & K. Soga

Engineering Department, University of Cambridge, Cambridge, UK

ABSTRACT: Ground deformation induced by tunnelling in shallow sandy ground can be reduced by placing some reinforcements such as facebolts and forepoling bolts from the tunnel. A series of centrifuge tests have been carried out in order to investigate the ground deformation pattern during tunnel excavation with reinforcements. Three dimensional numerical analysis of the problem was also performed using FLAC3D and the simulation results show good agreement with the centrifuge data.

### 1 INTRODUCTION

Tunnel reinforcement has been applied to bored tunnel excavation in order to keep the cutting face stable, to reduce ground (sub)surface settlements and to avoid any adverse influence on adjacent structures. Forepoling and facebolts are the two most popular tunnel reinforcements; the former is often used in European countries, whereas the latter is frequently applied in Asian countries. However, the specifications of using them are based on local and empirical designs or on experience of past constructions in similar ground conditions.

In order to excavate a larger tunnel under poor ground conditions safely, it is necessary to establish a new design method for tunnel reinforcements such as forepoling and facebolts. In particular, in order to evaluate the relative merit of these two techniques for ground deformation control, it is important to compare them using the same modeling techniques (centrifuge tests or numerical analysis) under the same ground conditions.

## 2 CENTRIFUGE TESTS

In this study, the effect of tunnel reinforcements on ground deformation in shallow tunnels was investigated. Chambon and Corté (1994) performed centrifuge modeling of tunneling in sandy ground and showed the minimum pressure to support a cutting face was independent of cover diameter ratio (C/D, C: cover, D: tunnel diameter). They also showed, when a model tunnel was installed with C/D = 4 and P = 0.1D (P: unsupported length), the failure lines extended to the height of about 2.5D from the crown and did not

extend to the ground surface. The centrifuge tests by Leca and Dormieux (1990) show that the failure lines reached the ground surface when C/D = 1. These two studies indicate that ground deformation pattern, when C/D is less than 1.0, is different from that C/D exceeds 1.0. Hence, it was decided to perform centrifuge experiments with a tunnel model of C/D = 1.0.

#### 2.1 Models

A schematic view of the model is shown in Figure 1. The strong box shows the half of the prototype so that the ground deformation changes could be observed through a perspex window that is installed on the longitudinal section of the model. The ground deformation was analyzed using PIV program developed by White & Take (1996).

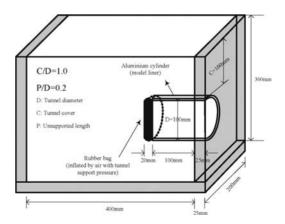


Figure 1. Schematic view of strong box and model tunnel.

Table 1. Matrix of reinforcement bolts.

Test	Туре	Arrangement*	Number
KDC10	No reinforcement	_	0
KDC04	Facebolts	FB01	14
KDC05	Facebolts	FB02	14
KDC06	Forepoling	FP01	14
KDC07	Forepoling	FP02	28

\* Each pattern is displayed in Figure 2.

The model box was filled with dry Leighton Buzzard Fraction E Silica sand with the relative density of 87% ( $\pm 2\%$ ), corresponding to a unit weight of 15.9 kN/m<sup>3</sup>.

The model tunnel, of diameter D = 100 mm (7.5 m)in prototype), is semicircular and the depth to the tunnel crown C was equal to the diameter D (C/D = 1.0). The excavation of the tunnel was simulated by decreasing the internal pressure of a rubber bag placed at the tunnel face. The bag was covered with an alminium rigid lining, which was installed at a distance of P (=0.2D) behind the tunnel heading. The internal pressure was reduced from 100 kPa to tunnel collapse pressure. The centrifuge tests were performed at 75 g.

A series of five tests were carried out as listed in Table 1. The model reinforcement bolts, which were made of alminium, were installed perpendicularly to the tunnelling head during the sand-pouring. They were coated with the same sand as used in the tests, the outer diameter of them were 2.4 mm (180 mm in prototype). The forepoling model bolts were attached to the model liner with glue. The number, the length and the arrangement of the reinforcements were varied as shown in Table 1 and Figure 2.

#### 2.2 Tunnel collapse

The model tunnel without tunnel reinforcement (KDC10) collapsed at the support pressure of 3.1 kPa, which agrees with the past centrifuge results by Chambon & Corté (1994). On the other hand, for the tunnels with reinforcements (KDC04-07), tunnels collapsed at lower pressures (2.2–2.7 kPa). This shows that not only facebolts but also forepoling contributes to reducing the minimum support pressure, but both techniques did not dramatically decrease the pressure required to keep the face stability.

The tunnel collapse mechanism on the longitudinal section is shown in Figure 3. When there is no reinforcement (KDC10), the front slippage line started at the bottom of the tunnel, extended upwards with a quasi-logarithmic curve, and then reached to the ground surface vertically. The line behind the tunnel was nearly vertical but a little inclined backwards. The failure mechanisms using facebolts are shown by KDC04 and 05 in Figure 3. The distinct difference from the non-reinforcement case was found at the front failure line, which started from some point at the upper face and then did not extend ahead of the face but extended upwards almost vertically. This showed that facebolts were effective to improve the face stability. In other words, this change in failure mechanism led to the reduction of the collapse volume. When facebolts were installed only at the upper face (KDC05), a chimney-like collapse was observed. This may be because the tunnel collapse was more dominated by P, that is, the stress relaxation at the crown rather than the height of the model tunnel, H.

When forepoling was introduced in a sparse manner (KDC06), the front slippage line was similar to that of non-reinforcement case (KDC10), but the back slippage line did not develop outward but inward toward the tunneling direction. This back line pattern was similar to that when a denser pattern of forepoling was introduced in KDC07. However, the geometry of the front line was totally different. In KDC07, the slippage line developed from the middle of the heading, and extended up to the horizontal line where forepoling bolts are embedded. Then the line suddenly developed vertically upwards to the ground surface.

When forepoling bolts are densely installed, they divide the surrounding ground into two zones; (a) the outside zone of invisible arch consisting of forepoling bolts, and (b) the inside zone of the forepoles. The mechanisms of collapse of the two zones have to be considered separately. It appears that the collapsed area shifted forward in the longitudinal direction.

# 2.3 Changes in displacement vectors with the decrease in tunnel support pressure

The ground deformation in sandy ground is quite small even when tunnel support pressure decreases to about half of the initial pressure. However, once it starts to develop, it abruptly increases and then reaches the collapse rather instantaneously. Therefore, in the past, it has been difficult to obtain the displacement vectors in sandy ground as the tunnel support pressure decreases. The PIV analysis, developed by White et al. (2003), was used to monitor subtle changes in ground deformation in sandy ground.

Figure 4 shows that the changes in the distribution of displacement vectors on the longitudinal section in KDC10 as the tunnel support pressure decreases. The displacement vectors were difficult to be detected even by using the PIV analysis until the internal stress was unloaded to around 25 kPa. Initially, the region deformed by the stress release was widespread. As shown in Figure 4, with decrease in the tunnel support pressure, the deformation became more localized around the face rubber bag. The first large ground movement was observed at 5.4 kPa and it was an earlike shape on the longitudinal section. The observed

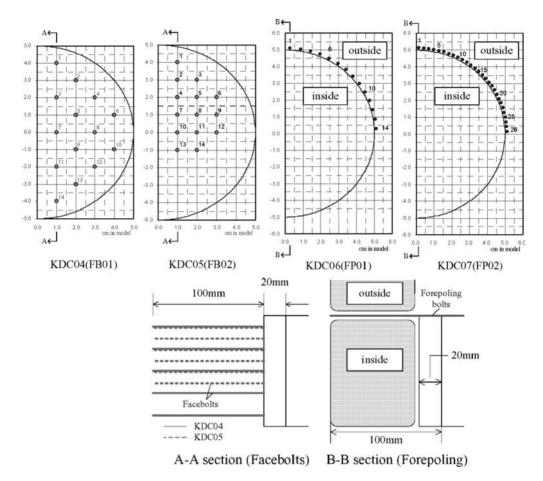


Figure 2. The arrangement patterns of bolts.

shape is similar to the three dimensional tunnel failure mechanism proposed by Leca & Dormieux (1990).

# 2.4 Displacement vectors near collapse in centrifuge tests

Figure 5 shows the displacement vectors at pressure close to the tunnel collapse, which varies from 3.5 to 5.0 kPa.

The distributions are comparable to the failure shapes shown in Figure 3. It was obvious from KDC04 and KDC05 that facebolts were effective to reduce the front area affected by tunnel excavation and also the ground movements, in particular, for the horizontal one. In the forepoling cases (KDC06 and KDC07), it appears that the ground moved under different mechanisms at the inside and outside of the half arches created by forepoling bolts as shown in Figure 1 and Figure 2. Some continuity between the inside and the outside is observed in KDC06, but they are discontinuous in KDC07. Hence, it was concluded that forepoling bolts can be effective to reduce the displacement outside the forepoling arch as long as they are installed densely enough to divide the surrounding ground into two zones.

Figure 6 shows the distribution of horizontal displacement along the cutting head at different internal pressures for KDC10, KDC04 and KDC07. Results show that facebolts (KDC04) contributed to the reduction of face extrusion. For the case of densely installed forepoling bolts (KDC07), the effect to reduce the face extrusion was not so apparent as that with facebolts, but the bulging pattern changed. That is, the maximum extrusion was found at the location beneath the crown, although it was found at about 1/4 the height of the face from the crown in KDC10 and KDC04. Ground deformation started at 20-25 kPa for all cases, but the deviation from the non-reinforcement case (KDC10) to the reinforcement cases (KDC04 and KDC07) became evident when the face pressure was 10-15 kPa.

In actual construction, great care must be taken in order to avoid fatal ground (sub)surface settlement above a tunnel, and hence it is essential to study how the settlement trough develops with the decrease in the tunnel pressure especially at locations just above the tunnel crown.

Figure 7 shows the subsurface settlement at just above the tunnel crown in KDC10, KDC04 and KDC07. These troughs appeared when the face pressure was 20–25 kPa for all cases, but the differences among them became apparent when the face pressure was 10–15 kPa. At around 5.5–6.5 kPa, the maximum settlements in KDC04 and KDC07 were about half

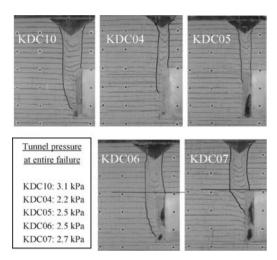


Figure 3. Tunnel failure patterns on the longitudinal section.

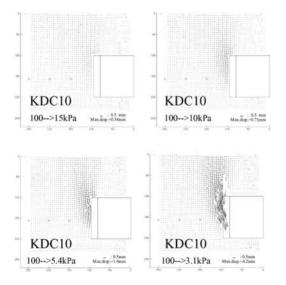


Figure 4. Displacement vector changes with the decrease in tunnel support pressure.

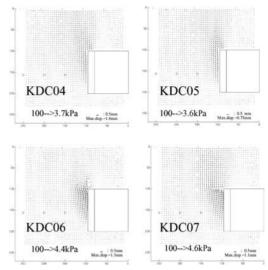


Figure 5. Displacement vector distribution on the longitudinal section.

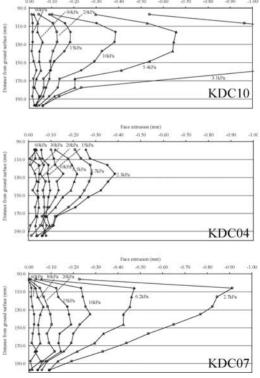


Figure 6. Distribution of face extrusion at the face.

as large as those in KDC10. Hence, both facebolts

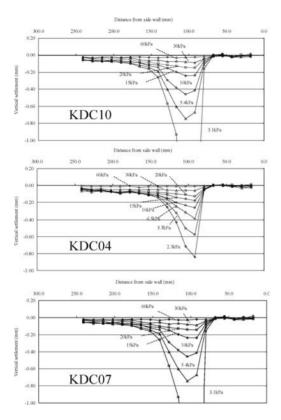


Figure 7. Subsurface settlement just above crown.

and forepoling were effective to reduce ground settlements. The shapes of the troughs are sharper in KDC10 and KDC04 than in KDC07. The maximum settlement in KDC4 was positioned behind that in KDC10. The trough in KDC07 was wider than the troughs observed in KDC04 and 10, and, as a result, the position of maximum settlement shifted ahead of the maximum settlement occurred in KDC10. Densely installed forepoling bolts were capable of reducing the influences from the stress release at both the face and the crown, while facebolts only contributed to counteract the effect of the stress release at the face.

# 3 SIMULATION MODELING AND ANALYSIS

#### 3.1 Modeling

In order to simulate the centrifuge test results, 3D analyses were performed using FLAC3D. The geometry was identical to the internal size of the strong box used in the centrifuge modeling tests;  $400 \times 200 \times 300$  h in mm, as shown in Figure 8. For the boundary conditions at side walls, roller conditions were applied. The analysis basically followed the order of the centrifuge

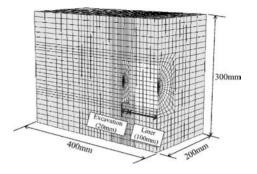


Figure 8. Geometry of 3D numerical analysis model.

Table 2. Soil properties for test simulation with FLAC3D.

Model	Young's modulus E	c(Pa)	φ(°)	ψ(°)
Mohr-Coulomb-1*		0.1	40	0, 15
Mohr-Coulomb-2**	$p' = (\sigma_v + \sigma_h + \sigma_h)/3$ $\sigma_h = \sigma_v * \nu/(1 - \nu)$		See Fi	gure 9

\* Mohr-Coulomb model 'without' strain softening/hardening model

\*\* Mohr-Coulomb model 'with' strain softening/hardening model

tests; swing-up to 75 g, and then decrease of the internal pressure inside the rubber bag. The tunnel lining was assumed to be rigid.

#### 3.2 Soil property

Mohr-Coulomb without strain softening/hardening model (MC), one of the standard and basic models, was used. Two dilation angles were used as shown in Table 2.

In addition, in order to simulate more accurately the results from the centrifuge tests, Mohr-Coulomb with strain softening/hardening model (SSH) was also used. The changes in parameters with plastic shear strain were derived from the triaxial tests by Coelho et al.(2007). As shown in Figure 9, the simulation of triaxial test results using FLAC3D showed good agreement with the test performed under the initial confining stress of 120 kPa.

# 3.3 Simulation results for the non-reinforcement case

Figure 10 shows the development of the maximum face extrusions with the decrease in the tunnel support pressure. The centrifuge exhibited the entire collapse at 3.1 kPa. The prediction with the SSH model was in good agreement with the experimental results. When the MC models were used, the deviation from the experimental results starts at around 30–40 kPa and

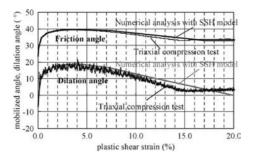


Figure 9. Simulation results of triaxial test at  $\sigma_3 = 120$  kPa.

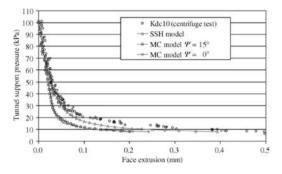


Figure 10. Simulation of the maximum face extrusion in KDC10 with 3D numerical analysis with the Mohr-Coulomb models with/without strain softening/hardening model.

then abrupt increase in face extrusion was found at around 15 kPa.

This is because the SSH model is able to simulate the plastic behaviour at and after small strain levels, corresponding to the face extrusion larger than 1.5 mm. As a result, the SSH model can follow the gradual increase in face extrusion with the decrease in tunnel support pressure.

Figure 11 shows the simulation results of the distribution of face extrusion in KDC10 when the tunnel support pressure was 8.0–9.0 kPa. Both constitutive models were successful in estimating the extrusion curve just before tunnel collapse, but, in order to simulate the deformation pattern observed in the experiments more precisely, the SSH model was found to be more appropriate than Mohr-Coulomb model.

However, there was a difference in position where the maximum extrusion occurred in Figure 11. In the numerical analyses, the peak was found at the middle and this location is lower than the location of the peak observed in the centrifuge test. This may be due to the difference in the shape of the excavation ranges in between centrifuge tests and numerical analyses. The model ground in centrifuge tests can move smoothly at the corner near the crown and the face because the rubber bag is flexible enough to be smoothly deformed. However, as for the numerical

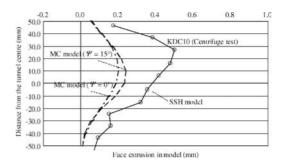


Figure 11. Simulation of the face extrusion bulges in KDC10 when the support pressure was 8.0–9.0 kPa.

Table 3. Input parameters for facebolts.

Model	Density	Young's modulus	Poisson's ratio
Pile	2700(kg/m <sup>3</sup> )	7*10 <sup>4</sup> (MPa)	0.2

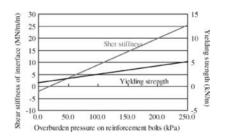


Figure 12. Changes in shear stiffness and yielding strength of soil-bolt interface with overburden pressure.

analyses, rectangular meshes might prevent the model ground from extruding inwards smoothly.

Further improvement in the numerical analysis (e.g. mesh making) is required in order to illustrate not only the development of the maximum extrusion but also its distribution along the face.

#### 3.4 *Reinforcing effects of facebolts on reducing face extrusion*

The reduction of face extrusion by facebolts was also simulated. The SSH model was adopted for both simulations. The interface property between ground and bolt surface, which is shown in Table 3 and Figure 12, was derived from the pull-out tests reported in Date et al. (2007). Figure 13. shows the simulation results of KDC04. Results from KDC10 are also presented for comparison purpose.

The deformation pattern simulated by the numerical analysis was similar to that of the centrifuge test, but the magnitude predicted by the numerical analysis was

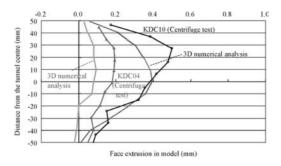


Figure 13. Simulation of face extrusion in KDC4 with 3D numerical analysis with the SSH model.

smaller than the centrifuge data. This may be due to the mesh problem mentioned above, the difference in the soil-bolt interaction properties and the difference in soil properties during between loading and excavating. Further investigation is needed.

## 4 CONCLUSIONS

A series of the centrifuge tests showed that introduction of facebolts and forepoling bolts for tunneling in shallow sandy ground yielded different shapes of tunnel collapse and contributed to some reduction in the tunnel support pressure to keep the cutting head stable. These techniques were also effective in reducing the vertical settlement at locations just above the tunnel crown. Facebolts were able to reduce the risk of face extrusion and hence can make the ground ahead of the face stiffer. Forepoling bolts can divide the ground around the tunnel face into two zones; the inside and the outside of the arch of forepoling bolts.

Numerical analysis results show that the SSH model (Mohr-Coulomb model with strain softening/hardening) gave better match to the centrifuge data than the MC model (Mohr-Coulomb model without strain softening/hardening). However, in order to simulate more realistic behaviour observed in the centrifuge tests, further investigation of the settings of numerical analysis such as mesh-making and soil-bolt interaction properties needs to be conducted.

# REFERENCES

- Chambon, P. & Corté, J.F. 1994. Shallow tunnels in cohesionless soil: stability of tunnel face. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 120(7): 1150–1163 1994.
- Coelho, P.A.L.F. 2007. In situ densification as a liquefaction resistance measure for bridge foundations. *PhD Thesis*, *Cambridge University, UK.*, 2007.
- Date, K., Mair, R.J., Soga, K. Centrifuge tests for tunnel reinforcement. Soils and Foundations, 2008. (in preparation)
- Leca, E., Dormieux, L. 1990. Upper and lower bound solutions for the face stability of shallow circular tunnels in frictional material. *Géotechnique* 40(4), 581–606 1990.
- White, D., Take, A. Bolton, M.D. 2003. Soil deformation measurement using particle image velocimetry (PIV) and photogrammetry. *Géotechnique* 53(7), 619–631 2003.

# Mechanical behavior of closely spaced tunnels – laboratory model tests and FEM analyses

J.H. Du & H.W. Huang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: Reduced-scale model tests and corresponding finite-element analyses were performed to investigate the effect of different excavation sequences for twin tunnels under the unsymmetrical pressure in weak rock. In these tests, the alternative sequences of the excavation of the left tunnel first and then the right one, or the right first and then the left, were simulated. The displacement of ground and the earth pressure in the rock were measured. Particular attention was paid to the different behaviors of the displacements and the stresses caused by the different excavation sequences. The result of the present study indicates that the state of stresses, the displacements around the tunnels and the ground surface settlements are all different when the excavation sequence is alternated. When the right one is excavated firstly, the stratum condition is more deteriorated. It can be concluded that the sequence of the left first and then the right one is better for such twin tunnels.

# 1 INTRODUCTION

With the development and the upgrade of infrastructures such as highway, subway, railway, and many other facilities, tunnel constructions are gradually increasing in the recent years. The preference in China, and as well as in other countries, is to use twin tunnels for the new transportation lines rather than a single larger diameter tunnels, when the space is limited.

Model test and numerical simulation are two key methods to investigate the tunneling problems. In the past two decades, the rapid advances have been made in tunnel model tests, many of which are investigations on the twin tunnels. Dhar et al. (1981) performed the fracture pattern around twin openings in weak materials of sand wax and sand wax mixtures with the plaster of Paris under controlled loading conditions. Twin openings for different orientations in respect to loading directions were studied. Adachi et al. (1993) also used model tests to analysis the interaction between twin tunnels. The tunnel excavation was simulated by tailor-made diameter reducible device. Chu et al. (2006) performed model tests of twin circular tunnels in homogenous material, two-layered formations, and three-layered formations to understand the mechanical behavior of a twin-tunnel of circular cross section in multi-layered formations, and a two-dimensional numerical simulation was also developed.

Using 2D and 3D numerical simulations, Ghaboussi (1977), Soliman (1993), Addenbrooke (1997), Ng

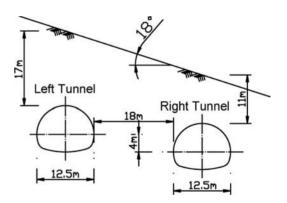


Figure 1. Typical cross section of Pingnian Tunnel.

(2004) et al. investigated some aspects of interaction mechanisms.

Pingnian Tunnel is a typical twin-tunnel, which was constructed in accordance with the principles of the New Austrian Tunneling Method in weak rock, being part of Luofu expressway in Yunnan Province. Tunnel width is 12.5 m, and the length is about 360 m. Spacing between two tunnels changes from 18 m (at Luochunkou site) to 25 m (at Funing site). The tunnels are under the inclined ground surface and the two tunnels are not constructed in the same level, and the vertical distance between them is from 3 m to 4 m. The typical cross section is shown in figure 1.

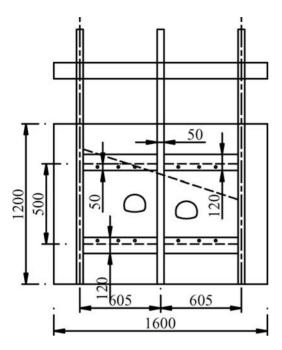


Figure 2. Sketch of steel frame with model tunnel (Unit: mm).

So far, the behavior of surrounding ground during the single tunnel excavation has been extensively investigated, however, the case that a tunnel is driven paralleling to another adjacent tunnel is of interest to tunnel engineers. During the Pingnian Tunnel construction, a collapse happened at the left line portal, and the slip also affected the adjacent portal of right line because of the small space between the two tunnels. In order to study the actual interaction between two tunnels, two-dimensional laboratory model tests were performed to capture the earth pressure transfer and the displacement of surrounding rock during construction. The particular attention was paid to the evolution of the displacements and stresses caused by the different excavation sequence. The numerical simulations by the FEM were also conducted for the problem to compare the analytical results by model tests.

## 2 SIMULATION METHODS

#### 2.1 Two-dimensional plane strain test

Although actual tunneling is a three dimensional problem and actual ground is heterogeneous as well as anisotropy in nature, in this study, plane strain tests were conducted in a homogeneous ground by simplification. The box used in tests was made of a steel frame, as shown in figure 2, with dimensions of  $1.6 \text{ m} \times 1.2 \text{ m} \times 0.4 \text{ m}$ . The four sidewalls of the test

Table 1. Properties of the ground material for the model tests.

26.9
38
25
39

box were assembled using steel sheets. For the case of the observation of the ground movement patterns during testing, two transparent Perspex plates were used at the front and back sides.

The tests were conducted according to the typical cross section in the steel frame as shown in figure 2. The tunnel width is 156 mm, the horizontal distance between two tunnels is 225 mm, the vertical distance is 500 mm under inclined ground with angle 18°, and the cover depths of left and right tunnel are 212 mm and 138 mm respectively. All dimensions are controlled by the geometrical similarity ratio of 1/80.

The ground material used in the model test was composed of barite powder, sand and plaster mixed with water, and the quality proportion was achieved through a series of material tests. The main parameters of the ground material are shown in table 1, determined by the following expressions according to the similarity theory.

$$C_{\phi} = 1,$$
 (1)

$$C_{\gamma} = 1,$$
 (2)

$$C_{\sigma} = C_{E} = C_{e}, \qquad (3)$$

$$C_{\sigma} = C_{\gamma} \cdot C_{l} = C_{l} = 80, \qquad (4)$$

where  $C_{\varphi}$ ,  $C_{\gamma}$ ,  $C_{\sigma}$ ,  $C_E$ ,  $C_c$ ,  $C_l$  are the similarity ratios for friction angle  $\varphi$ , stress  $\sigma$ , young's modulus E, cohesion c, unit weight  $\gamma$  and geometrical dimension l, which represent relationship for the parameters between the model test and the real situation.

The excavation was simulated by removing the ground inside the excavation zones manually, with the two different excavation sequences conducted: the left tunnel excavated first and then the right one (named test A), and the right first and then the left (named test B). A monitoring program was utilized to record the process of the ground movements and the earth pressure changes around the tunnels. Monitoring items included earth pressure cells, deep rods and micrometer gauges, and the locations of items shown in figure 3 and figure 4. All monitoring items were attached to the high-speed instrument of static strain gauge (YE2539), collecting data at any interval operator set.

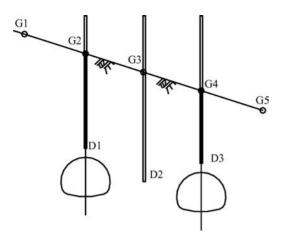


Figure 3. Locations for displacement gauges.

L6,

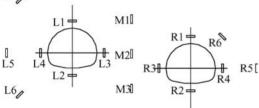


Figure 4. Locations for earth pressure gauges.

#### 2.2 Numerical simulation

Corresponding numerical simulation was performed to compare the above experimental tests. The simulations were based on the model test conditions and the material properties. The finite element software Msc.Marc was adopted for the simulations. In the analyses, the ground material was assumed to be elastic-plastic confirming to the Druker-Prager failure criterion together with the associated flow rule. The analysis consisted of two phases: the first phrase is to create an initial geostatic condition and the second phase to simulate the excavation process. During the first phase, the initial geostatic condition was achieved by applying gravity forces to all ground elements. In the second phase, the excavation process was simulated by removing the elements inside the excavation zone. The simulation of the test A and the test B performed in laboratory model tests were conducted accordingly.

### 3 RESULTS

# 3.1 Displacement

Figure 5 is a surface displacement-time diagram, which is obtained in the model test. From the figure we

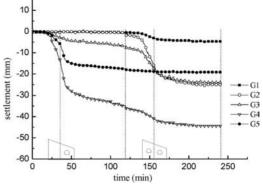


Figure 5. Surface settlement curves (obtained in model test).

can find that the settlement of every test point increases step by step along with the advancement of excavation. A substantial increase of the displacement occurs when excavation section passes the test section. The settlement develops continuously after excavation finished and reaches the steady-state in a period of time. The creep displacement is the major part during the period of the excavation termination, accounting for about 10%  $\sim$ 30% of the total displacement.

There is still some ground surface settlement (at G2, G4) increasing during the excavation process of adjacent tunnel, which reveals the existence of an interaction between the twin tunnels. G1 located above and left to G2, whose horizontal distance to the left wall of the left tunnel is one time of width of the tunnel, doesn't undergo a distinct displacement during the excavation of the right tunnel while it does during the excavation of the left one. The displacement of the lower G5, located in the right to the G4, is totally different, which is mainly affected by the excavation of right tunnel. For the spot G3, located at the surface above the pillar centerline, the settlement is still in the development state and the excavation of both the right and the left tunnels will produce effect on it; the two abrupt changes of the displacement happened in the excavation section passing the test one. The displacement change of the spot G3 appears smaller than the displacement of the spot G2 and G4.

We can make a conclusion from the above: the influence of tunnel excavation decreases with the increase of the distance to the excavated tunnel. The zone influenced mostly is those above the tunnels. Under such leaning stratum, the influence of the excavation to rocks in one time of the width of the tunnel region still exists, but it is small when the distance between the tunnel and the rock is more than three times of the width of the tunnel. Excavations of left and right tunnel all have effect on surface above the pillar.

The comparison between deep settlement (settlement of the inner stratum, indicated by D1, D2 and D3,

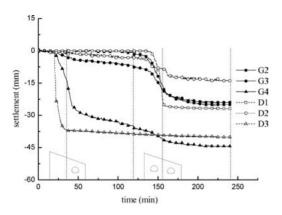


Figure 6. Surface and deep ground displacement curves (obtained in model test).

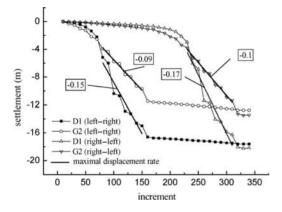


Figure 7. Surface and deep ground displacement curves (obtained in FEM analyses).

located above the left tunnel crown, in the pillar center line parallel to the right tunnel spring line and above the right tunnel crown respectively) and corresponding surface settlement is shown in figure 6. The development trends are coincident, however, deep settlement occurs prior to that of the surface, and its displacement rate is also relatively bigger. Time to steady state needed by the deep displacement is shorter than that of the surface settlement. When it comes to compare two tunnels, the displacement rate of the left tunnel is bigger than the right one.

In the test A, settlements of all measure points are more than those got in the test B. The maximal displacement is 35.9 mm when the left tunnel excavated first and then the right one, and it is 44.4 mm when the right one first and then the left. So it indicates that the left tunnel excavated first can control surface displacement more effectively. The comparison of surface settlements between two opposite construction sequences achieved in numerical simulations are

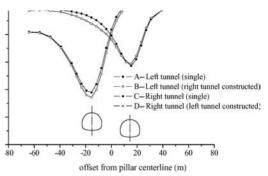


Figure 8. Transverse settlement trough corresponding to different excavation sequences (obtained in FEM analyses).

shown in figure 7. It reaches the same conclusion that the ultimate surface settlement occurred by the right tunnel excavated first is bigger than that occurred in the reverse excavation sequence. The evolution law of the displacement rate is the same of the settlement. So it is not beneficial for the tunnel stabilization to adopt the construction sequence with the right tunnel excavated before the left tunnel under such locations as the Pingnian twin tunnel under the inclined stratum.

The whole surface settlement curves can be obtained through FEM analyses, shown in figure 8. Based on these curves it can be concluded that, with the right tunnel finished, the excavation of the left tunnel (the curve B in figure 8) will generate more displacement than that caused by the excavation in green field (the curve A in figure 8). The disturbance due to the right tunnel construction lowers the stiffness of the rock around the left tunnel and when the left is constructed, the rock in the previously disturbed zone is disturbed again, and it will move more than what is expected. So, the existing of the right tunnel is undesirable for the left one. However, when there is the adjacent tunnel existing, additional ground surface settlement caused by right tunnel excavated (the curve D in figure 8) is less than that caused in green field (the curve C in figure 8). The existing of the left tunnel which is higher than the right one is an advantage for the right tunnel to a certain extent.

#### 3.2 Earth pressure

Earth pressure near the crown and the invert (R1 is above the crown and R2 is under the invert, refer to figure 4) of the right tunnel are given in figure 9. There is the same development rule for the earth pressure of R1 and R2; the pressure increase a bit due to the previous tunnel excavation, and decrease rapidly as excavation section passes the test section because of the load release caused by removing rock of the tunnel space.

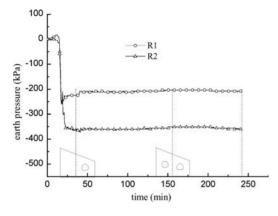


Figure 9. Earth pressure near crown and invert of right tunnel (obtained in model test).

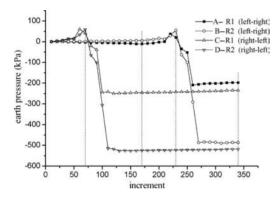


Figure 10. Earth pressure at crown and invert of the right tunnel.

The pressure change at R2 near the crown is bigger than that at R1 near the invert. The phenomena should be associated with the locations of two measuring points, the invert is deeper than the crown and initial stress near the invert is bigger than that near the crown. The same conclusion can be drawn by numerical simulation and the results are shown in figure 10. The stress changes caused by the excavation of the right tunnel in green field are more than that caused with the existing left tunnel. And it comes to opposite conclusion for the left tunnel. So, depending on the stress changes, the same conclusion can be drawn: the existing of the right tunnel is undesirable for the left one; and the existing left tunnel is beneficial to the right one.

#### 4 CONCLUSION

The reduced-scale laboratory tests and their corresponding numerical analyses were performed to investigate the transfer of the earth pressure and the displacement of the surrounding rock during the construction. At the same time the effect of different excavation sequences was analyzed.

The degree of the interaction between the two tunnels is proportional to the distance between them. The changes of stresses and displacements of the rocks along the tunnel are mainly affected by its excavation while the effect caused by the adjacent tunnel is small. The interaction effects on the two tunnels are different: the existing of the right tunnel deteriorates the properties of rocks around the left tunnel, which is harmful to the left tunnel; whereas the existing left tunnel reduces the asymmetric degree, and therefore benefits to the right tunnel.

Stress decreases rapidly with the excavation of the tunnel, which causes settlements of the surrounding rocks and consequently expands to surface.

Displacements of the rock increase gradually along with the excavation advancement, and the main portion occurs when the excavation section passes the test section. After the excavation finished, a creep displacement takes place, which accounts for about  $10\% \sim 30\%$  of the total displacement.

The difference of the excavation sequence causes the different development rule of the stress field and the displacement field. The values of displacements and stresses increase on adopting right tunnel excavated firstly. For the tunnels like the Pingnian tunnel, which is a shallow tunnel under unsymmetrical pressure with the vertical distance between the tunnels, it is profitable to excavate the left tunnel firstly.

#### REFERENCES

- Adachi, T., Kimura, M. & Osada, H. 1993. Interaction between multi-tunnels under construction. *Eleventh SoutheastAsian Geotechnical Conference, Singapore, 4–8 May 1993*: 51–60.
- Addenbrooke, T.I. 1996. Numerical analysis of tunnelling in stiff clay. Ph. Degree thesis, London: University of London.
- Chu, B.L., Hsu S.C., Chang Y.L. et al. 2006. Mechanical behavior of a twin-tunnel in multi-layered formations. *Tunnelling and Underground Space Technology*.
- Dhar, B.B., Ratan, S., Sharma, D.K. et al. 1981. Model study of fracture around underground excavations. *Proceedings* of the International Symposium on Weak Roc: 267–271.
- Ghaboussi, J. & Ranken, R. E. 1977. Interaction between two parallel tunnels. *International Journal for Numerical and Analytical Methods in Geomechanics*. 1:75–103.
- Ng, C.W.W., Lee, K.M. & Tang, D.K.W. 2004. Threedimensional numerical investigations of new Austrian tunnelling method(NATM)twin tunnel interactions. *Geotechnique*. 41:523–539.
- Soliman, E., Duddeck, H. & Ahrens, H. 1993. Two and three dimensional analysis of closely spaced double-tube tunnels. *Tunnelling and Underground Space Technology*. 8(1):13–18.

# Stability analysis of masonry of an old tunnel by numerical modelling and experimental design

# J. Idris & T. Verdel

Laego - Ecole des Mines de Nancy, Parc de Saurupt, Nancy Cedex, France

# M. Alhieb

Ineris – Ecole des Mines de Nancy, Parc de Saurupt, Nancy Cedex, France

ABSTRACT: The present paper proposes two numerical models of an old tunnel supported by masonry; these models were developed by the well-known Universal Distinct Element Code (UDEC). A masonry mechanical behaviour analysis and numerical simulation of the masonry ageing phenomenon will also be addressed by means of an experimental design to study the influence of masonry block physical properties on the mechanical behaviour of tunnel support structure.

# 1 INTRODUCTION

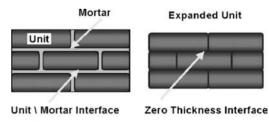
Masonry vaults are one of the most familiar structural shapes present in architectural heritage worldwide. Historical domed buildings arched stone bridges and vaulted tunnels are among the most famous examples.

Over the last few years, the development of numerical tools in the field of structural analysis has enabled researchers to establish approaches for numerically modelling masonry structures, yet an analysis of the mechanical behaviour of such blocks and joints structures remains challenging due to the extent of correlated factors. This paper seeks to study tunnels masonry behaviour used in old tunnels by numerical modelling and experience design.

# 2 MASONRY STRUCTURES NUMERICAL MODELLING APPROACHES

Several modelling approaches to masonry structures (continuous and discontinuous modelling) are currently under development by several research teams. A number of bridge arch models have been proposed to study the behaviour and certain instability problems, such as collapse (Ford et al., 2003; Sumon et al., 1995; Hughes et al., 1998; Bicanic et al., 1995; Brookes et al., 2004). Many historical masonry construction simulations and numerical analyses were presented by (Valluzzi et al., 2004; Lourenço, 2001; Giuriani et al., 2001; Bicanic et al., 2002). Yet no single publication actually focuses on or addresses the analysis of old tunnel masonry structures. Three basic modelling strategies for masonry structures can at present be identified (see figure 1):

- Detailed micro-modelling: blocks and mortar in the joints are represented as a continuum, whereas the unit/mortar interfaces are modelled by discontinuous elements;
- 2 Simplified micro-modelling: "geometricallyexpanded" continuum units, with discontinuum elements incorporating the behaviour of both mortar joints and interfaces;
- 3 Macro-modelling: all three principal features of structural masonry are represented by an equivalent continuum.



Homogenised continuum



Figure 1. Basic approaches to masonry structures.

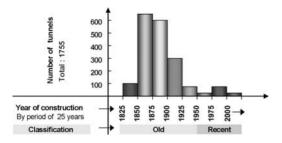


Figure 2. French tunnel classification (Idris et al., 2007).

# 3 TUNNELS AGEING PHENOMENA

Old underground structures, especially tunnels, display specific characteristics regarding behavioural evolution over time. The infrastructure environment, surrounding soils and construction materials used all contribute to this evolution. Consequently, several types of disorders may appear and develop in these underground structures due to specific ageing processes that, in reality, are complex phenomena. One impact is the alteration in mechanical properties of the various construction materials.

A majority of the world's tunnels are currently more than 100 years old; these would all be considered as ancient infrastructure. Figure 2 illustrates the classification of tunnels in France, where the oldest one exceeds 180 years.

The majority of old tunnels are supported by masonry elements. The type of masonry support or lining depends upon utilisation of the high compressive strength in the stones, which explains the vaulted section shape of old tunnels supported by masonry.

Apart from the environment and evolution in surrounding soil and in the absence of an effective isolation system for such underground structures, subsoil water can easily penetrate the masonry joints and circulate within.

Over time and in the presence of other aggressive ambient factors, several physical, chemical and biological processes may develop inside the masonry structure; this phenomenon and its impacts are collectively called the tunnel-ageing phenomenon.

As a result, various types of disorders appear inside old tunnels; these would include: longitudinal or transverse structural cracks, convergence and partial masonry collapse. Figure 3 provides some images of such disorders.

# 4 NUMERICAL MODELLING OF ANCIENT TUNNELS

A tunnel masonry structure is a discontinuous medium consisting of blocks bonded to each other by mortar;



Figure 3. Sample disorder types within old tunnels (CETU Tunnel Study Centre, 2004).

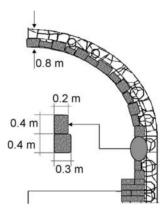


Figure 4. A typical old tunnel cross-section (extracted from Kerisel, 1975).

in addition, such a structure forms an interface with the surrounding soil. The Distinct Element Method (DEM) is a suitable technique for modelling these structures. By means of the Universal Distinct Element Code (UDEC), two simplified micro-models of an ancient tunnel have been derived. The geometric and geomechanical properties of both models were inspired from previous cases of ancient tunnels available in the "old tunnel" sub-database (Idris et al.2004, 2007) as well as from other bibliographical sources, including Kerisel, 1975 (see Figure 7, in which the thickness of the masonry support structure equals 80 cm).

The two representative models are positioned at a shallow depth of 20 m; they both display a vaulted section shape. In the first model, the masonry-supporting section consists of a regular rectangular and square limestone blocks (Fig. 5a). In the second, the masonry consists of two layers of limestone blocks: regular and irregular (Fig. 5b). Masonry blocks are bonded by a lime mortar. The masonry support thickness is 80 cm and the sidewall height in each model amounts to 3 m. All other geometric details are shown on Figure 5.

By taking into account model section symmetry, only half of each set-up needed to be modelled. Figure 6 shows the tunnel position within the

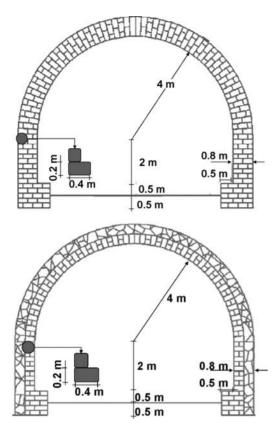


Figure 5. Two proposed models (a and b) for old tunnels.

surrounding soil, along with the selected boundary conditions, the chosen dummy joints and the mesh. Dummy joints were introduced in order to obtain a suitable grid around the excavated tunnel.

The soil surrounding the tunnel consists of a homogeneous mix of clay and sand. Table 1 lists the basic mechanical properties assigned to the surrounding soil, masonry and masonry joints, based on the work by (Verdel et al., 1999), (Hoek, 2000) and (Janssen, 1997).

Calculations were carried out in plane strain: the soil and masonry follow a perfect elasto-plastic Mohr-Coulomb plasticity criterion. The calculation step proceeded by two main stages: model consolidation in the initial stress condition prior to tunnel excavation; and tunnel excavation and simultaneous installation of masonry support. The calculation could then be continued until reaching model equilibrium.

# 5 MECHANICAL BEHAVIOUR ANALYSIS OF MASONRY BLOCKS

Figure 8 presents Mohr circles for every grid zone of the masonry block used in the initial model (according to which, every masonry block is divided into 2 or

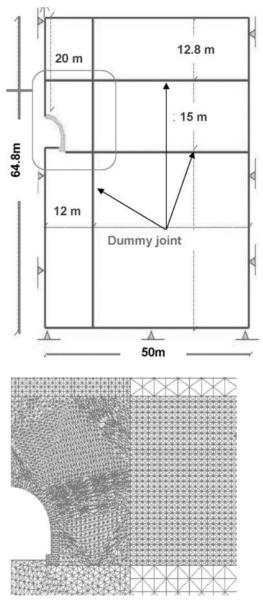


Figure 6. Tunnel position, boundary conditions and mesh.

4 grid zones). In the initial model, no block stress exceeds the Mohr-Coulomb failure criterion (as shown on Figure 7), and all masonry blocks display perfectly elastic behaviour. Based on the stress distribution of masonry blocks, we are able to approximate the critical values for masonry block cohesion, tensile strength and friction angle, all of which can modify the masonry behaviour from elastic to plastic.

This analytical approach has enabled us to define the variation range for these three parameters over

Table 1. Selected mechanical properties of the surrounding soil, masonry and masonry joints (M: Volumic mass; E: Young's modulus; v: Poisson's ratio; C: Cohesion;  $\varphi$ : Friction angle; Tr: Tensile strength Jkn, Jks: Normal, Tangential joint stiffness; JC: Joint cohesion; J $\varphi$ : Joint friction angle; JTr: Joint tensile strength).

Surrounding soil			Masonry		
Parameter	Unit	Value	Parameter	Unit	Value
М	Kg/m <sup>3</sup>	1900	М	Kg/m <sup>3</sup>	2100
E	MPa	200	Е	MPa	6000
ν		0.3	ν		0.2
С	MPa	0.50	С	MPa	3
φ	0	20	φ	0	30
Ťr	MPa	0.10	Ťr	MPa	1

Masonry joints

Parameter	Unit	Value	Parameter	Unit	Value
Jkn	GPa/m	150	$J\varphi$	0	25
Jks	GPa/m	69.7	JTr	MPa	0.4
JC	MPa	1.2			

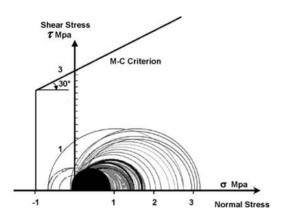


Figure 7. Mohr circles for masonry block grid zone stresses in the initial model.

which it is appropriate to study how the three chosen parameters influence the overall behaviour of masonry structures. As an example, the decrease in both cohesion and friction angle can serve to significantly increase the number of plastic zones within the masonry structure.

# 6 PROPOSED EXPERIMENTAL DESIGN FOR MASONRY SUPPORT AGEING SIMULATION

Construction materials are submitted to various modifications and degradations under several physical,

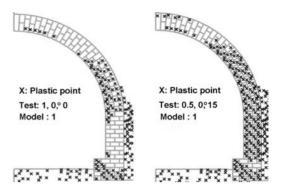


Figure 8. A sample of numerical simulation results for the first model (test, cohesion, tension and friction).

chemical and biological combined processes. A number of physical phenomena, such as dilation, contraction, freezing, hydration and desiccation cycles, potentially lead to a considerable evolution in material mechanical properties. Construction materials ageing processes detail were explained by (Idris et al. 2007).

The behaviour of masonry is dictated by the physical and mechanical properties of both blocks and joints. Our survey focused on evolution in the mechanical behaviour of masonry blocks due to ageing; special attention was therefore devoted to the parameters that define block failure, such as cohesion, tensile strength and friction angle.

To evaluate the influence of each chosen factor on masonry structure behaviour, it proved necessary to observe significant changes in model behaviour once factor values had been changed. The response factor selected herein is the number of blocks that change status from elastic to plastic behaviour.

For this purpose, a complete factorial design was proposed; this three-level design is written as  $K^n$  factorial design (with K = 3: the studied factor number, n: level number). This nomenclature means that 3 factors are considered, each one at 3 distinct levels (Barrentine, 1999).

Consequently, a complete factorial design with 27 experiments was proposed. Table 2 contains all of the experimental results (i.e. changed experimental factors and observed responses), and Figure 8 provides some of the results for both models.

We have to indicate that the obtained results from the two developed models are quite similar.

# 7 RESULTS ANALYSIS BY MULTIPLE LINEAR REGRESSION

The general purpose of a multiple regression analysis is to establish a relationship (criterion or model)

Table 2. Numerical experimental design results for masonry block parameters in the first developed model.

				DI C
Test (Number)	Cohesion (MPa)	Tensile strength (MPa)	Friction angle (°)	Plastic block (model 1) (Number)
1	1	1	30	1
2	1	1	15	9
3	1	1	0	24
4	1	0.5	30	1
5	1	0.5	15	9
6	1	0.5	0	24
7	1	0	30	16
8	1	0	15	22
9	1	0	0	55
10	0.5	1	30	29
11	0.5	1	15	76
12	0.5	1	0	112
13	0.5	0.5	30	29
14	0.5	0.5	15	77
15	0.5	0.5	0	112
16	0.5	0	30	46
17	0.5	0	15	96
18	0.5	0	0	116
19	0.2	1	30	116
20	0.2	1	15	116
21	0.2	1	0	116
22	0.2	0.5	30	116
23	0.2	0.5	15	116
24	0.2	0.5	0	116
25	0.2	0	30	116
26	0.2	0	15	116
27	0.2	0	0	116

between a dependent variable and one or more independent or predictor factors (Pedhazur, 1997). In our case, the dependent variable is the number of plastic masonry blocks *PN* while the exploratory variables are: masonry block cohesion (*C*), masonry block tensile strength (*Tr*), and masonry block friction angle ( $\varphi$ ), respectively. The general form of the adopted linear model including interaction terms is as follows:

$$PN = \beta_0 + \beta_1 C + \beta_2 Tr + \beta_3 \varphi + \beta_{12} (C, Tr) + \beta_{23} (Tr, \varphi) + \beta_{13} (C, \varphi) + \beta_{123} (C, Tr, \varphi)$$
(1)

Where *PN* is the dependent variable,  $\beta_0$  the constant and  $\beta_1$ ,  $\beta_2$ ,  $\beta_3$ ,  $\beta_{13}$ ,  $\beta_{23}$  and  $\beta_{123}$  the regression coefficients of the various terms to be solved by the regression technique.

The diagram in Figure 9 provides all of the calculated multiple regression factor coefficients, with which a multiple regression was applied to the standardised experimental design factors. The standardisation process changes all factor values over the interval [-1, +1]. This multiple regression was performed using the well–known Mathematica software.

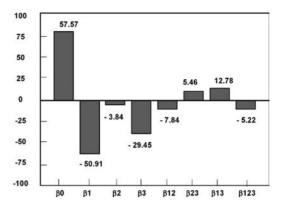


Figure 9. Calculated regression factor coefficients and their interactions (model 1).

The proposed linear model for the initial masonry block shape thus becomes:

$$PN = 57.57 - 50.91C - 3.84Tr - 29.45\varphi - 7.84(C, Tr) + 5.46(Tr, \varphi) + 12.78(C, \varphi) - 5.22(C, Tr, \varphi)$$
(2)

For the second shape of masonry blocks (model), the linear model becomes:

$$PN = 67.12 - 57.05C - 4.53Tr - 28.76\varphi - 9.34(C, Tr) + 7.03(Tr, \varphi) + 13.90(C, \varphi) - 2.32(C, Tr, \varphi)$$
(3)

These two linear models reveal that among the three studied factors, only cohesion ( $\beta_1$ ) and friction angle ( $\beta_3$ ) significantly influence masonry block mechanical behaviour, though the model does show that cohesion remains the single most significant factor of all those studied. The interaction coefficients indicate that only cohesion and friction angle exhibit a significant interaction capable of influencing the response.

# 8 CONCLUSION

In this paper, an experimental design was proposed to simulate ageing effects on old tunnel behaviour; a complete factorial experimental design, which expresses the evolution of three selected masonry mechanical properties, was then forwarded. The factors selected for the present study were: masonry block cohesion, tensile strength and friction angle.

All experimental design tests were modelled by means of the distinct element method, executed with the UDEC code.

Results analysis indicated that among the three studied masonry block mechanical factors, only cohesion and friction angle along with their interaction may exert a significant influence on masonry mechanical behaviour; furthermore, cohesion proved to be the most significant of all factors studied. The noninfluence of tensile strength factor is explained by the fact that the masonry structure is mainly loaded in compression due to its vaulted section shape.

For a typical model of an old tunnel, only three mechanical properties (factors) of masonry support in old tunnels were studied. It is important to point out however that several other mechanical properties of masonry must be taken into account in order to study their influence on the behaviour of older tunnel supports. Our subsequent research goal will focus on involving other masonry block and joint parameters in studies on old tunnel behaviour.

### REFERENCES

- Barrentine, L.B. 1999. An Introduction to Design of Experiments: A Simplified Approach. *Published by the American Society for Quality*, Chapter 3: experiments with three factors: 27–35.
- Brookes, C.L. & Mullett, P.J. 2004. Services load testing, numerical simulation and strengthening of masonry arch bridges, *CIMNE*, *Barcelona*: 10 pages.
- Bicanic, N., Stirling, C. & Pearce, C.J. 1995. Discontinuous modelling of masonry bridges, *Computational Mechanics*, V. 31: 293–314.
- Bicanic, N., Stirling, C. & Pearce, C.J. 2002. Discontinuous Modelling of Structural Masonry. *Fifth World Congress* on Computational Mechanics, Vienna, Austria: 18 pages.
- CETu (Centre d'Etude des Tunnels). 2004. Guide de l'inspection du génie civil des tunnels routiers, *Ministère de l'équipement, des transports se du logement direction des routes France*. ISBN: 2-11-084749-2 : 95 pages.
- Kerisel, J. 1975. Old structures in relation to soil conditions. *Geotechniques* 25, No. 3, p. 433–484.
- Lourenço, P.B. 2001. Analysis of historical constructions: From thrust-lines to advanced simulations, *Historical Constructions P. Roca (Eds.) Guimarãe* : 91–116.
- Ford, T.E., Augarde C.E. & Tuxford, S.S. 2003. Modelling masonry arch bridges using commercial finite element software, the 9th International Conference on Civil and Structural Engineering Computing, Netherlands: 20 pages.

- Giuriani, E., Gubana, A. & Arenghi, A. 2001. Structural rehabilitation of masonry vault, *Proceedings from the* UNESCO-ICOMOS Millennium Congress, Paris: 6 pages.
- Hoek, E. 2000. Practical Rock engineering, Chapter 11, Rock mass properties. rocscience.com on line: 161–203.
- Hughes, T.G. & Davies, A.W. 1998. The influence of soil and masonry type on the strength of masonry arch bridges, proc, sec, int. arch bridge conf., A. sinopoli Ed: 321–330.
- Idris, J., Verdel, T. & Al Heib, M. 2004 1. Feedback from a database created for reporting accidents in tunnels and galleries. Urban Transport X. Urban Transport and the Environment in the 21st century. Dresden, Germany, WIT press, ISBN 1-85312-716-7: 41–50.
- Idris, J., Al Heib, M. & Verdel, T. 2004 2. Base de données des accidents et des incidents survenus dans les ouvrages souterrains. *Tunnels et ouvrages souterrains*, No. 182: 363–368.
- Idris, J., Verdel, T. & Al Heib, M. 2007. Numerical modelling and mechanical behaviour analysis of ancient tunnel masonry structures. *Tunnelling and Underground SpaceTechnology*.(Article in press) :13 pages.
- Janssen, H.J.M. 1997. Structural Masonry. Structural Masonry, Numerical studies with UDEC. Centre for civil engineering research and codes. A.A. Balkema, Roterdam, Netherlands, ISBN 90 5410680 8: 96–106.
- Olofsson, T. 1985. A non-linear model for the mechanical behaviour of continuous rock joints. In Proc. of International Symposium on Fundamental of Rock Joints, Ed Stephansson O. Björkliden, Sweden: 395–404
- Pedhazur, E.J. 1997. Multiple regression in behavioral research, third edition, Chapter 5, Multiple regression, *New York: Harcourt Brace College* Publishers. ISBN 0-03-072831-2: 95–135.
- Sumon, S. K. & Ricketts, N. 1995. Strengthening of Masonry Arch Bridges. *Chapter in Arch Bridges*. Publ Thomas Tellord, London.
- Valluzzi, M.R., Binda, L. & Modena, C. 2004. Mechanical behaviour of historic masonry structures strengthened by bed joints structural repointing. *Construction and Building Materials* V.19: 63–73.
- Verdel, T. & Bigarre, P. 1999. Modélisation de tunnels anciens avec le logiciel UDEC, *Rapport INERIS, Société SIMECSOL*: 1–12.

# Excavation with stepped-twin retaining wall: Model tests and numerical simulations

N. Iwata, H.M. Shahin, F. Zhang, T. Nakai, M. Niinomi & Y.D.S. Geraldni Department of Civil Engineering, Nagoya Institute of Technology, Nagoya, Japan

ABSTRACT: Braced excavation using stepped-twin retaining wall is becoming popular in Japan. As it is a new technique used to prevent movements of double-elevated ground, mechanism of deformation due to excavation and change of stresses have not been fully understood. The design methodology of this technique is also not properly established. In this research, two-dimensional model tests are conducted to investigate the deformation mechanism of the ground and the earth pressure of the stepped-twin retaining wall. Numerical simulations with finite element method are also carried out for the same scale of the model tests. The aim of the research is to make clear the mechanism of the braced excavation using stepped-twin retaining wall and to establish an effective way to evaluating the mechanical behaviors of the retaining wall and the surrounding ground.

# 1 INTRODUCTION

In urban area, open excavation often cause problems to surrounding ground and adjacent structures. In practical daily design works, however, earth pressure is usually predicted by conventional methods such as a frame model together with Rankine's earth pressure theory. There is also no appropriate method to predict surface settlements of ground, which is usually predicted by empirical method and/or elastic finite element method. For braced excavation using stepped-twin retaining wall, conventional method cannot taken into consideration properly the influence of nearby structures as well as the construction sequence in evaluating the ground movements and the earth pressure.

In this research, two-dimensional (2D) model tests are conducted to investigate the deformation mechanism of the ground and the earth pressure of the stepped-twin retaining wall. Numerical simulations with finite element method are also carried out for the same scale of the model tests. In the finite element analyses, subloading tij model (Nakai & Hinokio 2004), is used in the analysis to model the ground material. This model can describe typical stress deformation and strength characteristics of soils such as the influence of intermediate principal stress, the influence of stress path dependency of plastic flow and the influence of density and/or confining pressure. Mass of aluminum rods is used in the model ground. Several patterns of the model tests are performed varying the length of the retaining wall and changing the distance

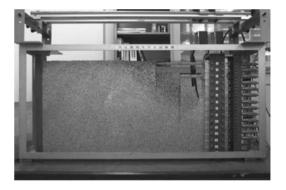


Figure 1. 2D Model test device.

between the two walls. The results of stepped-twin retaining wall are compared with the single retaining wall.

# 2 OUTLINE OF MODEL TESTS AND NUMRICAL SIMULATIONS

2D Model tests and the corresponding numerical simulations of braced excavation using stepped-twin retaining wall, were carried out to make clear the mechanical behavior the problem. Figure 1 shows the outline of the 2D Model test device. Four cases of model tests with different length of outer retaining wall and different spacing of stepped-twin retaining walls were considered, as shown in Figure 3.

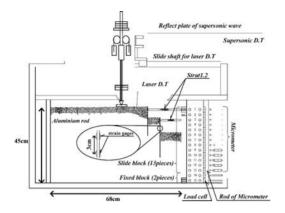


Figure 2. Outline of 2D Model test device.

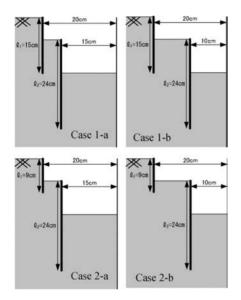


Figure 3. Cases of study.

Figure 2 shows the schematic diagram of the twodimensional apparatus. The size of the model ground is 68 cm in width and 45 cm in height. Aluminum rods of 5 cm in length, having diameters of 1.6 mm and 3.0 mm and mixed in the ratio of 3:2 in weight, are used as the model ground (unit weight of the mass is  $20.4 \text{ kN/m}^3$ ). In the experiment, the model ground was excavated with a thickness of 1.5 cm each time and two struts, located at the levels of -1.5 cm and -7.5 cm respectively, were set into place at the time when excavating level reached -1.5 cm below its position. The retaining walls, with different length and spacing, were set before the ground was excavated. Table 1 shows the material parameters of the model ground, the retaining wall (Aluminum plate) and the struts. A laser type

Table 1. Material Parameters.

Ground	Aluminum rods Unit weight $\gamma = 20.4 (\text{kN/m}^3)$
Retain wall	Aluminum plate EI = $0.88 \text{ (kN*m}^2/\text{cm)}$ EA = $4.22 \times 10^4 \text{ (kN/cm)}$
Strut	Upper: $k_I = 3.64 (kN/m/cm)$ Lower: $k_I = 4.13 (kN/m/cm)$

Table 2. Parameters of ground made of aluminum rod mass.

0.008	
	same parameters as
0.5	Cam-clay model
1.8 0.2	
1.2	shape of yield surface (same as original Cam-clay at $\beta = 1$ )
1300	influence of density and confining pressure
	0.004 0.3 1.8 0.2 1.2

displacement transducer is used to measure surface settlement of the ground. By taking photographs with a digital camera and using image processing of the photos, the distribution of movement and consequently the strain of the ground can be measured.

Numerical analyses are carried out with the same scale of the model test considering plane strain condition using isoparametric element. An elastoplastic constitutive model, named as subloading  $t_{ij}$  model (Nakai & Hinokio, 2004) is used in the finite element analyses to simulate the mechanical behaviors of the model ground. The model can take into consideration automatically the influence of the intermediate principal stress, by introducing a modified stress  $t_{ij}$ (Nakai and Mihara, 1984; Nakai and Matsuoka, 1986). Subloading surface concept proposed by Hashiguchi (1980) was also adopted in the model to consider the influence of density of ground materials. Detailed description about the performance and the reasoning of the model can be referred to aforementioned references.

Table 2 lists the parameters of model ground made of aluminum rod. Figure 4 shows the performance of the model. Figure 5 shows the finite element mesh of Case1-b. Smooth boundary condition is assumed for side boundaries, and the bottom of the meshes is kept fixed. The initial stresses of the ground are calculated by simulating the self-weight consolidation by applying body forces, starting from a negligible confining pressure ( $p_0 = 9.8 \times 10^{-6}$  kPa) and an initial void ratio

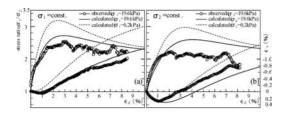


Figure 4. Stress-strain-dilatancy curves for aluminum rod mass.

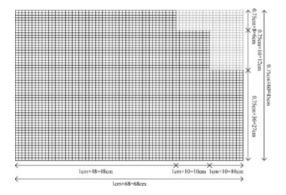


Figure 5. Finite element mesh (Case1-b).

e = 0.35. The retaining wall was simulated with beam element and the strut is simulated with spring element. Between the ground and the retaining wall, joint element whose mechanical behavior is simulated by a perfect-plastic joint elements (Nakai, 1985), was introduced to consider possible sliding between the ground and the wall.

# 3 RESULTS AND DISCUSSION

Figure 6 shows the observed surface settlements at different excavation stages in four cases. The length of outer retaining wall does not affect too much the settlement, while the spacing of twin walls has a great influence on the settlement. The shorter the spacing is, the larger the settlement will be. The same tendency can be obtained in the correspondent calculations, as shown in Figure 7.

Figures 8 and 9 show the observed and calculated deflections of the retaining walls during the excavation. Similar to the surface settlement, the main factor affecting the deflection is the spacing of twin walls.

From Figures 6 to 9, it is also known that the numerical calculation conducted in this paper can not only well describe the deformation patterns of the ground and the retaining wall qualitatively but also quantitatively to some extent.

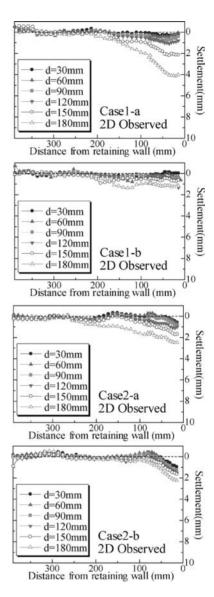


Figure 6. Surface settlements (Observed).

In the calculation, the frictional angle of joint elements which are used to simulate the friction between the ground and the wall is determined with constant normal stress frictional test and is found to be 17 degree. In the calculation, however, the displacement of the ground along the wall does not fit the observed one well. This is due to the fact that a perfect-plastic model is used for the joint elements which do not allow any elastic deformation before the joint element reaches yielding state. The reason why we do not use elasto-plastic model is that it is difficult to determine

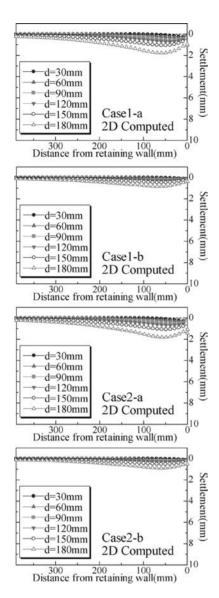


Figure 7. Surface settlements (Computed).

the shear stiffness of joint element. Further research needs to be down in future.

Figures 10 and 11 show the displacements of the ground surrounding the excavated area, obtained both from model test and numerical calculation. The deformation of the ground is limited to the area near the excavation. The calculated results agree well with the observed ones.

Figures 12 and 13 show the distribution of shear strain (The second invariant of strain tensor), obtained both from model test and numerical calculation. It is found that the shear strain of the ground is also limited

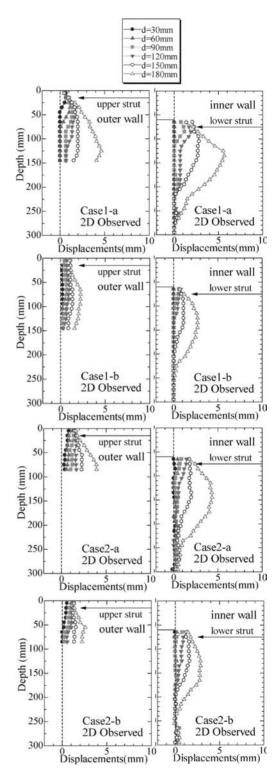


Figure 8. Displacements of retaining wall (Experiment).

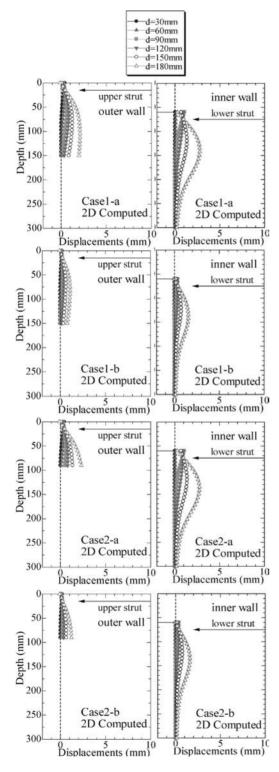


Figure 9. Displacements of retaining wall (Computed).

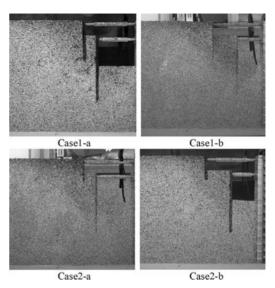


Figure 10. Displacement of g d = 180 mm).

ground (Observed,

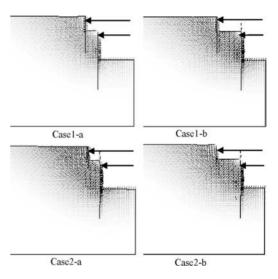


Figure 11. Displacement of ground (Computed, d = 180 mm).

to the area near the excavation. The calculated results agree well with the observed ones.

Figure 14 gives a comparison between the observed and the calculated results of axial forces within the struts at different excavating stages. Similar to the surface settlement, the main factor affecting the axial force is the spacing of twin walls instead of the length of the outer wall. The closer the spacing is, the higher the axial force of the second strut will be. The first

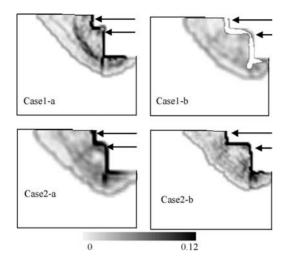


Figure 12. Distribution of shear strain (Observed, d = 180 mm).

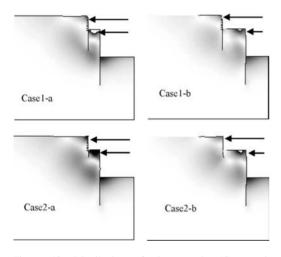


Figure 13. Distribution of shear strain (Computed, d = 180 mm).

strut, however, is not affected too much by these two factors, that is, the length of wall and the spacing. The numerical calculation can well describe the test results, both qualitatively and quantitatively.

Figure 15 gives a comparison of calculated change of earth pressures on retaining walls during excavation for the cases of 1-a and 1-b. It is found that the earth pressures on inner retaining wall do not show much evident difference. The outer wall, however, does behave quite differently, that is, the passive earth pressure on the bottom in case 1-a (narrow spacing) is much smaller than those in 1-b (wide spacing), implying that the ground between the twin wall cannot be expected to resist the deflection of the outer wall as what we

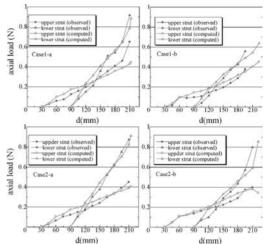


Figure 14. Comparison of axial forces in struts.

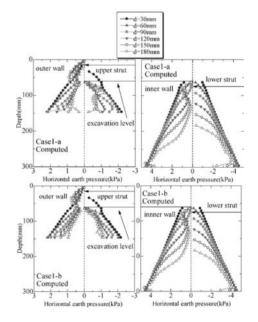


Figure 15. Change of earth pressures on retaining walls during excavation.

usually expected to be a passive earth pressure when the spacing of the two wall is enough narrow.

Figure 16 shows a comparison of calculated results of axial forces in struts in different excavation methods. In single-wall excavation, the axial force in upper strut increases firstly and then decreases after the lower strut comes into action. In twin-wall excavation, however, both the axial forces in upper and lower struts increase during the excavation. Meanwhile, the

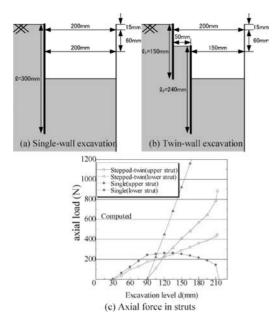


Figure 16. Comparison of different excavation methods.

axial force of lower strut increases much fast in twinwall excavation than those in single-wall excavation. Therefore the mechanism of twin-wall excavation and single-wall excavation is much different and should be considered carefully in daily design.

# 4 CONCLUSIONS

Laboratory model tests and the corresponding numerical simulations are conducted for investigating the deformation mechanism of the ground and earth pressure of the stepped-twin retaining wall. From this research the following points can be concluded:

- The displacements of the walls are inversely proportional to the distance between the walls.
- The surface settlement follows the same tendency of the wall displacements, and it is very much dependent on the distance between the two walls.
- 3. The distance between the walls is more important factor than the embedded length of the wall.
- Unlike the single retaining wall the struts of the stepped-twin retaining wall share axial load a more efficient way.

Finite element analysis conducted in this paper, which is based on subloading  $t_{ij}$  model and, is capable to describe the mechanical behaviors of the twin-wall excavation qualitatively and quantitatively.

## REFERENCES

- Hashiguchi, K. 1980. Constitutive equation of elastoplastic materials with elasto-plastic transition. *Jour. of Appli. Mech.* ASME 102(2): 266–272.
- Nakai, T. 1985. Finite element computations for active and passive earth pressure problems of retaining problems. *Soils and Foundations* 25(3): 98–112.
- Nakai, T. & Hinokio, M. 2004. A simple elastoplastic model for normally and over consolidated soils with unified material parameters. *Soils and Foundations* 44(2): 53–70.
- Nakai, T. & Matsuoka, H. 1986. A generalized elastoplastic constitutive model for clay in three-dimensional stresses. *Soils and Foundations* 26(3): 81–98.
- Nakai, T. & Mihara, Y. 1984. A new mechanical quantity for soils and its application to elastoplastic constitutive models. *Soils and Foundations* 24(2): 82–94.

# Stability of an underwater trench in marine clay under ocean wave impact

T. Kasper & P.G. Jackson COWI A/S, Kongens Lyngby, Denmark

ABSTRACT: A long section of the immersed tunnel of the Busan-Geoje Fixed Link in South Korea is to be constructed in a trench in soft marine clay. The site is exposed to large typhoon waves and the trench is to be left open for approximately one year. The trench profile was chosen as a balance between excavated volume (costs) and risk of slope instability under large waves. To evaluate this risk, far-shore wave data have been transformed into near-shore wave conditions by means of numerical modelling and wave flume tests have been carried. Potentially critical waves have been identified and their impact has been analysed by means of coupled hydro-mechanical numerical simulations. Based on the strength reduction method, safety factors for the slope stability during wave impact have been determined. The results and the consequences for the economic design of such a trench subject to large waves are discussed.

# 1 INTRODUCTION

The Busan-Geoje Fixed Link between South Korea's second largest city Busan and the island of Geoje consists of a 3.2 km long immersed tunnel (Odgaard et al. 2006), two rock tunnels and two 1.7 and 1.9 km long cable-stayed bridges. About 2.2 km of the immersed tunnel is constructed in a 12 to 15 m deep trench in soft marine clay at water depths between 20 and 50 m. The area is prone to frequent raids of typhoons and the trench is dredged approximately 1 year before construction of the tunnel. When the tunnel is placed in the trench, the lower half of the trench is filled with tunnel protection material (backfill). It was therefore necessary to design the trench profile as a balance between the excavated volume, i.e. construction costs on the one hand and the risk of slope failure under wave impact within a 1 years period on the other hand.

This paper discusses in Section 2, how the wave characteristics at the site have been determined and how the most critical waves with regard to trench slope stability have been identified. Based on the geotechnical characterization of the soft marine clay presented in Section 3, the impact of the critical waves on the trench is investigated by means of coupled hydro-mechanical numerical simulations (Section 4). Strength reduction analyses are made in the simulations in regular intervals of one second to determine the failure modes and factors of safety (Dawson et al. 1999). The last part of the paper contains a discussion of the results and general conclusions.

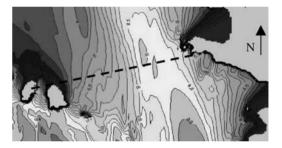


Figure 1. Numerical wave modelling: Calculated significant wave heights for a 10,000 years return period. The tunnel is indicated by the dashed line.

# 2 DERIVATION OF DESIGN WAVES

# 2.1 Numerical wave modelling

Based on a statistical analysis of wave and wind data in the area, the extreme far-shore boundary waves and extreme wind speeds have been derived. From these input data, the wave conditions at the location of the tunnel have been determined by means of numerical wave modelling with the program MIKE 21 (DHI Water & Environment).

The obtained maximum significant wave heights along the tunnel alignment are 6.2 m for a return period of 10 years, 8.0 m for a return period of 100 years and 9.2 m for the 10,000 years wave event (Figure 1). The large waves come from southerly direction with a wave velocity of approximately 13 m/s and pass the tunnel



Figure 2. Effect of typhoon waves on a breakwater in Busan (typhoon Maemi, 12.9.2003): (a) before and (b) after the storm.

Wave pressure profiles at seabed

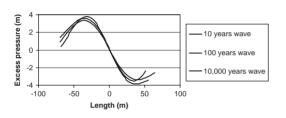


Figure 3. Most critical seabed pressure profiles for the trench stability determined from the wave flume tests.

trench almost perpendicularly. The potential devastating effect of such large waves is illustrated for a breakwater in Figure 2.

#### 2.2 Wave flume model tests

Based on the results of the numerical wave modelling, wave flume tests have been carried out for tunnel element TE7, which is subject to the highest waves at a relatively shallow water depth of 20 m. Each test consisted of a series of several hundreds of waves. From the pressure measurements in the tests, the seabed pressure profiles with the largest pressure difference across the 32 m wide trench slopes have been identified. These pressure profiles shown in Figure 3 are considered to be most critical for the slope stability. It should be noted that due to hydrodynamics, the seabed pressures under waves do not simply correspond to the physical wave height above.

#### **3 GEOTECHNICAL PROPERTIES**

Extensive ground investigations and laboratory tests have been carried out for the project (Steenfelt et al.

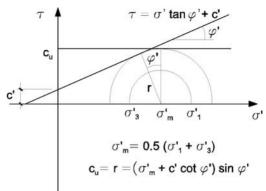


Figure 4. Representation of undrained strength in the numerical model with the Mohr-Coulomb model based on effective strength parameters.

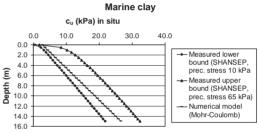


Figure 5. Comparison between measured and modelled undrained strength of the marine clay for in situ conditions.

2008). The undrained strength of the clay has been assessed from 30 cone penetration tests along the alignment and can be described according to the SHANSEP approach (Steenfelt & Foged 1992) as

$$\mathbf{c}_{u} = \mathbf{0.28} \, \boldsymbol{\sigma}_{vo}^{'} \left( \frac{\boldsymbol{\sigma}_{vo}^{'} + \Delta \boldsymbol{\sigma}}{\boldsymbol{\sigma}_{vo}^{'}} \right)^{0.76} \tag{1}$$

with the preconsolidation stress  $\Delta \sigma$  of the marine clay typically ranging between 10 and 30, max. 65 kPa. The effective strength parameters  $\varphi' = 25^\circ$ , c' = 3 kPa have been derived from 19 consolidated undrained triaxial tests. In the numerical model, the material strength is described with the Mohr-Coulomb model. Due to the fact that the clay is only slightly overconsolidated, the Mohr-Coulomb model can also be used without modification to correctly model the undrained strength in case of undrained conditions (Figure 4 and Figure 5).

The stiffness properties and permeabilities have been derived from oedometer tests. The material parameters are summarised in Table 1.

Table 1. Material parameters for the numerical simulations.

	Marine clay	Alluvium
Material model	Mohr-Coulomb	Mohr-Coulomb
$\gamma_{\rm sat}~({\rm kN/m^3})$	14.7	20
K <sub>0</sub> (-)	0.6	0.426
E (kPa)	8000	50000
ν(-)	0.2	0.25
$\varphi'(^{\circ})$	25	35
c'(kPa)	3	0
k (m/s)	$1 \cdot 10^{-9}$	$1 \cdot 10^{-5}$

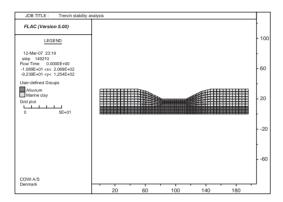


Figure 6. FLAC model of the trench.

# 4 NUMERICAL SIMULATION OF WAVE IMPACT

#### 4.1 Model description

The effect of waves on the trench is modelled by means of numerical simulations using the finite difference program FLAC (Itasca 2005). The two-dimensional, plane-strain model is shown in Figure 6.

The vertical boundaries of the model are fixed in horizontal direction, while the bottom of the model is fixed both in horizontal and vertical direction. The vertical boundaries and the bottom are assumed to be impermeable.

Soil stresses and pore water pressures are initialised according to the self-weight of the soils,  $K_0 = 1 - \sin \varphi'$  and a still water table at + 63 m model coordinate (still water pressures  $p_{still}$ ).

In a first step, the trench excavation is modelled either as a drained or undrained process. This is done to allow for modelling of wave impact shortly after trench excavation (undrained excavation modelling) and wave impact 1 year after trench excavation, where the excess pore pressures in the clay in the relevant area next to the trench slopes will almost completely be dissipated (drained excavation modelling).

Afterwards, wave impact is modelled by means of coupled hydro-mechanical analyses with transient

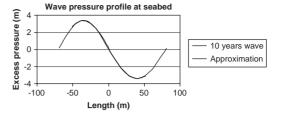


Figure 7. Approximation of the 10 years wave pressure profile in the FLAC model ( $A_1 = 3.4 \text{ m}$ ,  $L_1 = 70 \text{ m}$ ,  $A_2 = 3.4 \text{ m}$ ,  $L_2 = 80 \text{ m}$ ).

hydraulic and mechanical boundary conditions along the seabed and the trench surfaces. The wave pressure profile is described by means of two sine curves, one for the wave crest and one for the wave trough. This allows to consider different shapes of the wave crest and wave trough according to

$$p = A_1 \sin\left(\frac{\pi}{L_1}x\right) + p_{still}$$
 (wave crest) (2)

$$p = -A_2 \sin\left(\frac{\pi}{L_2}x\right) + p_{still}$$
 (wave trough) (3)

The described approach is able to model the wave pressure profiles quite accurately, as illustrated for the 10 years wave in Figure 7.

Based on Eqs. (2) and (3), the wave pressures are evaluated for each point in time at each gridpoint along the seabed and the trench surfaces and the corresponding total normal stress and pore water pressure boundary conditions

$$\sigma_n = -p \quad \text{and} \quad p_w = p \tag{4}$$

are applied. It should be noted that water pressures are defined positive, while compressive stresses are defined negative. According to the principle of effective stresses, these boundary conditions imply that the effective normal stresses at the seabed and the trench surfaces remain 0. Based on a linear variation of the boundary conditions along each zone edge, the whole wave pressure profile is approximated as a piecewise linear function with the correct values at each gridpoint. The model is defined such that a wave train with an arbitrary number of waves can be simulated. As a compromise between computational effort and concern about a possible change in safeties from wave to wave, a wave train with 2 waves is considered in the basic simulations. Due to the fact that each of the investigated large waves passes the trench within a period of approximately 6 seconds, the simplified assumption of quasi-static conditions in the simulation appears to be adequate. The compressibility of the pore water is

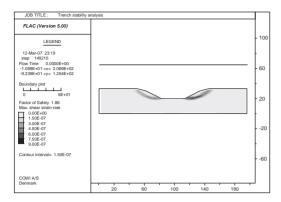


Figure 8. Predicted failure mechanism without wave impact after full consolidation of the trench excavation (drained conditions): Safety factor 1.86.

taken into account by a bulk modulus  $K_w = 2 \cdot 10^9$  Pa (pure water), while the compressibility of the soil grains is neglected. Failure mechanisms and safety factors are determined in time intervals of 1 second during wave impact by means of strength reduction analyses (Dawson et al. 1999).

#### 4.2 Results

Figure 8 shows the result of a safety analysis without wave impact after full consolidation of the trench excavation. The failure mechanism is illustrated by means of maximum shear strain rates at the end of the strength reduction analysis.

The FLAC result can be verified by comparison with other solutions (Table 2 and Figure 9). The drained results can be compared with a traditional limit equilibrium solution and a finite element stress based solution obtained with the program SLOPE/W (GEO-SLOPE) as well as with a strength reduction analysis with the finite element program PLAXIS (Brinkgreve & Bakker 1991). In the case of undrained conditions, a FE calculation is needed with all programs in order to correctly predict the effective stresses and excess pore pressures. The shapes of the slip surfaces and the safety factors show satisfying agreement in all cases. It is obvious that the safety factors decrease with consolidation. The reason is that the excavation represents an unloading process leading to a time-dependent reduction in effective vertical stresses, which are crucial for the soil strength. According to the Mohr-Coulomb failure criterion the shear strength is linearly dependent on the effective normal stress. It is estimated that after one year, the conditions in the clay in the critical area close to the slope surfaces may be nearly drained.

Figure 10 shows the evolution of the computed safety factors during wave impact after full consolidation of the trench excavation. It can be observed that

Table 2. Comparison of safety factors without wave impact.

	FLAC	GEO-SLOPE	PLAXIS
Undrained excavation	2.67	2.62 <sup>2</sup>	2.64
Drained excavation	1.86	1.83 <sup>1</sup> , 1.83 <sup>2</sup>	1.87

<sup>1</sup> General limit equilibrium solution

<sup>2</sup> Solution based on finite element stresses

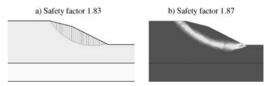


Figure 9. Illustration of results for drained conditions: (a) Traditional limit equilibrium solution (SLOPE/W), (b) Finite element strength reduction solution (PLAXIS).

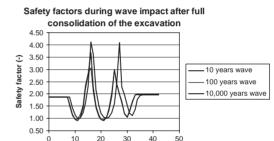


Figure 10. Computed safety factors during wave impact after full consolidation of the trench excavation.

Time (s)

the safety factors show a large variation in the range between 0.91 and 4.12 during passage of the waves.

The safety factors exceed 1.86 obtained for still water when a wave crest is above the trench. Then, increased pressures act along the bottom of the trench and decrease along the trench slopes towards the crown points of the trench (Figure 11). This pressure distribution in the trench counteracts the destabilizing gravity forces, thus leading to increased safety factors of up to 4.12.

On the other hand, a wave trough above the trench causes high pressures on the seabed beside the trench which decrease down along the trench slopes (Figure 12). This pressure distribution adds to the gravity forces, thus leading to decreased safety factors of down to 0.91. The prediction of safety factors smaller than 1 for the 100 and 10,000 years wave has been facilitated by scaling the clay strength (tan  $\varphi'$  and c') up and scaling the obtained safety factors down.

According to Table 3, the difference in minimum safety factors between the 10, 100 and 10,000 years

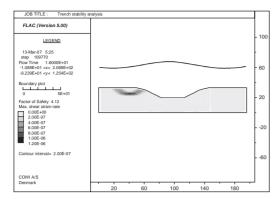


Figure 11. 10 years wave, t = 16 s: Illustration of wave pressure profile and predicted failure mechanism with the largest safety factor of 4.12 for wave impact after full consolidation of the trench excavation.

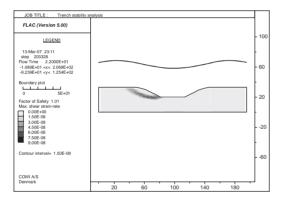


Figure 12. 10 years wave, t = 22 s: Illustration of wave pressure profile and predicted failure mechanism with the smallest safety factor of 1.01 for wave impact after full consolidation of the trench excavation.

Table 3. Summary of minimum safety factors.

	Wave impact shortly after trench excavation	Wave impact after full consolidation of the trench excavation
10 years wave	1.38	1.01
100 years wave	1.29	0.95
10,000 years wave	1.24	0.91

waves is relatively small and the safety factors for wave impact after full consolidation of the trench excavation are approximately 36% lower than the corresponding safety factors for wave impact shortly after trench excavation.

Figure 10 shows the safety factors for wave impact after full consolidation of the excavation (drained modelling of the excavation). Graphs with the same general pattern, but with larger safety factors are obtained for wave impact shortly after excavation of the trench (undrained modelling of the excavation).

It may be concluded from the calculated safety factors that failure of the slopes may occur if these extreme waves would pass the trench after 1 year consolidation. However, it can be argued that sliding of slopes is a dynamic process which takes place in the range of at least several seconds. Figure 11 and Figure 12 illustrate that the maximum and minimum safety factors are only a few seconds apart. Even if the exceptional 100 and 10,000 years waves would pass the open trench after 1 year consolidation, larger sliding movements may be prevented by the fact that critical safety factors occur only within about 1 or 2 seconds. Although water wave impact is a slower phenomenon, the situation may be compared with earthquake impact where the exceedance of yield accelerations in shorter periods induces some irreversible plastic deformations which do not necessarily lead to complete failure. An estimate of expected sliding movements during wave impact could be obtained e.g. by means of dynamic coupled hydro-mechanical simulations with the dynamic version of FLAC.

A simulation of the 10 years wave event with  $K_w = 2 \cdot 10^7$  Pa (representing pore water with a high content of dissolved/entrapped air) yields a minimum safety factor of 0.99 for wave impact after full consolidation of the excavation and 1.35 for wave impact shortly after excavation of the trench. These safeties for highly compressible pore water are slightly smaller than the corresponding safeties of 1.01 and 1.38 for the basic case. The influence of pore water compressibility on the behaviour of different soil structures under draw down and wave impact has been investigated in detail e.g. by Köhler (2000).

No perceptible change in safety factors can be observed between the first and second wave in the simulations (Figure 10). This has further been confirmed by a simulation with a wave train of 6 waves.

## 5 DISCUSSION AND CONCLUSIONS

Due to frequent raids of typhoons in the area, it was necessary to consider the possible effect of large waves in the design of the 2.2 km long and 12 to 15 m deep trench for the immersed tunnel of the Busan – Geoje Fixed Link in South Korea. Safety factors for the trench stability under extreme waves with return periods of 10, 100 and 10,000 years have been determined by means of numerical simulations. The minimum values have been found to be between 1.24 (10,000 years return period) and 1.38 (10 years return period) for wave impact shortly after trench excavation and between 0.91 (10,000 years return period) and 1.01

(10 years return period) for wave impact at the end of the construction period (after consolidation of the trench excavation). These safety factors have been considered to be acceptable for the following reasons:

The investigated situation is a temporary construction phase, where the trench is open for about 1 year. Under these circumstances, the 10 years wave event is considered to be the relevant design case.

Although the minimum safety factors for the 100 and 10,000 years waves are slightly below 1 (Table 3), failure is unlikely even under these extreme waves due to the fact that the minimum safety factors occur within only 1 or 2 seconds (Figure 10). This period is considered to be too short for complete failure to occur. High strain rates would be required for failure to develop and it is a well-established fact that the strength of clay is actually higher for high strain rates. Minimum and maximum safety factors are only a few seconds apart (Figure 10).

Failure of the trench slopes represents a financial risk and would require a clean-up afterwards, but would neither affect lifes nor the technical success of the project. The costs for excavation of a trench in offshore conditions at water depths between 20 and 50 m are high and should be limited to a minimum.

#### 6 PROJECT PROGRESS

The excavation of the trench began in July 2006 and was finished in September 2006. Busan only experienced a minor tropical storm in summer 2006. The production of the soil improvement (cement deep mixing) of the marine clay in the trench for the foundation of the tunnel began in November 2006 and has been finished in May 2007. The offshore construction of the tunnel is scheduled to start at the end of 2007 and finish in 2010.

#### ACKNOWLEDGEMENTS

The authors gratefully acknowledge the permission from Daewoo Engineering & Construction to publish this paper.

#### REFERENCES

- Brinkgreve, R.B.J. & Bakker, H.L. 1991. Non-linear finite element analysis of safety factors. In G. Beer, J.R. Booker & J.P. Carter (eds), *Computer Methods* and Advances in Geomechanics: 1117–1122. Rotterdam: Balkema.
- Dawson, E.M., Roth, W.H. & Drescher, A. 1999. Slope stability analysis by strength reduction. *Géotechnique* 49: 835–840.
- DHI Water & Environment, Denmark. MIKE 21. www. dhisoftware.com/mike21/
- GEO-SLOPE International, Ltd. GeoStudio 2004, version 6.19. www.geo-slope.com.
- Itasca Consulting Group, Inc. 2005. FLAC Fast Lagrangian Analysis of Continua, version 5.0. www.itascacg.com.
- Köhler, H.J. 2000. Pressure spreading at soil water interfaces and its influence on soil structure design. In A. Cancelli et al. (eds), EUROGEO2000; Proc. Second European Geosynthetics Conference, Bologna, 15–18 October 2000: Vol. 2, 687–694. Bologna: Patron Editore.
- Odgaard, S.S., Jensen, O.P., Kasper, T., Yoon, Y.H., Chang, Y., & Park, R.Y. 2006. Design of long immersed tunnel for highway in offshore conditions – Busan – Geoje Fixed Link. *Tunnelling and Underground Space Technology* 21(3–4), Special Issue: Safety in Underground Space (CD-ROM Proceedings of the ITA-AITES 2006 World Tunnel Congress, Seoul, Korea).
- PLAXIS b.v. PLAXIS version 8.5. www.plaxis.nl.
- Steenfelt, J.S. & Foged, N. 1992. Clay till strength SHANSEP and CSSM. NGM-92, 81–86.
- Steenfelt, J.S., Jackson, P.G., Christensen, C.T., Lee, J-S., & Ha, Y.-B. 2008. Ground investigations for the Busan – Geoje Immersed Tunnel. Submitted to 3rd International Conference on Site Characterisation, ISC'3, Taipei, 2008.
- Zienkiewicz, O.C., Humpheson, C. & Lewis, R.W. 1975. Associated and non-associated visco-plasticity and plasticity in soil mechanics. *Géotechnique* 25(4): 671–689.

### A study on behavior of 2-arch tunnel by a large model experiment

S.D. Lee

Department of Civil Engineering, Ajou University, Korea

K.H. Jeong, J.W. Yang & J.H. Choi Dodam Engineering and Construction Co., Korea

ABSTRACT: It is tendency that the parallel tunnels are constructed close to each other in order to diminish civil complaints and environmental damage. The 2-Arch tunnel is similar to two parallel tunnels with very short centre-to-centre distance. Recently, construction of 2-Arch tunnel is increasing. However, it is executing without enough studies for behavior of the 2-Arch tunnel. In this research, a study for behavior of 2-Arch tunnel is examined using large model test machine. At first we make the model ground with horizontal joint-set. Then embody in-situ stress by applying pressure to boundary of model ground. Then excavate model ground according to construction steps of the 2-Arch tunnel. As a result, during excavation of pilot tunnel, measured ground displacements are about  $40 \sim 50\%$  of whole displacement, which is concentrated in 0.25D (D: diameter of tunnel) region around tunnel. Height of loosen area by construction of the 2-Arch tunnel is 0.15w (w:centre-to-centre distance between left and right tunnel). These results are compared with DEM to conform the reliability of results.

#### 1 INTRODUCTION

The 2-Arch tunnel is similar to two parallel tunnels with very short centre-to-centre distance. The 2-Arch tunnel has been increasing in order to diminish environmental damage and ensures linking with structure close by tunnel. There have been a series of researches on the behavior of 2-Arch tunnel in soil and weathered rock. However, the use of 2-Arch tunnel in korea has been rapidly increasing in hard rock. So, researches of 2-Arch tunnel in hard rock are urgently needed. The stability of center upper part of 2-Arch tunnel is weak comparing with other parallel tunnels; therefore it is necessary to install pillar and secure structural stability. In 2-Arch tunnel, mutual effect of precedence and afterward tunnel is bigger than general parallel tunnel. The stability of pillar and mutual effect are very important in research of the 2-Arch tunnel.

Therefore, this study is conducted to model characteristic of sedimentary rock that similar with ground condition of prototype tunnel. Step in different stage displacement tendency, precedence tunnel stress transition by excavate afterward tunnel, pillar and lining behavior examine by achieving an experiment according to carrying out construction steps of 2-Arch tunnel. With this, compare with numerical analysis (DEM) in same condition and confirm the reliability of results.

#### 2 SUMMARY OF EXPERIMENT

#### 2.1 The model package

The dimensions of the model ground were 3 m  $\times$  3 m  $\times$  0.27 m (B  $\times$  H  $\times$  T). The main characteristic of the test machine is application of both the vertical and lateral loading system to embody in-situ stress.

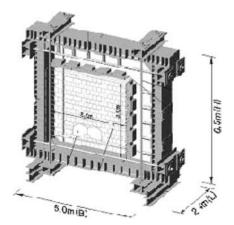


Figure 1. Large model test machine.

Table 1.	Property of	mode	l ground.
----------	-------------	------	-----------

Block					Joint su	rface
E (MPa)	ν	γ (KN/m <sup>3</sup> )	C (MPa)	φ (°)	C (MPa)	φ (°)
980	0.25	19.8	2.55	35	0	32

The inside of the model container was coated with grease to reduce side friction so that shear stress transfer at the ground and the container interface could be minimized.

#### 2.2 The model ground

It has been well known that rock mass is not homogeneous and exists discontinuities. The behaviour of tunnels and surrounding ground are heavily dependent on the characteristic of the rock mass. In this study to model characteristic of sedimentary rock, the model ground was made of a number of concrete bricks. The model ground was classified by Rock Mass Rating (RMR) used in tunnel design in Korea widely. The model ground used in the model tests had slightly rough surface with aperture thickness less than 1 mm. Overall the model ground had total RMR rating of 67 and hence the model ground was classified as class number II, i.e., Good rock.

# 2.3 Tunnel cross section and stiffness reduction rate

There is a problem in problem in passing of equipment and construct of pillar, because center tunnel is small and narrow (Fig. 2). Therefore, an experiment achieved into improved 2-Arch tunnel section that considers equipment exit and entrance center tunnel of approximately enlarge, and improve waterproof sheet establishment location and carrying out order (Fig. 3).

Usually, the property of the material and size are decided by through the suitable stiffness reduction rate when achieve a model experiment. The property of model tunnel was decided by the method of Duddeck and Erdmann (1985) that use stiffness ratio ( $\alpha$ ) of lining. Actuality stiffness ratio of research object tunnel and model tunnel depended on stiffness of each ground and property of tunnel lining. Those are expressed with below way. The reduction rate applications are same with table 2.

Stiffness ratio of prototype tunnel :  $\alpha = (E_k \cdot R^3)/(E_b \cdot I_b)$ Stiffness ratio of model tunnel :  $\alpha = (E_{km} \cdot R_m^3)/(E_{bm} \cdot I_{bm})$ 

 $E_k(E_{km})$ : Modulus of elasticity of ground (MPa) R(R<sub>m</sub>): Diameter of prototype(model) tunnel (cm)

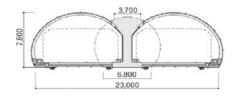


Figure 2. The exist 2-Arch tunnel cross-section.

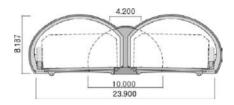


Figure 3. Improved 2-Arch tunnel cross-section.

Table 2. Application of reduction ratio.

Tunnel	E(ground) (MPa)	E(lining) (MPa)	Diameter (m)	Thickness lining (cm)
Prototype tunnel	9.8*10 <sup>3</sup>	1.96*10 <sup>5</sup>	23	40
Model tunnel	9.8*10 <sup>3</sup>	1.96*10 <sup>5</sup>	1.2	0.6

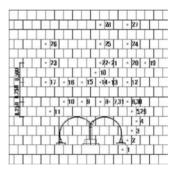


Figure 4. Measure point of ground displacement.

 $E_b(E_{bm})$ : Modulus of elasticity of lining (MPa)  $I_b(I_{bm})$ : Second moment of area lining  $t(t_m)$ : Thickness of prototype(model) tunnel lining (cm)

#### 2.4 Instrumentation

In the model test the behaviour of the tunnel and the ground due to excavation was monitored using LVDT (Linear Variable Displacement Transducer, ground displacements) and load cell (pillar load). The locations of LVDTs are shown in Figure 4.



1) Make a ground



2) Exca-pilot tunnel



3) Exca-upper part of the precedence tunnel.



4) Exca-upper part of the afterward tunnel.



5) Completion of the tunnel excavation.

Figure 5. The test procedures.

#### 2.5 The center pillar

In order to measure the rock load acting on the centre pillar, load cell was installed prior to the formation of the model ground. In the real tunnel excavation, the centre pillar is to be installed after completion of the pilot tunnel. However, to measure loosening load during the pilot tunnel excavation the centre pillar was installed prior to the assembly of the model ground. Thereafter the pillar load has been arranged to zero to measure rock load, exclusively associated with the main tunnel excavation.

#### 2.6 Experiment condition and method

The model test was conducted under a uniform surface loading of 980 kPa. An achieved experiment considers actual construction step as follows.

- 1. Made up horizontality being stratiform rock mass by the use concrete block of fixed size.
- 2. The model test was conducted under a uniform surface loading of 980 kPa with lateral earth pressure of 1470 kPa and hence the earth pressure coefficient was 1.5.
- 3. Excavation of upper part of the pilot tunnel.
- 4. Excavation of lower part of the pilot tunnel.
- 5. Arrangement of load cell reading to zero.
- 6. Excavation of precedence tunnel.
- 7. Excavation of afterward tunnel.

#### **3 THE RESULTS OF THE EXPERIMENT**

This research is that investigate behavior of 2-Arch tunnel that is conducted through a large model test and DEM analysis from the stratified rock. The main examination contents are same as follows.

- 1. Displacement tendency at different steps
- 2. Precedence tunnel stress transition by afterward tunnel excavation
- 3. Estimate of behavior of center pillar and loosen area rock load.

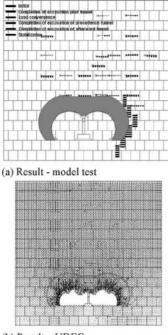
#### 3.1 Displacement tendency of different steps

The ground displacement is the main factor for tunnel and adjacent structure's stability judgment by tunnel excavation. The displacements that appear in experiment results are compared with numerical analysis (DEM) in equal condition and analyzed. Boundary condition of this numerical analysis set limits to horizontality displacement in side wall and lower part set limits to perpendicular displacement. Input data of numerical analysis is decided by axial compression test, direct shear test and RMR value of model ground.

#### 3.1.1 Displacement concentration region

In the model test, most of the ground displacement was examined by large model test and numerical analysis occurred within 0.25D. Experiment consequences and numerical analysis consequences of displacement were variance more or less.

This is judged by joint action that is happen vertically joint between block and block that can produce cause and experiment special quality upper that did



(b) Result - UDEC

Figure 6. Displacement concentration region.

not accord with correctly properties value of actuality model ground in numerical analysis.

But, a model experiment and numerical analysis result of displacement concentration extent and tendency are similar, therefore in identical branch result was judged that it is meaning that compare mutual and analyzed displacement to different steps.

#### 3.1.2 Horizontal displacement of different steps

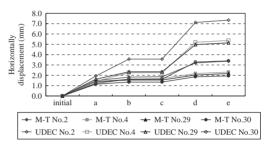
In order to evaluate stability of the side wall, the lateral ground displacements at the right part of the tunnel have been monitored using LVDTs. The direction of displacement is positive to tunnel and displacement of opposite direction is negative.

- a. Excavation upper part of the pilot tunnel.
- b. Excavation lower part of the pilot tunnel.
- c. Installation of pillar.
- d. Excavation of the precedence tunnel.
- e. Excavation of the afterward tunnel.

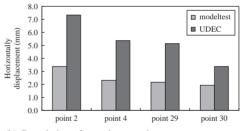
During excavate to pilot tunnel, most of displacement occurred. In general the stress condition of the ground after tunnel excavation may be different from the in-situ condition. In particular, direction and magnitude of the principle stresses will be changed due to stress re-distribution through the arching effect, hence the measured results are indicate that majority of the

Table 3. H-displacement of different steps (mm).

		Horizontal displacement (mm)					
Measure point		a	b	c	D	e	
Large model test	2 4 29 30	1.49 1.36 1.4 1.14	1.83 1.62 1.54 1.33	1.85 1.69 1.55 1.33	3.21 2.18 2.02 1.87	3.38 2.31 2.17 1.94	
UDEC	2 4 29 30	1.943 1.423 1.617 1.231	3.578 2.254 2.344 1.618	3.577 2.254 2.344 1.618	7.127 5.203 4.984 3.280	7.349 5.382 5.141 3.387	



(a) H-displacement of different steps



(b) Completion of tunnel excavation

Figure 7. H-displacement.

stress change occurred during stage b after the stress condition may not be changed much. Also, The measured results show that the longer the distance from the tunnel base the smaller the lateral ground displacement.(measure point 2,4's horizontal displacement) > measure point 29,30's horizontal displacement). Overall the measurements suggest that the stability of the 2-arch tunnel depends mainly on the excavation of the pilot tunnel. Therefore, the pilot tunnel should be stabilized prior to excavation of the other part of the tunnel. A large model experiment and numerical analysis result is same with table 3.

#### 3.1.3 Vertical displacement of different steps

In order to measure vertical ground displacements during the tunnel excavation at different steps, ground

Table 4. V-displacement of different steps (mm) - M.T.

Mea poir	asure nt	а	b	c	d	e
7	0.25D	-1.05	-1.53	-1.55	-1.84	-2.33
13	0.50D	-0.82	-1.21	-1.16	-1.38	-1.89
8	0.25D	-1.24	-1.66	-1.67	-2.01	-2.99
14	0.50D	-0.99	-1.25	-1.35	-1.59	-2.38
9	0.25D	-1.32	-1.72	-1.74	-2.52	-2.74
15	0.50D	-0.99	-1.31	-1.39	-1.94	-2.11
10	0.25D	-1.2	-1.59	-1.59	-1.94	-2.21
16	0.50D	-0.86	-1.24	-1.3	-1.51	-1.81

Table 5. V-displacement of different steps (mm) - UDEC.

Mea poir	isure it	а	b	c	d	e
7	0.25D	-0.95	-1.44	-1.45	-1.75	-4.39
13	0.50D	-0.95	-1.63	-1.63	-1.95	-4.26
8	0.25D	-1.43	-2.01	-2.01	-2.60	-5.89
14	0.50D	-1.29	-2.05	-2.05	-2.67	-5.41
9	0.25D	-1.8	-2.43	-2.43	-3.99	-6.14
15	0.50D	-1.43	-2.22	-2.22	-3.66	-5.63
10	0.25D	-0.95	-1.44	-1.44	-3.26	-4.09
16	0.50D	-0.97	-1.65	-1.66	-3.09	-4.05

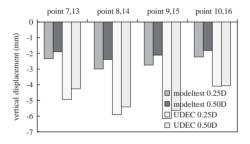
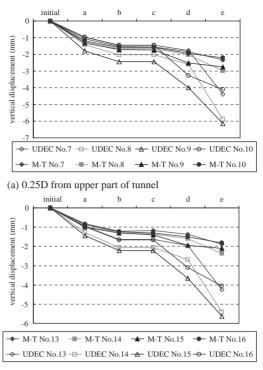


Figure 8. V-displacement (completion of excavation).

displacements at 0.25D and 0.5D above the tunnel crown have been monitored (0.25D: points 7-100.5D: points 13-16), where D is the width of the tunnel (Table 4).

The majority of the displacements occurred at excavation of pilot tunnel after insignificant changes of the ground displacement had developed. The Vertical displacement in 0.25D was observed greatly 120% when compare with displacement in 0.5D and most displacement was concentrated on 0.25D extent.

Horizontal displacements more than 40% of whole displacements and vertical displacements more than 20% are occurred during excavation of pilot tunnel. The displacement of model test is larger than numerical analysis result (Table 5) at pilot tunnel excavation steps. The displacement was concentrated on



(b) 0.5D from upper part of tunnel

Figure 9. V-displacement of different stages.

0.25D extent. Such result means that stability of whole tunnel is dominated by stability of pilot tunnel excavation in 2-Arch tunnel. This suggests that rock bolt length should be longer than 0.25D to prevent the rock loosening.

# 3.2 Precedence tunnel stress transition by afterward tunnel excavation

Precedence tunnel stability by afterward tunnel excavation is main concerns to establishment 2-arch tunnel that parallel tunnel is very near. In the case of precedence tunnel, displacement is converged at completion of excavation and establishment of support. But ground displacement and additional loading to support is increased by excavation of afterward tunnel. Additional load that happen to precedence tunnel at afterward tunnel excavate is  $10 \sim 30\%$  of whole load to the tunnel. (Lee and kim – 2001, The consideration of improve 2-Arch tunnel Design and construction method).

At the large model test result, displacement of precedence tunnel is occurred  $8 \sim 12\%$  of whole displacement by excavation of afterward tunnel. And  $20 \sim 35\%$  of UDEC analysis result increased at the same point. It is considered similar result if take into

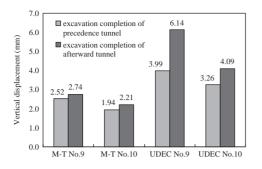


Figure 10. Displacement of precedence tunnel by excavation of afterward tunnel.

account difference in behavior of joint that happen in experiment and joint properties value that apply in analysis. This is similar with suggested result by Lee, kim. This result means that effect to support of precedence tunnel by excavation of afterward tunnel. It can become problem in stability secure of precedence tunnel or support amount excess of afterward tunnel that such conduct has precedence tunnel support and afterward tunnel supports equally. Therefore hereafter designs need to establish suitable support pattern plan in conduct of 2-Arch tunnel.

#### 3.3 Behavior of the center pillar

Matsuda (1998) proposed an empirical equation of rock load for a tunnel in soils and weathered rocks by considering soil depth (H) and tunnel centre-tocentre distance (B). However, to date studies of rock load that can be used in preliminary tunnel design for 2-arch tunnel in rock is rather limited. In the current study, an empirical relation for rock load based on the measurement is proposed for the rock when RMR is greater than 60.

- The load width act on pillar is distance apart two tunnel. (W)
- When distance of tunnel to ground surface (H) is longer than tunnel width (D)

 $P = \gamma \times D \times W$ 

 $\gamma$ : unit weight of ground

• When distance of tunnel width (D) is longer than tunnel to ground surface (H)

 $P = \gamma \times H \times W$ 

- 1. Initial stage
- 2. Excavation completion of upper part of pilot tunnel.
- 3. Excavation completion of lower part of pilot tunnel
- 4. Load convergence
- 5. Excavation completion of precedence tunnel
- 6. Excavation completion of afterward tunnel

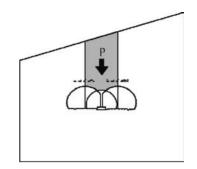


Figure 11. The load acting on pillar of 2-Arch tunnel in soil and weathered rocks (Matsuda, 1998).

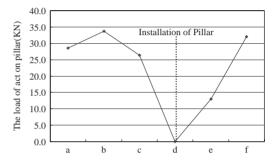


Figure 12. Change of the load acting on pillar at different steps.

Table 6. The load acting on pillar (KN).

a	b	с	d	e	f
28.6	33.0	26.4	0	13.2	31.2

The maximum load acting on pillar of 2-Arch tunnel is 33.0 KN (Table 6). In Figure 12, the load of until converge step that act on pillar of 2-Arch tunnel is supported by rock bolt and shotcrete. The final load of about 31.2 KN is the actual rock load to be supported by the center pillar. In the current study, an empirical relation for rock load based on the measurement is proposed for the rock when RMR is greater than 60. The empirical equation proposed from the current research is H = 0.15 W, where W is the centre-to-centre distance between left and right tunnels. Although more study is obviously required, this relation may be used for the preliminary design of the centre pillar for the 2-arch tunnel.

#### 4 CONCLUSIONS

The 2-Arch tunnel is similar to two parallel tunnels with very short centre-to-centre distance. However, it

is executing without enough studies for behavior of the 2-Arch tunnel. In this research, a study for behavior of the 2-Arch tunnel is examined using large model test machine. Then embody in-situ stress by applying pressure to boundary of model ground.

- The ground displacements mainly occurred within 0.25D, where D is the tunnel width. Horizontal displacements more than 40% of whole displacements and vertical displacements more than 20% occurred during excavation of pilot tunnel. Such result means that stability of whole tunnel dominated by stability of pilot tunnel excavation in the 2-Arch tunnel. This suggests that rock bolt length should be longer than 0.25D to prevent the rock loosening.
- 2. Based on the limited information obtained from the measurement, the rock load acting on the centre pillar may be H = 0.15 W, where W is the centre-to-centre distance between left and right tunnels when RMR is more than 60. However, obviously more research is required to generalize the proposed relation.

#### REFERENCES

- Amadei, B. & Stephanson, O. 1997. Rock stress and its measurement, Kluwer Academic Publishers.
- Brady, B.H.G. & Brown, E.T. 1985. Rock mechanics for underground mining, Kluwer Academic Publishers.
- Goodman, R.E. 1988. *Introduction to Rock Mechanics*, John Wiley and Sons.
- Hoek, E. & Brown, E.T. 1980. Underground excavation in Rock, Instn. of Mining & Metallurgy.
- Hoek, E. 2000. Practical Rock Engineering, Course note.
- Lee, S.D., Choi, S.I. & Gu, J.G. 1994. Design and construction of stability underground structure.
- Lee & Kim. 2001. The consideration of improve 2-Arch tunnel Design and construction method.
- Lee, I.M. 2001. Principle of rock mechanics.
- Matsuda, T. 1998. Ground behavior and Settlement control of twin tunnels in soil ground. *Tunnels and Metropolies*: pp1193–1198
- Matsuda, T. 1998. Discussion behavior and settlement control of twin tunnel in soil ground, *Tunnels and Metropolises*.
- Priest, S.D. 1993. Discontinuity Analysis for Rock Engineering, Kluwer Academic Publishers.
- Yun, J.S. 1991. Investigation and test of rock mass.

# Behavior of tunnel due to adjacent ground excavation under the influence of pre-loading on braced wall

#### S.D. Lee

Department of Environmental, Civil and Transportation Engineering, Ajou Univ, Suwon, Gyeongi-Do, Korea

#### I. Kim

Sambon Eng. Anyang, Gyeonggi-Do, Korea

ABSTRACT: Pre-loads could be imposed on the braced wall to prevent its horizontal displacements during the ground excavation even though a tunnel exists adjacent to the braced wall. New pre-loading system for large loads was developed and applied to the large scale model tests. Model tests were performed in the sandy ground, which was homogeneously and isotropically constructed in the test pit and numerically analyzed by Finite Element Method. It was found that the stability of existing tunnel was greatly enhanced when the horizontal displacements of a braced wall was reduced by applying pre-load, which was larger than the design load.

#### 1 INTRODUCTION

Horizontal displacement of a braced wall during the ground excavation could soften the rear ground, which could cause the existing structure in the rear ground to be unstable (Lee, 1999). Pre-loading method is widely used to reduce the lateral displacement of braced wall.

O'Rourke (1976) found that lateral displacement of a braced wall could be reduced by pre-loading in halfsize of the design force. But too large pre-loading could jeopardize the strut. O'Rourke (1981) also proposed that the effective rigidity could be increased by imposing pre-load.

To establish the optimum range of pre-loading in clayey soil, Mana and Clough (1981) performed numerical analyses and found that displacement could be reduced by pre-loading the design force. But they pointed out that too large pre-loading could cause local deformation of the steel joint of the supporting structure and could damage it.

Canadian Geotechnical Society (1997) found that pre-loading which amounted to the design force should be imposed to reduce the displacement of a braced wall.

Most studies on the pre-loading have focused on the ground behavior, the effective rigidity of strut, and the displacement of a braced wall.

To keep the tunnel in the rear ground to be stable during the adjacent ground excavation, horizontal displacement of braced wall should be reduced. Horizontal displacement, however, could not be sufficiently reduced by pre-loading the design force and it is difficult to be convinced of the tunnel stability. In this paper, behavior of the braced wall and the tunnel in rear ground was studied, when pre-loading was imposed on a braced wall during the ground excavation. For this purpose, large scale model tests were conducted and numerically analyzed.

#### 2 LARGE SCALE MODEL TESTS

#### 2.1 Summary

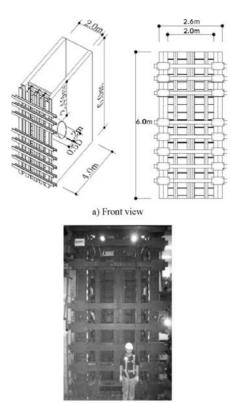
Pre-loading method was verified through large scale model tests. They were performed by imposing preload on a strut during the ground excavation by a new pre-loading system, through which large preload could impose.

Model tests were conducted in a large test box (2.0 m wide, 4.0 m long, 6.0 m tall) under 2dimensional boundary conditions. Lateral wall of test box was specially treated to minimize the friction. Existing tunnel was detached 0.5D from the braced wall.

Until excavation was completed, wall displacement was kept to be zero  $(1 \sim 8 \text{ stages})$  by pre-loading. Model tests without pre-loading were also conducted.

#### 2.2 Test ground

For model tests, homogeneous and isotropic model ground was constructed. Sand was put into test pit in 0.3 m depth at a time and compacted by a plate vibrator. Relative density of test ground was checked by extracting a sample using DIN 4021 Core Cutter (Lee, 1998).



b) General view

Figure 1. Model test box.

Physical characteristics of test ground for the model tests were confirmed through grain size distribution test, specific gravity test, field unit-weight test, and water content test.

According to the Unified Soil Classification System (USCS), test ground was poorly graded Sand SP. Mechanical properties of test ground were obtained through a direct shear test. Internal friction angle ( $\varphi$ ) was 38° and cohesion (c) was 6.0 kPa. Modulus of elasticity (E) was 28,000 kPa.

Test results are shown in Figure 2 and Table 1:

#### 2.3 Model tunnel and model braced wall

Specifications of the model tunnel were determined from the rigidity ratio by Duddeck and Erdmann (1985). Rigidity ratio between the ground and the tunnel lining for actual and model tunnel was as follows.

Rigidity Ratio of Actual Tunnel;

$$\alpha = \frac{E_k R^3}{E_b I_b} \tag{1}$$

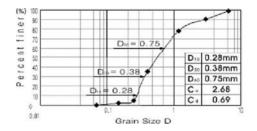


Figure 2. Grain size distribution curve.

Table 1. Physical properties of test ground.

Parameter	Value
Max. Dry Unit Weight ( $\gamma_{\text{dmax}}$ )	16.86 (kN/m <sup>3</sup> )
Min. Dry Unit Weight ( $\gamma_{dmin}$ )	$13.82 (kN/m^3)$
Dry Unit Weight of Test Ground ( $\gamma_d$ )	$15.39 (kN/m^3)$
Relative Density (D <sub>r</sub> )	56 (%)
Water Content $(\omega)$	6.8 (%)
Specific Gravity (Gs)	2.63

Rigidity Ratio of Model Tunnel;

$$\alpha = \frac{E_{km}R^3_m}{E_{km}I_{km}} \tag{2}$$

Here, subscript *m* means model tunnel.

where  $E_k(E_{km}) =$  Young's Modulus of Ground (kPa);  $R(R_m) =$  Radius of Tunnel (m);  $E_b(E_{bm}) =$  Young's Modulus Lining (kPa);  $I_b(I_{bm}) =$  Moment of Inertia of Tunnel Lining; and

Thickness of model tunnel  $t_m$  was estimated under the assumption that the rigidity ratio of real and model tunnel was equal (Soliman et al. 1993).

$$\alpha = \frac{E_k R^3}{E_b I_b} \equiv \frac{E_{km} R^3_m}{E_{bm} I_{bm}} = \frac{E_{km} R_m^3}{E_{bm} b t_m^3 / 12}$$

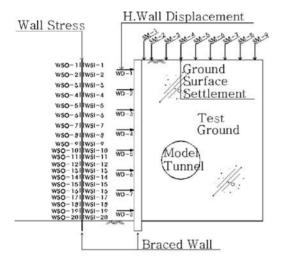
where  $t(t_m) =$  Thickness of tunnel lining (m)

$$t_m = \left(\frac{12E_{km}R_m^{-3}}{aE_{bm}}\right)^{\frac{1}{3}}$$
(3)

Modulus of elasticity of the actual ground ( $E_k$ ) was 400,000~500,000 kPa. But modulus of elasticity of model ground ( $E_{km}$ ) was 20,000~30,000 kPa. For the model tunnel lining with the diameter of 0.6 m, 6 mm-thick steel was used, which was equivalent to 0.3 m-thick concrete lining. For a braced wall, 16 mm-thick steel was used, which was equivalent to an 0.8 m-thick concrete slurry wall.

Hobbs (1966) estimated the time reduction rate under the assumption that the acceleration of gravity was constant. Reduction rate of length was established first and then reduction rate for time, density, weight,

Property	Dimensions	Reduction tate (Tunnel lining and braced wall)
Length	[L]	1/10
Time	[T]	1/3.16
Weight	[M]	1/3.120
Density	$[ML^{-3}]$	1/3.12
Stress	$[ML^{-1}T^{-2}]$	1/31.24
Acceleration of Gravity	[LT <sup>-2</sup> ]	1.0



a) Braced Wall and Ground Surface

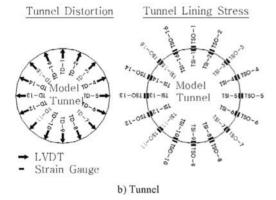


Figure 3. Measuring points for model tests.

and stress was estimated. The law of similarity could be based on weight, time, and acceleration. Steel with unit weight  $78 \text{ kN/m}^3$  was used for model tunnel lining and the braced wall (Yang et al. 2007, Shim et al. 2007).

Table 3. Properties of ground.

Parameter	
Modulus of Elasticity (E)	20,000 kPa
Poisson's Ratio ( $\nu$ )	0.25
Unit Weight ( $\gamma$ )	16.39 kN/m <sup>3</sup>
Internal Friction Angle ( $\varphi$ )	38°
Cohesion (C)	6.0 kPa

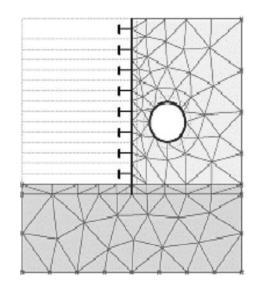


Figure 4. Finite element mesh and boundary conditions.

#### 2.4 Measurement

Moment of braced wall (40 points), lateral displacement (8 points), rear ground surface settlement (9 points), lining moment (32 points), and tunnel distortion (16 points) was automatically measured and saved at every 30 minutes. Measuring points are shown in Figure 3 and Table 3.

#### 3 NUMERICAL ANALYSIS

#### 3.1 Analysis program

Tunnel was detached 0.5D from the braced wall. Tests were numerically analyzed using FEM program PLAXIS Ver. 8.2.

Triangular plane strain elements with fifteen nodal points were used for the ground. Interface was adapted on the boundary at the braced wall and the ground.

Finite element mesh and boundary conditions for the numerical analysis are shown in Figure 4.

Vertical displacement at the lateral boundary and horizontal displacement at the lower boundary were possible.

Table 4. Material properties for braced wall and tunnel lining.

Parameter	Braced wall	Tunnel lining
Axial Rigidity (EA)	3,293,000 kN/m	1,929,000 kN/m
Flexural Rigidity (EI)	70.0 kNm <sup>2</sup> /m	5.788 kNm <sup>2</sup> /m
Thickness (d)	0.016 m	0.006 m
Poisson's Ratio (ν)	0.3	0.3

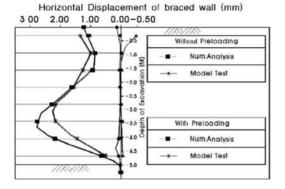


Figure 5. Horizontal displacement of braced wall.

#### 3.2 Input data

Properties of ground and structure members used in numerical analysis are shown in Tables 3 and 4:

#### 4 RESULTS OF LARGE SCALE MODEL TESTS AND NUMERICAL ANALYSES

#### 4.1 Horizontal displacement of braced wall

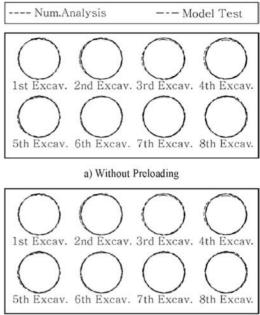
Horizontal displacement of a braced wall in the large scale model tests and numerical analyses is shown in Figure 5. At the final excavation stage, maximum lateral displacement ( $\delta$ max) of braced wall decreased from 2.290 mm to 0.03 mm by preloading, while the result of numerical analysis decreased from 2.914 mm to 0.193 mm (Fig. 5).

#### 4.2 Tunnel distortion

Distortion of tunnel lining is shown in Figure 6. Without preloading, tunnel distorted gradually due to ground excavation. Tunnel was pressed vertically and expanded horizontally. By pre-loading tunnel distortion greatly decreased. Maximum displacement of the tunnel lining decreased from 2.22 mm to 0.21 mm in the tests and from 2.670 mm to 0.337 mm in the numerical analysis (Fig. 6) by pre-loading.

#### 4.3 Member stress of tunnel lining

By pre-loading, maximum flexural moment decreased from 0.15 kNm/m to 0.068 kNm/m and maximum



b) With Preloading

Figure 6. Distortion of tunnel lining.

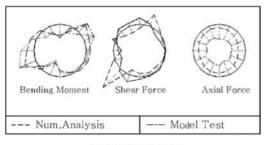
shear force decreased from 1.160 kN/m to 0.436 kN/m. Axial force, however, increased from -17.436 to -22.837 kN/m in the numerical analysis (Fig. 6). Maximum flexural moment at the final excavation stage decreased from -0.195 kNm/m to -0.021 kNm/m by pre-loading while the shear increased from -0.331 kN/m to -0.178 kN/m. Maximum axial force increased from -34.377 kN/m to -124.863 kN/m, in the model tests.

In the numerical analyses, on the contrary, by preloading, maximum flexural moment decreased from 0.150 kNm/m to 0.068 kNm/m, maximum shear decreased from 1.160 kN/m to 0.436 kN/m. Maximum axial force, however, slightly increased from -17.463 kN/m to -22.837 kN/m, which was in an allowable range. Moment of the tunnel lining is shown in Figure 7.

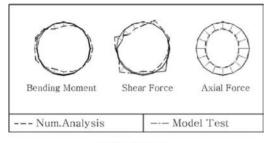
# 4.4 Lateral displacement of braced wall and tunnel distortion

Without pre-loading, the largest displacement and tunnel distortion were observed in the 6th excavation stage, in the tunnel level.

Significant horizontal displacement of braced wall and tunnel distortion would not occur throughout the entire process, when pre-loading was imposed. If braced wall was not deformed laterally, tunnel shape would hardly change (Fig. 8).

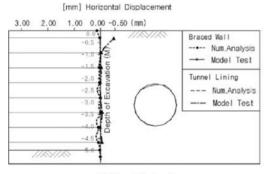


a) Without Preloading

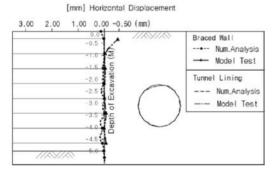


b) With Preloading

Figure 7. Bending moment and stress of tunnel lining.



a) Without Preloading



b) With Preloading



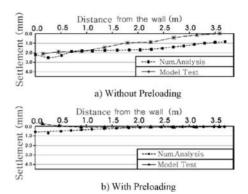


Figure 9. Rear ground surface settlement.

#### 4.5 Ground surface settlement

By pre-loading maximum ground surface settlement at final excavation stage decreased from 2.1 mm to 0.12 mm in the model tests and from 2.506 mm to 0.726 mm in the numerical analysis. Rear ground surface settlement is shown in Figure 9.

#### 5 CONCLUSION

It was investigated how to ensure the stability of the adjacent tunnel during the ground excavation by reducing the horizontal displacement of a braced wall by gradually imposing pre-loading. A new preloading system, through which large pre-loading could impose was developed and applied to large scale model tests. Performed tests were numerically analyzed.

The results are as follows:

- 1 If lateral displacement of the braced wall was greatly reduced by imposing pre-loading, which was larger than design force, the stability of the adjacent tunnel at the rear ground would greatly increase.
- 2 Maximum flexural moment and maximum shear of tunnel lining were decreased but the axial force was increased by imposing pre-loading in both large scale model tests and numerical analyses.
- 3 Largest horizontal displacement of a braced wall occurred in the excavation stage, which was the tunnel level, when pre-loading was not imposed.
- 4 Tunnel was pressed vertically and expanded horizontally during the ground excavation without preloading. But by imposing pre-loading no significant distortion in its shape was observed throughout the entire excavation process.
- 5 Most of the ground surface settlement was decreased by imposing pre-loading.

#### REFERENCES

- Canadian Geotechnical Society. 1997. Foundation Engineering Manual. 3rd. ed.
- Duddeck, H. & Erdmann, J. 1985. On structural design models for tunnels in soft soil. *Underground Space*, Vol. 9, Pergamon Press; 246–259.
- Hobbs, D.W. 1966. Scale model study of strata movement around mine roadways. Apparatus, technique and some preliminary results, *Int. J. of Rock Mech. Min. Sci.*, 3.
- Lee, S.D. 1998. Soil Mechanics (2nd. edition). Saeron Press; 353–354.
- Lee, S.D. 1999. Foundation Engineering. Saeron Press; 252–254.
- Lee, S.D. 2003. Understanding of the Latest Tunneling Technology. *Ajou University Geotechnical Engineering*; 1–22.
- Mana, A.I. & Clough, G.H. 1981. Prediction of movement for braced cuts in clay. J. Geotech. Engineering. Div., ASCE, vol. 107, No. 6; 759–778.

- O'Rourke, T.D., Cording, E.J. & Boscardin, M. 1976. The Ground movements related to braced excavation and their influence on adjacent buildings. U.S Department of Transportation, Report no. DOT-TST 76, T-23.
- O' Rourke, T.D. 1981. Ground Movements Caused by Braced Excavation. Journal of Geotechnical Engineering Division. ASCE. Vol. 107. NO. GT9; 1159–1178.
- Shim, H.J. and others. 2007. Model Test and Test Blasting to Design Adjacent Tunnel. KTA 2007 Annual Conference April 20–21 Seoul Korea; 267–278.
- Soliman, E., Duddeck, H. & Ahrens, H. 1993. two-and three-dimensional analysis of closely spaced double-tube tunnels. *Tunnelling and Underground Space Technology*, Vol. 8, No. 1; 13–18.
- Yang, H.S. and others. 2007. A Study on Characteristics of Materials for Scaled Model Test. *Ministry of Construction & Transportation R&D Performance Forum Journal*; 33–36.

# Two distinctive shear strain modes for pile-soil-tunnelling interaction in a granular mass

#### Y.J. Lee

Soil-Structure Interaction Group, Steel Structure Research Laboratory, Research Institute of Industrial Science & Technology (RIST), South Korea

#### C.S. Yoo

Department of Civil & Environmental Engineering, Sungkyunkwan University, South Korea

ABSTRACT: There are many high-rise buildings in urban areas, which are normally supported by piled foundations. Consequently, a major concern for geotechnical engineers is the construction of a tunnel adjacent to the piled foundations, since ground behaviour between an existing loaded pile and tunnelling has not been understood well so far, particularly for granular soils rather than clay soils. In order to figure out such complicated ground behaviour, the interactive shear strain patterns are generated by two-dimensional laboratory model tests and finite element analyses (FEA). For the model testing, a multi-sized aluminium rod mixture considered as a continuum granular mass and close range photogrammetric technique for obtaining displacement data within the rods are introduced. Two distinctive shear strain modes, viz. connective and isolated modes, are presented through the comparison of the model tests and FEA according to the pile tip locations.

#### 1 INTRODUCTION

Tunnel excavation work in the soft ground results in significant reduction in total stress in the vicinity of a tunnel boundary. The reduction in stress causes ground movements which affect adjacent building foundations consisted of a row of loaded piles.

In order to identify the pile-soil-tunnelling interaction behaviour, small-scale physical model tests and numerical methods were employed in this study. Careful assessment of the pile-soil-tunnelling interaction problems is relatively new and only limited information is currently available.

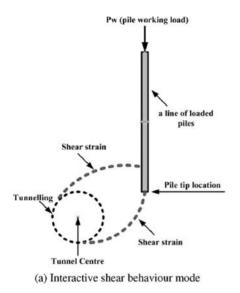
This study focuses on two distinctive shear strain modes of the ground between the existing pile and a new tunnel construction as shown in Figure 1. The laboratory model tests were conducted by strain control rather than stress control, and then comparison with the finite element analyses was carried out to identify the shear strain modes. This research incorporates strain controlled tests with idealised two-dimensional aluminium rods – considered as a granular mass – taken to very high volume loss (up to about 20%) to highlight the full shear failure formation. Digital image processing technique has allowed overall deformation patterns of ground movements to be obtained from the analysis of digital images. Detailed shear strain patterns of the ground can be obtained which give a clear insight into the pile-soil-tunnelling interaction events.

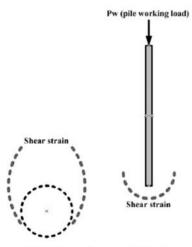
The model pile was principally an end-bearing pile where most of the pile load is concentrated on the pile tip rather than pile shaft. The pile working load was maintained constant during the test. This working load was chosen by reference to a displacement-controlled pile-load test (the pile working load, 3.6 kN, is 77% of the ultimate pile load). In this study, in order to avoid complexity of pile loading the influence of lateral loading was not considered.

#### 2 LABORATORY MODEL TEST

#### 2.1 Test equipment

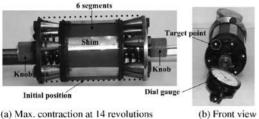
The displacement-controlled model tunnel consists of 6 segments, the two ends of a segment being carved on tapered cones. Each segment moves inward as the tapers are withdrawn, simulating the two-dimensional volume loss during tunnelling operations. The tunnel diameter is reduced by rotating the two knobs as shown in Figure 2. The outer diameter of the tunnel is initially 100 mm. The reduction of the tunnel diameter gives directly a 2D volume loss ( $V_L$ ). This 2D volume loss per revolution is determined from the calibration result (Lee, 2004).





(b) No interactive shear behaviour mode

Figure 1. Schematic illustration of shear strain modes for the pile-soil-tunnelling interaction.



(a) Max. contraction at 14 revolutions

Figure 2. Diameter reduction system of model tunnel.

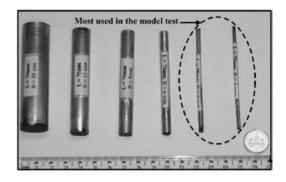


Figure 3. Multi-sized aluminium rod material.

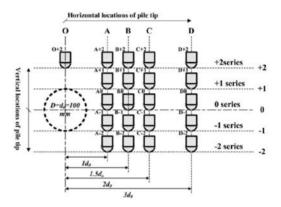


Figure 4. Identification of pile tip location.

The aluminium rod mixture consists of six different diameters (viz. 2 mm, 3 mm, 6 mm, 9 mm, 12 mm and 20 mm), which have the same length of 75 mm. It represents a well graded, idealised two-dimensional granular material, as shown in Figure 3. The test chamber is rigid, a rectangular steel frame (width: 1058 mm, height: 930 mm). The model pile ( $25 \text{ mm} \times 75 \text{ mm}$  in cross section, embedded length, L: 370 mm) is made of aluminium alloy. No significant effects of the boundary conditions were observed during the test, since the smallest rods (2 mm and 3 mm diameters) were mainly used in the interactive regions and the largest rods were used in the vicinity of the steel frame boundaries.

#### 2.2 Pile-soil-tunnelling interaction test

The pile tip identification system adopted was to label the distances of the pile tip away from the tunnel centre line (O) as A ( $1d_0$ : model tunnel diameter), B ( $1.5d_0$ ),  $C(2d_0)$ ,  $D(3d_0)$  and the pile tip level as 0 (on the centre line), +1 (on the crown level), +2 (at 1d<sub>0</sub> above the centre line level), -1 and -2 being similarly below the centre line level. One final location (O + 2) was with the pile directly above the centre line of the tunnel on the +2 level (Figure 4).

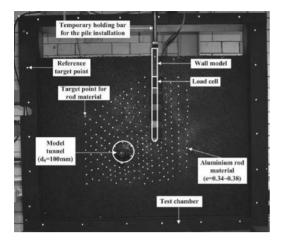


Figure 5. Set-up stage with test equipment and reflective target points.

The pile was in place and loaded before any of the pile-soil-tunnelling interaction occurred. The model tests, therefore, simulated new tunnelling adjacent to a row of existing loaded piles.

Each model test normally consisted of three stages as follows: (1) set-up stage, material 'compacted' as before, with reflective nodes shown in Figure 5 (both pile and tunnel installed in positions but with no pile loading and tunnel at initial diameter,  $d_0$ ); (2) pile loaded to working load by dead weight (P<sub>w</sub> from the pile-load test); (3) tunnelling stage (reduction of the tunnel diameter to a maximum value of 12 revolutions, i.e. V<sub>L</sub> = 18.65%).

The tests using aluminium rods in this study are much quicker to carry out in terms of testing time than any conventional test using a real soil such as sand or clay. In addition, detailed observation of the side section can easily be carried out, enabling details of the failure mechanisms, which are associated with strain fields rather than stress fields, to be examined.

#### 3 CLOSE RANGE PHOTOGRAMMETRY

The close range photogrammetric technique for determining strains used in this study has recently been applied to a number of geotechnical engineering problems. A Kodak DC 290 digital camera was used to capture both the frame (or chamber) reference points and the target points fixed to the centre part at the ends of the smaller size rods within the multi-sized rod matrix. The digital camera can provide acceptably high resolution. Ten independent images were taken at each epoch (or stage) using the Kodak DC 290 digital camera as shown in Figure 6.

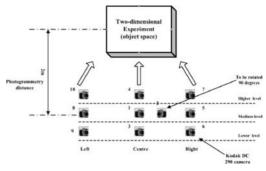


Figure 6. Typical digital camera positions for capturing 10 images.

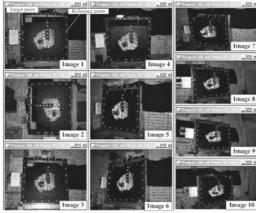
A pixel resolution of  $1792 \times 1200$  was available. Each retro-reflective target was identified and the position of its image within each of the ten photographs was measured by the VMS (vision metrology system) program. The measured 2D x-y coordinates of the target points from the VMS are arranged into a triangulation mesh by means of the EngVis program. Figure 7 shows an example of the imaging processing by the VMS and the subsequent triangulation by the EngVis respectively.

The system and its application were described by Woodhouse et al. (1999), Woodhouse (2000) and Kwok and Swajani (2001). From the measured displacements, strains were calculated based on the assumption of linear strain in each triangular element (Lee and Bassett, 2006). More detailed model test procedures matched with the image capturing stages are shown in Figure 8.

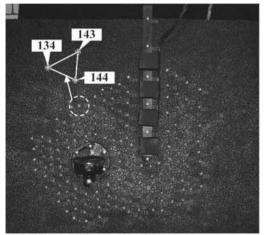
#### 4 NUMERICAL ANALYSIS

In order to carry out the finite element analyses (FEA), the 3D pile-soil-tunnelling interaction situation was idealised to two-dimensional plane-strain conditions and to compare with the model pile-soil-tunnelling test result, the same tunnel geometry, pile size, and location of the pile relative to the tunnel were adopted for the FEA. The FE analyses were carried out using the continuum finite element program CRISP (Britto and Gunn, 1987; Woods and Rahim, 2001).

Ground behaviour was assumed to be governed by an elastic-perfectly-plastic constitutive model based on the Mohr-Coulomb failure criterion with a nonassociated flow rule. The critical state angle of friction  $(\phi'_{cs})$  and the angle of dilation  $(\psi)$  were determined to be 23° and 15° for the model granular material, respectively (based on a best FEA fit of load-settlement relationship for the model pile loading tests, Lee, 2004). It should be noted that later these parameters were also obtained from shear box tests (area void



(a) VMS process for identification of target points



(b) EngVis process for generation of triangular elements

Figure 7. Digital image analysis by VMS and EngVis programs.

ratio,  $e = 0.34 \sim 0.38$ ,  $\phi'_{cs}$  and e values are found to be similar to those given by Yamamoto and Kusuda, 2001). Parameter values from the shear box tests were comparable to the best fit values. The effective cohesion (c') was assumed to be 0.1 kPa. The variation of Young's modulus (E) was assumed to increase linearly with depth. The ground parameters are summarised in Table 1.

The tunnel support and pile was modelled as a twonode bar element and linear elastic material respectively. The Young's modulus and Poisson's ratio for both the tunnel support and pile were assumed to be 15.5 GPa and 0.2, respectively. The unit weight of the pile and the cross section area of the tunnel support ring were  $23 \text{ kN/m}^3$  and  $0.003 \text{ m}^2$ , respectively. The model parameters for the tunnel support and the pile are summarised in Table 2.

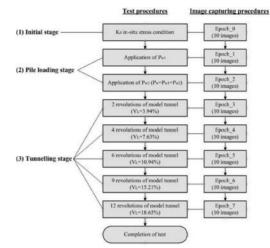


Figure 8. Relationship between model test steps and image capturing stages.

#### Table 1. Ground parameters used in the FEA.

Ground surface level, $Y_0^*(m)$ Young's modulus at $Y_0$ , $E_0^*(kPa)$ Gradient of Young's modulus, $m_E^{**}(kPa/m)$ Poisson's ratio, $v$ Unit weight of soil, $\gamma$ (kN/m <sup>3</sup> ) Effective cohesion, c' (kPa) Critical state angle of friction, $\phi'_{cs}$ (degrees) Angle of dilation, $\psi$ (degrees)	$\begin{array}{c} 0.72 \\ 1600 \\ 10,000 \\ 0.35 \\ 24 \\ 0.1 \\ 23 \\ 15 \end{array}$
------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------	----------------------------------------------------------------------------------------

Note: \*from bottom to top of mesh; \*\*varying with depth

Table 2. Parameters for tunnel support and pile.

Parameters	Tunnel support	Pile
E (GPa)	15.5 0.2	15.5 0.2
Unit weight, $\gamma$ (kN/m <sup>3</sup> ) Cross section area, A (m <sup>2</sup> )	- 0.003	23

FNR (Full Newton-Raphson) iterative solution scheme was adopted together with a tolerance of 0.05 and a maximum iteration number of 100.  $K_0$  (0.66) was applied as the initial in-situ stress conditions. Double convergence check based on both the displacement and force norms and a total of 1055 increments were used. It should be noted that the largest 2D volume loss values (7.63% to 18.65%) were generated in order to capture the interaction failure patterns between the pile and the tunnel.

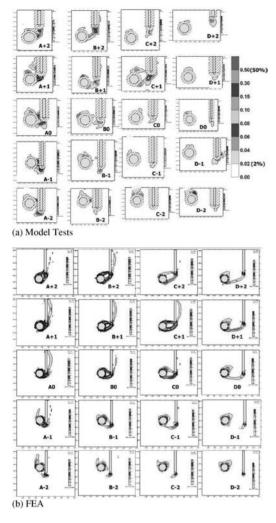


Figure 9. Comparison of maximum shear strain contours at  $V_L = 3.94\%$ .

#### 5 RESULTS

Figures 9 and 10 show the maximum shear strain contours of the model tests at  $V_L = 3.94\%$  and  $V_L = 10.94\%$  respectively. The location and intensity of shear strain clearly identify the developing shear failure formation between the pile base and the tunnel. It is noted that the case of O + 2 was omitted in this comparison (this case is not a normal practice for considering a proper tunnel position adjacent to a row of loaded piles).

A clear neutral or dead block X - an area with low or no strain – was observed clearly in case C + 1 at 10.94% of volume loss as shown in Figure 10.



(a) Model Tests

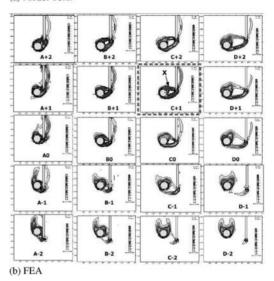


Figure 10. Comparison of maximum shear strain contours at  $V_L = 10.94\%$ .

The shear failure formation appeared to comprise two distinctive shear strain modes: (1) one that includes a "neutral soil block" X, separating a formation running from the pile base to the tunnel invert area and a second mechanism running from the pile shaft to the tunnel crown area, and (2) the other one that is an independent shear behaviour mode, i.e. no interactive shear strain mode between the pile and the tunnel. In summary, the two different shear strain modes shown in Figure 1 can be identified according to pile tip locations as shown in Figure 11.

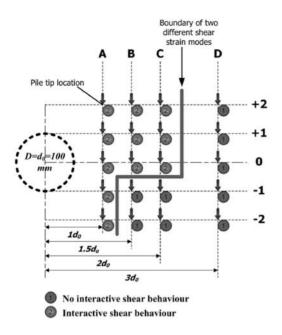


Figure 11. Boundary of two different shear strain modes according to pile tip locations.

Similar shear strain mode behaviour was observed at both small and large magnitudes of the volume loss.

While the model tests and FEA are twodimensional, it is expected that similar behaviour would be found in real projects where the tunnelling presents a three-dimensional problem. However, the true boundary of two different shear strain modes in the latter may be different from the former tests and analyses.

#### 6 CONCLUSIONS

Comparison between the physical model tests and the finite element analyses showed many successful points of agreement in terms of shear strain data. Based on the maximum shear strain data, it was observed that the shear strain modes developed are strongly dependent on the pile tip location and the magnitude of the volume loss. Through this study, it is recognized that the boundary of two different shear strain modes may be a useful guide for the tunnel planners who need to make a decision on the proper positioning of tunnel construction adjacent to a row of loaded piles in urban areas.

#### REFERENCES

- Britto, A.M. & Gunn, M.J. 1987. Critical state soil mechanics via finite elements. Chichester, UK: Ellis Horwood Limited.
- Kwok, H.Y. & Swajani, C. 2001. Precise measurement technique. 3rd year project report, Department of Civil and Environmental Engineering, University College London, University of London.
- Lee, Y.J. 2004. Tunnelling adjacent to a row of loaded piles. PhD Thesis, University College London, University of London.
- Lee, Y.J. & Bassett, R.H. 2006. Application of a photogrammetric technique to a model tunnel. *Tunnelling and Underground Space Technology* 21(1): 79–96.
- Woodhouse, N.G. 2000. Geometric models appropriate for engineering analysis from vision metrology data. PhD thesis, University College London, University of London.
- Woodhouse, N.G., Robson, S. & Eyre, J.R. 1999. Vision metrology and three dimensional visualisation in structural testing and monitoring. *Photogrammetric Record* 16(94): 625–641.
- Woods, R. & Rahim, A. 2001. SAGE-CRISP Technical Manual, Version 4. The CRISP Consortium Ltd. http://www.mycrisp.com/demo/TECHMAN.pdf
- Yamamoto, K. & Kusuda, K. 2001. Failure mechanisms and bearing capacities of reinforced foundations. *Geotextiles* and Geomembranes 19(3): 127–162.

### Stability analysis of large slurry shield-driven tunnel in soft clay

Y. Li & Z.X. Zhang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, School of Civil Engineering, Tongji University, Shanghai, P.R. China

F. Emeriault & R. Kastner *INSA-Lyon, LGCIE, France* 

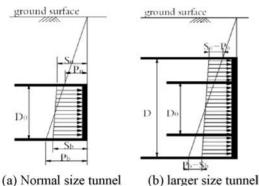
ABSTRACT: The possibility of partial failure of large slurry shield-driven tunnels is investigated by an upperbound approach in limit analysis and a three-dimensional numerical modeling for the Shanghai Yangtze River Tunnel. The results of the upper-bound limit analysis failure mechanisms and the 3D numerical modeling have shown that the partial blow-out of the upper part of the tunnel face occurs when the slurry pressure is too large while the global collapse of the whole tunnel face occurs when the slurry pressure is too small. The failure mechanisms and critical slurry pressures obtained from both approaches are presented and discussed.

#### 1 INTRODUCTION

In recent years, the rapid growth in urban development has resulted in an increased demand for the construction of tunnels for electric and communication cables, and transportation systems. For obvious practical reasons such as accessibility, serviceability and economy, these tunnels are constructed by shield machines of large diameter and at shallow depths. The Groene Hart Tunnel, constructed in 2005 in Netherlands, was carried out by a slurry-shield machine with an outside diameter of 14.87 m. The M-30 Tunnel in Madrid excavated by EPB shield machine, 15.2 m in diameter, was until recently the biggest shield tunnel completed in the world. In September 2006, two massive 15.43 m diameter slurry shield machines began work on Shanghai Yangtze River Tunnel. With the increase of the shield tunnel diameter, the excavated volume is increased dramatically and the probability of excavation in complicated stratum with different types of soil layers increases greatly too. The stability of the soil itself decreases at the same time. Thus, in recent years, more and more attention was paid to the face stability of large shield-driven tunnels.

The analysis of the face stability of shallow circular tunnels driven by the pressurized slurry shield requires the determination of the pressure to be applied by the shield. This pressure must avoid both the collapse (active failure) and the blow-out (passive failure) of the soil mass near the tunnel face. A number of studies have concerned tunnel face stability. Most results are analytical and are based on limit equilibrium method (Horn, 1961; Anagnostou & Kovári 1994; Broere, 2001) and limit analysis method (Davis et al. 1980; Leca & Dormieux, 1990; Chambon & Corté, 1994; Soubra 2000, 2002; Subrin & Wong, 2002). A rational and well-defined approach for the computation of the supporting pressure is translational multiblock failure mechanism of the upper-bound method, which is presented by Soubra (2002) based on the 3D limit analysis model of Leca and Dormieux (1990). This mechanism allows the slip surface to develop more freely in comparison with the available mechanisms given by Leca and Dormieux, and thus, improves the best upper-bound solutions given by these authors. The multiblock mechanism is convenient for a constant tunnel pressure which is an acceptable assumption for small to medium tunnel diameters (<10 m). However, studies on face stability of very large slurry shielddriven tunnels, for which the hypothesis of a constant slurry pressure is not applicable, are fairly few.

The non-constant supporting pressure of slurry shield-driven tunnel is caused by the density of the slurry. Since the density of the slurry should remain within a certain range to obtain high-quality filter cakes, and it is always smaller than the density of soil, there will be a pressure difference between the slurry pressure and the earth pressure at tunnel crown and invert. This pressure difference increases with the tunnel diameter increase, shown in Figure 1, where,  $S_u$  and  $S_b$  are the slurry pressures at the level of the tunnel crown and invert;  $P_u$  and  $P_b$  are the corresponding



(a) Normai size tunner

Figure 1. Pressure difference in different size tunnels.

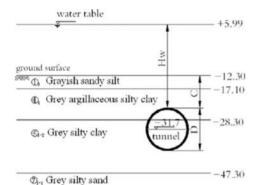


Figure 2. Geological condition with C/D = 0.7.

earth pressures;  $D_0$  is the diameter of the small tunnel and D is the diameter of the larger one. This pressure difference may induce a different failure mechanism from the one corresponding to a constant slurry pressure, especially in large tunnels.

In this paper, a simplified computation scheme considering the non-constant slurry pressure is adopted, in which the multiblock failure mechanism suggested by Soubra (2002) is employed to investigate the possibility of partial failure in large size slurry shield-driven tunnel. Also, a more rigorous 3D numerical modeling is carried out to compare with the obtained results of critical slurry pressures and the corresponding soil mass at failure.

#### 2 CASE STUDY

The main part of Shanghai Yangtze River Tunnel is 7.5 km river-crossing tunnel connecting the Pudong and Changxing Island in Shanghai, China. Excavation of the tunnels is carried out by a pressurized slurry shield machine with an outside diameter of 15.43 m, which is the world's largest until now.

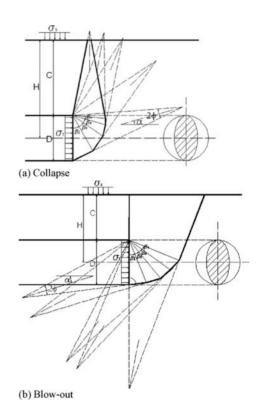


Figure 3. Multiblock failure mechanisms.

The tunnel is mainly excavated below the river bed of Yangtze River, which is composed of muddy clay and soft clay, with some local lenses of silty fine sand. The tunnel will be driven at a depth up to 65 m. A hydrostatic pressure up to 650 kPa is anticipated. The shallowest ground section is under the river with a cover-to-depth ratio of C/D = 0.7, where C and D is the cover depth and diameter of the tunnel. Because of the unfavorable geological condition and the large dimension of excavating face, the face stability of the tunnel is one of the key technical aspects in this project. The most dangerous profile with C/D = 0.7 is chosen for the study, as shown in Figure 2, where  $H_w$  is the height of the water table.

#### 3 PARTIAL FAILURE MECHANISM

#### 3.1 Multiblock failure mechanism

The problem of the face stability analysis relevant to a circular rigid tunnel of diameter *D* driven under a depth of cover *C* could be idealized, as shown in Figure 3. A surcharge  $\sigma_s$  is applied on the ground surface, and  $\sigma_t$  is the uniform supporting pressure on the tunnel face. The multiblock failure mechanism considered in this

paper is described in Soubra (2002). It is composed of several truncated rigid cones with circular crosssections and with opening angles equal to  $2\phi$ , where  $\phi$  is the friction angle of the soil. The different blocks of this mechanism move as rigid bodies. These rigid cones translate with velocities of different directions, which are collinear with the cones' axes and make an angle  $\phi$  with the discontinuity surface. The velocity of each cone is determined by the condition that the relative velocity between the cones in contact has the direction that makes an angle  $\phi$  with the contact surface. The present mechanism is completely defined by *n* angular parameters  $\alpha$  and  $\beta_i$  (i = 1...n-1), where *n* is the number of rigid blocks. The geometrical construction of this mechanism is similar to that of Leca and Dormieux (1990). The present mechanism is characterized by more freedom angles than that of Leca and Dormieux (1990) and thus improves the solutions of the critical supporting pressure.

The external forces contributing to the rate of external work consist of (i) the self-weight of the truncated rigid cones; (ii) the surcharge loading  $\sigma_s$  (in case of outcrop of the upper rigid block) and (iii) the pressure  $\sigma_t$  at the face of the tunnel. The rate of energy dissipation occurs along the lateral surfaces and radial planes of the failure mechanism. By equating the total rate of external work to the total rate of internal energy dissipation, the pressure  $\sigma_t$  at the face of the tunnel is obtained as follow:

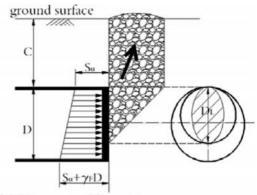
$$\sigma_t = N_s \sigma_s + (N_s - 1) \frac{c}{\tan \phi} + N_\gamma \gamma D \tag{1}$$

where,  $N_s$  and  $N_{\gamma}$  are surcharge and soil weight coefficient; c,  $\phi$  are the cohesion and friction angle of the soil and  $\gamma$  is the soil unit weight.

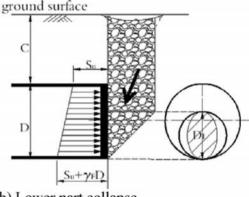
#### 3.2 Critical slurry pressures

In order to study the possibility of partial failure, two partial failure mechanisms are used as shown in Figure 4, corresponding to the blow-out of the upper part of tunnel and the collapse of the lower part of tunnel. The partial failure is assumed to occur in an area with a vertical axis  $D_L$ , where  $D_L \in (0, D)$ . In upper part blow-out mechanism, as shown in Figure 4(a), the top of the failure area passes through the tunnel crown. In the lower part collapse mechanism, as shown in Figure 4(b), the bottom of the failure area passes through the tunnel invert. The upper part collapse and lower part blow-out are not considered because they are less dangerous.

The critical slurry pressure corresponding to partial upper part blow-out and lower part collapse are computed in a simplified approach using the multiblock failure mechanism as follow: the tunnel pressure obtained from the multiblock mechanism for different



(a) Upper part blow-out



(b) Lower part collapse

Figure 4. Two kinds of partial failure mechanisms.

prescribed values of the tunnel diameters (corresponding to different values of  $D_L$  in the present analysis) are computed. The value of  $D_L$  giving the minimal (respectively maximal) blow-out (respectively collapse) pressure is considered as the critical pressure causing a partial failure. A five-block model (i.e. n = 5) is employed for this study (as shown in Figure 3) since it was shown in Soubra (2002) that n greater than 5 will not significantly improve the results. Water above the ground surface is considered as a surcharge. Because the excavation is executed quickly compared to the soil consolidation, a typical undrained analysis is employed. Table 1 summarizes the characteristics of the soil. The unit weight of slurry is  $\gamma_F = 12 \text{ kN/m}^3$ . In case 1, the cohesion is considered to be a constant. In case 2, cohesion of the soil increases with depth as  $c_u = -0.95z + 0.4$ , where z is the depth of the soil layer. The mean value from the level of tunnel invert to the ground surface in case 2 is equal to the  $c_u$  in case 1. Case 2 is chosen to take into account the variation of the undrained cohesion between the tunnel

Table 1. Soil parameters used in numerical modeling.

	unit weight ½0 (kN/m <sup>3</sup> )	cohesion c <sub>u</sub> (kPa)	friction angle $\phi_u$ (°)	Young's modulus E (MPa)	Poisson's ratio v
case1		24.5	0.01	3.21	0.495
case2		-0.95z+0.4	0.01	3.21	0.495

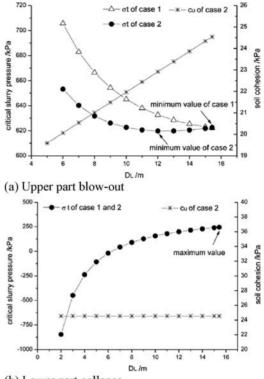
crown and invert in extra large tunnel, especially in normally consolidated clay.

According to the basic assumption of the multiblock failure mechanism, the intersection of the failure block and the tunnel face will be an ellipse with a long axis of  $D_L$  in vertical direction. The critical slurry pressure  $\sigma_{t0}$ obtained by multiblock model is a uniform pressure (as shown in Figure 3). Considering the total force balance on the failure face of diameter  $D_L$ , an equivalent slurry pressure is obtained. The slurry pressure  $\sigma_t$  at tunnel crown can be computed as follow:  $\sigma_t = \sigma_{t0} - \gamma_s D_L/2$ . Actually, since the slurry pressure increase with depth, the total force due to the slurry is acting on the tunnel face with eccentricity e. When using uniform pressure  $\sigma_{t0}$ , a moment should be taken into consideration at the same time. However, the moment is not considered here since the normalized eccentricity e/D is very small in this study (about  $2\% \sim 4\%$ ).

The critical slurry pressure  $\sigma_t$  is calculated for partial diameter  $D_L$  varying in 1 m steps, which leads to the maximum (or minimum) value of  $\sigma_t$ , as shown in Figure 5. In case 2, the cohesion for different  $D_L$  is the mean value from the top to the bottom of the failure block. In upper part blow-out, cohesion decreases with the decrease of  $D_L$ , since the failure block becomes more and more shallow. While in lower part collapse, the failure block is always starting from the tunnel invert to the ground surface, the mean value of cohesion for different  $D_L$  is equal to the one in case 1. Therefore, there is no difference between the value of  $\sigma_t$  in case 1 and 2 in collapse.

Figure 5 shows that in case 1, the maximum and minimum critical slurry pressure is obtained when partial failure diameter  $D_L = D = 15.43$  m, which means that the global failure will happen both in collapse and blow-out cases.

In case 2, the minimum slurry pressure in blowout is obtained when partial failure diameter is 12 m, which means that in blow-out failure, the partial failure of upper part tunnel face will be more dangerous than the global failure of the whole tunnel face. However, there are no great differences among critical slurry pressures when  $D_L \in (10, 15.43)$ . Therefore, in practice, global failure of the entire face and partial failure with  $D_L > 10$  m will have the same probability. For collapse, the maximum slurry pressure is obtained when



(b) Lower part collapse

Figure 5. Critical slurry pressure of partial failure mechanism.

 $D_L = 15.43$  m. With the decrease of the partial failure diameter, the critical slurry pressure will decrease at the same time: global failure of the whole tunnel face is more dangerous than partial failure. The critical slurry pressure at the tunnel crown level is 619.9 kPa and 265.9 kPa corresponding to blow-out (partially) and collapse (globally), which is 712.5 kPa and 358.5 kPa at the level of tunnel center.

#### 4 NUMERICAL ANALYSIS WITH FLAC<sup>3D</sup>

### 4.1 FLAC<sup>3D</sup> numerical modeling

In order to investigate the behavior of the tunnel face during failure, numerical analysis is carried out with the commercially available finite-difference code FLAC<sup>3D</sup>, which is an effective program for situations where physical instability may occur.

In the model, due to symmetry, only one half is included. The model is sufficiently large to allow for any possible failure mechanism to develop and to avoid any influence from the model boundaries (as shown in Figure 6). The water table is 2D above the tunnel crown. In order to focus the analysis on

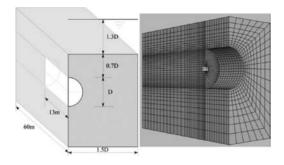


Figure 6. Studying profile and 3D numerical model.

the face failure in front of the shield machine, the excavation process was simulated using a simplified single-step excavation scheme, assuming that the tunnel is excavated 13 m (the length of the shield machine) instantaneously. Such a simplified modeling scheme had been successfully adopted in previous studies (Gioda & Swoboda, 1999).

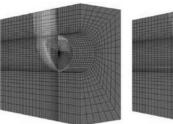
The different soil layers are assumed to be elastic perfectly-plastic materials conforming to the Mohr-Coulomb failure criterion. An undrained analysis is carried out by using the undrained parameters of the soil as shown in Table 1.

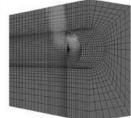
The initial slurry pressure  $\sigma_{s0}$  is equal to the earth pressure at rest at center of tunnel face (in this case,  $\sigma_{s0} = 537.6$  kPa). The slurry pressure is increasing with the depth according to the density of the slurry. Slurry pressure at the center of the tunnel face  $\sigma_{si}$  will be increased (or decreased) by multiplying  $\sigma_{s0}$  by a pressure factor *M* in every construction phase *i*, until blow-out (or collapse) occurs. The failure criterion is defined as follows: the construction phase is considered as the beginning of failure, where for the first time considerable value of unbalance force (not equal to 0) and velocity (greater than  $e^{-11} \sim e^{-12}$  m/step in this case) is observed. Then the critical slurry pressure corresponding to collapse and blow-out will be obtained by the following equation:

$$\sigma_{si} = M \cdot \sigma_{s0} \tag{2}$$

#### 4.2 Numerical modeling results

For case 1, collapse occurs at the construction phase when M = 0.65 and blow-out occurs at M = 1.30; M = 0.70 and M = 1.30 for case 2 respectively. Maximum and minimum critical slurry pressure could be obtained by equation (2). Displacement contours at the failure of case 2 is plotted in Figure 7. A global collapse failure of the whole tunnel face is observed. This mechanism is well coincident with the centrifugal experiment result of Chambon and Corté (1994). However failure mechanism of blow-out is a partial failure





(a) Collapse (M=0.70)

(b) Blow-out (M=1.30)

Figure 7. Displacement contours when failure is observed in case 2.

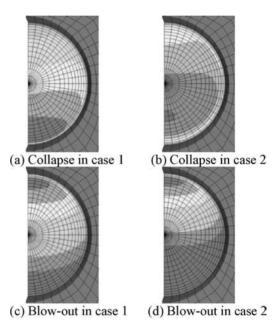


Figure 8. Velocity contours of tunnel face when failure occurs.

in the upper part of the tunnel face. Partial failure mechanism is more obvious in case 2 than in case 1.

Two cases are compared in Figure 8. In collapse, as shown in Figure 8 (a) and (b), the total tunnel face will have considerable value of velocity; and the most dangerous point with maximum velocity is near the tunnel invert. The failure area on the tunnel face is the whole face, which is circular in shape. In blow-out, as we see in Figure 8 (c) and (d), the failure is constrained in upper 3/4D and 1/2D part of the tunnel face, corresponding to the case 1 and case 2 respectively. The failure mechanism in both cases has an elliptic shape, with a long axis in the horizontal direction. The most dangerous point with the maximum velocity is the point near the tunnel crown. The failure mechanism of case 2 could be obviously observed by the plot of

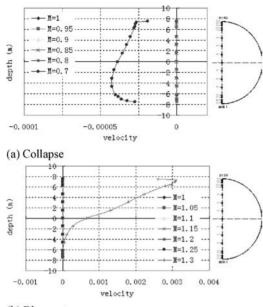




Figure 9. Velocity development of blow-out along the center vertical axis of the tunnel face of case 2 (unit: m/step).

velocity evolution of the monitored points along the center vertical axis of the tunnel face, as shown in Figure 9.

#### 5 COMPARISON OF FAILURE MECHANISMS

The failure mechanisms of five-block model and 3D numerical modeling are compared in case 2 (as shown in Figure 10). In five-block model, two sets of  $\alpha$  and  $\beta_i$  are obtained by optimization of the coefficients  $N_{\gamma}$  and  $N_s$ . As shown in Figure 10, there are only small differences between the two soil mass at failure corresponding to the two sets of  $\alpha$  and  $\beta_i$ .

Basically speaking, the failure mechanisms of fiveblock model and 3D numerical modeling well agree with each other. Both of them well predict the partial failure on the upper part of the tunnel face in blow-out, and global failure of whole tunnel face in collapse.

#### 6 CONCLUSIONS

 Both the results of partial failure mechanism based on multiblock model and 3D numerical analysis show that the partial failure mechanism will happen in blow-out but global failure with entire tunnel face will dominate the collapse failure, especially in case where cohesion is changing with depth.

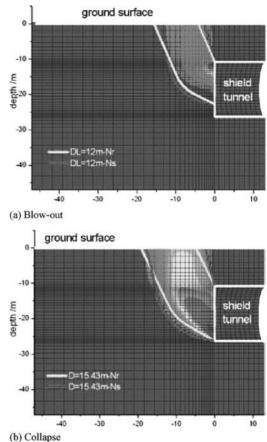


Figure 10. Comparison of failure mechanisms of Case 2 (velocity contour for FLAC<sup>3D</sup> analysis).

Table 2.	Comparison	of critical	slurry	pressure	at	tunnel
center leve	el in case 2.					

	Critical sl pressure /		Pressure factor M		
	Collapse	Blow-out	Collapse	Blow-out	
5-Block Model 3D Numerical	358.5 376.3	712.5 698.9	0.67 0.70	1.33 1.30	

2. The critical slurry pressures of both multiblock partial failure mechanism and 3D numerical analysis are as shown in Table 2. The results well agree with each other. It should be noted that the multiblock model is the upper-bound solution; it is possible that a smaller value of the critical slurry pressure for blow-out and greater one for collapse could be found.

- 3. The results of 3D numerical analysis show that, in blow-out, the failure area on the tunnel face is described by an elliptic shape, with a long axis in horizontal direction, and a short axis in vertical direction. However, this shape of failure mechanism is different from the assumption of the multiblock model of upper-bound theorem, which is an elliptic shape with long axis in vertical direction.
- 4. In multiblock model analysis of case 2, a mean value of cohesion  $c_u$  from the top to the bottom of the failure block is employed. However, according to the shape of the failure block, the changing cohesion will influence the energy dissipation along the lateral surface of failure block. A more precise calculation of energy dissipation along the failure surface with  $c_u$  changing with depth is necessary to take into account. Also, the rotation of failure block caused by the eccentricity of the slurry pressure should be considered in multiblock model to obtain better results.

#### ACKNOWLEDGEMENTS

The research were conducted with funding provided by the National High Technology Research and Development Program (863 Program) of China and Shanghai Leading Academic Discipline Project, Project Number: B308. The first author is grateful to EGIDE to provide the French funding scholarship of her doctoral stay in INSA Lyon. Particular thanks are due to Prof. Soubra. A. H. for important discussion and the use of multiblock software.

#### REFERENCES

- Anagnostou, G. & Kovári, K. 1994. The Face Stability of Slurry-Shield-Driven Tunnels. *Tunnelling and Underground Space Technology*. Vol.9, No. 2:165–174.
- Broere, W. 2001. Tunnel Face Stability & New CPT Applications. PhD thesis, Delft University of Technology. Delft University Press, the Netherlands.
- Chambon, P. & Corté, J.F. 1994. Shallow tunnels in cohesionless soil: Stability of tunnel face. ASCE Journal of Geotechnical Engineering. 120: 1148–1165.
- Davis, E.H., Gunn, M.J., Mair, R.J. & Seneviratne, H.N. 1980. The Stability of Shallow Tunnels and Underground Openings in Cohesive Material. *Geotechnique*. 30 (4): 397–416.
- Gioda, G. & Swoboda, G. 1999. Developments and Applications of the Numerical Analysis of Tunnels in Continuous Media. *International Journal for Numerical and Analytical Methods in Geomechanics*. 23: 1393–1405.
- Horn, M. 1961. Horizontal earth pressure on perpendicular tunnel face. Hungarian National Conference of the Foundation Engineer Industry, Budapest. (In Hungarian)
- Leca, E. & Dormieux, L. 1990. Upper and lower bound solutions for the face stability of shallow circular tunnels in frictional material. *Géotechnique* 40, 4: 581–606.
- Soubra, A.H. 2000. Three-dimensional face stability analysis of shallow circular tunnels. *International Conference* on Geotechnical and Geological Engineering, 19–24 November 2000. Melbourne, Australia.
- Soubra, A.H. 2002. Kinematical approach to the face stability analysis of shallow circular tunnels. 8th International Symposium on Plasticity, 443–445. Canada, British Columbia.
- Subrin, D. & Wong, H. 2002. Tunnel face stability in frictional material: a new 3D failure mechanism. C. R. Mecanique, 330: 513–519. (In French)

### Effects of soil stratification on the tunneling-induced ground movements

F.Y. Liang, G.S. Yao & J.P. Li

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: The available analytical predictions for tunneling-induced ground movements are usually based on the assumptions that the ground is homogeneous and elastic. Actually, effects of soil stratification should be taken into account. In order to clarify this problem, a FLAC3D computational model was conducted to investigate the effect of vertical stratification in soil on the tunneling-induced ground movements. The applicability of the presented model is verified with other available published results as well as the field case histories. Surface settlements and lateral displacements induced by tunneling with layered soil model were compared with those based on homogeneous soil. The results show that the stratification of soil should not be neglected in the analysis of the tunneling-induced ground movements, especially when the soil is assumed to be an elastic medium.

#### 1 INTRODUCTION

Tunneling process will inevitably result in ground movements, and as a result tunnel constructions in urban area may cause serious damage to the adjacent preexisting buildings or public facilities. Therefore one of the important issues of tunneling in urban areas is the estimation of potential ground movements. Methods for estimating the ground deformations due to tunneling may be classified into three categories: empirical methods, numerical methods, and analytical methods.

In the engineering practices, ground deformations are often described as a normal distribution curve with empirical formulas based on field observations (e.g., Peck 1969; New & O'Reilly 1991). These methods are limited in their applicability to cope with different ground conditions and construction techniques, and the information they can provide is limited in the horizontal movements and subsurface settlements.

Some attempts have been made to develop simple closed-form analytical solutions for tunnelinginduced ground movements in clays with the assumption of a uniform radial or oval-shaped ground deformation pattern around the tunnel section (e.g., Sagaseta 1987; Verruijt & Booker 1996; Loganathan & Poulos 1998; Park 2004). However, the effect of soil stratification was not considered in these analytical methods.

Based on the three-dimensional (3-D) nature of the stress changes and deformations, numerical methods such as finite element method and boundary element method could be used to analyze the effect of soil stratification on the tunneling-induced ground movements (Mair et al. 1996) and 3-D finite element simulation models for shield-driven tunnel excavation, taking into account relevant components of the construction process, have been developed (e.g., Lee & Rowe 1991; Rowe & Lee 1992).

In order to analyze the effects of soil stratification on tunneling-induced ground movements, a 3-D computational model was used to simulate the tunnel excavation in soft soil in this paper. Firstly, the factors of the computational model taking into account the soil stratification were described. Secondly, the simulation of the Thunder Bay Tunnel in layered soft soil using FLAC3D with Mohr-Coulomb material model was carried out. The predicted ground movements were investigated in detail. In order to check the applicability of the proposed computational model, the results were compared with those from Lee & Rowe (1991) and the field data observed by Belshaw & Palmer (1978). In the end, another simulation for the tunnel using the elastic model was carried out. The results of the ground movements were compared with the analytical results of Loganathan & Poulos (1998) and the field data observed by Belshaw & Palmer (1978) and Lee et al. (1992). The effects of soil stratification were discussed and some suggestions for future research were also put forward.

#### 2 METHOD OF ANALYSIS

#### 2.1 Simulation of the ground loss

The settlements caused by tunneling are often characterized by the term "ground loss". The main cause to the ground loss is that there is the gap between the

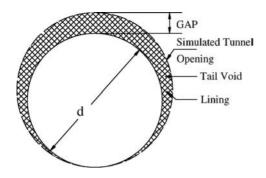


Figure 1. Definition of GAP (Lee et al. 1992).

construction section and the design section. Then the actual ground movement at the tunnel opening section is oval shaped. These movements can be approximately incorporated to the volume of ground loss through the heading and over the shield (as shown in Figure 1). Thus the gap parameter (GAP) can be considered as the maximum settlement at the tunnel crown, and it may be expressed as Lee et al. (1992).

$$GAP = G_p + u_{3p}^* + \omega \tag{1}$$

where  $G_P$  = physical gap ( $G_P = 2\Delta + \delta$ ) that represents the geometric clearance between the outer skin of the shield and lining;  $\Delta$  = thickness of the tailpiece;  $\delta$  = clearance required for erection of the lining;  $u_{3D}^*$  = equivalent 3-D elasto-plastic deformation at the tunnel face; and  $\omega$  = value that takes into account the quality of workmanship.

Applying tail void grout will lead to a much smaller value of *GAP* than indicated here, but this value is a possible maximum.

#### 2.2 Principal assumptions

The medium around the tunnel almost always exists in the form of stratification features, and the tunneling analysis is a matter of three-dimensions rather than two-dimension. FLAC3D is used to simulate the tunneling-induced ground movements in this paper. FLAC3D is a finite difference three-dimensional software package based on the Lagrangian difference method. FLAC3D can offer an ideal analysis tool for solution of three-dimensional problems in geotechnical engineering. The assumptions of the analytical method are presented in brief as follows.

- 1. A plane contact on the interface of different soil layers is assumed. The transverse movements between the soil layers are neglected, and then the friction force of the soil layers can be ignored.
- 2. The inner material is homogeneous in the same soil layer. The influence of the ground water is neglected.

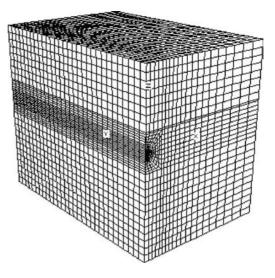


Figure 2. 3-D model geometry used in analysis.

- Uniform pressure is applied to the ground surface as the external loads. The ground stress field is generated by the gravity force.
- The medium is simplified as the continuum, ideal elasto-plastic material, and the Mohr-Coulomb material model is adopted.
- The oval-shaped ground deformation pattern is imposed as the boundary condition at the tunnel opening.

#### 3 EFFECTS OF SOIL STRATIFICATION USING ELASTO-PLASTIC MODEL

#### 3.1 Calculation model and boundary conditions

A 3.3-km-long, 2.47-m-diameter section of the sanitary trunk sewer tunnel in the city of Thunder Bay, Ontario, Canada, was constructed in 1976. The tunnel was constructed through soft clay using a tunnelboring machine together with a segmented precast concrete tunnel lining. The details of tunnel construction, soil condition, and the tunnel dimensions were published elsewhere by Belshaw & Palmer (1978) and Lee & Rowe (1991).

Because of the symmetry of the structure and the influence range of the tunnel excavation, only half the tunnel is considered. The depth from the ground surface to the tunnel axis is 10.7 m. The model geometry is 30 m long (y direction), 25 m deep (z direction) and 20m wide (x direction). Figure 2 illustrates the 3-D mesh used in this analysis. There are 21240 elements and 23464 nodes. The lateral and bottom surface of the model are displacement boundaries. The model is

fixed in the horizontal direction at lateral surface, and the vertical displacement is limited at the bottom surface. The top surface of the model is the ground surface and is free in all directions.

For the Thunder Bay tunnel, the estimation of the gap parameter proposed by Lee et al. (1992) has been used and the physical gap  $G_P$  is 90 mm. The calculation has two stages. Firstly, the radial oval displacement as a boundary condition is applied on the tunnel crown. The 3D movements ahead of the face into the zone to be excavated can then be determined, and then  $u_{3D}^*$  can be evaluated and used to calculate the total gap GAP. Secondly, the model based on the tunnel diameter 2R + GAP, is established. The radial oval displacement as a boundary condition is applied on the tunnel diameter displacement around the tunnel can be simulated.

In this study, the structural behavior of the tunnel shield and lining are neglected. In order to ensure the accuracy of the simulation, three cases are assumed.

- The tunnel is assumed to be excavated under perfect alignment, and the tunnel machine is pressed hard against the face so that 3-D movement will be minimized.
- 2. Full release of axial stress at the tunnel face is assumed, which would result in the full development of 3-D ground loss ahead and over the tunnel shield.
- 3. The excavation of the tunnel is the instantaneous excavation, and the interrelationship between the supporting and lining of the tunnel is neglected. Results of the ground movements (surface and sub-surface settlements, lateral displacement) from the numerical simulation are defined as the final ground movements after the tunnel excavation.

# 3.2 Comparison with observed results and other numerical methods

In this section, the soil is assumed to have an elasticperfectly plastic constitutive relationship and a Mohr-Coulomb material model. The silt and silty sand strata for the tunnel are divided into four sublayers according to the data in Lee & Rowe (1991). The layers were assigned soil parameters on the basis of soil densities, laboratory testing, and empirical correlations with SPT data. The parameters adopted for these materials in the present analysis are based on the research of Lee & Rowe (1991) and summarized in Table 1. The material parameters for the uniform soil model are the average values of the four layers respectively.

Results from the presented analysis were compared with those from Lee & Rowe (1991) as well as field measurements recorded by Belshaw & Palmar (1978). The analysis model used by Lee & Rowe (1991) was also based on elasto-plastic constitutive relationship.

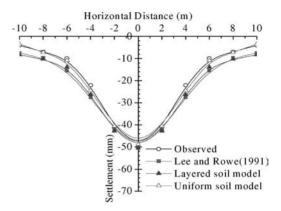


Figure 3. Surface settlement troughs at various distances from tunnel face.

3.2.1 *Surface settlement along transverse section* The development of transverse surface settlement troughs (i.e., settlement profile along the ground surface perpendicular to the direction of tunneling) for various calculation methods at 2.2 m behind the tunnel face are shown in Figure 3.

The shape of surface settlement troughs calculated with layered soil model is consistently slightly wider than the observed troughs, and its values are approximate to the results from Lee & Rowe (1991). From the curves, the settlement values based on the uniform soil model slightly underestimate the values based on the layered soil. Figure 3 shows that consideration in lamination of the soil has only a small effect on the calculated surface settlement trough.

#### 3.2.2 Variation of lateral displacement

The lateral displacement values when the tunnel face was 15 m away from the monitoring points are shown in Figure 4. The monitoring data are 2.2 m away from the vertical centerline of the tunnel. The present results based on the layered soil are very similar to those obtained by Lee & Rowe (1991) using the 3-D finite element analysis. The results from the two methods are in good agreement with the field data. From the curves in Figure 4, the horizontal movement values based on the uniform soil model greatly underestimate the field data above the tunnel horizontal centerline, and the latter are almost  $1.5 \sim 8$  times larger than the former. Under the tunnel horizontal centerline, the lateral displacements based on uniform soil model decrease rapidly and are close to those based on layered soil and the field data. As well known, the lateral displacement above the tunnel centerline has an important influence on the movement of structures located close to the tunnel. Figure 4 shows that consideration of soil lamination has a significant effect on the lateral displacement.

### 3.2.3 Variation of subsurface settlement with depth above the tunnel axis

The observed and calculated subsurface settlement distributions with depth at 15 m from the tunnel face are shown in Figure 5. Results based on the layered soil are very close to the results obtained by Lee & Rowe (1991), and they are also in good agreement with the field data. The settlement values based on the uniform soil model underestimate the field data in the whole measurement field, and the latter are almost  $1.06 \sim 1.30$  times larger than the former.

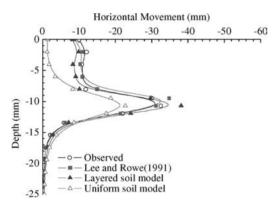


Figure 4. Lateral displacement 15 m behind the tunnel face.

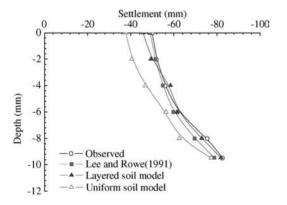


Figure 5. Subsurface settlements with depth above tunnel axis.

From the above analyses, results from this model
taking into account the stratification of the soil are in
good agreement with those obtained by Lee & Rowe
(1991) as well as the field data, and more accurate than
those based on the uniform soil model. Therefore the
FLAC3D model established in this paper proves to be
reasonable and reliable for the simulation of tunneling-
induced ground movement.

#### 4 EFFECTS OF SOIL STRATIFICATION USING ELASTIC MODEL

Loganathan & Poulos (1998) redefined the ground loss parameter with respect to the gap parameter and incorporated this into the closed form solution derived by Verruijt & Booker (1996). The proposed approach tended to overpredict the ground loss parameter for tunnels in soft clay, and the predicted surface settlement troughs were wider than the field observations. Nevertheless, in general, the observed and calculated settlement and horizontal movement are in good agreement for tunnels in uniform clay. However, the applicability of the solution based on uniform soil model should be discussed when it was applied to layered soils.

In this section, the soil is assumed to have a linear elastic constitutive relationship. The ground around the tunnel is also divided into four sublayers. The parameters adopted for these materials in the present analysis are summarized in Table 1. The material parameters of uniform soil model are the average values of the four layers respectively.

Results from the present analysis are compared with those from Loganathan & Poulos (1998) as well as field measurements recorded by Belshaw & Palmar (1978) and Lee et al. (1992). The effects of the stratification of the soil on the ground movements are analyzed in the following.

#### 4.1 Surface settlement along transverse section

Settlement results from this calculation, Loganathan & Poulos (1998) and the field data (Lee et al. 1992) are shown in Figure 6. Apparently, the values predicted by Loganathan & Poulos (1998) overestimate the field

No.	Description of soil layers	Thickness (m)	$\gamma (kN \cdot m^{-3})$	$E_u$ (MPa)	$E_b$ (MPa)	c (kPa)	$\varphi\left(^{o} ight)$	$K_0$
1	Peat	1.0	14.00	12.3	213.0	39	30	0.5
2	Loose silty sand	7.0	19.77	15.4	119.3	40	32	0.52
3	Silty clay	5.2	17.45	9.2	110.5	29	26	0.88
4	Varved clay	11.8	18.70	32.0	240.0	59	34	0.8

Table 1. Mechanical parameters of materials.

Notes:  $\gamma$ —unit weight of soil;  $E_u$ —shear modulus;  $E_b$ —bulk modulus; c—cohesive strength;

 $\varphi$  —internal friction angle;  $K_0$ —coefficient of static earth pressure.

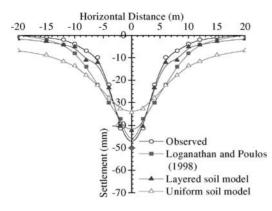


Figure 6. Surface settlement troughs at various distances from tunnel face.

data. The values based on the layered soil underestimate the values predicted by Loganathan & Poulos (1998) and are close to the field data despite the fact that they are slightly greater than the field data. From the curves, the settlement values based on the uniform soil model are greater than the values based on the layered soil in most areas and the trough of the former is obviously wider than that of the latter. But the maximum settlements obtained by this model underestimate the value obtained by Loganathan & Poulos (1998) and the field data at the tunnel crown.

#### 4.2 Variation of lateral displacement

The values of the lateral displacements monitoring points are shown in Figure 7. The values obtained by Loganathan & Poulos (1998) overestimate the field data (Belshaw & Palmer 1978) within the depth range from 5 m to 15 m. The values based on the layered soil are in good agreement with the field data. From the curves, when adopted the uniform soil model, the values are less than the field data at the range of depth from 0 to 7 m, and close to the field data at the range of depth from 7 m to 11 m and greater than the field data at the range of depth from 11 m to 25 m. Figure 6 shows that the prediction of the ground lateral displacements based on the layered soil model is better than the uniform model.

### 4.3 Variation of subsurface settlement with depth above the tunnel axis

The results of the subsurface settlement above the tunnel centerline are shown in Figure 8. The values obtained by Loganathan & Poulos (1998) underestimate the field data (Lee et al. 1992) greatly in spite that the trends of both profiles are similar. The present results based on layered soil show larger values for the surface settlement than those from Loganathan & Poulos (1998) and close to the field data except that

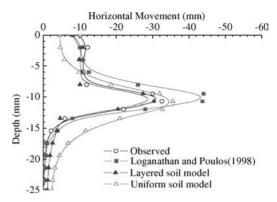


Figure 7. Lateral displacement 15 m behind the tunnel face.

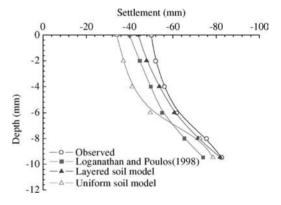


Figure 8. Subsurface settlements with depth above the tunnel axis.

there is an error of  $7 \sim 11$  per cent at the range of depth from 0 to 2 m. At other depths the error is less than 5 per cent. The subsurface settlement values based on the uniform soil model obviously underestimate the field data and there is an error of  $20 \sim 30$  per cent between them.

#### 5 CONCLUSIONS

A FLAC3D computational model for simulating the displacement caused by excavation of the Thunder Bay tunnel has been performed, and the effects of the stratification of the soil on the tunneling-induced ground movements are analyzed sufficiently.

Results from the model based on layered soils are in good agreement with those obtained by Lee & Rowe (1991) as well as the field data. Comparatively, results disregarding the stratification of the soil will have distinct errors. Consequently, in engineering practice, the stratification of soils should not be neglected in the analysis of the tunneling-induced ground movements. Furthermore, the results based on the elasto-plastic model can simulate the deformation curves better than those based on the elastic model.

Based on the elasto-plastic model, the analysis results showed that the soil stratification has very few effects on the ground surface settlement, but it has greater effects on the ground lateral displacement and sub-surface settlement.

While based on the elastic model, the analysis results showed that the soil stratification has some distinct effects in the ground surface settlement, lateral displacement and sub-surface settlement respectively.

In order to obtain more reasonable analytical solution to the tunneling-induced ground movements, it is necessary to incorporate the stratification of soils into the analytical solution proposed by Loganathan & Poulos (1998).

#### ACKNOWLEDGEMENTS

This work reported here is partly supported by National Natural Science Foundation of China (Grant No.: 50708078) and Program for Young Excellent Talents in Tongji University (Grant No.: 2006KJ043). The authors wish to express their gratitude for the financial assistances.

#### REFERENCES

Belshaw, D.J. & Palmer, J.H.L. 1978. Results of a program of instrumentation involving a precast segmented concretelined tunnel in clay. *Canadian Geotechnical Journal* 15: 573–583.

- Lee, K.M. & Rowe, R.K. 1991. An analysis of threedimensional ground movements: the Thunder Bay tunnel. *Canadian Geotechnical Journal* 28: 25–41.
- Lee, K.M., Rowe, R.K. & Lo, K.Y. 1992. Subsidence owing to tunnelling. I. Estimating the gap parameter. *Canadian Geotechnical Journal* 29: 929–940.
- Loganathan, N. & Poulos, H.G. 1998. Analytical prediction for tunnelling-induced ground movements in clays. J. Geotech. Geoenviron. Eng. 124(9): 846–856.
- Mair, R.J. 1996. Prediction of ground movements and assessment of risk of building damage due to bored tunneling. *Proceedings of geotechnical aspect of underground construction in soft ground*, Rotterdam, Balkema Press: 713–718.
- New, B.M. & O'Reilly, M.P. 1991. Tunnelling induced ground movements, predicting their magnitude and effects. Proc., 4th Conf. on Ground Movements and Structures, Cardiff, Wales, Pentech Press: 671–697.
- Park, K.H. 2004. Elastic solution for tunnelling-induced ground movements in clays. *Int. J. Geomech.*, 4(4): 310–318.
- Peck, R.B. 1969. Deep excavations and tunnelling in soft ground. Proc., 7th Int. Conf. on Soil Mech. and Found. Engrg.: 225–290.
- Rowe, R.K. & Lee, K.M. 1992. Subsidence owing to tunnelling. II. Evaluation of a prediction technique. *Canadian Geotechnical Journal* 29: 941–954.
- Sagaseta, C. 1987. Analysis of undrained soil deformation due to ground loss. *Geotechnique* 37: 753–756.
- Verruijt, A. & Booker, J.R. 1996. Surface Settlements Due to Deformation of a tunnel in an elastic half plane.*Geotechnique* 46(4): 753–756.

# Centrifuge modelling to investigate soil-structure interaction mechanisms resulting from tunnel construction beneath buried pipelines

#### A.M. Marshall & R.J. Mair

Department of Engineering, University of Cambridge, Cambridge, UK

ABSTRACT: New underground construction is undertaken increasingly close to existing buried structures. The resulting effects on these structures must be properly evaluated. This paper examines the case of tunnel construction built transversely to existing buried continuous pipelines using data obtained from tests within the University of Cambridge Geotechnical Beam Centrifuge. This research aims to visually validate hypothesized soil-structure interaction mechanisms that account for pipeline behaviour. This is accomplished by placing the tunnel-soil-pipeline system directly against a Perspex wall within the centrifuge package so that digital images can be taken of the soil and buried structures. Particle Image Velocimetry (PIV) is used to measure displacements and provide a complete description of the soil-structure interactions.

#### 1 INTRODUCTION

Beneath the surface of any major city is an intricate and increasingly congested series of tunnels, pipelines, and buried structures. New tunnels are constructed for various purposes and form an important part of urban infrastructure. Tunnel construction has varying effects on surrounding ground depending on soil type and construction process (Mair and Taylor, 1997). The design of new tunnels must account for likely effects on nearby buried structures.

Analytical methods for this problem typically fall within three categories: [1] finite element (FE) or finite difference (FD) methods, [2] Winkler-type models, and [3] continuum solutions. Winkler models and continuum solutions generally provide quick solutions (see for example Attewell et al., 1986; Klar et al., 2005; Vorster et al., 2005; Klar et al., 2007) however they all have similar simplifying assumptions that limit their applicability. Klar & Marshall (2007) validated some of the simplified assumptions of the continuum method by comparison with more rigorous shell structure solutions. FE and FD methods are best suited for the problem because they can incorporate the complex tunnel-soil-pipeline interactions. For these models to be applicable, they must be validated against real soil and soil-structure behaviour.

Vorster et al. (2005) proposed a series of global and local soil deformation mechanisms to account for observed pipeline behaviour above a tunnel during centrifuge testing. The proposed mechanisms are based on deformation and stress measurements made within the soil mass.

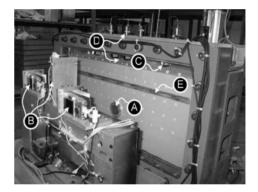


Figure 1. Centrifuge package.

The current research aims to elucidate the soilstructure interaction mechanisms that account for the behaviour of existing tunnels and buried pipelines located above new tunnel construction by providing visual evidence of soil and structure deformations. This is accomplished by performing similar tests to those described in Vorster et al. (2005) but with a new centrifuge package that places the tunnel-soil-pipeline system adjacent to a Perspex wall such that digital images of the system can be analyzed using Particle Image Velocimetry (PIV).

#### 2 CENTRIFUGE PACKAGE

A centrifuge package was developed that allowed visual observations to be made of the soil, model

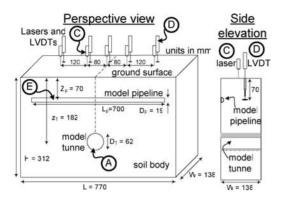


Figure 2. Schematic of centrifuge package.

tunnel, and buried pipeline. In doing so, digital images could be taken during the test and PIV (White et al., 2003) could be used to accurately measure soil and structure displacements.

Figures 1 and 2 present the developed centrifuge package. The relevant components are labelled as follows:

A: The model tunnel: composed of a stiff inner cylinder, fixed within the box walls, and sealed within a flexible rubber membrane. Fluid is extracted from within the sealed rubber membrane to replicate tunnelling volume loss;

B: Three 8 megapixel digital cameras. These are used to photograph the soil and buried structures for PIV analysis;

C: Five lasers used to measure surface vertical displacements within the middle third of the box where boundary effects are negligible (see Figure 2 for locations);

D: Five linear variable differential transformers (LVDTs) used to measure subsurface vertical displacements within the middle of the box and at a depth of 70 mm, corresponding to the axis of the model pipeline (see Figure 2 for locations);

E: The model pipeline: consists of an aluminium half-cylinder with a length of 700 mm, an outer radius of 9.5 mm, and a wall thickness of 1.6 mm.

All tests were performed at 75 g using dry Leighton Buzzard fraction E sand at a relative density of approximately 90%.

#### **3 BASELINE TESTS – NO PIPELINE**

Two tests were completed in which no model pipeline was included. These tests served two purposes: first, to investigate the effects of placing a layer of glass at the Perspex interface (thought to reduce boundary effects) and second, to obtain baseline displacement data for the simulation of tunnelling in sand.

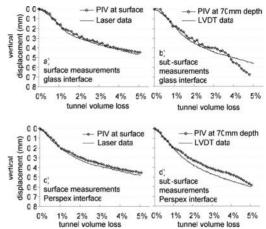


Figure 3. Vertical soil displacement data for a) surface measurements with glass interface, b) sub-surface measurements with glass interface, c) surface measurements with Perspex interface, and d) sub-surface measurements with Perspex interface.

Volume loss (VL) is used as a reference throughout the following text. Two methods of determining volume loss are referred to and are defined as follows:

- 1 Tunnel volume loss: the change in tunnel volume (or volume of fluid extracted from the model tunnel) divided by the original total tunnel volume, expressed as a percent (VL<sub>tunnel</sub> =  $\Delta V_{tunnel}/V_{tunnel} \times 100$ )
- 2 Soil volume loss: calculated by integrating the soil settlement profile at a given depth and dividing by the original tunnel volume, expressed as a percent  $(VL_{soil} = V_{trough}/V_{tunnel} \times 100)$ .

#### 3.1 Assessment of boundary friction using glass and Perspex

Boundary effects are an issue when obtaining PIV data from centrifuge tests. An assessment of the boundary effects on the obtained PIV results was carried out by comparing the PIV displacement data to that obtained from the lasers and LDVTs which were placed within the middle third of the thickness of the box (where boundary effects are negligible). Glass is often used as an interface between the soil and the Perspex wall when performing PIV tests because it is harder than Perspex and therefore prevents sand grains from scraping into the Perspex at high stress levels (a phenomenon which causes increased boundary friction).

Two tests were carried out: one with glass and one with Perspex alone. Figure 3 presents a comparison between the PIV centreline data (directly above the tunnel) and the surface (laser) and sub-surface (LVDT) data, for both the glass and a Perspex interface tests. Figure 3 shows very good correlation between the PIV data and that obtained from measurements made within the middle of the box for both glass and Perspex interfaces, indicating very little friction at the interface.

The use of glass caused optical problems (coloured zones caused by refraction of light) which affected the quality of the PIV analysis. It was therefore decided that glass would not be used since the PIV displacement data was effectively the same for glass and Perspex.

The reason for the departure of the PIV data from the general trend in Figure 3b) at a tunnel volume loss of about 3.7% is not known. It may be a result of soil loss between the glass and the Perspex around the model tunnel. This potential problem was addressed in subsequent tests by sealing circumference of the tunnel ends with grease.

#### 3.2 Tunnelling in sand

The baseline tests provided useful data regarding the effects of tunnelling in sand. Figure 4 presents the variation of volume loss at various depths within the soil for a) the baseline test with a glass interface, and b) the baseline test with a Perspex interface. The 1:1 line represents the volume loss of the model tunnel. When the slope of a line is greater than 1, the soil is experiencing a volume loss that is larger than that provided by the volume loss in the tunnel.

Figure 4 illustrates that the value of volume loss for sands is not unique. In clays, it is correctly assumed that volume loss does not change with depth (undrained case, constant volume) and therefore calculation of volume loss derived from surface settlements should give an accurate estimate of volume loss within the tunnel. However, estimation of tunnel volume loss using soil displacement data is not so simple for sands. As Figure 4 illustrates, it depends very much on the magnitude of tunnel volume loss achieved. The trend of soil volume loss presented in Figure 4 was also noted by Vorster (2005) and Jacobsz (2002) for similar centrifuge tests in sands.

Examining Figure 4, we observe that the surface and sub-surface volume loss are very similar up to a tunnel volume loss of about 0.5%, after which the volume losses calculated at different depths within the soil diverge. The maximum ratio of surface volume loss to tunnel volume loss is 1.38 for the glass interface test and 1.67 for the Perspex interface test. This maximum ratio occurs at a tunnel volume loss of between 1.0 and 1.2%. As tunnel volume loss is increased further, the rate of change of soil volume loss relative to tunnel volume loss begins to reduce. Surface volume loss falls below tunnel volume loss at a tunnel volume loss of between 2.3 and 2.6%. The slope for the subsurface soil nearest to the tunnel is shown to increase to about 1 after a tunnel volume loss of approximately 3%.

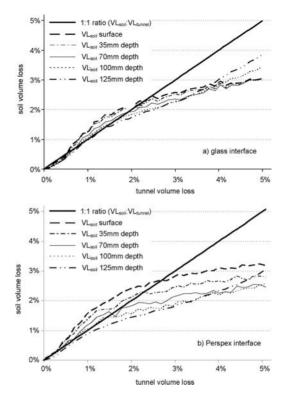


Figure 4. Soil volume loss at various depths compared to tunnel volume for test with a) glass interface, and b) Perspex interface (VL<sub>soil</sub> calculated using PIV displacement data).

Figure 5 presents a probable explanation for some of the results shown in Figure 4 by examining volumetric strains within the soil (derived from the PIV analysis of the Perspex interface test). Figure 5a) illustrates that at a low volume loss of 0.54%, the soil is generally in a contractive mode in a fan-shaped zone above the tunnel. This explains the tendency towards larger soil volume losses at lower tunnel volume losses in Figure 4. As tunnel volume loss is increased to 1% and higher, a local zone of dilation develops in the soil above the tunnel. As this dilatant zone grows in size and magnitude, it compensates for the contractive soil above it and results in lower calculated values of soil volume loss, thus reducing the slope of the lines in Figure 4.

#### 4 PIPELINE ABOVE TUNNEL

Centrifuge testing of a model pipeline placed transversely above the tunnel was performed in order to investigate soil displacements and structural behaviour of the pipe.

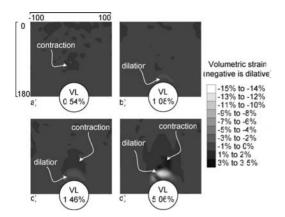


Figure 5. Development of volumetric strain as tunnel volume loss is increased (from PIV data of Perspex interface test).

The relevant dimensions of the test are given in Figure 1. In prototype scale, the test represents a 4.65 m diameter tunnel constructed at a depth of 13.65 m and running transversely beneath a 1.43 m pipeline buried at a depth of 5.25 m. The tunnel cover to diameter ratio  $(C_T/D_T)$  is 2.4 while the pipeline cover to diameter ratio  $(C_p/D_p)$  is 3.1.

Bending moment data was obtained using the displacement data of the pipeline provided by the PIV analysis (Fig. 6). The vertical deflection data of the pipeline was found to fit well to a modified Gaussian curve (Equation 1) in the form presented by Vorster et al. (2005).

$$S_{v}(x) = S_{\max} \frac{n}{(n-1) + \exp\left[\alpha \left(\frac{x}{i}\right)^{2}\right]}$$
$$n = \exp(\alpha) \frac{2\alpha - 1}{2\alpha + 1} + 1$$
(1)

where  $S_v =$  vertical displacement of pipe lining; x = offset from tunnel centreline;  $S_{\text{max}} =$  maximum vertical displacement of pipe lining; i = distance to inflexion point of the displacement curve; n = shape function parameter to control the width of the displacement curve; and  $\alpha =$  parameter to ensure i is consistent with the Gaussian curve presented by Peck (1969). The optimum fit was obtained using i = 55 mm and  $\alpha =$  0.01 (resulting in n = 0.03). Note that when  $\alpha = 0.5$ (n = 1), Eq. 1 becomes the Gaussian curve.

Bending moments were derived using elastic beam theory whereby moments are related to the deformed shape of a beam by:

$$M(\mathbf{x}) = E_p I_p \frac{d^2 S_v(\mathbf{x})}{dx^2}$$
(2)

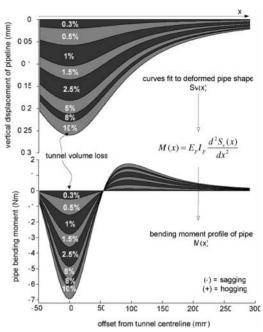


Figure 6. Derivation of bending moments from deformed shape of pipeline (from PIV data).

where M = bending moment;  $E_p =$  Young's modulus of the pipe material; and  $I_p =$  second moment of the cross-sectional area of the pipe. Note that these curves represent an approximation of the true bending moments since the fitted curve can not match exactly the actual deformed shape of the pipe.

There is a concern that tunnelling may cause gaps below certain overlying structures. Figure 7 presents PIV displacement data to illustrate that a gap does indeed form below pipelines affected by tunnelling. Soil displacements directly below the pipeline are compared to pipeline displacements at the tunnel centerline. The upper and lower pipe linings are shown to displace the same amount (i.e. no cross-sectional distortion of the pipe occurs). Gap formation is shown to commence at a tunnel volume loss of between 1 and 2%. The length of the gap (along the pipe) varied from 2.9 pipe diameters at low volume loss to 3.4 pipe diameters at 10% tunnel volume loss.

Figure 7 also shows a plot of the maximum sagging moment of the pipe versus tunnel volume loss. The data suggests that bending moments increase substantially with the onset of tunnel volume loss, however pipe bending behaviour does not appear to be very sensitive to the formation and growth of the gap that forms below the pipe. The height of the gap (i.e. the separation between the soil and the lower pipe lining) is shown to increase significantly at a volume loss of about 6% with no equivalent response in the pipe

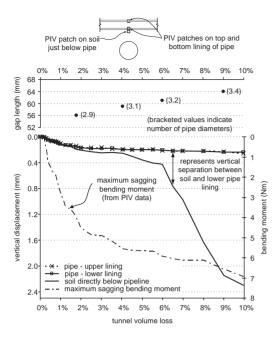


Figure 7. Gap formation below pipeline and increase in maximum sagging moment as tunnel volume loss is increased.

bending moment. The width of the gap does not grow substantially and is the likely reason why the bending moments do not respond to sudden increases in gap height at the tunnel centreline.

#### 5 CONCLUSIONS

Soil-structure interaction mechanisms are important when evaluating the response of buried structures to ground movements. The research described in this paper has elucidated the soil-structure interaction mechanisms that occur when tunnelling beneath buried pipelines.

The results presented illustrate some important aspects of tunnelling within sandy ground (i.e. drained material). These include:

- soil volume loss will not always be the same as tunnel volume loss;
- the value of soil volume loss depends on the magnitude of the tunnel volume loss;
- soil volume loss calculated at the surface can be greater or lower than tunnel volume loss;

- the maximum ratio of surface volume loss to tunnel volume loss was as high as 1.67 and occurred at a tunnel volume loss of between 1 and 1.2%;
- a local zone of dilation formed just above the tunnel at a tunnel volume loss of about 1%.
- The zone of dilation effectively reduces the soil volume losses above it.

The pipeline test illustrated that a gap forms below the pipe at a volume loss of between 1 and 2%. The length of the gap grew from 2.9 tunnel diameters at low volume loss to 3.4 tunnel diameters at 10% tunnel volume loss. Bending moments induced in the pipe increase from the onset of tunnel volume loss and do not appear to be sensitive to the growth of the gap height.

#### REFERENCES

- Attewell, P.B., Yeates, J. & Selby, A. R. 1986. Soil movements induced by tunnelling and their effects on pipelines and structures. Blackie and Son Ltd, UK.
- Jacobsz, S.W. 2002. The effects of tunnelling on piled foundations. PhD Thesis, University of Cambridge.
- Klar, A., Vorster, T. E. B., Soga, K. & Mair, R. J. 2005. Soil Pipe Interaction due to tunnelling: Comparison between Winkler and Elastic Continuum Solutions. *Geotechnique*, 55(6), 461–466.
- Klar, A. & Marshall, A.M. 2007. Shell versus beam representation of pipes in the evaluation of tunneling effects on pipelines. accepted, *Tunnelling and Underground Space Technology*.
- Klar, A., Marshall, A.M., Soga, K. & Mair, R.J. 2007. Tunneling effects on jointed pipelines. accepted, *Canadian Geotechnical Journal*.
- Mair, R. J. & Taylor, R. N. 1997. Bored Tunnelling in the urban environment. Proceed. 14th International conference on soil mechanics and foundation engineering. Hamburg: Balkema. 4:2353–2385.
- Peck, R.B. 1969. Deep excavation and tunnelling in soft ground. Proceed. 7th International conference on soil mechanics and foundation engineering. Mexico City. 266–290.
- Vorster, T. E. B. 2005. The effects of tunnelling on buried pipes. PhD Thesis. Cambridge University.
- Vorster, T. E. B., Klar, A., Soga, K. & Mair, R. J. 2005. Estimating the Effects of Tunneling on Existing Pipelines. *Journal* of Geotechnical and Geoenvironmental Engineering, 131 (11), 1399–1410.
- White, D. J., Take, W. A. & Bolton, M. D. 2003. Soil deformation measurement using particle image velocimetry (PIV) and photogrammetry. *Geotechnique*, 53(7), 619–631.

## Ground movement and earth pressure due to circular tunneling: Model tests and numerical simulations

H.M. Shahin, T. Nakai, F. Zhang, M. Kikumoto, Y. Tabata & E. Nakahara Nagoya Institute of Technology, Nagoya, Japan

ABSTRACT: Two-dimensional (2D) model tests on tunnel excavation using a newly developed circular tunnel apparatus are carried out. Numerical simulations are also conducted using finite element method under planestrain and drained conditions. In the finite element analyses, elastoplastic subloading tij model is used as a constitutive model of the ground material. From the model tests it is revealed that in the case of the same volume loss due to tunnel excavation, surface settlement and earth pressure around tunnel are significantly influenced by the displacement applied at the tunnel crown for the same overburden. The volume loss is less significant compare to the crown drift in the case of shallow tunneling. The numerical results show very good agreement with the results of the model tests.

#### 1 INTRODUCTION

Shallow tunneling is one of the essential methods to make underground space in urban area. With the ongoing demand of tunneling technology research works on ground movements and earth pressures due to tunnel excavation have been potentially increased. The trap door apparatus has been used to investigate the mechanism of tunneling problems by many researchers (Murayama and Matsuoka, 1971; Adachi et al. 1994, Nakai et al., 1997; Shahin et al., 2004). In our previous work, we have been carried out laboratory model tests using trap-door tunnel apparatus to investigate the deformation mechanism and redistribution of stress surrounding the tunnel. To investigate the deformation mechanism and earth pressure of the ground more precisely, a new and more realistic tunnel apparatus has been developed where the cross section of the tunnel is circular. The apparatus can simulate various excavation methods such as full face excavation, top and side drift and bench cut excavation. This paper reports 2D model tests using the newly developed circular tunnel apparatus and numerical analyses using the subloading  $t_{ii}$  model. This model can consider influence of intermediate principal stress on the deformation and strength of soils, Dependence of the direction of plastic flow on the stress paths, Influence of density and/or confining pressure on the deformation and strength of soils. Model tests are performed with different soil covers and for different excavation patterns. The deformation mechanism is described focusing the ground movements and shear strain patterns.

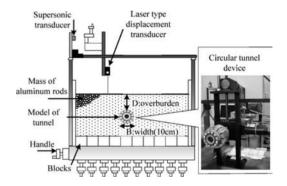


Figure 1. Schematic diagram of 2D tunnel apparatus.

#### 2 DESCRIPTION OF 2D MODEL TEST

#### 2.1 Apparatus of model test

Figure 1 shows a schematic diameter of 2D tunnel apparatus. Figure 2 represents a newly developed model tunnel with circular cross section. It consists of a shim at the center of the tunnel surrounded with 12 segments. The segments are strongly tightened all around the shim with rubber band. One motor is attached with the shim to pull it out in the horizontal direction. The model tunnel is kept in space with a vertical shaft, and can be moved in the vertical direction with another motor. Therefore, the device consists of two motors one is for shrinking the tunnel and the other for moving the tunnel vertically to fix it at a chosen ground

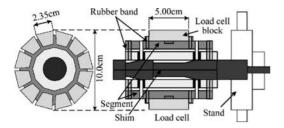


Figure 2. Circular tunnel device.

depth. It is possible to make these motors work simultaneously and individually together with controlling the speed of the motors. With the motor the shim is pulled out gradually which changes the diameter of the shim, consequently the segments move inward and the diameter of the tunnel is reduced. Changing the shape of the shim different kinds of excavation process, such as full face excavation, top and side drift and bench cut excavation can be reproduced with this apparatus. The reduction of tunnel diameter and the amount of radial shrinkage are obtained from a dial gauge reading which is determined from the calibration result. The vertical movement (if requires to impose) of the tunnel is also measured with another dial gauge. Therefore, the shrinkage of the tunnel can be attained in a controlled manner, which can simulate the condition of a real tunnel construction.

In the apparatus 12 load cells are used to measure earth pressure acting on the tunnel. The load cells are attached with the block which is placed surrounding the segments of the tunnel. Each load cell block is 2.35cm in width and 5.0 cm in length. The blocks are tightly fastened with rubber band. Therefore, earth pressure can be obtained at 12 points on the periphery of the tunnel at a time. Earth pressure can also be obtained at other positions by rotating the tunnel. However, in this case it will be required to make the model ground once again. Including the load cell bocks the total diameter of the model tunnel is 10.0 cm. The circular tunnel device is placed on an iron table that was used for the trap door tunnel apparatus (Nakai et al., 1997; Shahin et al., 2004). It has 10 moveable blocks above which the ground is made. The reason of using this type of base is to adjust the initial stress condition of the ground such a way that the stress distribution becomes similar to the ground without tunnel. The surface settlement of the ground is measured using a laser type displacement transducer with an accuracy of 0.0 1mm and its position in the horizontal direction is incurred with a supersonic wave transducer. Photographs are taken during experiments which are later on used as input data for the simulation of ground movements with Particle Image Velocimetry.

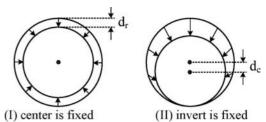


Figure 3. Schematic explanation of excavation patterns.

#### 2.2 Model ground and excavation patterns

Firstly, the tunnel device is set at a height of 10cm; the height is measured from the bottom boundary to the tunnel invert. Varying the distance between the tunnel invert and the bottom boundary, several experiments were conducted. It was found that this height (10 cm) is free from the influence of the bottom boundary. After setting the tunnel device, mass of aluminum rods, having diameters of 1.6 and 3.0 mm and mixed in a ratio of 3:2 in weight, is stacked up to a prescribed depth. The unit weight of the aluminum rod mass is  $20.4 \text{ kN/m}^3$ , and the length is 5.0 cm. The initial ground is made in such a way so that the earth pressure becomes similar to the earth pressure at rest adjusting the bottom moveable blocks of the apparatus. Great care is taken to make a uniform ground and not to apply any undesired load in the ground.

In this study two types of excavation patterns are considered. Pattern 1 (full face excavation) corresponds to the excavation where the center of the tunnel is kept fixed and the diameter of the tunnel is reduced applying shrinkage of 4 mm all around the tunnel as shown in Figure 3. Pattern 2 represents the excavation pattern where the invert is kept fixed (top drift excavation). This is obtained by descending the tunnel during the application of shrinkage. Here, the same amount of shrinkage (4 mm) is applied. However, as the center is moved downward by 4mm the amount of imposed displacement at the tunnel crown is 8 mm. In the both excavation patterns the volume loss of the ground is the same, which is equal to 15.36%. The model tests have been conducted for four kinds of overburden ratio, D/B equals 0.5, 1.0, 2.0 and 3.0, where D is the depth from the ground surface to the top of the tunnel and B(10 cm) is the width of the tunnel. In Figure 3,  $d_r$  represents the amount of shrinkage in the radial direction towards the center of the tunnel, and  $d_c$ indicates the amount of descended of the tunnel center.

#### 3 NUMERICAL ANALYSES

Figure 4 shows the mesh used in the finite element analyses. Isoparametric 4-noded elements are used in the mesh. Both vertical sides of the mesh are free in the

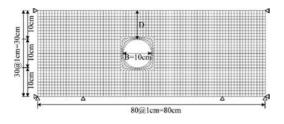


Figure 4. FEM mesh (D/B=2.0).

Table 1. Material parameters for aluminum rods.

0.008	
	-
0.3	Same parameters as
	Cam-clay model
1.8	
0.2	
1.2	Shape of yield surface
	(same as Cam-clay
	at $\beta = 1$ )
1300	Influence of density and confining pressure)
	0.004 0.3 1.8 0.2 1.2

vertical direction, and the bottom face is kept fixed. To simulate the tunnel excavation, horizontal and vertical displacements are applied to the nodes that correspond to the tunnel block. Analyses are carried out with the same conditions of the model tests. In the case of the fixed invert excavation, both horizontal and vertical displacements are applied to the nodes correspond to the blocks of the tunnel in the experiment except the bottom most blocks, where only horizontal displacements are applied to the corresponding nodes. The displacements patterns have been decided from the actual movements of the ground in the experiments. Two-dimensional finite element anlyses are carried out with FEMtij-2D using the subloading  $t_{ii}$  model (Nakai and Hinokio, 2004). Model parameters for the aluminum rod mass are shown in Table 1. The parameters are fundamentally the same as those of the Cam clay model except the parameter a, which is responsible for the influence of density and confining pressure. The parameter  $\beta$  represents the shape of yield surface. The parameters can easily be obtained from traditional laboratory tests. Figure 5 shows the results of the biaxial tests for the mass of aluminum rods used in the model tests. The figure shows the positive and negative dilatancy of aluminum rod mass; and it is clear that the strength and deformation behavior is very similar to those of dense sand. The dotted lines rep resent the numerical results for a confining pressure of 1/100 times the confining pressure of experiments. From the stress-strain behavior of the element tests simulated with subloading  $t_{ij}$  model, it is noticed that

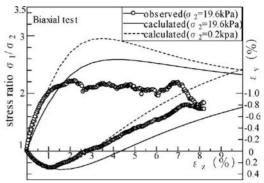


Figure 5. Stress-strain-dilatancy relation.

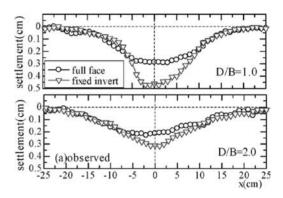


Figure 6. Observed surface settlement profiles.

this model can express the dependency of stiffness, strength and dilatancy on the density as well as on the confining pressure. The initial stresses, correspondent to the geostatic (self-weight) condition, are assigned to the ground in all numerical analyses.

#### 4 RESULT AND DISCUSSION

#### 4.1 Surface settlement

Figure 6 shows the observed troughs of surface settlement in the full surface excavation and top drift excavation for the amount of shrinkage  $d_r = 4$  mm in the case of D/B = 1.0 and 2.0. Figure 7 represents the computed results corresponding to the observed ones. The abscissa represents distance from the center of the tunnel, while the vertical axis shows the amount of surface settlement. For both patterns of excavation the maximum surface settlement occur vertically above the tunnel crown. Surface settlements become smaller with the increase of the tunnel depth, but they extend over a wider region similar to the previous research conducted with trap door tunnel apparatus (Shahin et al., 2004). The shape of the surface settlement

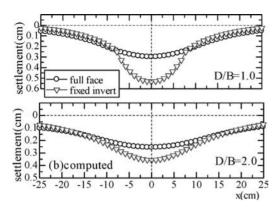


Figure 7. Computed surface settlement profiles.

profiles is the same for all the soil covers in the case of full face excavation. For the same volume loss the maximum surface settlement is larger in the case where the invert is fixed than that of the full face excavation. This is because the applied displacement at the crown is 8 mm for the fixed invert and 4 mm for the full face excavation though the volume loss is the same. Surface settlement occurs locally for 8 mm applied displacement in the case of fixed invert, consequently the shape of surface settlement profile varies with the soil cover in this case. The tendency of larger surface settlement for the fixed invert is more significant up to D/B = 2.0. However, in the case of D/B = 3.0 (not shown here) the difference of the surface settlement significant. From these results it is revealed that for the same volume loss surface settlement profiles vary with the excavation patterns in the case of shallower tunneling. Therefore, the surface settlement may not be properly estimated using the method of volume loss (Mair et al., 1993) for shallow tunneling. The results of numerical analyses show the same tendency of model tests not only in shape but also in quantity.

#### 4.2 Shear strains

The distribution of shear strain of the model tests are obtained from the simulation of Particle Image Velocimetry (PIV) technique. The PIV is originally developed in the field of fluid mechanics (Adrain, 1991). In this paper, two images are divided into a finite area; the average movement rate of the aluminum rods of each area is being output as nodal displacement. The strain for one grid is calculated from these displacements by using the shape function and the Jacobian matrix that are used in finite element method for displacement and strain relationship. Figures 8 and 9 show the distribution of shear strain for full face excavation and fixed invert excavation, respectively, in the case of D/B = 1.0 and 2.0 for  $d_r = 4$  mm. It is seen in Figure 8 that the shear band of the ground

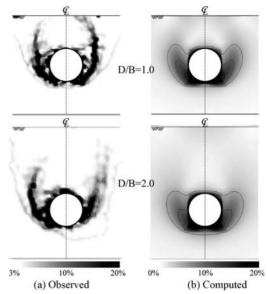


Figure 8. Distribution of shear strain: center is fixed.

is developed from the tunnel invert and covered the entire tunnel during tunnel excavation for the full surface excavation. From Figure 9 it is seen that shear band develops from the side of the tunnel not from the tunnel invert. In this case the length of the shear band is longer than that of the full face excavation. The range of the deformed region for the fixed invert is narrower compare to the full face excavation. The different patterns of shearing strain due to the different types of the tunnel excavation lead the change of the ground behavior. Moreover, the shear strain of the numerical analyses shows very good agreement with the results of the model tests.

#### 4.3 Earth pressure

Figures 10 and 11 show the observed and computed earth pressure distributions for D/B = 1.0 and 2.0 in the case of full face excavation and fixed invert excavation, respectively. The plots are drawn in the 12 axes corresponding to the radial direction of the 12 load cells towards the center of the model tunnel. The figures represent the value of earth pressure in Pascal corresponding to the amount of applied displacement (amount of shrinkage). It is seen in Figure 10 that earth pressure decreases all around the tunnel for the full face excavation due to the arching effect. The results appear to be in agreement with the results of tunnel experiments performed by Murayama and Matsuoka, 1971; Adachi et al., 1994; Shahin et al. 2004. As shear band develops surrounding the entire tunnel (Fig.8) the surrounding ground undergoes to the

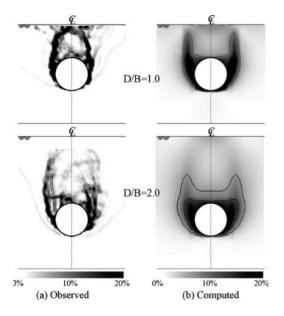


Figure 9. Distribution of shear strain: invert is fixed.

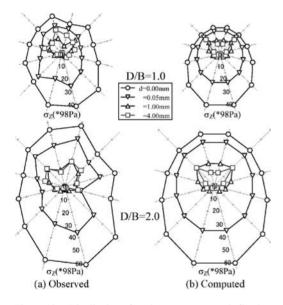


Figure 10. Distribution of earth pressure: center is fixed.

loosen state which reduces stresses in that place. It is also noticed that the earth pressure decreases suddenly after applying shrinkage of the tunnel within 0.00 to 0.20 mm. Further shrinking the tunnel, earth pressure decreases gradually at a lower rate up to a certain extent after which the earth pressure becomes almost constant. Sudden change in earth pressure is due to soil arching, immediately after disturbing the ground. For the fixed invert excavation earth pressure

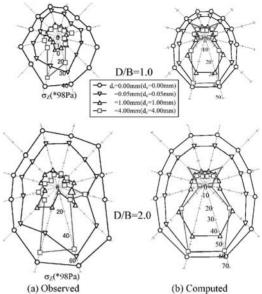


Figure 11. Distribution of earth pressure: invert is fixed.

distributions are different from the full surface excavation. In this case earth pressure decreases all around the tunnel till  $d_r = 1$  mm, for further shrinkage of the tunnel it increases in the bottom part of the tunnel while it remains almost same in the upper part of tunnel. It can be explained with the shear strain distribution shown in Figure 9. As the ground becomes loosen only in the upper part of the tunnel after  $d_r = 1$  mm, the confining pressure in the bottom part increases, therefore, the increase of earth pressure in the bottom part of tunnel can be speculated. From the above discussions it can be said that the distribution of earth pressure is highly dependent on the excavation patterns.

Figure 12 and 13 illustrates the change of earth pressure at load cells 4 and 9 against the amount of shrinkage of the tunnel for different soil covers. Load cell 4 is located in the vicinity of the tunnel crown, and load cell 9 is in the part of the tunnel invert. This figure confirms the sharp change of earth pressure during tunnel excavation. In the case of fixed invert excavation (Fig. 13), at the position of load cell 9 earth pressures increases after around  $d_r = 1$  mm, which is different from the results of the full face excavation (Fig. 12) where earth pressure remain constant after that amount of shrinkage. The phenomenon of the earth pressure increase after the reduction to some extent can be described as the change of arching effect due to the non-linear and elastoplastic behavior of the ground materials. It can not be described with a usual linear elastic model. The results of numerical analyses are in good agreement with the results of model tests. Therefore, it can be said that a proper elastoplastic

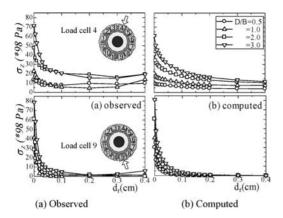


Figure 12. Earth pressure history: center is fixed.

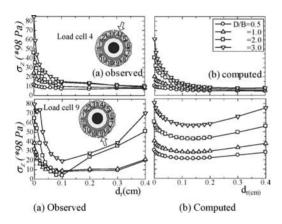


Figure 13. Earth pressure history: invert is fixed.

constitutive model is required to predict earth pressure around tunnel for lining design.

#### 5 COMPARISONS WITH THE ELASTIC ANALYSES

In this study numerical analyses with a linear elastic theory has been carried out to compare the results with the elastoplastic analyses. This section describes some typical results of the analyses. Young's modulus for the elastic analyses is calculated from the stressstrain relation (Fig. 14) of biaxial test performed in laboratory for the mass of aluminum rods. The value of E = 5500 kPa is chosen from the figure, and the assumed value of Poisson's ratio is 0.33 for the ground of aluminum rods mass.

Figure 15 shows the surface settlement profiles of the model test, elastoplastic analysis and elastic analysis for soil cover D/B = 1.0. It is seen in this figure that elastoplastic analysis can precisely express the results

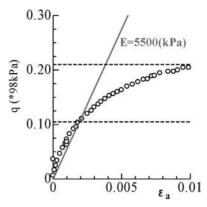


Figure 14. Stress-strain relation of aluminum rods mass.

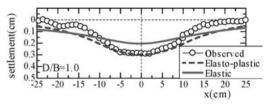


Figure 15. Comparisons of surface settlement profiles.

of the model test. The elastic analysis produces a wider surface settlement profiles compare to the observed one, the maximum surface settlement is smaller as well. As there is no yield point in a liner elastic model it can not express the deformation occurred locally. In this analysis displacement is applied to simulate tunnel excavation, therefore, there is no relation of the magnitude of the Young's modulus in the shape of the settlement trough except the value of Poisson's ratio.

#### 6 CONCLUSIONS

To investigate the deformation mechanism and earth pressure of the ground, a new tunnel apparatus has been developed. With this apparatus 2D model test and elastoplastic finite element analyses have been carried out. From the model tests and numerical analyses, the following points can be concluded:

- Surface settlement and earth pressure around tunnel are significantly influenced by the displacement applied at the tunnel crown for the same overburden and the same volume loss.
- 2. The volume loss is less significant compare to the crown drift in the case of shallow tunneling.
- The full face excavation produces wider range of the deformation region compare to the top drift (fixed invert) excavation.

- 4. The ground deformation mechanisms are different for the same volume loss with different excavation patterns.
- 5. The distribution of earth pressure is highly dependent on the excavation patterns.

The finite element analysis with subloading  $t_{ij}$  model is a useful tool to predict earth pressure and ground behavior during tunnel excavation.

#### REFERENCES

- Adachi, T., Tamura, T., Kimura, M. & Aramaki, S. 1994. Earth pressure distribution in trap door tests: *Proc. of 29th Japan National Conference of SMFE*, 3, 1989–1992 (in Japanese).
- Adrain, R. J. 1991. Particle imaging techniques for experimental fluid mechanics: Ann. Rev. Fluid Mech. 23, 261–304.

- Murayama, S. and Matsuoka, H. 1971. Earth pressure on tunnels in sandy ground: *Proc. of JSCE*, 187: 95–108 (in Japanese)
- Mair, R.J., Taylor, R.N. & Bracegirdle, A. 1993. Subsurface settlement profiles above tunnels in clays. Geotechnique 43, No.2, 315–320.
- Nakai, T., Xu, L. & Yamazaki, H. (1997): 3D and 2D model tests and numerical analyses of settlements and earth pressure due to tunnel excavation, *Soils and Foundations*, 37(3), 31–42.
- Nakai, T., and Hinokio, M. 2004. A simple elastoplastic model for normally and over consolidated soils with unified material parameters. *Soils and Foundation*. 44(2): 53–70.
- Shahin, H. M., Nakai, T., Hinokio, M., Kurimoto, T., and Sada, T. (2004): Influence of surface loads and construction sequence on ground response due to tunneling, *Soils* and Foundation, 44(2), 71–84.

# Analysis of pre-reinforced zone in tunnel considering the time-dependent performance

#### K.I. Song, J. Kim & G.C. Cho

Department of Civil & Environmental Engineering, Korea Advanced Institute of Science and Technology, Daejeon, South Korea

ABSTRACT: Auxiliary support systems such as the reinforced protective umbrella method are applied before tunnel excavation to increase ground stiffness and to prevent the large deformation in soft ground and shallow depth tunnelling. This study suggests a method to characterize the time-dependent behavior of pre-reinforced zones around the tunnel using elastic waves and direct shear test. The results obtained from the laboratory tests are applied to numerical simulations of a tunnel considering its construction sequences. According to numerical analyses, the time-dependent tunnel stability is most critical in the initial installation part of pre-reinforced zone and the portal of tunnel. However, time-dependent effect on tunnel behavior is not significant during construction as long as a proper overlap length is applied. Finally, the suggested analysis method of combining experimental and numerical procedures that consider the time-dependent effect on the pre-reinforced zone on the tunnel behavior will provide reliable and practical design and analysis for tunnels in soft ground.

#### 1 INTRODUCTION

Recently, instances of the construction of large underground structures in soil and soft rock layers are increasing. Auxiliary support systems such as the reinforced protective umbrella method are applied before or during tunnel excavation to ensure the stability of the tunnel face, to increase ground stiffness and to prevent large deformations in soft ground during shallow depth tunneling. These auxiliary supports change the state of stress and derive the arching effect around the tunnel. However, the effects of pre-reinforcement on tunnel stability and waterproofing of large section tunnels in soft ground have yet to be clearly and quantitatively defined.

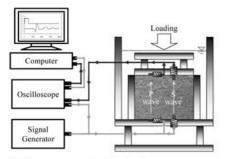
In conventional tunneling, there are typically 1 to 2 days between one face and the next face after the pre-reinforcement step. During this time interval, it is known that changes of the material properties caused by the effect of curing of the grouting material exist. However, 28 days of stiffness and strength after construction are generally applied to the material properties of the pre-reinforced zone in design stage without considering the effect of the time-dependent behavior of the injected grout material.

The present study suggests a method to characterize the time-dependent behavior of pre-reinforced zones around a large section of tunnel in soft ground using elastic waves. An experimental analysis was performed to characterize the time-dependent behavior of the pre-reinforced zone. Direct shear tests were performed at different time stages to obtain the timedependent strength parameters. In addition, elastic wave velocities (i.e., Vp and Vs) were continuously measured using piezoelectric bender elements to obtain the time-dependent stiffness parameter. Timedependent strength and stiffness parameters obtained from laboratory tests were applied in the numerical simulation of a large section tunnel in soft ground, taking into account its construction sequence. The proposed analysis method, which combines experimental and numerical procedures while considering the timedependent effect on the pre-reinforced zone on the tunnel behavior, will provide a reliable and practical design basis and means of analysis for large section tunnels in soft ground.

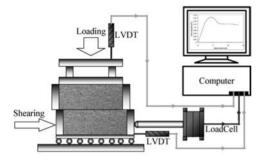
## 2 TIME-DEPENDENT EXPERIMENTAL ANALYSIS

## 2.1 Measurement of elastic wave velocity and shear strength

As shown in Figure 1(a), S-wave and P-wave velocity were measured in this study using bender elements. The propagation direction was varied by arranging the installation direction of the bender element such that it was parallel for the p-wave and perpendicular for the s-wave. Wave velocities were measured continuously



(a) Measurement setup for elastic wave



(b) Direct shear test

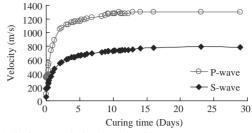
Figure 1. Schematic drawing of test setup.

Table 1. Material properties fo samples used in test.

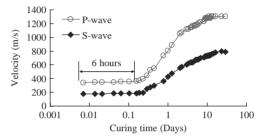
Material properties	Joomunjin Standard Sand	Portland Cement
Friction angle (°)	39	_
Min. void ratio	0.62	-
Max. void ratio	1.13	-
Density (kN/m <sup>2</sup> )	25.9	30.8
Realtive density (%)	60 W/C ratio (%)	170
Vertical Stress	160 Dimension	$10(W) \times 10(D)$
(kPa)	(cm)	× 7(H)

for 28 days under 160 kPa vertical stress. From the wave propagation transition, arrival time was calculated. The wave velocities were calculated by dividing the height of the specimen by the measured arrival time (Dano, 2004; Khan, 2006).

A schematic drawing of the motorized direct shear apparatus is shown in Figure 1(b). The shear speed of the direct shear apparatus was maintained at 0.5 mm/min. A load cell of 2 ton capacity was installed to calibrate the load during shearing. Measured vertical and horizontal displacements were saved in a computer automatically using a LVDT. All material properties of samples used in the tests are listed in Table 1.



(a) Elastic wave velocity (Normal scale)



(b) Elastic wave velocity (Log scale)

Figure 2. Time-dependent characteristics of elastic wave velocities.

#### 2.2 Results and analyses of experimental tests

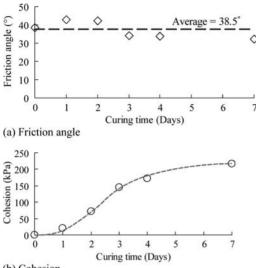
### 2.2.1 *Time-dependent characteristics of wave* velocities

The results show that the wave velocity increases exponentially according to curing time and becomes almost constant after 7 days. Figure 2(a) shows typical velocity data for two types of measured waves. It is found that the strength of the mixture becomes an important factor from about 6 hours. As shown in Figure 2(a), P-wave velocity is faster than S-wave velocity and represents the wave that travels through the skeleton of sand and cement mixture. From Figure 2(b), it is seen that the wave velocity starts to increase when the curing time is  $4\sim 6$  hours. This means that the mixture takes  $4\sim 6$  hours to reveal a cementation effect.

### 2.2.2 Time-dependent characteristics of shear strength and strength parameters (c and $\phi$ )

The direct shear tests performed for specimens less than 7 days of curing time correspond to the wave velocity measurement. The results of the direct shear test were very similar to those of the wave velocity measurements. However, the tests were performed with intervals of a day or more, and thus the process of the strength revelation through cementation could not be observed.

Figure 3 shows the time dependent characteristics of shear strength parameters obtained from direct shear test. In Figure 3(a), the friction angle does not change in accordance with curing time. On the other hand,



(b) Cohesion

Figure 3. Time-dependent characteristics of shear strength parameters.

Figure 3(b) shows time-dependent characteristics of cohesion. Through Figure 3(b), it is noticeable that the cohesion increases with curing time and after a certain amount of curing time the cohesion converges. It can be deduced that the bonding of cement causes this increase in cohesion. Therefore, strength is controlled by the normal stress (i.e., frictional) at the early stages and then by cohesion (Schnaid et al., 2001). In the numerical analysis, it is assumed that the friction angle does not change and the cohesion increases as curing time increases.

Ultimately, the cohesion controls the increase of the shear strength, and cohesion depends on time in a similar manner as elastic wave velocity. Hence, Young's modulus and cohesion are selected as improvement properties to reflect time-dependent characteristics in order to facilitate accurate simulation via the numerical analysis program in this study.

### 2.3 Verification of testing results and acquirement of time-dependent design properties

An empirical determination method of pre-reinforced zones to investigate the stability in a tunnel is presented and the results are compared with characteristics of elastic wave velocity from laboratory test results.

As shown in Eq. (1), cohesion is related to the unconfined strength and friction angle. Herein, unconfined strength of grout bulb at 7 days of curing time is about 8 MPa from the uniaxial compressive strength tests. When the friction angle of a soil layer in natural condition is about 39 degrees, the cohesion of the pre-reinforced zone is about 1.91 MPa according to Eq. (1).

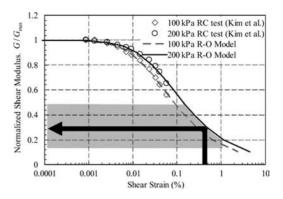


Figure 4. Normalized shear modulus of sandy soil (Kim et al. 2004).

Sandy soil in natural condition has no cohesion in the direct shear test. However, after 7 days, it has a cohesion value of about 2.17 MPa, which is similar to the result obtained by Eq. (1).

$$c_g = \frac{q_{u(design)}}{2 \times \tan\left(45^\circ + \frac{\phi}{2}\right)} \tag{1}$$

Young's modulus of the grout bulb can be presented as a function of the weight density and design criteria strength in the concrete design specification, as given by Eq. (2).

$$E_g = W^{1.5} \times 4270 \times \sqrt{q_u} \tag{2}$$

Here, when the weight density is  $18 \text{ kN/m}^2$  and the unconfined strength at a curing time of 7 days is 8 MPa, Young's modulus is about 8300 MPa. The initial Young's modulus (i.e., 45 Mpa) can be calculated from equation (3)~(6), this represents an increase of 184 times.

Regarding the characteristics of elastic wave propagation, the maximum shear modulus can be calculated by Eq. (3) from the measured shear wave velocity.

$$(G_{max})_{Field or Lab.} = \rho \times V_s^2$$
(3)

The normalized shear modulus in the range of the tunnel strain level can be obtained by a resonant column and torsional shear test and the Ramberg-Osgood model (1943), as shown in Figure 4. Typically, the strain level of a tunnel is about  $0.1 \sim 1\%$ . Hence, the normalized shear modulus of sandy soil is about 0.3, as shown in Figure 4. Accordingly, the shear modulus of the pre-reinforced zone can be calculated by the maximum shear modulus from Eq. (3) and the normalized shear modulus in Eq. (4). Furthermore, the Young's modulus of model can be derived by Eq. (5) and Eq. (6), which delineates the relationship among Young's modulus, shear modulus, and Poisson's ratio.

$$G_{\text{model}} = (G_{\text{max}})_{\text{Field or Lab.}} \times (G/G_{\text{max}})_{at \gamma}$$
(4)

$$v = \mathbf{f}(V_p, V_s) \tag{5}$$

$$E_{model} = 2 \cdot (G_{model})_{Field \, or \, Lab.} \cdot (1+\nu) \tag{6}$$

Although it is clear that the measured shear wave velocity has characteristics of infinitesimal deformation, it is reasonable that the increase in Young's modulus can vary according to any increase in the elastic wave velocity depending on the time.

With these equations and theory, Young's modulus of sandy soil in natural condition is 45 MPa and Young's modulus for a curing time of 7 days is 7400 MPa. Therefore, Young's modulus of a prereinforced zone increases 165 times compared to that before pre-reinforcement. It is apparent from the equations (3)  $\sim$  (6) presented here that stiffness increases at a similar rate to the result calculated by Eq. (2). Moreover, the Young's modulus of pre-reinforced sandy soil after 28 days of curing time increases by about 200 times in comparison with normal condition sandy soil. Based on these results and equations, numerical analyses should be accompanied to evaluate the timedependent characteristics of pre-reinforced zones in a tunnel.

Young's modulus and cohesion of a pre-reinforced zone should be expected to increase by 200 times increase after a curing time of 28 days.

#### 3 NUMERICAL SIMULATION OF LARGE SECTION TUNNEL IN SOFT GROUND

### 3.1 Large section tunnel model and boundary conditions

Comparative analyses were performed to analyze the time-dependent effect of a pre-reinforced zone on the large section tunnel behavior while tunneling. The material properties of the pre-reinforced zone have five different conditions that are either time-dependent conditions or constant time conditions (i.e., 1d, 2d, 3d, and 28d). A commercial 3D FEM analysis program (i.e., MIDAS-GTS) was used as a numerical simulation tool. Elasto-plastic ground material, which is linear elastic and perfectly plastic with following Mohr-Coulomb yield criterion and non associated flow rule, was used for analyses.

For the application of the stiffness and strength parameters depending on the time obtained from the experimental study, a tunnel model should be located at the same stress level of the experiment condition, as propagation characteristics of elastic waves are affected by the state of the stress.

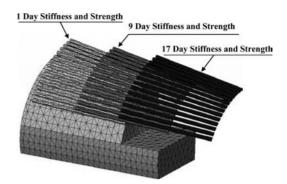


Figure 5. 3D tunnel model and time-dependent material properties of the pre-reinforced zone after 12 m excavation.

Figure 5 shows the simulated 3D four-lane tunnel model, which was constructed 15 m under the surface of the ground. It has a radius of 9.4 m and a height of 10.4 m. The boundary of 3D FE model is horizontally fixed at the 4 end sections and vertically fixed at the bottom section. Water table is located at the top of the surface. This simulated tunnel is an actual section of a four-lane tunnel that was designed and constructed in weathered rock in Korea. Four sub-sectional excavations with a length of 0.75 m per stage were modeled. Steel pipes 114.3 mm in diameter were modeled as a beam element: a 12 m steel pipe was installed at an inclination of 11° with a 6m overlap, with a 0.7m transversal interval between the steel pipes that were installed along the tunnel crown 60° from the center. The grout expansion radius is 0.3 m from the pipe center.

### 3.2 Numerical analysis of a tunnel considering the time-dependent behavior

The time-dependent behavior of a pre-reinforced zone can be modeled using the procedures described below. The material properties (i.e., stiffness and strength) of the pre-reinforced zone are registered as the boundary conditions from Day 1 to Day 28. The registered initial boundary conditions were applied to a pre-assigned mesh in the pre-reinforcement construction. The boundary conditions were applied and updated according to the field construction process. Figure 5 shows the conceptual time-dependent stiffness and strength characteristics of the pre-reinforced zone after the third pre-reinforcement and 12 m of excavation at an excavation speed of 0.75 m/day.

The tunnel behaviors were analyzed considering the time-dependent effect of the pre-reinforced zone in terms of the vertical displacement and the horizontal displacement. The material properties used for the analysis are tabulated in Table 2. Figure 6 shows the time-dependent elastic modulus and cohesion values obtained from the experimental study as well as those used with the numerical analysis.

Table 2. Material properties used for the numerical analysis.

Ground	$\gamma$ (kN/m <sup>2</sup> )	E (MPa)	υ	c (kPa)	φ (°)
Weathered soil	18	45	0.32	0	39
Weathered rock	21	200	0.3	250	35
Steel Pipe	33	210000	0.3	_	_

where  $\gamma$ : unit weight, E: elastic modulus,  $\upsilon$ : Poisson's ratio, c : cohesion,  $\phi$ : friction angle.

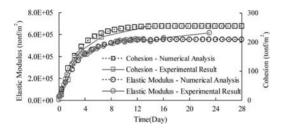


Figure 6. Time-dependent stiffness and strength for numerical analysis and obtained experimental result.

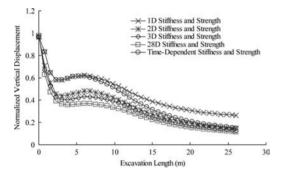


Figure 7. Normalized vertical displacement variation.

#### 4 COMPARATIVE ANALYSIS OF THE NUMERICAL SIMULATION RESULT

For quantitative analysis, displacements of each case are normalized with the result of a pipe-only case. With the normalized displacements, the effect of the time-dependent behavior of the pre-reinforced zone was examined.

#### 4.1 *Time-dependent effect on vertical displacement*

Figure 7 shows the normalized vertical displacement at the portal according to the time-dependent condition and constant time conditions. The normalized vertical displacement behavior of the time-dependent condition is similar to the results of one day of the constant time conditions at the initial excavation section. As the excavation length increases, the results of time-dependent condition become nearly identical to

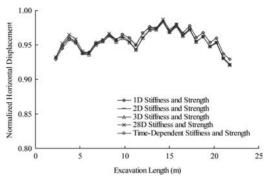


Figure 8. Normalized horizontal displacement variation at tunnel face.

the analysis result of  $2\sim3$  days of constant time condition, and the vertical displacement converges. The stiffness and strength of the pre-reinforced zone of  $1\sim2$  days of the constant time condition correspond to  $30\sim50\%$  of 28 days of the constant time condition. In other words, a reduction of the material properties of the pre-reinforced zone makes it possible to model the time-dependent effect of the pre-reinforced zone on the global tunnel behavior at the initial tunnel excavation.

#### 4.2 Time-dependent effect on horizontal displacement at the tunnel face

Figure 8 shows the normalized horizontal displacement at the tunnel face according to the timedependent condition and constant time conditions. From the analysis result, variation of the normalized displacement at the time-dependent condition ranges from  $0.94 \sim 0.98$ , which is very similar to other cases during excavation. Therefore, pre-reinforcement can be considered as prevention of a collapse rather than a displacement reduction control at the tunnel face. It can be concluded that grouting reduces the horizontal displacement by approximately  $2\sim 6\%$  at the tunnel face with the pre-reinforcement method.

#### 4.3 Time-dependent effect on horizontal displacement at the tunnel side wall

Figure 9 shows the horizontal displacement variation at the tunnel side wall according to the time-dependent condition and constant time conditions. From the analysis result, the normalized horizontal displacement variation of the time-dependent condition is similar to the results of one day of the constant time conditions at the initial excavation section. The horizontal displacement variation of the time-dependent condition is located between 2 and 3 days of the constant time condition as the excavation length increases. Therefore, between 2 and 3 days of a constant stiffness and strength condition can be used for a time-dependent analysis of a pre-reinforced zone for the horizontal

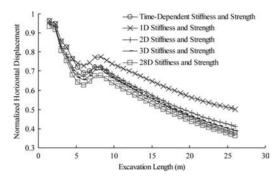
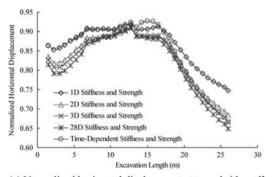
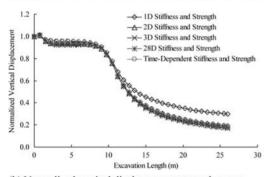


Figure 9. Normalized horizontal displacement at tunnel side wall.



(a) Normalized horizontal displacement at tunnel side wall



(b) Normalized vertical displacement at tunnel crown

Figure 10. Normalized displacement at 10.5 m from the portal.

displacement of a tunnel, which is identical to the variation of the vertical displacement.

Figure 10 shows the normalized vertical and horizontal displacements 10.5 m from the portal. Figures 10(a) and 10(b) are in good agreement with the result of 28 day of the constant time condition as well as the result of the time-dependent condition of the normalized vertical and normalized horizontal displacement. When the excavation length is 10.5 m, the pre-reinforced zone initially has 14 days of stiffness and strength; secondly, the pre-reinforcement zone had 6 days of stiffness and strength.

The stiffness and strength levels of 6 and 14 days of the constant time condition correspond to 87.8% and 99.3%, respectively, of 28 days of the constant time condition. Therefore, the time-dependent characteristics of the pre-reinforced zone do not affect tunnel displacement due to the sufficiently overlapped pre-reinforced zone. However, the time-dependent characteristics of the pre-reinforced zone should be considered in the case of a portal and an initial support section of a weak layer, which can cause a large displacement and unsafe construction conditions. Therefore, it can be concluded that  $2\sim 3$  days for the stiffness and strength of pre-reinforced zones is appropriate to model the time-dependent behavior. In addition, a properly overlapped section of pre-reinforced zone can be assumed as 3 or 7 days of constant stiffness and strength without a time-dependent effect due to sufficient exposure of the stiffness and strength during the excavation stages in conservative designs.

#### 5 CONCLUSIONS

Based on an experimental study of a pre-reinforced zone, the time-dependent characteristics were analyzed in terms of the strength and stiffness. Results show that the shear strength and the elastic wave velocities increase as the time stage increases. Shear strength and strength parameters (i.e., the cohesion and friction angle) can be uniquely correlated to elastic wave velocities. The results obtained from laboratory tests were applied to a numerical simulation of a tunnel, taking into account its construction sequence. According to the results of the numerical simulation, vertical displacement and horizontal displacement results for fewer than  $2 \sim 3$  days of constant time boundary conditions are nearly identical to the analysis results of the time-dependent condition. Therefore, it can be concluded that  $2\sim3$  days for the stiffness and strength of pre-reinforced zones is appropriate to model the time-dependent behavior of a large section tunnel. Finally, the suggested analysis method combining experimental and numerical procedures that consider the time-dependent effect on the pre-reinforced zone on tunnel behavior will provide a reliable and practical design basis and means of analysis for tunnels in soft ground.

#### **ACKNOWLEDGEMENTS**

This paper was supported by the Korea Institute of Construction and Transportation Technology Evaluation and Planning under the Ministry of Construction and Transportation in Korea (Grant No. 04-C01) and Brain Korea 21 Project in 2006.

#### REFERENCES

- Dano, C., Hicher, P.Y. & Tailliez, S. 2004. Engineering Properties of Grouted Sands. *Journal of Geotech. and Geoenviron. Engineering*, 130(13): 328–338.
- Khan, Z., Majid, G., Cascante, G., Hutchinson, D.J. & Pezeshkpour, P. 2006. Characterization of a cemented san with the pulse-velocity method. *Canadian Geotechnical Journal*, 43: 294–309.
- Kim, D.S., Kwon, K.C. & Oh, S.B. 2004. Development of Experiment-Analysis Integrated System for the Evaluation of Deformation Behavior of Geotechnical Structures on Weathered Residual Soils at Whole Strain

Range. Research Report Korea Science and Engineering Foundation.

- MIDAS-GTS. 2005. Geotechnical & Tunnel Analysis System, MIDAS Information Technology Co., Ltd.
- Ramberg, W. & Osgood, W.R. 1943. Description of stressstrain curves by three parameters. *Technical Note 902*, *National Advisory Committee for Aeronautics, Washing*ton, D.C.
- Schnaid, F., Prietto, P.D.M. & Consoli, N.C. 2001. Characterization of Cemented Sand in Triaxial Compression. Journal of Geotech. and Geoenviron. Engineering, 127(17): 857–868.

### Vault temperature of vehicle fires in large cross-section road tunnel

#### K.S. Wang, X. Han & Z.X. Li

Shanghai Institute of Disaster Prevention and Relief, Tongji University, Shanghai, P.R. China

ABSTRACT: On the basis of CFD models, the simulation analysis of a lorry and car fire in large cross-section road tunnel was carried out in this paper. The vault temperature was simulated and compared with the value calculated by correlative empirical equation. The corresponding results would be contributed to provide effective technique for the fire protection measures of the major structure of road tunnel.

#### 1 INTRODUCTION

Large scale tunnel fires often caused considerable material damage, not only to vehicles, but also to the tunnel facilities as well. The damage was brought about by the massive development of heat and aggressive combustion gases, which led to enormous difficulties to reconstruction of tunnel. In order to provide the road tunnel with effective fire protection measures, it is important to predict the vault temperature in different road tunnel fire scenarios. There were some experiments on testing the structure characteristics under fire. But most of them were tested by following the standard temperature-time curve (Yasuda et al., 2004). Very few data are available on maximum smoke temperature under the tunnel ceiling with different ceiling heights, longitudinal ventilation velocities and fire intensities. Based on CFD models, the simulation analvsis of a lorry and car fire in large cross-section road tunnel was carried out in this paper. The vault temperature was simulated and compared with the value calculated by correlative empirical equation. The corresponding results would be contributed to provide effective technique for the fire protection measures of the major structure of road tunnel.

#### 2 DESIGN OF FIRE SCENARIO FOR CFD

#### 2.1 Brief introduction of FDS

In these simulations, FDS (Fire Dynamics Simulator) 4.06 which was released by NIST (National Institute of Standards and Technology, USA) was used. FDS is a Computational Fluid Dynamics (CFD) model with LES (Large Eddy Simulation) of fire-driven fluid flow. The model solves numerically a form of the Navier-Stokes equations appropriate for low-speed, thermally-driven flow with an emphasis on smoke and heat transport from fires. The partial derivatives of the conservation equations of mass, momentum and energy are approximated as finite differences, and the solution is updated in time on a three-dimensional, rectilinear grid. Thermal radiation is computed using a finite volume technique on the same grid as the flow solver.

#### 2.2 Simulation scenarios of tunnel fire

In this paper, CFD simulation of the maximum vault temperature was conducted by FDS (4.06) in different fire scenarios with constant longitudinal ventilation velocity set as 3 m/s. The simulation length of the model which was three-driveway large cross-section road tunnel with diameter of 15 m was 100 m, as shown in Figure 1. The Heat Release Rate (HRR) of the fire

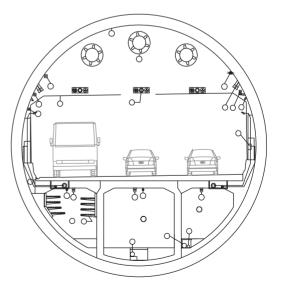


Figure 1. Cross-section of road tunnel.

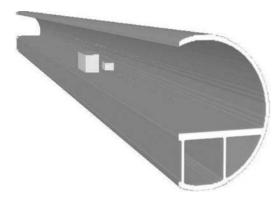


Figure 2. CFD analysis model in scenario 1.

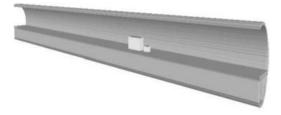


Figure 3. CFD analysis model in scenario 2.

source was based on the EUREKA project conclusion, which was completed by nine Europe countries in antiquated road tunnel. The fire position was located at 37 m from the upstream ventilation cross-section and 63 m from the downstream ventilation cross-section. In this simulation, the car was ignited by the lorry and the design scenarios include: scenario 1: the car in the flank of the lorry; scenario 2: the car downstream the lorry. In these two scenarios, the spacing between the car and the lorry was 2 m, as shown in Figures 2 and 3.

In the simulation process, the lorry firstly burned and for about 220s the car was ignited by the lorry which received more heat radiant flux than its critical value of  $16 \text{ kw/m}^2$ . The maximum HRR of the lorry and the car was 20 MW and 5 MW separately. The total HHR generated by these two vehicles was shown in Figure 4.

#### 3 DESIGN OF FIRE SCENARIO FOR CFD

#### 3.1 HRR curve

As shown in Figure 4, HRR curves in these two scenarios were constructed. The relative position of the lorry and the car had less influence on total HHR. During the simulation, the HHR was approximately 23 MW and less than the algebra sum of 25 MW which was calculated by the HHR of these two vehicles respectively.

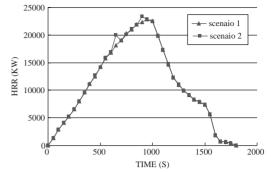


Figure 4. Heat release rate of fire source in scenario 1, 2.

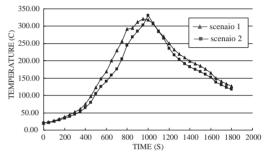


Figure 5. Vault temperature-time curves in scenario 1, 2.

#### 3.2 Vault temperature-time curve

As shown in Figure 5, the vault temperature reached the maximum value of  $321^{\circ}$ C at the position of 4 m downstream from the fire source after 941 s in scenario 1 and achieved  $331^{\circ}$ C at the position of 5 m downstream from the fire source after 980 s in scenario 2. The vault temperature–time curves were basically of superposition in these two scenarios with the same ventilation velocity of 3 m/s and geometry properties. It was indicated that the relative position of these two vehicles had less influence on vault temperature curve.

As shown in Figures 6 and 7, because of large crosssection of the road tunnel and relatively smaller HRR value of the fire source, the simulated vault temperature was not much high. On the other hand, under the longitudinal ventilation velocity of 3 m/s, the fire flames were sloped towards downstream far from the vault, so it attenuated the environmental temperature and made less contribution to vault temperature.

#### 3.3 Comparative analysis of vault temperature

An empirical equation which was derived to predict the maximum smoke temperature under the tunnel ceiling is presented as follows (Kurioka et al., 2003):

$$\frac{\Delta T_{\max}}{T_a} = \gamma \left(\frac{Q'^{2/3}}{Fr^{1/3}}\right)^c \tag{1}$$

Smokeview 4.0.6 - Sep 15 2005

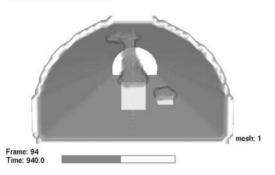


Figure 6. Fire flames in scenario 1 after 940s.

Smokeview 4.0.6 - Sep 15 2005



mesh: 1



Figure 7. Fire flames in scenario 2 after 980s.

$$Q' = Q / \left( \rho_a C_p T_a g^{1/2} H_d^{5/2} \right)$$
 (2)

$$Fr = u^2 / (gH_d) \tag{3}$$

 $Q'^{2/3} / Fr^{1/3} < 1.35, \ \gamma = 1.77, \varepsilon = 6/5$  (4)

$$1.35 \le Q'^{2/3} / Fr^{1/3}, \ \gamma = 2.54, \varepsilon = 0$$
 (5)

where Q = heat release rate of the fire, kW; Q' = dimensionless heat release rate; Fr = dimensionless Froude number;  $\gamma$ ,  $\varepsilon$  = experimental constant;  $C_p$  = specific heat capacity of air at constant pressure, kJ kg<sup>-1</sup> K<sup>-1</sup>;  $\rho_a$  = ambient air density, kg m<sup>-3</sup>;  $T_a$  = ambient air temperature in tunnel, K;  $\Delta T_{max}$  = maximum excess temperature of smoke under the tunnel ceiling, K; u = representative longitudinal ventilation velocity, m s<sup>-1</sup>; g = acceleration due to gravity, ms<sup>-2</sup>;  $H_d$  = height from the surface of fire source to tunnel ceiling, m.

For validating the availability of equation (1) to the vault temperature in large cross-section road tunnel fires, three other scenarios were introduced to the simulation analysis, including: (1) scenario 3: car fire; (2) scenario 4: passenger car/lorry; (3) scenario 5: heavy goods vehicle. The heat release rates of the fire source were also in accordance with the EUREKA project conclusion, as shown in Figure 8. The fire flames in scenario 3, 4, 5 were shown in Figure 9, 10, 11 respectively.

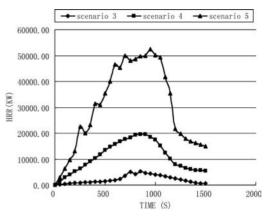
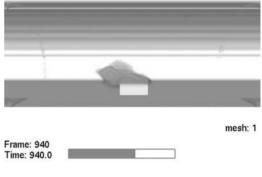
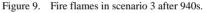


Figure 8. Heat release rate of fire source in scenario 3, 4, 5.

Smokeview 4.0.6 - Sep 15 2005





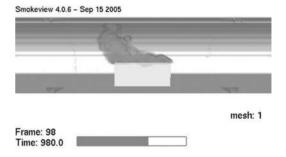
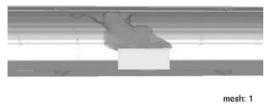


Figure 10. Fire flames in scenario 4 after 980s.

The vault temperature reached the maximum value at the position of 4 m downstream from the fire source in scenario 3, 4, 5 with the same longitudinal ventilation velocity of 3 m/s. The simulation results and empirical equation value of the vault temperature in these scenarios were shown in Figure 12 and Table 1.

As shown in Figure 13 and Table 1, there were rather differences between the simulation results and

Smokeview 4.0.6 - Sep 15 2005



Frame: 98 Time: 980.0

Figure 11. Fire flames in scenario 5 after 980s.

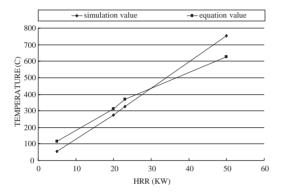


Figure 12. The simulation value and the equation value of vault temperature in different scenarios.

Table 1. The simulation value and the equation value of vault temperature in different scenarios.

Scenario	3 (5 MW)	4 (20 MW)	1, 2 (23 MW)	5 (50 MW)
Sv	54	275	(321 + 331)/2 = 326	753
Ev	116	312	369	628
Re	53.4%	11.9%	11.7%	16.6%

Sv = simulation value; Ev = equation value; Re = relative error.

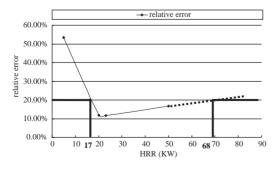


Figure 13. The relative error among the simulation value and the equation value of vault temperature in different scenarios.

the empirical equation values of the vault temperature. The relative error exceeded 20% for the HRR which were less than 17 MW and greater than 68 MW. It should be mentioned that for small scale fire, such as HRR of 5 MW, the relative error approached 53.4%. Therefore, much more simulation analysis and practical tests need to be carried out. On the other hand, it was suggested that the empirical equation could be further improved to be applicable for small scale fire in large cross-section road tunnel. Perhaps some other factors had to be considered during the calculation of vault temperature, involving different kinds of ventilation conditions and geometrical properties of the road tunnel.

#### 4 CONCLUSION

- As for lorry and car fires in large cross-section road tunnel and under longitudinal ventilation velocity of 3 m/s, the relative position of vehicles had less influence on total HHR which was approximately 23 MW and less than the algebra sum of 25 MW.
- 2. Owing to large cross-section of the road tunnel, small scale vehicle fire and longitudinal ventilation velocity of 3 m/s, the simulated vault temperature was not very high.
- 3. In connection with the calculation of the vault temperature, there was limitation to the related empirical equation. The relative error which was between the simulation analysis results and the empirical equation value exceeded 20% for the HRR less than 17 MW and greater than 68 MW. Obviously it should be further improved especially for small-scale fire of the road tunnel.

#### ACKNOWLEDGMENTS

The support of the Natural Science Foundation of China (Grant No. 50678124) is gratefully appreciated.

#### REFERENCES

- Gu, Z.P. & Cheng, Y.P. 2003. Research on temperature field distribution in transportation tunnel fires. *Paper collections of first session international meeting on city and industrial safety in China (Nan jing)*: 265–268 (in Chinese).
- Haack, A. 1998. Fire protection in traffic tunnels: general aspects and results of the EUREKA project. *Tunnelling* and Underground Space Technology 4(13): 377.
- Kurioka, H., Oka, Y., Satoh, H. & Sugawa, S. 2003. Fire properties in near field of square fire source with longitudinal ventilation in tunnels. *Fire Safety* 38 (4): 319–340.
- Yasuda, F., Ono, K. & Otsuka, T. 2004. Fire protection for TBM shield tunnel lining. *Tunnelling and Underground Space Technology* 19 (4–5): 317.

# Effects of different bench length on the deformation of surrounding rock by FEM

X.M. Wang, H.W. Huang & X.Y. Xie

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R.China

ABSTRACT: Bench cut method has been extensively used in mountain tunneling. This is mainly due to its flexibility to adapting to different ground conditions. Induced displacements are empirically controlled by adjusting the speed of excavation, the bench length, partial-face excavation and closure of invert. In this paper, a series of three-dimensional, numerical, elastoplastic analyses were conducted to investigate the effects of different bench length on the deformation of surrounding mass in soft rock. The closure of invert was also investigated to their role in controlling the final displacement. When bench cut method adopted in soft rock, the bench length should not be too long or too short and 0.5 times of tunnel diameter around for bench length is appropriate.

#### 1 INTRODUCTION

In mountain tunneling, the bench cut method is used extensively due to its simplicity and flexibility to adapting to different ground conditions. And the method provides an advantage of simultaneous excavation of the upper and lower sections. The key issue of adopting this method is selecting a length and a shape of bench to assure the stability of the face, especially for tunneling in soft rock. In addition, auxiliary methods are used as required. When the ground is good enough, having enough self-supporting properties, the bench length can be reduced to 0 m (full face cut). In the case of a weak ground, the bench length is empirically decided. It is therefore important to evaluate and compare the effect of different bench length on the deformation of surrounding rock and on the stability of excavation front.

Usually, numerical analyses work as a kind of model test in which many relevant design variables can be investigated in parametric studies (Ng & Lee, 2005; Karkus & Fowell, 2003; Galli et al., 2004). In this way it is possible to quantify the relative importance of each possible intervention in order to choose the most effective measures from the economic and safety point of view.

Seki et al. (1994) conducted a series of threedimensional finite-element elastic analyses of unlined tunneling to determine the effect of bench length and shape. The initial stress was given by external force, ignoring the dead weight effect. They found that the longer the bench, the smaller the displacement due to squeezing at the face. Moreover, they found that the bench length scarcely exerted influence upon settlement of the crown. Finally they concluded longer bench tended to offer a great safety factor and leaving the core was effective for increasing the face stability.

Farias et al. (2004) conducted a series of threedimensional elastic finite element analyses of tunneling by the New Austrian Tunneling Method (NATM) to investigate relative importance of relevant techniques for settlement control. The techniques included partial-face excavation, free span distance and support activation. The tunnel has 9.6 m of diameter and the soil cover is 10 m. They found that tunnel support lining including free span and closure of invert was the most relevant single factor analyzed in reducing induced settlements. The closer to face the lining was concreted, the smaller the displacements. Moreover, they found the bench helped to keep horizontal pressure in the excavation face.

Eberhardt (2001) conducted a series of comprehensive, three-dimensional, elastic and elasto-plastic, numerical analyses of tunneling in the central Swiss Alps with different assumed initial stress states to demonstrate three-dimension stress rotation ahead of an advancing tunnel face. The diameter of tunnel was 10 m and the bench length was 10 m. They found that high stress concentrations in association with large rotation of the maximum principle stress were

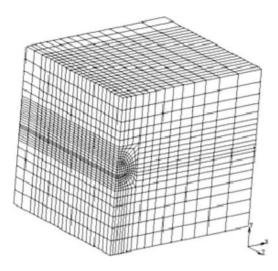


Figure 1. Three dimensional finite element mesh.

observed when the initial maximum principle stress alignment was horizontal and parallel to the tunnel axis.

In this paper, a series of three-dimensional, elastoplastic, numerical analyses were carried out to investigate the effects of different bench lengths on the deformation of surrounding mass in soft rock. The objectives of these analyses are to find optimum bench lengths for different rock mass.

#### 2 THREE-DIMENSIONAL NUMERICAL MODELLING

#### 2.1 Numerical approximations

A hypothetical tunnel excavation in soft rock was modeled in this three-dimensional numerical study. The diameter of the tunnel (D) was taken as 12 m, with a constant cover depth 30 m. The tunnel was assumed to be a bench excavation and lined with spray shotcrete and bolt. The finite element program, MARC, was adopted to model the tunnel excavation.

The three-dimensional finite element mesh used in the present analyses is shown in Figure 1. The bedrock was set 37.2 m below the bottom of the tunnel and the domain expands laterally 80 m from the tunnel centerline. The model took advantage of symmetry of the problem. Boundary conditions are totally fixed at the bottom of the model and only vertical displacements are free in the vertical sides. The model consisted of 10114 elements and 10373 nodes. Eight-noded brick elements and four-noded shell elements were used to model the rock and concrete lining, respectively. And two-noded truss elements were used to model bolt. The mesh was divided into 22 longitudinal blocks of

Table 1. Rock parameters used in the finite element analyses.

Rock	E/GPa	ν	$\gamma/KN^*m^{-3}$	C/MPa	$\varphi/(^{\circ})$
IV <sub>upper</sub>	6	0.3	23	0.7	39
IV <sub>lower</sub>	2	0.35	20	0.2	27
$V_{lower}$	1	0.35	17	0.05	20

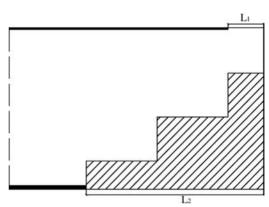


Figure 2. Schematic representation of  $L_1$  and  $L_2$ .

variable sizes. The first 20 blocks length corresponds 1/4 of the tunnel diameter (D). The last 2 block length is 1/2 D. A monitored section, located in the middle of the mesh (i.e., at z = -30 m), was studied during every stage of excavation and construction. To account for the relatively large stress and strain gradients near the tunnel opening, small finite elements were used.

An elastic – perfectly plastic rock model, using Drucker – Prager failure criterion with a nonassociated flow rule, was adopted in this study. The tunnel lining and bolt were modeled as linear elastic. The Young's modulus and Poisson's ratio for the tunnel lining were taken to be 25 GPa and 0.2, respectively. The unit weight of the tunnel lining was 22 kN/m<sup>3</sup>. For bolt, the Young's modulus of 210 GPa with Poisson's ratio of 0.3, were adopted. Table 1 provides the rock parameter used in this study. These values were based on Code for design of Road Tunnel (2004).

#### 2.2 Numerical modeling procedures

A given cross-section was divided into three parts: upper section, lower section and invert section. For simplicity, upper bench length equals to lower bench length ( $L_b$ ). The free distance between excavation face and the support heading will be referred as free span ( $L_1$ ). The distance between the excavation face and the first whole lining section will be referred as full support distance ( $L_2$ ). The schematic representation of these was shown in Figure 2.

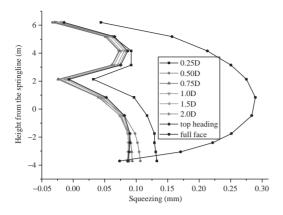


Figure 3. Distribution of squeezing at the face (IV  $_{\mbox{upper}}$  rock).

Tunnel excavation and construction were simulated by deactivating the rock elements within the proposed tunnel excavation zone and by activating the support. The tunnel excavation rate was modeled 3.0 m (i.e., D/4) per day, which was used as a step size in the numerical analyses. No support was applied to the tunnel face. The unsupported length equals 3 m (i.e.,  $L_1 = 3$  m). The excavation sequences are:

- 1 Excavate upper section rock until the upper bench length equal  $L_b$ , and install tunnel lining to the previously excavation span simultaneously. Leave free span of 3 m.
- 2 Excavate upper section rock and lower section rock until the lower bench length equal L<sub>b</sub>, and install tunnel lining to the previously excavation span simultaneously. Leave free span of 3 m.
- 3 Excavate upper section rock, lower section rock and invert section rock, and apply lining.
- 4 Advance the tunnel by repeating step 3 until the upper tunnel face has passed 3.0D from the monitoring section.

The initial stress was given by gravity because of shallow tunnel. The different bench lengths (0-2D), top heading cut and full face cut were studied in the analyses.

#### 3 ANALYTICAL RESULTS

#### 3.1 Squeezing at the tunnel face

When a large displacement is created because of squeezing at the tunnel face, toppling or collapse by slipping of the tunnel face is very likely to take place. Figures 3–5 show, for each bench length, the distribution of squeezing (in the Z direction, i.e., tunnel driving direction) along the tunnel centerline at the monitoring section. Figure 6 shows the relationship between

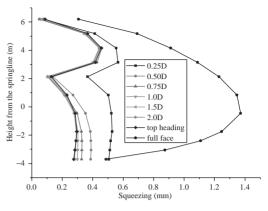


Figure 4. Distribution of squeezing at the face (IV  $_{lower}$  rock).

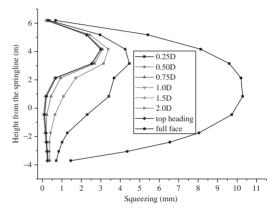


Figure 5. Distribution of squeezing at the face (V<sub>lower</sub> rock).

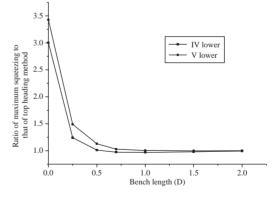


Figure 6. Relationship between bench length and maximum squeezing displacement.

bench length (IV<sub>lower</sub> and V<sub>lower</sub> rock) and maximum squeezing displacement.

For different rock, as the bench length increases, the squeezing distribution tends to the distribution of

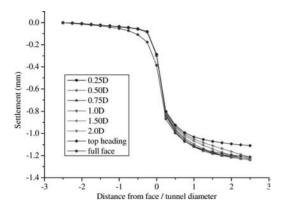


Figure 7. Crown settlement versus face distance for different bench length (IV<sub>upper</sub> rock).

top heading cut. An increase in bench length greatly decreases the maximum squeezing. The longer the bench, the smaller the squeezing: for  $V_{lower}$  type rock, compared with the results for the top heading method, the squeezing of the full face method is 3.43 times, with 0.25D bench length 1.49 times, with 0.50D bench length 1.13 times, and with 0.75D bench length 1.03 times. For another two types of rock, as the bench lengths increase, the changing trends of squeezing are the same as the trend of  $V_{lower}$  type rock. However, with the deterioration of rock, the stable value of bench length (when the bench longer than stable value, there is no significant change of squeezing) is different. For the IV<sub>upper</sub>, IV<sub>lower</sub> and V<sub>lower</sub> rock, the stable values of bench length are 0.50D, 0.50D and 0.75D, respectively.

It is very interesting to note that for  $IV_{upper}$  rock the computed squeezing displacements at the top and bottom of upper section are negative values. That is due to load transfer in longitudinal direction resulted from arcing in the unsupported zone. That has an effect similar to the classical Terzaghi's "trap door" experiment.

#### 3.2 Crown settlement

When a large displacement of the crown at the face is produced, the tunnel face or its vicinity is prone to failure. Figures 7–9 show the crown settlements at the monitoring section versus the normalized distance of the excavation face to monitoring section, for different bench lengths. A negative value of distance indicates that the excavation face has not reached the monitoring section yet. A significant percentage of the final stabilized settlement is induced before face passage. This can only be adequately reproduced in three-dimensional analyses.

The crown settlement before the face passage does not significantly depend upon the bench length. However, compared with full face method, the crown

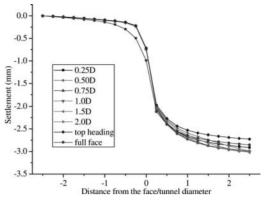


Figure 8. Crown settlement versus face distance for different bench length (IV<sub>lower</sub> rock).

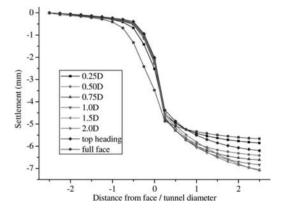


Figure 9. Crown settlement versus face distance for different bench length ( $V_{lower}$  rock).

settlement of bench method before the face passage has a remarkable decrease. As the bench length increases, the stabilized settlement becomes larger and closer to the value of settlement of top heading cut. For V<sub>lower</sub> rock, the stabilized settlement is 5.85 mm for 0.25D bench length, 6.41 mm for 0.50D bench length, and 6.83 mm for 0.75D. When the bench length is greater than 1.0D, the stabilized settlement value is larger than 7.0 mm, but the "exact" value could not be obtained in this study for the limitation of the model. The reason for settlement decreasing mainly attributes to full activation with invert closure. As the bench length decreases, the value of  $L_2$  becomes smaller too. From this point of view, to keep tunnel face stability, the bench length should not be too long for soft rock. On the other hand, to prevent tunnel squeezing, it is necessary to make bench length long enough. Consequently, the bench length too long or too short is not helpful to stability of excavation face, and there is a reasonable value of bench length for some rock. For good rock (i.e.,  $IV_{upper}$  rock), because the stabilized settlement is small, it is not necessary to reduce bench length to decrease the settlement and the bench length can be set to one larger value which benefits the simultaneous excavation of the upper and lower sections. For bad rock (i.e.,  $IV_{lower}$  and  $V_{lower}$  rock), the bench length should not be too long, and 0.5 times of tunnel diameter around for bench length is appropriate.

As mentioned in the introduction, Seki et al. (1994) conducted three-dimensional finite-element analyses to investigate the effect of bench length and shape on the tunnel face stability. In their studies, they concluded that longer benches are made in poorer grounds. However, in this study, the contrary result can be obtained. This is because Seki's modeling was elastic and the tunnel was unlined. The stabilized settlement is the same for different bench length.

#### 4 CONCLUSIONS

The bench cut method by which the upper and lower sections are excavated at the same time is used extensively in mountain tunneling. Results were presented from a detailed three-dimensional finite element analyses directed towards the effects of different bench lengths on the deformation of surrounding mass in soft rock.

The longer the bench, the smaller the displacement due to squeezing at the face. When the bench length is longer than stable value, there is no significant change of squeezing, compared with the squeezing of top heading method. The stable values of bench lengths are different for different rock.

Compared with full face method, the crown settlement of bench method before the face passage has a remarkable decrease, and it does not significantly depend upon the bench length. There is a reasonable value of bench length for some rock. The bench length should not be too long or too short and 0.5 times of tunnel diameter around for bench length is appropriate for soft rock.

#### ACKNOWLEDGMENTS

This work was supported by Shanghai Leading Academic Discipline Project, Project Number: B308.

#### REFERENCES

- Eberhardt, E. 2001. Numerical modeling of three-dimension stress rotation ahead of an advancing tunnel face. *International Journal of Rock Mechanics & Mining Sciences* 38(4): 499–518.
- Farias, M.M., Junior, A.H., Assis, A.P. 2004. Displacement control in tunnels excavated by the NATM: 3-D numerical simulations. *Tunneling and Underground Space Technol*ogy 19(3): 283–293.
- Galli, G., Grimaldi, A., Leonardi, A. 2004. Threedimensional modeling of tunnel excavation and lining. *Computers and Geotechnics* 31(3): 171–183.
- Karakus, M. & Fowell, R.J. 2003. Effects of different tunnel face advance excavation on the settlement by FEM. *Tunneling and Underground Space Technology* 18(5): 513–523.
- Ministry of Communications of the People's Republic of China. 2004. *Code for design of Road Tunnel*. Beijing: China Comunications Press.
- Ng, C.W.W. & Lee, G.T.K. 2005. Three-dimensional ground settlements and stress-transfer mechanisms due to openface tunneling. Canadian Geotechnical Journal 42(4): 1015–1029.
- Seki, J., Noda, K., Washizawa, E. et al. 1994. Effect of bench length on the stability of tunnel face. In Abdel Salam (ed.), *Tunnelling and Ground Conditions*: 531–542. Rotterdam: Balkema.

### The effects of loaded bored piles on existing tunnels

J. Yao, R.N. Taylor & A.M. McNamara *City University, London, UK* 

ABSTRACT: This paper presents the development of a series of centrifuge tests carried out to investigate the effects of loading of bored piles on existing tunnels. The apparatus was designed to monitor the tunnel lining deformation while a live loading was applied. Four facts were considered: rate of loading the pile, pile base level relative to the existing tunnel, the ratio of the depth of clay cover to tunnel diameter, and the distance between the pile and tunnel.

#### 1 INTRODUCTION

High-rise buildings with deep pile foundations are more and more used in the fast developing urban environment. Inevitable disturbance to the ground and surrounding underground structures caused by their construction and subsequent loading may have significant impact in terms of settlement and deformation.

Over the last thirty to forty years, tunnel owners have become concerned about this possibility, and an exclusion zone was introduced to protect the tunnels. However, these guidelines, are mainly based on their empirical correlations from similar projects, and generally apply limits on the minimum distance between the existing tunnels and new pile foundations. (Chudleigh et al. 1999).

Many researchers have summarised the pile-tunnel interaction problems (Schroeder, 2002; Yao et al 2006), which can be categorised into two groups: effects of tunnelling on piles and effects of piling on tunnels, where the second group received less attention. Some field case studies have been presented, which investigated the effect of piling on existing tunnels (Chapman et al, 2001, Higgins et al, 1999, and Benton & Phillips, 1991). Schroeder (2002a, 2002b) also conducted a set of FE analyses to investigate the interaction between pile foundations and existing tunnels. Yao et al (2006) described a series of centrifuge tests designed to investigate the effect of bored pile excavation on existing tunnels.

The effect of bored pile foundations on existing tunnels can be categorised into two parts: pile installation and the post piling period. In this project, the main objective is focused on the post piling period, which is the investigation on the effect of pile loading on existing tunnels. The paper presents the centrifuge apparatus design, the centrifuge test results and some preliminary analysis of the results. All the tests presented are solely with regards to post piling; the influence of pile excavation is not considered.

#### 2 CENTRIFUGE MODELLING

The apparatus was designed to meet five requirements: the pile could be loaded at anytime, a range of loading rates, the applied load could be recorded, pile settlement could be recorded and the rate of loading could be changed during the test.

#### 2.1 Model overview

Figure 1 shows a picture of the model on the centrifuge swing. The centrifuge strongbox is constructed from aluminium with a transparent Perspex window in the front to enable a view of the experiment in progress. A tunnel lining with an internal deflection gauge system installed in the centre of the sample, two rub bags were used to seal the tunnel at the two ends of the tunnel lining (Yao et al, 2006) and a pile loading apparatus was fastened on top of the model container. Druck PDCR81 pore pressure transducers were used to monitor the pore water pressure in the model and air pressure in the tunnel lining system during the centrifuge testing. Digital image processing was used to trace the deformation at the front surface of the model.

All the tests were conducted in the Geotechnical Engineering Research Centre at City University, London. The Acutronic 661 centrifuge with a maximum payload of 200 kg at 200 g and radius of 1.8 m is described in detail by Schofield and Taylor, (1988). All tests presented in this paper were conducted at

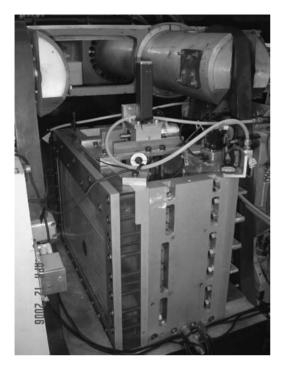


Figure 1. Model on centrifuge swing prier to test.

an acceleration level of 100 g, according to scaling law (Taylor, 1995), 1 cm in the model equals to 1m at prototype scale. Kaolin clay was used for modelling (Al Tabba, 1987).

#### 2.2 Pile loading unit

A pile loading system was designed to load the preinstalled pile during the centrifuge test, and also provide the facilities to measure the settlement of the pile and applied load. Figure 2 shows a schematic of the pile loading unit, which consist a model pile (preinstalled in the model), an actuator, two LVDTs, a load cell, and support units.

For ease of centrifuge operation, the pile was pre-installed into the model during model preparation. Having a pile pre-installed has two important requirements:

- No settlement would occur due to the self weight of pile
- The pile must be strong enough to withstand the applied load

A 22 mm outer diameter, 10 mm internal diameter aluminium tube with two ends sealed was selected as the model pile. Two different lengths were made for different pile base levels and cover to tunnel diameter (C/D) ratios. The model pile was made to have a lower density than the clay sample, so it would not cause

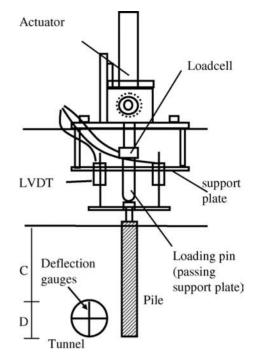


Figure 2. Schematic of the loading unit. C: depth of the cover above tunnel, D: diameter of the tunnel.

any settlements during centrifuge testing without any applied load. As can be seen from Figure 2, a loading pin sitting on top of the pile cap, which had two LVDTs (Linearly variable differential transformers) installed on either of it to monitor the settlement of the pile. The entire loading unit will stop the pile being pushed out due to the lighter density.

The actuator contains a ø35 mm 90 watt motor, RE35-118783, a planetary Gearhead GP 42 C gear box, and an aluminium screw jack, MSZ-Alu. The actuator fitted directly above the pile was supported by a channel unit sitting on top of the strongbox as can be seen from Figures 1 and 2. The loading pin was attached to it, to apply the load to the pile. A load cell was used to monitor the load applied to the pile. It was mounted between the loading pin and the screw jack.

#### 2.3 Deflection gauge tunnel system

Figure 3 shows the cross section of the model in plan, which has similar layout to the pile excavation tests carried out by Yao et al (2006). The tunnel diameter was 50 mm with the thickness of 0.15 mm, and was made of carbon fibre. A 16 mm diameter titanium tube was used to support the tunnel unit and provide the pathway for all the wires and cables. The tunnel was sealed by two air-pressurised rubber bags at the two ends. Tunnel lining deformation detector was attached to it.

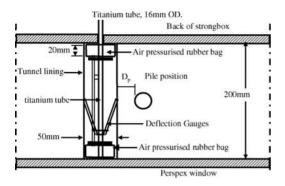


Figure 3. Cross section of the model, showing the deflection gauges and pile borehole(s).  $D_p$  is applied pile-tunnel spacing during test (22 or 44 mm).



Figure 4. Assembled deflection gauges system.

Figure 4 shows the deflection detector unit developed by Yao et al (2006). That was made of four deflection gauges, which were calibrated strain gauged carbon fibre cantilevers, and their output was recorded via an onboard PC. In order to measure both inward and outward deformation, the deflection gauges were given a 4 mm pre-deformation.

#### 2.4 Other equipment

Strongbox: The centrifuge model was set up within an aluminium alloy strongbox, which had an inner plan area of  $550 \text{ mm} \times 200 \text{ mm}$ , and can contain a solid model up to 300 mm high, a 80 mm width Perspex window at the front to enable the model to be viewed during the test.

Water was supplied from a stand-pipe sitting on the centrifuge swing, and the pore pressure in the clay model was monitored using pore pressure transducers pre-installed into the sample before the test. Details of this equipment have been well discussed by a number of researchers at City University, London. (Taylor et al, 1998, McNamara, 2001).

A pile cutter set was used to excavate the bore hole for the model pile. The cutter was made of 22 mm stainless steel thin walled tube.

#### 2.5 Sample preparation and test procedure

Kaolin slurry with a water content of 120% was poured into the strongbox, which had a 3 mm thick porous plastic sheet with a 0.5 mm thick filter paper in the bottom. The strongbox was transferred into a consolidation press, and loaded up to a vertical effective stress of 500 kPa. Upon completion of normal consolidation it was swelled back to 250 kPa. The bulk unit weight of the kaolin was about 17.44 kN/m<sup>3</sup>. A de-aired and calibrated pore pressure transducer was installed into the sample after the swelling period.

A typical sample set-up and test procedure consisted of the following steps:

- Free water at the top of the model was removed after closing the drainage taps closed at the base of the strongbox; this was to avoid clay swelling back.
- The applied vertical stress was reduced to zero and the strongbox removed from the consolidation press.
- The front wall was removed, so the front surface of the Kaolin sample could be cleaned to ensure a better image process, and the top of the sample was trimmed to the required height.
- Tunnel was cut and pre-assembled tunnel unit was installed into the model.
- Marker beads for image processing were pushed into the sample front surface on a 10 mm grid.
- The Perspex window was then bolted onto the strongbox.
- The pile shaft hole was excavated and pile was preinstalled into the model.
- The pile loading unit was mounted on top of the strongbox; loading pin was driven down to the pile cap.
- The strongbox was weighed and placed on centrifuge swing.
- The model was accelerated to 100 g on the centrifuge, and left over night to reach the pore pressure equilibrium.
- Air pressure in the rubber bags at the ends of the tunnel was reduced
- Load applied to the pile using the actuator.

The tunnel lining deformations, pore pressure in the model, load applied, pile settlements and air pressure in the rubber bags, were monitored and data stored on the computer in the control room. Global movement around the tunnel was measured using the image processing system. However, the results of digital image analysis will not be discussed in this paper.

#### 3 TEST RESULTS AND DISCUSSION

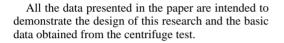
Eight tests were carried out, of which 2 were trial tests used to commission and improve the apparatus. Table 1 summarises all the tests:

- Pile was installed at two different tunnel-pile clear spacing: 22 mm and 44 mm.
- Two different C/D ratios were used: 2 and 3.
- Pile base was installed at two different levels relative to the tunnel position: tunnel crown level and invert level.
- Only a single pile was tested.

Table 1. Table of tests.

Test ID	C/D	Pile Length	Pile base	Spacing
JY13	2	100 mm	Crown	2.2
JY14	2	100 mm	Crown	2.2
JY15	2	150 mm	Invert	4.4
JY16	2	100 mm	Crown	2.2
JY17	2	150 mm	Invert	2.2
JY18	2	100 mm	Crown	4.4
JY19	3	150 mm	Crown	2.2
JY20	3	150 mm	Crown	4.4

\*Spacing is the minimum distance between the outsides of the tunnel and pile (m in prototype scale).



#### 3.1 Rate of loading the pile

The main purpose of two initial trial tests was to testify the loading unit and to determine the best rate at which to load the pile.

Burland et al (1966) produced a simple method to predict the load/settlement curve. It is assumed that the curve is linear up to full mobilisation with takes place at a settlement of about 0.5% of the shaft diameter. Brown et al (2002) presented a series of Statnamic tests to investigate the influence of loading rate on pile behaviour in clay. The pile bearing capacity was found to be particularly sensitive to pile deformation rate. Dayal & Allen (1975) found the similar response. Frischmann & Fleming (1962) stated that the recorded settlement was largely elastic. All settlement was assumed to be as a direct result of shear strains. Skempton (1951) presented the similar result:

$$\rho_s = \frac{p_s \cdot \ln(\frac{r_m}{r_o})}{2 \cdot \pi \cdot L \cdot G_{avr}}$$

where:  $\rho_s =$  shaft settlement,  $p_s =$  load applied to pile shaft-soil interface, L = effective length of pile shaft,  $G_{ave} =$  mean shear modulus of soil along pile shaft,  $r_m =$  radius from pile at which strain becomes negligible (Randolph & Wroth, 1978) and  $r_o =$  pile radius. Whitaker & Cooke (1966) stated that when the settlement is about 0.5 per cent of the shaft diameter, the pile shaft frictional resistance develops rapidly and linearly with the settlement.

The speed of pile displacement also affects the pore pressure at the base of the pile, which can be understood as on increase in magnitude of excess pore pressure with increasing penetration rate (Brown et al,

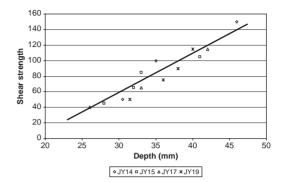


Figure 5. Undrained shear strength measured against depth in model.

2002). The rate of loading used in the centrifuge tests were designed to create undrained conditions. The loading speed of 2 mm per minute was selected on the basis of the trial tests.

The ultimate load of a pile can be defined as either the load at which the pile settlement continues to increase without further increase of resistance, or the load at which pile settlement reaches 10% of the pile base diameter (Fleming et al, 1992). For most soil conditions, the second category is more likely to be the controlling factor for end bearing resistance (Burland et al, 1966). In this research, the ultimate load is defined as the load which causes a settlement of 10% of pile foundation base diameter. The ultimate load capacity of piles can be estimated in terms of undrained strength  $(S_u)$ , in this research measured from quick undrained tests direct taken after the test, and undrained pile shaft adhesion factor, which was chosen as 0.6 in this paper. Figure 5 shows the measured undrained shear strength again depth directly taken after four tests.

Figure 6 shows the recorded load applied in the test JY16. Half of the maximum calculated load reached as soon as the load applied, then increased more gradually towards the maximum. The load only achieved 85% of the designed ultimate load. Applying the factor of safety of 1.5 to 2, the achieved load was acceptable for our research purposes.

#### 3.2 Tunnel lining deformations

Tables 2 and 3 summarise the maximum deformations measured by the deflection gauges for tests JY13 to JY20, and positive values indicate movement towards the tunnel centre. During the pile excavation test carried out by Yao et al 2006, the tunnel crown was subjected to the greatest deformations for all cases, and moved towards the tunnel centre, but there was less effect at tunnel invert. As it can be seen, while the pile was under load, the crown was still affected the

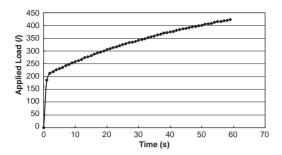


Figure 6. The recorded increasing of load against time.

Table 2.Summary of maximum tunnel lining deformationsrecorded during pile loading period (mm at prototype scale).

ID	Crown	Left	Right	Invert
JY15	11	-1	2	-8
JY16	18	-1	2	-12
JY17	22	-2	3	-19
JY18	10	-1	1	-9
JY19	24	-3	4	-18
JY20	16	-2	3	-14

Table 3. Summary of maximum deformations in percentage: deformation over tunnel diameter  $(\delta/D)$ .

ID	Crown	Left	Right	Invert
JY15	0.22	-0.02	0.04	-0.16
JY16	0.36	-0.02	0.04	-0.24
JY17	0.44	-0.04	0.06	-0.38
JY18	0.2	-0.02	0.02	-0.18
JY19	0.48	-0.06	0.08	-0.36
JY20	0.32	-0.04	0.06	-0.28

most, followed in turn by the invert, the right side and the left side of the tunnel. (The right side of the lining is the nearest site to the pile). Both right and left sides were always moved away from the pile.

Figure 7 shows the recorded tunnel lining deformation for test JY16. It can be seen that the tunnel lining did not start to move until the load reached half of its maximum. Together with Figure 6, it shows the relationship between the applied load, pile settlement and tunnel lining deformation. Similar results were found for most of the tests.

Figure 8 shows the relationship between applied load and pile settlements. The pile did not start to move until the load achieved half of its maximum.

Table 4 lists the change of diameter of the tunnel lining in the vertical and horizontal directions. Figure 9 shows the relationship between the tunnel lining deformation and pile-tunnel clear spacing and Figure 10

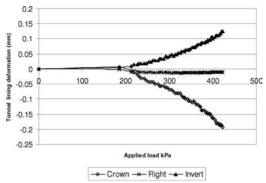


Figure 7. The tunnel lining deformations against the applied load. In all cases, positive values indicate movement towards tunnel centre, deformations at model scale, for test JY16.

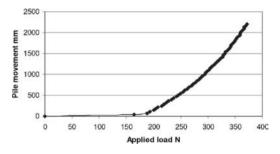


Figure 8. Applied loads against pile settlements, for test JY16.

Table 4. Change of tunnel diameter and tunnel centre position (in prototype).

ID	δν	$\delta v/D$	$\delta h$	$\delta h/D$	Cv	Ch
JY15	3	0.06	1	0.02	9.5	-1.5
JY16	6	0.12	1	0.02	15	-1.5
JY17	3	0.06	1	0.02	20.5	-2.5
JY18	1	0.02	0	0	9.5	-1
JY19	6	0.12	1	0.02	21	-3.5
JY20	2	0.04	1	0.02	15	-2.5

 $\delta_{\nu}$  and  $\delta_{h}$ : Change of tunnel diameter at vertical/ horizontal direction (mm);  $C_{\nu}$  and  $C_{h}$ : Change of tunnel centre position at vertical/ horizontal direction (mm),  $C_{\nu}$  move towards pile opening and  $C_{h}$  move downward.

presents the relationship between the lining deformation and pile base position. It can be seen from changes of the diameter, that deformation of the tunnel lining is non-uniform. The changes of the tunnel centre position in the vertical and horizontal direction are also summarised in Table 4. Figure 11 shows the change in positions of the tunnel centre for each case. Both crown and invert were subjected to significant impact for test JY17 and JY19 where long pile with larger C/D ratio

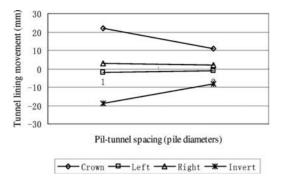


Figure 9. The change of movement due to change of the pile-tunnel clear spacing(C/D = 2).

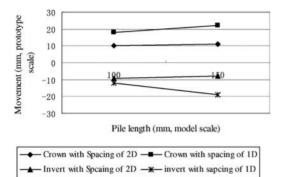


Figure 10. The change of movement due to change of pile base position (C/D = 2).

was used. Movements up to 0.48% of tunnel diameter were recorded on the crown when the long pile was installed together with deeper cover. For the same C/D ratio and tunnel-pile clear spacing, in test JY16 and JY17 a movement of 0.08% of the tunnel diameter was recorded for the longer pile. Summarizing the tables and the figures, it can be see that in general, as clear space increased, the deformation reduced; the deeper the pile bases level or the longer the pile length, the more the effect of the pile at the tunnel.

## 4 CONCLUSIONS

This paper presents a study of the influence of pile loading on an existing tunnel, which is a part of a research on the effect of bored pile installation and subsequent loading on an existing tunnel. The design of the model and preliminary analysis of the results were presented.

Based on the literature review and the centrifuge model tests, the following observations were made.

 By reviewing previous research and based on two trial tests, the rate of the loading was chosen as

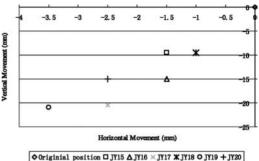


Figure 11. The positions of tunnel centre for all cases. Movement on X axis is towards pile opening, movement on Y axis is towards the bottom of the strongbox.

2 mm per minute. The trial tests also confirmed that increasing in the rate of loading will increase the load achieved on the pile.

- Pile settlement has a linear relationship with increasing of the applied load once the load exceeds half of the maximum designed ultimate load.
- Tunnel lining response to the pile movement as observed from the results shows that the tunnel centre always move downwards and away from the pile.
- Increasing the pile-tunnel clear spacing will reduce the deformation of the tunnel lining.
- Using the long length pile will have more effect on the tunnel lining regardless of the C/D ratio.
- Tunnel crown is always subject to significant movement due to either pile excavation (Yao et al, 2006) or pile loading. Tunnel invert affected more from pile loading than pile excavation.

#### REFERENCES

- Al-Tabbaa, A. 1987, Permeability and stress-strain response of Speswhite kaolin, PhD thesis, University of Cambridge.
- Benton, L. J. & Phillips, A., 1991, The Behaviour of two tunnels beneath a building on piled foundations Proc. 10th European Conference on Soil Mechanics and Foundation Engineering, Florence, 665–668.
- Chapman, T., Nicholson, D. & Luby, D. 2001, Use of the observational method for the construction of piles next to tunnels. Proc. Int. Conf. Response of Buildings to Excavation Induced Ground Movements, (ed F. M. Jardine) London:CIRIA.
- Chudleigh, I., Higgins, K. G., St John, H. D., Potts, D. M. & Schroeder, F. C. 1999. Piletunnel interaction problems. *Proc. Tunnel Construction & Piling '99, London*. The Hemming Group Ltd, 172–185.
- Higgins, K. G., Chudleigh, I., St John, H. D. & Potts D. M. 1999, An example of pile tunnel interaction problems. Proc. Int. Symp. Geotech. Aspects of Underground Construction in Soft Ground, IS-Tokyo

'99(eds O.Kusakabe, K. Fuita & Y. Miyazake) Rotterdam: Balkema, 99–103.

- Schroeder, F. C. 2002a, The influence of bored piles on existing tunnels: a case study. *Ground Engineering* 35, No 7, 32–34.
- Schroeder, F. C. 2002b, The influence of bored piles on existing tunnels. *PhD thesis*, Imperial college, University of London.
- Schofield, A. N. & Taylor, R. N. 1988, Development of standard geotechnical centrifuge operations. *Centrifuge* 88 (ed Corte), Balkema, 29–32.
- Taylor, R. N. 1995, *Geotechnical Centrifuge Technology*, Blackie Academic and Professional, Glasgow.
- Taylor, R. N. Grant R. J., Robson, S. & Kuwano, J. 1998, An image analysis system for determining plane and 3-D displacements in soil models. *Centrifuge* 98, (eds, Kimura, Kusakabe & Takemura), Balkema, 73–78.
- Yao, J, Taylor, R. N. & McNamara, M. A. 2006, The effects of bored pile installation on existing tunnels. *The proceeding* of 6th International Conference on Physical Modelling in Geotechnics, Hong Kong.

# 3D FEM analysis on ground displacement induced by curved pipe-jacking construction

G.M. You

Shanghai Municipal Engineering Design General Institute, Shanghai, P.R. China

ABSTRACT: The power tunnel of Tibet road in Shanghai is the most difficult pipe-jacking project in China with long distance and large diameter. The tunnel crosses through Suzhou river, underground pipelines and area of dense buildings, especially the underground passage of subway and the existing subway line 2, so it's vital important to protect the surrounding environment when the pipe-jacking is constructed. The deformation of ground surface and the existing tunnels during the construction of curved pipe-jacking is studied with 3D finite element methods. First, the calculated results are analyzed and compared with the measured data in site to verify the correctness of the 3D FEM model. Second, there are many factors which can affect ground displacement when curved pipe-jacking is constructed. Among these factors, slurry sleeves' qualities, soil pressure on the face of pipe-jacking, slurry injection pressure and earth resistance are discussed on the basis of finite element simulation. Then, the deformation of existing tunnels of subway line 2 is studied. Finally, the ground displacement formula of curved pipe-jacking is discussed. The results show that the continuity of slurry sleeves and the pressure of the face are very important factors to ground surface deformation during the construction of curved pipe-jacking. The ground displacements induced by curved pipe-jacking are not larger than those of linear pipe-jacking if the slurry sleeves around pipe are good and continual. Because of additional earth resistance, the maximum of the ground surface deformation perpendicular to pipe axis is at the side of the center of the pipe-jacking curve, and the distance of deviation depends on the radius of pipe-jacking curve.

### 1 INTRODUCTION

Curved pipe-jacking technology was applied in Japan, Europe and America many years ago, and some successful experiences were also acquired. Nomura et al. (1985) developed a pipe jacking method (D301) to facilitate long-span, curved and high-speed capabilities in the construction of small diameter (300 mm) tunnels. Nanno (1996) proposed a new curving method called "the unit curving method" in which four jointadjusters are installed between pipes. The joint angle is controlled by the adjusters and the thrust is also distributed uniformly in the four adjusters. The new method solved most of the technical problems in curved drives and performed many jobs successfully in actual construction sites. Vogler & Georg (2002) studied the stresses on the curved pipe. The prediction equation for a curved jacking area was analyzed in order to explain the characteristics of thrust and the friction resistance (Shimada et al. 2004). The thrusts in slurry pipe-jacking can be predicted accurately by using the resistance between the mud slurry and the concrete pipes and the resistance between the soil and the pipes in the curved jacking area.

In China, pipe-jacking technology has been developed rapidly in recent years and many successful constructions were obtained, such as the sewerage outfall project in Shenzhen (Mao, 2001), the secondary project of improving combined sewerage system in Shanghai (Ge, 2002), the project of Hefang street with sewage pipe and rainwater pipe in Hangzhou (Jin et al. 2002), the project of main trunk sewer pipe of Yangli wastewater treatment plant in Fuzhou (Liu, 2003). However, only single curve (horizontal curve or vertical curve) was used in these projects. Ding et al. (2001) calculated and analyzed the jacking force, joint stretching value, pipe internal force, stability of soil and earth resistance of curved pipe-jacking by means of pipe-joint mechanical model and beam on elastic foundation method.

Fang & Wang (1998) analyzed the ground settlement due to pipe jacking in soft soils and developed a method to predict the settlement profile of straight pipe-jacking. Wei et al. (2003) analyzed the mechanism and reason of ground deformation caused by pipe jacking construction. Wei et al. (2005) derived the computing formulas of ground deformation induced by bulkhead additive thrust, force of friction between shield and soil, and force of friction between follow-up pipes and soil by using the Mindlin solution in elastic mechanics. Furthermore, the formula of total ground deformation induced by pipe jacking construction was obtained by combining the formula of ground deformation induced by ground loss with the previous one. However, there is no study on the formula of ground deformation of curved pipe-jacking.

The power tunnel of Tibet road in Shanghai is the most difficult pipe-jacking project in China which is 3.033 kilometers long. This paper discusses the deformation of ground surface and the existing tunnels by means of the numerical analysis. Some important parameters during the construction and the ground displacement formula of curved pipe-jacking are also discussed.

## 2 3D NUMERICAL ANALYSIS OF CURVED PIPE-JACKING

### 2.1 Mechanical model

The radius of pipe-jacking plane curve is 600 m. The distance along the pipe axis range form 0 to 70 m. The distance perpendicular to pipe axis is 66 m and the height of the model is 40 m. The tunnel is located at a depth of 7.5 m (from the top of pipe to ground surface). The outer- and inner-diameter of the concrete pipe are 3,200 mm and 2,700 mm, respectively. In this analysis, we use the following assumptions:

- 1. The pressure on the excavation face is uniformly distributed with circular shape. The value is equal to the actual measurement pressure of soil and water cabin (face pressure). According to the actual observation record, the face pressure is 0.18 MPa.
- 2. During the construction of pipe-jacking, time dependent behavior of soil is not considered.
- 3. The frictional resistance between soil and pipe-ring is uniformly distributed along pipe axis. According to the actual observation record, the frictional resistance is about 2.0 kPa (You et al. 2006).
- 4. To distinguish with straight pipe-jacking, earth resistance should be accounted which induced by curved pipe-jacking. The relationship between earth resistance and radius of pipe-jacking curvature is approximately inversely-proportional linear (Ding et al. 2001). The earth resistance is 12.0 kPa at a radius of 600 m.

## 2.2 Boundary conditions

Displacement boundary conditions are applied to this model. The top side of the model is free boundary. Vertical displacements of the bottom side and normal displacements of the vertical sides are fixed, respectively.

### 2.3 Compute parameters

According to the geological report, the materials and their parameters that are used for this simulation are listed in table 1.

Table 1. Physical and mechanical parameters of the materials.

Materials	Unit weight (kN/m <sup>3</sup> )	Young's modulus (MPa)	Poisson's ratio	Cohesion (kPa)	Friction angle (deg)
Brown clay	18.6	26.7	0.35	21.6	14.5
Silty clay	17.3	14.2	0.37	14.4	13.0
Muddy clay	16.9	11.8	0.39	14.0	10.5
Gray clay	17.3	16.1	0.36	15.7	11.5
Silty clay	17.8	27.2	0.35	18.5	17.5

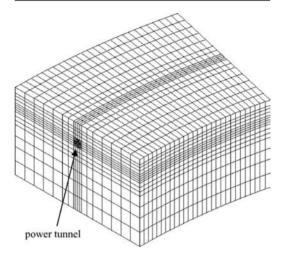


Figure 1. 3D-FEM mesh model.

The breaking criterion used in the model is Drucker-Prager criterion.

### 2.4 Initial stress state

Initial stress state is obtained by the FEM software directly. Only self-weight stress of soil is considered without tectonic stress.

## 2.5 Finite element mesh

The mesh, consisting of 11120 nodes and of 10696 elements, is subdivided into 6 regions, having different material properties. Eight node, solid element and four node, shell element are used to simulate soil and pipejacking ring, respectively. In order to simulate the support effect of slurry sleeves, contact surface element is engaged between pipe-jacking ring and outer soil. Figure 1 shows the 3D FEM model used in this study.

## 2.6 Analysis of numerical results

Figures 2 and 3 show the comparison between ground surface deformation obtained from numerical analysis and form measured data. The predictions from the FEM compare reasonably well with the observed results.

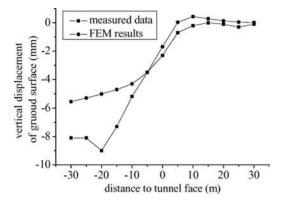


Figure 2. Displacements of ground surface along pipe axis.

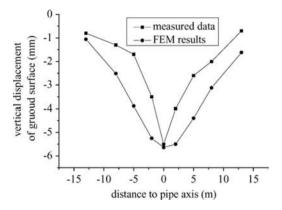


Figure 3. Displacements of ground surface perpendicular to pipe axis.

Because of the complex factors affecting the deformation of ground surface, it is impossible to consider every factor during the simulation of construction. Only several main factors are involved in this study. These factors are slurry sleeves, the pressure of the face and the earth resistance.

### 2.6.1 Contribution of slurry sleeves

The effect of slurry sleeves to reduce resistance between pipe and soil relates not only to the material and mixture ratio of slurry but also to the injection parameters such as the location of injection holes, injection pressure and slurry injection quantity. The location of injection holes and slurry injection quantity can be simulated by different locations of slurry sleeves around pipe. In this study, 5 cases are considered, as shown in Figure 4.

Figures 5 and 6 show the surface displacements with different locations of slurry sleeves. Figure 5 indicates that the continuity of slurry sleeves has vital important effect on the surface displacements. The surface displacements will be reduced sharply if good slurry sleeves can be formed around the pipe. This

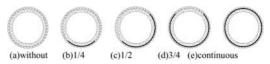


Figure 4. Different locations of slurry sleeves around pipe.

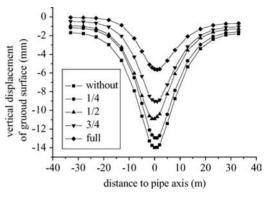


Figure 5. Displacements of ground surface perpendicular to pipe axis with different locations of slurry sleeves.

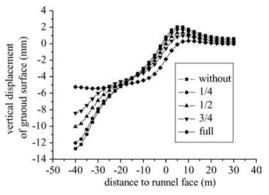


Figure 6. Displacements of ground surface along pipe axis with different locations of slurry sleeves.

phenomenon is more obvious at the top center of pipe. The range of deformation is wider if the slurry sleeves are less continuous. Figure 6 indicates that the less continuous the slurry sleeves, the lager is the ground surface uplift movement ahead the tunnel face and the less are the ground surface settlements above the tunnel face. The less continuous the slurry sleeves, the less are the ground surface settlements behind the tunnel face (in the range  $0 \sim -15$  m). The settlements are invariant after 15 m behind the tunnel face if the slurry sleeves are continuous. Moreover, the settlements will develop fast if the slurry sleeves are not continuous.

## 2.6.2 Contribution of face pressure

Figures 7 and 8 show the surface displacements with different face pressures. Figure 7 indicates that face

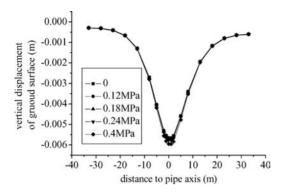


Figure 7. Displacements of ground surface perpendicular to pipe axis with different face pressures.

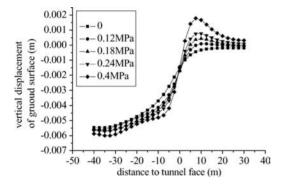


Figure 8. Displacements of ground surface along pipe axis with different face pressures.

pressure has important effect on the surface displacements at the top center of pipe. Figure 8 indicates that the larger the face pressure, the more obvious is the uplift movement of surface ground. The point with maximum uplift movement is closer to the tunnel face with increase of face pressure. The deformations ahead and behind the tunnel face will be increased with increase of face pressure. The settlement at the top of the tunnel face is invariant with different face pressure.

#### 2.6.3 Contribution of earth resistance

Figures 9 and 10 show the surface displacements with different earth resistances. The alteration of earth resistance does not change much the displacements perpendicular to pipe axis. The displacements are almost the same with different earth resistance. From this result, it means that if good and continual slurry sleeves can be obtained, the displacements of ground surface of curved pipe-jacking are almost the same as straight one. Moreover, because of additional earth resistance, the deformation profile perpendicular to pipe axis is not symmetric. The larger the earth resistance, the more obvious is the difference. Furthermore,

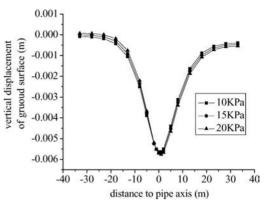


Figure 9. Displacements of ground surface perpendicular to pipe axis with different earth resistances.

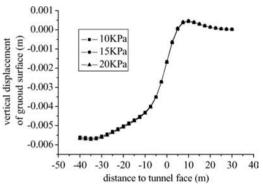


Figure 10. Displacements of ground surface along pipe axis with different earth resistance.

the point with maximum ground surface displacement perpendicular to pipe axis is not on the projection line of pipe axis but at the side of the center of the pipe-jacking curve.

### 3 NUMERICAL ANALYSIS OF SUBWAY LINE 2'S DEFORMATION

The power tunnel passed over the existing subway line 2, and the minimum clear distance between power tunnel and existing subway line 2 is 1.5 m. The total projected length of crossover zone is about 25 m long. The acute angle between power tunnel and subway line 2 is about 75 degree.

Figure 12 shows the relationship between vertical displacement of subway line 2 and jacking distance. Figure 13 shows the relationship between horizontal displacement of subway line 2 and jacking distance. In Figures 12 and 13 origin of jacking distance is 40 m away from the cross point. For the total projected length of crossover zone is about 25 m long, the jacking

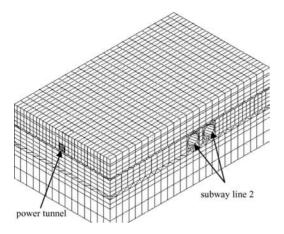


Figure 11. Mesh of power tunnel and subway line 2.

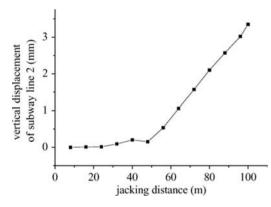


Figure 12. Relationship between vertical displacement of subway line 2 and jacking distance.

distance ranged from 40 to 65 m when the power tunnel passed through subway line 2.

Figure 11 shows the 3D finite element mesh of the power tunnel and subway line 2. The region of the model is: 100 m (along pipe axis) × 60 m (perpendicular to pipe axis) × 40 m (depth). Eight node, solid element and four node, shell element are used to simulate soil and pipe-jacking ring, respectively. Displacement boundary conditions are applied to this model. The top side of the model is free boundary. Vertical displacements of the bottom side and normal displacements of the vertical sides are fixed, respectively. The parameters that are used for this simulation are listed in table 1.

In Figure 12 positive displacement indicates heaving movement. It can be seen that vertical displacement of subway line 2 increased sharply after the jacking face passed through the subway. There was almost no displacement of subway line 2 before the pipe began to traverse the subway. Because the pipelines traversed above the subway line 2, only uplift displacements

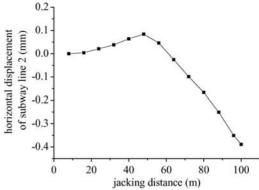


Figure 13. Relationship between horizontal displacement of subway line 2 and jacking distance.

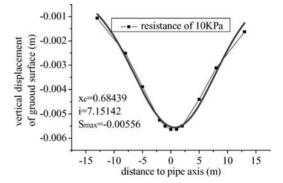


Figure 14. Regression curve of ground surface displacements with 10 kPa earth resistance.

occurred during the construction. Figure 13 indicates that the direction of horizontal displacement pointed to the tunnel face.

The FEM results show that the maximum of vertical and horizontal displacements were 3.3 mm and 0.39 mm, respectively. The values meet the demand of the normal operation of existing subway line 2.

#### 4 GROUND DISPLACEMENT EQUATION OF CURVED PIPE-JACKING

Because of earth resistance, the deformation profile perpendicular to pipe axis of curved pipe-jacking is not symmetric to the normal line of pipe axis. According to numerical results as well as measured data, the ground displacement formula of curved pipejacking is obtained by means of regression analysis (Fig. 14). The vertical displacements perpendicular to pipe axis are:

$$S_V = S_{\max} \exp\left(\frac{-\left(x - x_c\right)^2}{2i^2}\right) \tag{1}$$

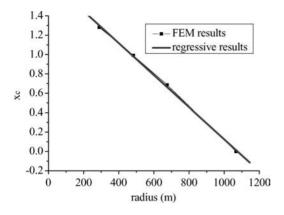


Figure 15. Relationship between curvature radius r and regressive coefficient  $x_c$ .

Where  $x_c$  is the regression coefficient related to the radius of pipe-jacking,  $S_{max}$  and *i* are the maximum settlement of ground surface (when  $x = x_c$ ) and standard deviation of the settlement curve, respectively.

Based on the research consequence (Ding et al. 2001), we can get the curvature radiuses correspondence to the earth resistances mentioned above (10 kPa, 15 kPa and 20 kPa). Then, according to the previous fitting result, the relationship between curvature radius r and regressive coefficient  $x_c$  is obtained (Fig. 15). The fitted linear regression line can be expressed as follows:

$$x_c = -0.00165r + 1.7789 \tag{2}$$

It should be noted that the equations (1) and (2) are obtained with the special geological conditions of this project. The equations' suitability for other areas and geological conditions need more practical verifications.

#### 5 CONCLUSIONS

- 1 Because of additional earth resistance, the maximum value of the ground surface displacement perpendicular to pipe axis is at the side of the center of the pipe-jacking curve, and the distance of deviation depends on the radius of pipe-jacking curve. The larger the radius of pipe-jacking plane curve, the less is the deviation.
- 2 The deformation profile perpendicular to pipe axis of curved pipe-jacking is not symmetric about the normal line of pipe axis.
- 3 Slurry sleeves have vital effect on ground displacements. If good and continual slurry sleeves can be obtained, the displacements of ground surface of

curved pipe-jacking are almost the same as straight one. The key to control the displacements of ground surface is to control the quality of slurry sleeves.

- 4 The face pressure is an important factor to ground surface displacements. During the construction of curved pipe-jacking, the pressure of soil and water cabin should be controlled strictly.
- 5 Only uplift displacements of subway line 2 occurred during the construction of pipe-jacking. The direction of horizontal displacements of subway line 2 pointed to the tunnel face.

#### REFERENCES

- Ding, W.Q. & Zhu, H.H. et al. 2001. Beam on elastic foundation method considering pipe-joint and analysis of pipe packing construction. *Journal of Tongji University* 29(5): 616–620.(in Chinese)
- Fang, C.Q. & Wang, C.D. 1998. An analysis and prediction of ground settlement due to pipe jacking. *Journal of Jiangsu* University of Science and Technology 19(4): 106–110.(in Chinese)
- Ge, J.K. & Zhang, Y. 2002. Application of jacking technology to sharply curved pile. *Chinese Journal of Geotechnical Engineering* 24(2): 247–250.(in Chinese)
- Jin, W.H. & Gong, F.X. et al. 2002. Application of long distance curve pipe-jacking to Hangzhou project. Special Structures 19(4): 62–65.(in Chinese)
- Liu, P.R. 2003. The Application of long-distance curved pipe jacking technology. *Fujian Construction Science and Technology* (2): 38–39.(in Chinese)
- Mao, B.Q. 2001. Technology of curved pipe-jacking with long-distance and large-diameter in sea area. *Special Structures* 18(3): 48–53. (in Chinese)
- Nanno, T. 1996. A method for driving curved pipe-jacked tunnels. *Tunnelling and Underground Space Technology* 11(2): 3–25.
- Nomura, Y. & Hoshina, H. et al. 1985a. Pipe jacking method for long curve construction. *Journal of Construction Engineering and Management* 111(2): 138–148.
- Nomura, Y. & Hoshina, H. et al. 1985b. Design and characteristics of D301 pipe jacking machine. *Denki Tsushin Kenkyusho Kenkyu Jitsuyoka Hokoku* 34(12): 1789–1799.
- Shimada, H. & Khazaei, S. et al. 2004. Small diameter tunnel excavation method using slurry pipe-jacking. *Geotechni*cal and Geological Engineering 22(2): 161–186.
- Vogler & Georg. 2002. Stresses on jacking pipes when driving curved alignments. Betonwerk und Fertigteil Technik/Concrete Precasting Plant and Technology 68(7): 50–61.
- Wei, G. & Huang, Z.Y. et al. 2005. Study on calculation methods of ground deformation induced by pipe jacking construction. *Chinese Journal of Rock Mechanics and Engineering* 24(Suppl. 2): 5808–5815.(in Chinese)
- Wei, G. & Xu, R.Q. et al. 2003. Analysis of ground deformation caused by pipe jacking construction. *Trenchless Technology* 20(6): 24–27.(in Chinese)
- You, G.M. & Ge, J.K. et al. 2006. Study on environmental protection and control techniques of 3-D curved pipe-jacking construction. *Rock and Soil Mechanics* 27(Suppl.): 398–401.(in Chinese)

*Theme 6: Calculation and design methods, and predictive tools* 

## Calculation of the three dimensional seismic stressed state of "Metro Station–Escalator–Open Line Tunnels" system, which is located in inclined stratified soft ground

R.B. Baimakhan, N.T. Danaev, A.R. Baimakhan, G.I. Salgaraeva, G.P. Rysbaeva, Zh.K. Kulmaganbetova, S. Avdarsolkyzy & A.A. Makhanova Scientific Center of Fundamental Research, Almaty, Kazakstan

S. Dashdorj Hokkaido University, Sapporo, Japan

ABSTRACT: This includes examples of a destructive effect of earthquakes on the underground structures and the new mechanic-mathematical model of the inclined-stratified massif, which differs from the like model in such a way that enables to research stressed state of an underground structure for arbitrary orientation of the extended axis. Besides, this includes the results of calculating a three-dimensional seismic stressed state of the underground system, which includes a metro station escalator open line tunnels in the soft ground that has inclined and stratified structure.

## 1 INTRODUCTION

A lot of cases of damage and destruction of underground structures during a strong earthquake are known. They include, for example, underground piping, mountain tunnels, stationary and metro open line tunnels of both shallow and deep shaft. There are many examples of considerable damage and strong destructions in underground structures, from piping to mines, all around the world. Underground structures seem to be stable. But this is not quite true. For example, as shown on the Figure 1, in Daikai, Japan, during earthquakes in Kobe in 1995 (Magnitude - 7.5) not only the roofs and sides of the metro station but also its columns were being destroyed laid at a epth of 50 m from the surface (Iida et al. 1996, Ishihara 1998). There occurred mass destruction in underground mines at a depth of 326 m near the town of Shurab during the Isfara-Batkent earthquake in 1977 (Middle Asia, magnitude - 6.5). (Rashidov et al. 1975).

Based on the analysis of damage of Based on the analysis of damage of 71 underground structures caused by earthquakes, Ch. Daudin, D.V. Monakhenko, S.G. Shulman and others propose to assess them by five categories: 1–shifts along contacts; 2–general distortion; 3–local cracks; 4–rock fracturing and failure and 5–partial failure (Monakhenko & Shulman 1980).

A general analysis shows that during intensive earthquakes underground structures may be destroyed



Figure 1. Characteristic features of destructions of the metro station during earthquakes in Kobe, Japan in 1995.

both in the shallow and deep shaft and also in the earth stratum and rock mass. Factors, which effect on such destructions are diverse. The strength of structure and fastening elements depends not only on manufactured materials, but also on physical & mechanical properties of the surrounding massif. However, one may note a characteristic feature that underground structures of deep shaft are destroyed owing to dynamic stresses surpassing the breaking point of fastening materials. The ground stratum surrounding an underground structure often is characterized by natural anisotropy. The existing methods allow us to consider static and seismic stressed state mainly in a flat state. It is connected, firstly, with restricted possibilities of analytical methods, and, secondly, with an undeveloped model of underground structures in a three-dimensional state subject to the arbitrary orientation of the extended axis in space in arbitrary directions.

Thirdly, to fully provide seismic stability of a complex system of underground structures, it is necessary to consider their interference (for instance, that of station, escalator and metro paired open line tunnels during seismic vibrations)

## 2 MECHANICAL–MATHEMATIC MODELS OF THE INCLINED–STRATIFIED TRANSTROPIC MASSIF

The Figure 2 includes different directions of wave propagation in relation to the elements of the inclined layered massif subject to the modeling by means of a transversal isotropic body with an inclined plane of isotropy. The variant I corresponds to the wave propagation along the line of layer spreading (intersection of the inclined plane of isotropy and the horizontal plane at the angle  $\varphi$ ), the variant II–across the line of layer spreading at the angle  $\phi$ . They were previously considered by J.S. Yerzhanov, Sh.M.Aitaliev, J.K.Massanov in the above-mentioned work.

Thereby underground openings are on the horizontal plane, though they are oriented in a different way related to the line of layer spreading (plane of isotropy): drifts and crosscuts, if they are oriented towards the line of layer spreading across and perpendicular, respectively; diagonal openings if they are oriented at an angle. The variant III corresponds to the common case, where the direction of seismic waves constitutes an arbitrary angle  $\chi$  with the line of layer

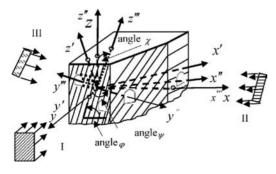


Figure 2. Different directions of seismic waives spreading in inclined-stratified transtropic massif relatively the lengthy axis of underground structure.

spreading. The longitudinal axis of an extended underground opening may have the same direction, i.e. the opening may be inclined to the horizon. This variant was first analyzed by R.B. Baimakhan (Baimakhan 2002).

He also has obtained the most general expressions of velocities of quasi-longitudinal and two quasitransversal waves that spread in arbitrary direction in the inclined layered transtropic massif with normal  $\bar{n} = \bar{n} \{\cos \alpha, \cos \beta, cjs\gamma\}$ .

For such massif, the equation of the generalized law in a matrix form should be written as follows

$$\{\sigma\} = [D]\{\varepsilon\}$$
(1)  
where  $\{\sigma\} = \{\sigma_x, \sigma_y, \sigma_z, \tau_{yz}, \tau_{zz}, \tau_{xy}\}$ ,  
$$[D] = [c_{y}] (i, j = 1, 2, ..., 6),$$
$$\{\varepsilon\} = \{\varepsilon_x, \varepsilon_y, \varepsilon_z, \gamma_{yz}, \gamma_{zz}, \gamma_{yz}, \gamma_{yz}\}.$$

The following values of elasticity factors  $c_{ij}$  for a horizontal stratified state are established. (Erzhanov et al. 1980).

$$c_{11} = \frac{E_{1}(n-v_{2}^{2})}{(1+v_{1})(n(1-v_{1})-2v_{2}^{2})},$$

$$c_{12} = \frac{E_{1}(v_{2}^{2}+nv_{1})}{(1+v_{1})(n(1-v_{1})-2v_{2}^{2})},$$

$$c_{13} = \frac{E_{1}v_{2}}{(n(1-v_{1})-2v_{2}^{2})},$$

$$c_{22} = c_{11}, c_{23} = c_{13},$$

$$c_{33} = \frac{E_{1}(1-v_{1})}{n(1-v_{1})-2v_{2}^{2}},$$

$$c_{44} = G_{2}, c_{55} = G_{2}$$

$$c_{66} = \frac{E_{1}}{2(1+v_{1})}.$$
(2)

where  $E_K$ ,  $v_K$ , (k = 1, 2) – Jung's modules and Poisson's ratios.  $G_2$  – shear modulus. (Lekhnitskiy 1965)

$$d_{ij} = \sum_{m=1}^{6} \sum_{n=1}^{6} c_{mn} q_{im}^{\varphi} q_{jn}^{\varphi} , \qquad (3)$$

$$d'_{ij} = \sum_{m=1}^{6} \sum_{n=1}^{6} d_{mn} q^{\phi}_{im} q^{\phi}_{jn} , \qquad (4)$$

$$\dot{x}_{ij} = \sum_{m=1}^{6} \sum_{n=1}^{6} \dot{d}_{mn} q_{im}^{\chi} q_{jn}^{\chi} .$$
(5)

where  $q_{im}^{p}q_{jn}^{p}$ ,  $(p = \varphi, \phi, \chi, i, j = 1, 2, ..., 6)$  – matrixes of cosines of turning angles. As seen, said calculations are interconnected. Based on the expression (5) in the work of R.B. Baimakhan, expressions are received for speeds of elastic wave propagation in the transtropic massif with a normal of  $\bar{n} = \bar{n} \{\cos \alpha, \cos \beta, \cos \gamma\}$  in an arbitrary direction as follows (Baimakhan 2002)

$$V_{p} = \sqrt{(2\sqrt{-p/3}\cos(\delta/3) - b/3)/\rho} ,$$
  

$$V_{p} = \sqrt{(2\sqrt{-p/3}\cos(\delta/3) - b/3)/\rho} ,$$
  

$$V_{p} = \sqrt{(2\sqrt{-p/3}\cos(\delta/3) - b/3)/\rho} .$$
 (6)

where  $\delta = \arccos(-0.5q(-p/3)^{-3/2})$ ,  $\rho$  – environmental density,  $\alpha$ ,  $\beta$ ,  $\gamma$  – angles between the normal of the wave front and the axes of the Cartesian coordinates *OXYZ*.

## 3 DESTRUCTIVE EARTHQUAKE ACCELEROGRAM CONSTRUCTION SUBJECT TO THE GROUND ANISOTROPY

The methods of reception of elastic equivalent characteristics for alternate isotropic strata are also available in the work of J.S. Erzhanov, Sh.M. Aitaliev and J. K. Masanov (Erzhanov et al. 1980).

For transtropic (transversal-isotropic) strata, elastic constants should be calculated by using the following formulas:

$$E_{1} = \frac{\sum_{k=1}^{n} h_{k} E^{k}}{\sum_{k=1}^{n} h_{k}}, \quad E_{2} = \frac{\sum_{k=1}^{n} h_{k}}{\sum_{k=1}^{n} \frac{h_{k}}{E^{k}}},$$

$$G_{2} = \frac{\sum_{k=1}^{n} h_{k}}{\sum_{k=1}^{n} \frac{h_{k}}{G^{k}}}, \quad \nu_{1} = \frac{\sum_{k=1}^{n} \nu^{k} h_{k} E^{k}}{\sum_{k=1}^{n} h_{k} E^{k}},$$

$$\nu_{2} = \frac{\sum_{k=1}^{n} h_{k}}{\sum_{k=1}^{n} \frac{h_{k}}{E^{k}}}, \quad \frac{\sum_{k=1}^{n} \nu^{k} h_{k}}{\sum_{k=1}^{n} h_{k} E^{k}}.$$
(7)

where  $h_k$  – thickness of the *k* stratum; *n* – the number of strata;  $E^k$ ,  $v^k$ ,  $G^k$  – elastic characteristics of the *k* isotropic stratum;  $2(1 + v^k)$ .

With provision for data of JSC "Almaty metrokurylys" and reference data on grounds (Bulychev 1989) some average values of elastic characteristics are systematised and adduced at in the Table 1. Calculated by formulas (7) values of elastic constants and for different locations of layers are adduced at the Table 2.

These data by value of deformation modulus perpendicular to the layer  $E_1$  conditionally is possible to refer to the three ground conditions of a city. Their minimum values correspond to alluvial grounds at the debris cones of rivers Big and Small Almatynka northward from the Raimbek avenue. Average meanings – to the grounds along the Abay avenue between two rivers Almatinka and Vesnovka. The Most values correspond to grounds of flood-plain areas of rivers. Values of elastic properties may be specified by way of detailed determination of lithologic thicknesses of grounds in regions of all stations and routes of driving tunnels.

If three components of the accelerogram on the surface are known, then to receive their modified values with a depth along the vertical line down, according to the work of R.B. Baimakhan, we have the following formulas (Baimakhan 2002)

$$\begin{aligned} G_{SV_{t}}(T_{SV_{t}}) &= 1 + \frac{1 + K_{SV_{t}}}{1 - K_{SV_{t}}} \left( 1 - \left( \frac{T_{SV_{t}-1}}{T_{SV_{t}}} \right)^{2} \right) + q \left( \frac{T_{SV_{t}}}{T_{SV_{t}-1}} \right)^{2}, \\ G_{P_{t}}(T_{P_{t}}) &= 1 + \frac{1 + K_{P_{t}}}{1 - K_{P_{t}}} \left( 1 - \frac{T_{P_{t-1}}}{T_{P_{t}}} \right)^{2} + q \left( \frac{T_{P_{t}}}{T_{P_{t}-1}} \right)^{2}, \\ G_{SV_{t}}(T_{SV_{t}}) &= 1 + \frac{1 + K_{SV_{t}}}{1 - K_{SV_{t}}} \left( 1 - \left( \frac{T_{SV_{t}-1}}{T_{SV_{t}}} \right)^{2} \right) + q \left( \frac{T_{SV_{t}}}{T_{SV_{t}+1}} \right)^{2}, \\ G_{SV_{t}}(T_{SV_{t}}) &= 1 + \frac{1 + K_{SV_{t}}}{1 - K_{SV_{t}}} \left( 1 - \left( \frac{T_{SV_{t}-1}}{T_{SV_{t}}} \right)^{2} \right) + q \left( \frac{T_{SV_{t}}}{T_{SV_{t}+1}} \right)^{2}, \end{aligned}$$

$$\tag{8}$$

Table 1. Elastic properties of grounds  $E^k$ ,  $v^k$  and thickness  $h^k$  of their layers at the underground rote.

Layers ground paper	<i>E<sup>k</sup></i> (m)	ν (m)	$h^k$
<ol> <li>Soft poured soil</li> <li>Loamy soil</li> <li>Gravel pebbles</li> <li>Loam with pebbles</li> <li>Boulder ground with gravel</li> <li>Boulder ground with pebbles</li> </ol>	7.0	0.40	2.8
	30.0	0.36	2.2
	25.0	0.28	3.3
	8.5	0.21	2.7
	120.0	0.27	3.2
	80.0	0.35	4.8
<ol> <li>7 Tick Loam</li> <li>8 Sand of average size</li> <li>9 Boulder with pebbles</li> <li>10 Tick Clay</li> </ol>	50.0	0.25	6.5
	22.0	0.36	4.5
	200.0	0.32	5.1
	300.0	0.31	5.9

Table 2. Calculated equivalent-transtropic properties of soft stratified ground  $E_1$ ,  $E_2$ ,  $G_2$ ,  $\nu_1$ ,  $\nu_2$ .

Layer	E <sub>1</sub> (Mpa)	<i>E</i> <sub>2</sub> (Мра)	G <sub>2</sub> (Mpa)	$\nu_1$	$v_2$
1	71.19	14.47	5.42	0.30	0.06
2	73.61	15.80	5.95	0.30	0.07
3	73.45	15.61	5.88	0.30	0.06
4	76.51	16.07	6.04	0.30	0.06
5	80.88	16.40	6.14	0.31	0.06
6	83.16	15.87	5.93	0.31	0.06
7	83.02	14.94	5.56	0.31	0.06
8	83.85	14.58	5.42	0.31	0.05
9	83.73	14.23	5.29	0.31	0.05
10	81.55	13.73	5.10	0.31	0.05

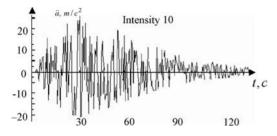


Figure 3. Synthetic accelerogram of a 10 Intensite earthquake.

where q = 0.009;

$$T_{P_{i}} = \frac{h_{i}}{V_{P,h_{i}}}, K_{P_{i}} = \frac{\rho_{h_{i-1}V_{P_{i-1}}}}{\rho_{h_{i}V_{SV_{i}}}},$$

$$T_{SV_{i}} = \frac{h_{i}}{V_{SV,h_{i}}}, K_{SV_{i}} = \frac{\rho_{h_{i-1}V_{SV_{i-1}}}}{\rho_{h_{i}V_{SV_{i}}}},$$

$$T_{SH_{i}} = \frac{h_{i}}{V_{SH,h_{i}}}, K_{SH_{i}} = \frac{\rho_{h_{i-1}V_{SH_{i-1}}}}{\rho_{h_{i}V_{SH_{i}}}},$$
(9)

Having divided the known values of the accelerogram on the surface of the ground by modification factors, we'll receive their values a depth along the vertical line down.

$$\begin{aligned} \ddot{a}_{P_{hi}}(t) &= \ddot{a}_{P,h_{i-1}} / G_{P_i}(T_{P_i}), \\ gg \ \ddot{a}_{SV_{hi}}(t) &= \ddot{a}_{SV,h_{i-1}} / G_{SV_i}(T_{SV_i}), \\ \ddot{a}_{SV_{hi}}(t) &= \ddot{a}_{SV,h_{i-1}} / G_{SV_i}(T_{SV_i}), \\ \ddot{a}_{SH_{hi}}(t) &= \ddot{a}_{SH,h_{i-1}} / G_{SH_i}(T_{SH_i}). \end{aligned}$$
(10)

Using the formulas (2) - (11) by the methods mentioned in the work of R.B. Baimakhan, design accelerograms of intensive 9 - 10 grade earthquakes are constructed for a particular building site as related both to the surface and the inner strata of a heterogeneous massif (Figure 3, Baimakhan 2003).

### 4 CALCULATION OF A SEISMIC STRESSED STATE OF THE UNDERGROUND STRUCTURE SPACE SYSTEM

By a method of finite elements, a seismic stressed state of a complex system of the metro tunnels is researched. The Figure 4 shows a design area. For clearness, the metro station is shown in a double enlarged form.

Paired open line tunnels come to the metro station and go away from the opposite side. Boundary conditions of the task are as follows: in the vertical planes ABDC and EFHG there are no horizontal displacements.

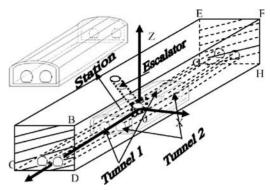


Figure 4. Design area of a complex system of underground structures.

 $\vartheta = 0$ ; in the vertical planes BFHD and AEGC there are no horizontal displacements u = 0; in the horizontal plane CGHD there are no vertical displacements w = 0. The upper horizontal plane AEFB is free of stresses. The roofing of the metro station is at a depth of 17 m from the surface. Geometric dimensions: height, width and length of said design – 125, 45 and 160 m respectively; those of the station – 9, 22 and 100 m; those of open line tunnels – the height and width – 2 m; diameter of the escalator tunnel –9, length –54 m; angle of inclination for the escalator against the horizontal axis 0X for the lying side –30°. Length, width and height of the gallery, which connects the lower end of the escalator to the station are –13, 8 and 5 m.

Said area is divided by eight-nods isoparametric prismatic elements with eight additional internal integration points for 3508 space elements with total nods of 4687. Physical & mathematical properties of the massif are as follows:  $E_1 = 1.028 \cdot 10^4 Mna$ ,  $v_1 = 0.31$ ,  $v_2 = 0.10$ ,  $E_2 = 0.292 \cdot 10^4 Mna$ ,  $G_2 = 0.11 \cdot 10^4 Mna$   $\gamma = 2.2m/m^3$ . Lining materials for the station and the open line escalator tunnels are:

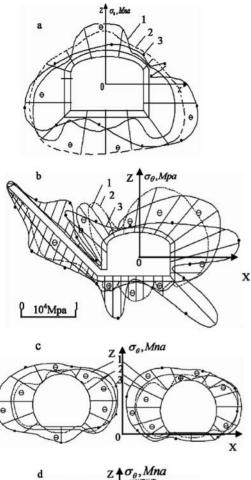
 $E^{o \delta \partial e \pi k u} = 2,5 \cdot 10^4 Mna, \quad v^{o \delta \partial e \pi k u} = 0,25,$  $\gamma^{o \delta \partial e \pi k u} = 2,5 m/m^3$  (timber–lining)

Values of the angles  $\varphi$ ,  $\phi$  and  $\chi$  vary widely. According to the metro project, the angle of inclination  $\chi$  cannot exceed  $\pm 4^{\circ}$  for open line tunnels and 30° for escalator and other special tunnels. The angles  $\varphi$  and  $\phi$  can change from 0 to 90°.

The Figure 5 shows seismic stresses diagrams at the tunnel contours if  $\varphi = 0$ ,  $\phi = 0$ ,  $\chi = 4^{\circ}$  and the angle of wave fall  $\alpha = 60^{\circ}$ ,  $\beta = 30^{\circ}$ ,  $\gamma = 0$ .

#### 5 CONCLUSION

Under the static load, the areas of the roofing and the lying side of the station are exposed to the main stress concentration. Seismic stress diagrams, though they



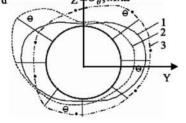


Figure 5. Seismic stresses diagrams at the tunnel contours in different times of a non-stationary seismic load of 10 grade earthquake: a-station section at a distance of 22 m from the front end; b-station section at a level of the escalator; c-section of open line tunnels at a distance of 35 m from the station; d-central section of the gallery between the escalator and the station. The curves correspond: 1-to the statifi loading of the mass of the massif upper strata; 2 and 3 – to the seismic loads at the times of 18.244 sec. and 35.132 sec.

are complex, however, show some regularities. In addition to the angular areas, side areas are exposed to the most intensive seismic load. Such case is typical of station, escalator and open line tunnels and galleries. The middle part of the escalator tunnel is in a more seismic stressed state.

The results of said research make it possible to strengthen elements of an underground structure and to reduce destructive effects cased by acts of God such as earthquakes.

#### REFERENCES

- Baimakhan, R.B. 2002. Analysis of seismic stability of underground structures in a heterogeneous thickness by a method of finite elements. Almaty: Daur.
- Baimakhan, R.B. 2003. Development of the analysis of seismic stressed state of underground structures subject to the peculiarities of the geodynamics of the region. Thesis for competition of an academic degree of Doctor of technical sciences. Almaty: Institute Mathematical and mechanical.
- Erzhanov, J.S. et. al. 1980. Seismic stability of underground structures in the stratified anisotropic massif. Almaty: Nauka (Science).
- Iida, H. et al. 1996. Damadge to Daikai Subway Station. Soil and Foundation. Special Issue: 283–300.
- Ishihara, K. 1998. Performance of tunnels and underground structures during earthquakes. *International Conference* on Soil–Structure Interaction in Urban Civil Engineering. 8–9 October: 19–31
- Lekhnitskiy, S.G. 1965. Theory of elasticity of an anisotropic body. Moscow: Gostechizdat.
- Monakhenko, D.V. & Shulman, S.G. 1980. Issues of seismic stability of underground structures. *News from institutes* of higher education. Series Construction & Architecture No. 8: 3–15.
- Rashidov, T.R. et al. 1975. Seismic stability of metro tunnel structures. Moscow: Transport.

## A complex variable solution for tunneling-induced ground movements in clays

H.L. Bao, D.M. Zhang & H.W. Huang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: A closed-form plane strain solution is presented for an elastic half-plane with a circular tunnel, where an oval- shaped ground deformation pattern is imposed as the boundary condition of the displacement around the tunnel opening. The solution translates complex variables from the research physical region onto a circular ring in the mirror image region by conformal mapping. The coefficients in the Laurent series expansions of the stress functions can be obtained from the boundary conditions and the convergence of the series. Finally, a case study is also performed based on the metro line No. 2 in Shanghai and a satisfactory agreement between the predicted settlement and the observed one is obtained.

#### 1 INTRODUCTION

Due to recent city developments within the limited land of urban areas, more and more complex facilities are developed under the ground surface, which may cause serious potential damage to adjacent overlying services and structures. Engineers responsible for the design and construction of tunnels should predict tunneling-induced ground movements in order to protect the existing structures and tunnels. There we three different approaches for estimating the potential tunneling-induced ground movements: empirical methods, numerical methods and analytical methods.

Empirical procedures have been widely used to assess potential ground movement owing to tunneling. In practice the ground deformations are often described based upon field observations, for instance, a normal Gaussian distribution curve proposed by Peck (1969), which has no theoretical basis. It is assumed that the surface settlement through can be approximated by the normal probability curve or error function. However in reality, ground movements depend on a number of factors such as tunnel geometry and depth, tunnel construction method, the quality of the workmanship and management, behavior of the soil around tunnel. Therefore the empirical methods are also subject to these important limitations.

Recently some attempts have been made to develop closed-form analytical solutions for tunneling-induced ground movements in soft ground. Verruijt and Booker (1996) presented a simple analytical solution for a tunnel in a homogeneous elastic half space by the virtual image technique. Loganathan and Poulos (1998) introduced an equivalent undrained ground loss parameter, which can be estimated using the gap parameter proposed by Lee et al. (1992). Verruijt (1997) proposed the complex variable solution for circular tunnel in an elastic half plane with the boundary condition of a prescribed uniform radial displacement at the cavity opening. Bobet (2001) presented another elastic solution for ground deformations of a shallow tunnel in a saturated ground which was made with the boundary condition of uniform radial displacement at the tunnel opening. However in practice the radial ground movement is not uniform but oval-shaped. Park (2004) proposed the elastic solution for the tunneling-induce ground deformation in clay by imposing the prescribed four different types of oval-shaped displacement at tunnel opening. Wang (2007) expanded Verruijt's solutions by incorporating four different oval-shaped deformation pattern at tunnel opening proposed by Park (2004). In this paper, the complex variable method by Verruijt (1997) is also used for the solution of elasticity problems for a half-plane with a tunnel of the third oval-shaped deformation pattern at tunnel opening. The metro line No. 2 in Shanghai is used to check the applicability of the analytical solution.

#### 2 STATEMENT OF THE PROBLEM

The problem deals with an elastic half-plane with a circular tunnel, see Figure 1. The radius of the tunnel is denoted by r, the depth of its centre by h, and the

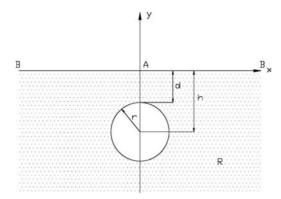


Figure 1. Half-plane with a circular tunnel.

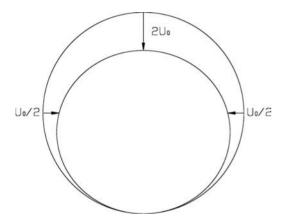


Figure 2. Oval-shaped deformation pattern of tunnel boundary.

cover by d. The upper boundary of the half-plane is free of stress. As in practice the radial ground movement around the tunnel is not uniform but oval-shaped. Park (2004) proposed four different oval-shaped ground deformation pattern as the inner boundary conditions. Wang (2007) suggested the third boundary conditions as the good agreement with the field measurements had been obtained. Therefore in this paper, loading takes place along the inner boundary of the tunnel, in the form of the third oval-shaped deformation pattern (Figure 2), proposed by Park (2004), which can be denoted by,

$$u_r = -u_0 \left( 1 + \sin\theta - \frac{1}{2}\cos^2\theta \right) \tag{1}$$

## **3 BASIC EQUATIONS**

The complex variable method for the solution of two-dimensional linearly elastic problem involves two analytic functions of complex variable,  $\Phi(z)$  and  $\Psi(z)$ .

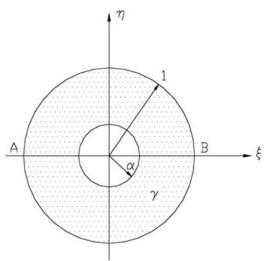


Figure 3. The region after conformal mapping.

The horizontal and vertical displacements,  $u_x$  and  $u_y$ , in the elastic region R can be expressed as

$$2\mu(u_{x} + iu_{y}) = \kappa\Phi(z) - z\overline{\Phi'(z)} - \overline{\Psi'(z)}$$
<sup>(2)</sup>

where  $\mu$  is the shear modulus of the clay, and  $\kappa$  is related to Poisson's ratio v by

$$\kappa = 3 - 4\upsilon$$
 (3)

As plane strain condition are assumed in this paper, the stresses can be expressed as

$$\sigma_{xx} + \sigma_{yy} = 2 \left\{ \Phi'(z) + \overline{\Phi'(z)} \right\}$$
(4)

$$\sigma_{yy} - \sigma_{xx} + 2i\sigma_{xy} = 2\left\{\bar{z}\Phi^{''}(z) + \Phi'(z)\right\}$$
(5)

It is convenient to express the boundary condition in terms of the integral of the surface traction, integrated along the boundary,

$$F = F_1 + iF_2 = iJ_0^{S}(t_x + it_y)ds$$
  
=  $\Phi(z) + z\overline{\Phi'(z)} - \overline{\Psi(z)} + C$  (6)

where C is an integration constant.

### 4 CONFORMAL MAPPING

We conformally map the region R in the Z-plane onto a ring region  $\gamma$  in the mirror image region, which is referred as  $\zeta$ -plane, bounded by the circles  $|\zeta| = 1$ and  $|\zeta| = \alpha$ , see Figure 3. The conformal mapping is given by

$$z = \omega(\zeta) = -i\hbar \frac{1-\alpha^2}{1+\alpha^2} \frac{1-\zeta}{1+\zeta}$$
(7)

Where

$$\alpha = \frac{1}{r} \left( h - \sqrt{h^2 - r^2} \right) \tag{8}$$

If  $\alpha \to 0$  the radius of the circular caving is practically zero, which indicates a very deep tunnel. If  $\alpha \to 1$  the covering depth is very small. For every value of d/h the corresponding value of  $\alpha$  can be determined from equation (8).

By virtue of the substitution  $z = \omega(\zeta)$  the function  $\Phi(z)$  and  $\Psi(z)$  can be written in term of  $\zeta$ ,

$$\Phi(z) = \Phi(\omega(\zeta)) = \Phi(\zeta)$$
(9)

$$\Psi(z) = \Psi(\omega(\zeta)) = \Psi(\zeta)$$
(10)

Because the conformal transformation function  $\omega(\zeta)$  is analytic in the ring region, the function  $\Phi(z)$  and  $\Psi(z)$  can be represented by Laurent series expansions,

$$\Phi(\zeta) = a_0 + \sum_{k=1}^{\infty} a_k \zeta^k + \sum_{k=1}^{\infty} b_k \zeta^{k}$$
(11)

$$\Psi(\zeta) = c_0 + \sum_{k=1}^{\infty} c_k \zeta^k + \sum_{k=1}^{\infty} d_k \zeta^{-k}$$
(12)

The coefficients of the series can be determined from the boundary conditions.

#### 5 BOUNDARY CONDITIONS

The first boundary condition is that the upper boundary y = 0 must be entirely free of stress. According to equation (6), Verruijt (1997) gave the relationship of the coefficients,

$$c_0 = -\overline{a_0} - \frac{1}{2}a_1 - \frac{1}{2}b_1$$
(13)

$$c_{k} = -\overline{b_{k}} + \frac{1}{2}(k-1)a_{k-1} - \frac{1}{2}(k+1)a_{k+1}, \quad k = 1, 2, 3 \cdots$$
(14)

$$d_{k} = -\overline{a_{k}} + \frac{1}{2}(k-1)b_{k-1} - \frac{1}{2}(k+1)b_{k+1}, \quad k = 1, 2, 3 \cdots$$
 (15)

Therefore, one half of the unknown coefficients have been expressed into the other half.

In this paper, the second boundary value problem is considered, in which the third oval-shaped displacement pattern proposed by Park (2004) is prescribed along the tunnel boundary. According to Verruijt's solution, if the function  $G'(\alpha\sigma)$ , which defines the boundary condition at the tunnel boundary, can be written as a Fourier series,

$$G'(\alpha\sigma) = \sum_{k=-\infty}^{+\infty} A_k \sigma^k$$
(16)

the coefficient must satisfy the equations

$$(1-\alpha^2)(\mathbf{k}+1)\overline{\mathbf{a}_{\mathbf{k}+1}} - (\alpha^2 + \kappa \alpha^{-2\mathbf{k}})\mathbf{b}_{\mathbf{k}+1} = (1-\alpha^2)\mathbf{k}\overline{\mathbf{a}_{\mathbf{k}}} - (1+\kappa \alpha^{-2\mathbf{k}})\mathbf{b}_{\mathbf{k}} + \mathbf{A}_{\mathbf{k}}\alpha^{-\mathbf{k}}, \quad \mathbf{k} = 1, 2, 3$$

$$(17)$$

$$(1+\kappa\alpha^{2k+2})\overline{\mathbf{a}_{k+1}} + (1-\alpha^{2})(k+1)\mathbf{b}_{k+1} = \alpha^{2}(1+\kappa\alpha^{2k}) + (1-\alpha^{2})k\mathbf{b}_{k} + \overline{\mathbf{A}_{k+1}}\alpha^{k+1}, \quad k = 1, 2, 3$$

$$(18)$$

and

$$\left(1-\alpha^2\right)\overline{a_1} - \left(\kappa + \alpha^2\right)b_1 = A_0 - (\kappa + 1)a_0 \tag{19}$$

$$(1+\kappa\alpha^2)\overline{a_1} + (1-\alpha^2)b_1 = \overline{A_1}\alpha + (\kappa+1)\alpha^2\overline{a_0}$$
(20)

Wang (2007) had given the expression of the coefficients of the Fourier series for the third oval-shaped displacement pattern,

$$A_0 = -(1+\alpha)^2 \,\mu u_0 i$$
 (21)

$$A_{1} = \left(\frac{3}{2} + 3\alpha + \alpha^{2} - \alpha^{3} - \frac{1}{2}\alpha^{4}\right) \mu u_{0} i$$
 (22)

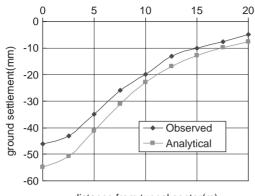
$$A_{k} = -\frac{(\alpha^{2}-l)^{2}(4\alpha+3)+(\alpha^{2}-l)^{3}(k+1)}{4}\alpha^{k-3}\mu u_{0}i \quad (k \ge 2)$$
(23)

$$A_{-k} = -\frac{(\alpha^2 - 1)^2}{4} \alpha^{k-1} \mu u_0 i \quad (k \ge 1)$$
(24)

Therefore all the coefficients of the Laurent series have been determined, except for  $a_0$ . This constant can be determined from the requirement of convergence of the Laurent series by the linear interpolation method given by Verruijt (1997).

#### 6 CASE STUDY

A case study is performed with the background of Shanghai metro line 2. The tunnel lining for Shanghai metro line 2 is 6.2 m in external diameter and 5.5 m in internal diameter. The average depth to the center-line of the tunnel is about 11 m. The tunnel was excavated by EPB shield machine. The shield body is 6.24 m long with 6.34 m in diameter. Therefore the clearance between the external diameters of shield body and the tunnel, which is usually named physical gap  $G_p$ 



distance from tunnel center(m)

Figure 4. Surface settlement.

according to the definition of Lee et al. 1992), will reach 140 mm (Zhang,2004). The Young's modulus of the clay is taken as 25 Mpa and Poisson's ratio as 0.3. Figure 4 is a plot of comparison between predictions of surface settlement from the complex variable solution and observed movements. The comparison is quite good, which proves that the analytical solution can give reasonable predictions of ground movement for shield driven tunnels in soft clays.

## 7 CONCLUSIONS

Current design practice to predict tunneling-induced ground movements is based primarily on empirical methods, which are subject to some important limitations. In practice, the geometry of the tunnel, soil properties and construction methods may affect the settlements of the ground. In this paper, complex variable method is used for the solution of elasticity problems for a half-plane with a tunnel of oval-shaped deformation. The metro line No. 2 in Shanghai is used to check the applicability of the proposed analytical solution. It gives good prediction of tunneling-induced ground movements. However the plasticity of soft soil is not considered in this paper, which means that the method in this paper can only be a preliminary design of tunnels in soft clays. More factors should be taken into account to improve the reliability of the method.

#### ACKNOWLEDGMENTS

The authors wish to acknowledge the financial support from National Science Foundation of China (No. 50608058) and Hi-tech Research and Development Program (863 Program) of China (No. 2006AA11Z118).

#### REFERENCES

- Bobet, A. 2001. Analytical solutions for shallow tunnels in saturated ground. Journal of Engineering Mechanics 127(12): 1258–1266.
- Lee, K.M., Rowe, R.K. & Lo, K.Y. 1992. Subsidence due to tunnelling: Estimating the gap parameter. Canadian Geotechnical Journal 29(6): 929–940.
- Loganathan, N. & Poulos, H.G. 1998. Analytical prediction for tunneling-induced ground movements in clays. Journal of Geotechnical and Geoenvironmental Engineering 124(9): 846–856.
- Park, K.H. 2004. Elastic solution for tunneling-induced ground movements in clays. International Journal of Geomechanics 4(4): 310–318.
- Peck, R.B. 1969. Deep excavations and tunnelling in soft ground. 7th Int. Conf. on Soil Mech. and Found. Engrg., Mexico, 1969.
- Verruijt, A. 1997. A complex variable solution for a deforming circular tunnel in an elastic half-plane. International Journal for Numerical and Analytical Methods in Geomechanics 21: 77–89.
- Verruijt, A. & Booker, J.R. 1996. Surface settlements due to deformation of a tunnel in an elastic half plane. Geotechnique 46(4): 753–756.
- Wang, L.Z. & Lv, X.J. 2007. A complex variable solution for different kinds of oval deformation around circular tunnel in an elastic half plane. Chinese Journal of Geotechnical Engineering 29(3): 319–327.
- Zhang, D.M., Huang, H.W. & Hicher, P. 2004. Numerical prediction of long-term settlements of tunnels in clays. ITA-AITES 2004 World Tunnel Congress and 30th ITA General Assembly., Singapore, May 2004.

## Simulation of articulated shield behavior at sharp curve by kinematic shield model

## J. Chen, A. Matsumoto & M. Sugimoto

Nagaoka University of Technology, Nagaoka, Niigata, Japan

ABSTRACT: Tunnelling cases at sharp curve have increased by using articulated shield with copy cutter. At sharp curve, the shield behavior, i.e., the position, the rotation angle, and the advance direction, should be precisely controlled to follow the planed alignment. However, it is sometimes difficult to control and predict the shield behavior without case records. To simulate the shield behavior, the kinematic shield model has been proposed by the authors, taking into account the excavated area, the tail clearance, the rotation direction of cutter face, the shield slide, and the dynamic equilibrium condition. This paper reports the simulation results of the articulated shield behavior during excavation at a sharp curve. This study yields the following findings: 1) The kinematic shield model simulates the measured path of the shield reasonably; 2) The articulated angle, the copy cutter area and length are the predominant factors affecting the shield behavior.

## 1 INTRODUCTION

Recently, aiming at the completeness of city functions in urban areas, many infrastructures such as roadways, water supply systems, sewerage and electric power lines, etc. have been constructed densely. Due to spatial limitation, sometimes it is difficult to choose simply an ideal route for a new tunnel. On the other hand, with new techniques adopted continuously, shield tunneling method has achieved greatly. Under these circumstances, many shield tunneling cases at sharp curve have been reported. In the case of excavation at a sharp curve, articulated shield becomes popular since conventional single shield met some difficulties in operational control and generated wide range of ground disturbance.

The direction control systems have been applied. However, these systems are based on empirical relationships and are lacking in the precise theoretical background (Shimizu & Suzuki 1994). Therefore it is sometimes difficult to control the shield at a sharp curve and to predict the shield behavior without case records.

To clarify the shield tunneling behavior and the behavior of the surrounding ground, numerical methods such as FEM, DEM have been adopted (Komiya et al. 1999; Kasper & Meschke 2004; Melis & Medina 2005). The enforced displacement by means of the gap between the excavated surface and the tunnel lining has been applied in FEM and this technique gave a very good prediction of the ground movement for shield tunneling (Rowe & Lee 1992). DEM is proved to be effective for soil stability problem at tunnel face. However, these numerical methods require the shield movement as one of known conditions. The immediate ground movement during excavation stage is difficult to be simulated, since it is related to the shield movement and the excavated area.

By taking into account ground displacement around the shield, the kinematic shield model had been proposed for single circular shield (Sugimoto & Sramoon 2002). This model was validated by the simulation of an earth pressure balanced (EPB) shield behavior along a straight alignment in a single layer of sandy gravel (Sramoon et al. 2002) and along a curve in a multilayered ground (Sugimoto et al. 2007). Extending this model to articulated shield, the articulated shield model was developed and validated by simulating the steady behavior of curve-only excavation (Sugimoto et al. 2002).

The transient behavior of an articulated shield in a way of sharp curvature movement is of interest to be cleared. In this study, the simulation results of the articulated shield behavior at a sharp curve with radius 20 m are reported. The validity of the model is examined by comparing the simulation results with the measurement data. Furthermore, the factors affecting shield behavior are also discussed.

### 2 KINEMATIC SHIELD MODEL

The articulated shield model is composed of five forces: force due to self-weight of machine,  $f_1$ ; force

on the shield tail,  $f_2$ ; force due to jack thrust,  $f_3$ ; force on the cutter disc,  $f_4$ ; and force on the shield periphery,  $f_5$ , as illustrated in Fig. 1 (Sugimoto & Sramoon 2002). Among them,  $f_1$  and  $f_5$  act on both sections of the shield.  $f_3$  comes from shield jacks and articulated jacks. f5 is composed of the ground reaction force and the dynamic frictional force on the shield skin, which are due to the earth pressure acting on the shield skin plate. Since earth pressure is reliant on ground deformation, and excavated cross section area is usually a little bit larger than the shield cross section area, the ground reaction force from the ground to the shield can be obtained by considering the coefficient of earth pressure K, which is given by a function of the distance between the original excavated surface and the shield skin plate  $U_n$ , as shown in Fig. 2. In Fig. 2, K at  $U_n = 0$  means the coefficient of earth pressure at rest  $K_0$ , and the gradient of K at  $U_n = 0$  represents the coefficient of subgrade reaction k. Here, note that the subscripts h and v mean horizontal and vertical directions, respectively; the subscripts min, o, and max define minimum, initial, and maximum, respectively; and  $\sigma_{v0}$  is overburden pressure. K in any direction  $K_n$  can be calculated by interpolation between  $K_h$ and  $K_{v}$ .

The shield behavior is represented by the movement of the shield in x, y, and z directions ( $\Delta x$ ,  $\Delta y$ ,  $\Delta z$ ), and the shield postures (yawing angle  $\phi_y$ , pitching angle  $\phi_p$ , and rolling angle  $\phi_r$ ). Since the change of  $\phi_r$  is limited in practice, the factor of shear resistance due to the cutter torque  $\alpha_{sg}$  was adopted as a parameter instead of  $\phi_r$ . The articulated shield behaviour can be obtained by solving the following equilibrium conditions of forces and moments:

$$\begin{bmatrix}\sum_{i=1}^{5} \left(\boldsymbol{F}_{Fi}^{M} + \boldsymbol{F}_{Ri}^{M}\right)\\\sum_{i=1}^{5} \left(\boldsymbol{M}_{Fi}^{M} + \boldsymbol{M}_{Ri}^{M}\right)\end{bmatrix} = 0$$
(1)

where F and M are the force and moment vectors and the subscripts F and R denote front and rear sections of the shield respectively. The moment vectors are generated by the cross product of the position

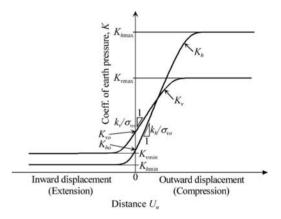
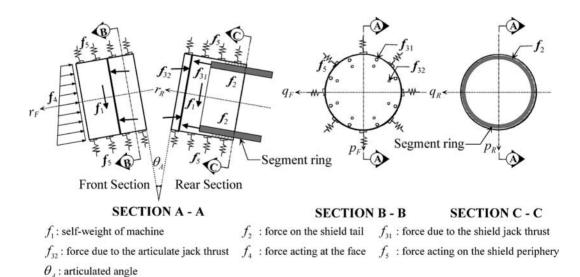
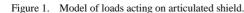


Figure 2. Ground reaction curve.





vector and the force vector. Here, the superscript M indicates the machine coordinate system, which can be transformed to total coordinate system by using transformation matrices.

#### 3 SHIELD TUNNELLING SITE

#### 3.1 Test site description

The test site was established at a cable tunnel. The total length of this cable tunnel is 264 m and the test site is about  $60 \text{ m} \log at$  a leftward sharp curve section with radius of 20 m. The inclined gradient of the tunnel alignment is upward of 0.2%.

Figure 3 shows the geological profile at the test section, where the overburden depth is about 34 m and the groundwater level is 2.8 m below the ground surface. Table 1 shows ground properties. The ground is composed mainly of alluvial layers (As1, Ac1, Acs, As2, and Ac2) and diluvial layers (Ds1, Dc2, Dc3, and

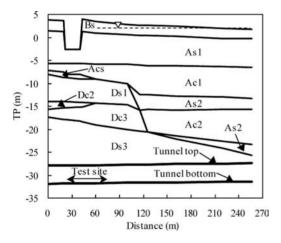


Figure 3. Geological profile at test site.

Table 1.	Soil	parameters	at	test	site.
----------	------	------------	----	------	-------

*M* Ds3). The tunnel is excavated in the diluvial sand layer Ds3 with *N*-values over 50 by standard penetration

test.

The articulated slurry shield with outer diameter of 3.95 m and 5.765 m in length was used. After excavation, the reinforced concrete segments with outside diameter of 3.8 m and 1.2 m in width were installed at the straight sections. The steel segments with 0.3 m in width were adopted at the curve section. The dimensions of the tunnel and the shield are summarized in Table 2.

Table 2. Dimension of tunnel and machine.

Item	Component	Value
Tunnel	Horizontal curve radius (leftward)	20 m
	Vertical slope gradient (ascending)	0.2%
	Overburden depth	34 m
	Groundwater level	G.L2.8 m
	Outer radius of segment	1.9 m
	Width of segments	1.2 m, 0.3 m
Shield	Outer radius	1.975 m
	Total length	5.765 m
	Length of front section	2.24 m
	Length of rear section	3.525 m
	Self-weight	810 kN
	Open ratio of cutter face	20.0%
	Thickness of cutter face	0.35 m
	Radius of chamber	1.943 m
	Length of chamber	0.80 m
	Radius of cutter face	1.98 m
Shield jack	Number of jacks	12
-	Cross-sectional area	346.361 cm <sup>2</sup>
	Radius of jack	1.67 m
Articulated jack	Number of jacks	12
•	Cross-sectional area	433.736 cm <sup>2</sup>
	Radius of jack	1.47 m

Soil layer	Unit weight $\gamma(kN/m^3)$	Cohesion c (kN/m <sup>2</sup> )	Friction angle $\phi(^{\circ})$	Coef. of subgrade reaction $k$ (kN/m <sup>3</sup> )	Coef. of earth pressure at rest $K_0$	Young's modulus E (kN/m <sup>2</sup> )
Bs	19.0	0	26	4737	0.562	8400
As1	19.0	0	38	9474	0.384	16800
Ac1	15.3	59	0	789	0.844	1400
Acs	18.1	0	24	3158	0.593	5600
As2	19.2	0	29	7895	0.515	14000
Ac2	15.4	50	0	2368	0.832	4200
Ds1	19.2	0	37	19737	0.398	35000
Dc2	15.8	91	0	8684	0.848	15400
Dc3	15.0	160	0	7895	0.847	14000
Ds3	19.7	0	45	67895	0.293	120400

#### 3.2 In situ data

Figure 4 shows the data of tunneling operation, shield behavior, and excavation condition measured continuously by automatic measurement system. The parameters of tunneling operation are articulated angle in horizontal direction  $\theta_{CH}$  (+: leftward), area of applied copy cutter CC range (measured from the invert of shield in clockwise direction, viewed from shield tail), jack thrust  $F_{3r}$ , horizontal jack moment  $M_{3p}$  (+: rightward), vertical jack moment  $M_{3q}$  (+: downward), and cutter torque CT (+: anticlockwise direction, viewed from shield tail). The parameters of observed shield behavior are defined as yawing angle  $\phi_v$  (+: rightward), pitching angle  $\phi_p$  (+: downward), and rolling angle  $\phi_r$  (+: clockwise direction, viewed from shield tail). The parameters of excavation condition are shield velocity  $v_s$ , slurry pressure  $\sigma_m$ , and slurry density  $\gamma_m$ in the chamber, which are usually controlled to stabilize the tunnel face. The mucking ratio  $R_v$  is the ratio of the measured discharged soil volume to the theoretical excavated soil volume.

The articulation of the shield was applied to negotiate the sharp curve. To follow the planed alignment, the applied articulated angle increased gradually from the beginning point of the curve, then it kept steady value of 500 minutes, which suits for the leftward curve of 20 m radius. CC was approximately 0.1 m in length and was applied mainly in range of 30°~180° (measured from the shield invert in clockwise direction, viewed from tail) to increase the excavated area around the cutter disc, which reduces the acting earth pressure on the shield skin plate and makes a shield advance easily.  $F_{3r}$  was applied to drive the shield forward against the earth pressure at the cutter face and the dynamic friction around the shield skin plate. After the transient section from curve to straight line (from the distance 75 m to the distance 80 m),  $F_{3r}$  increased obviously.  $M_{3p}$  was applied to turn the shield to follow the horizontal leftward curve.  $M_{3q}$  was mainly applied against the vertical moment due to the earth pressure on the cutter disc and the self-weight of the shield. CT was generated due to the shearing resistance on the cutter disc.

The observed  $\phi_y$  reveals the detailed yawing behavior of the shield. The observed  $\phi_p$  indicates that the shield negotiates the inclination of the tunnel alignment. The observed  $\phi_r$  fluctuates at the sharp curve alignment and is somewhat relative to *CT* and the rotation direction of the cutter disc. This points out that the shield rolls around its longitudinal axis in the opposite rotation direction of the cutter disc.

The shield velocity  $v_s$  decreased at the sharp curve. To stabilize the face,  $\sigma_m$  was applied based on the lateral earth pressure at the tunnel face, and  $\gamma_m$  was kept approximately  $12 \text{ kN/m}^3$ .  $R_v$  was close to unity throughout the test site, which indicates that excellent excavation control has been achieved.

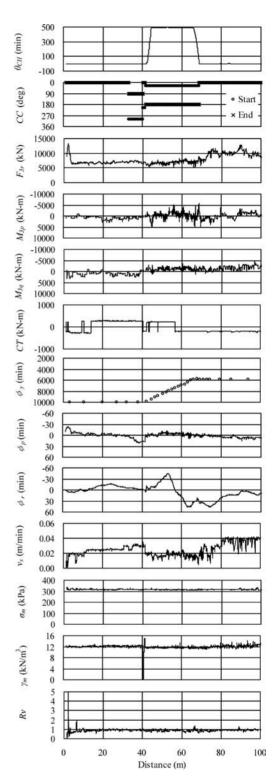


Figure 4. Shield tunneling measurement data.

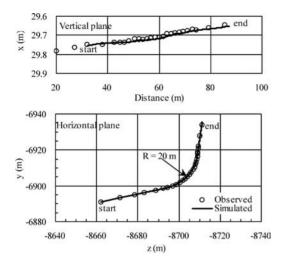


Figure 5. Simulated and observed shield traces.

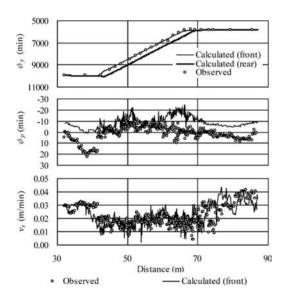


Figure 6. Simulated and observed shield behavior.

#### 4 SIMULATION RESULTS

#### 4.1 Shield behavior

Figure 5 shows the simulated and observed shield traces. From this figure, the maximum difference of 1 cm for vertical position and the maximum difference of 3 cm for horizontal position can be verified. The simulated and observed time dependent parameters  $\phi_y$ ,  $\phi_p$ ,  $v_s$  are compared in Fig. 6. The simulated  $\phi_y$  indicates that the shield performs good negotiation to the sharp curve. As for the simulated  $\phi_p$ , there

is about 15 minutes uplift at the first straight section (from the distance 30 m to the distance 42 m), compared with the observed data. From the distance 60 m to the end point of the test site, maximum 20 minutes uplift can be found. Considering the possibility of change of geological conditions at some locations and the 5 minutes precision of the inclinometer, these differences are acceptable. Except for the transient section from curve to straight line, the calculated  $v_s$  is generally consistent with the observed. At the transient section, the maximum difference of 0.01 m/min for  $v_s$  is revealed. According to the construction report, intermittent excavation is applied to confirm the tail clearance at this section, there is some possibility to record slower velocity than that obtained in the simulation.

#### 4.2 Ground-shield interaction

Ground-shield interaction is discussed by using the calculated distance between the original excavated surface and the shield skin plate  $U_n$  and the calculated normal effective earth pressure acting on the shield  $\sigma_{ns}$  at straight line and sharp curve. Figs. 7 and 8 show the distribution of  $U_n$  and  $\sigma_{ns}$  around the shield periphery at the straight line with distance of 39.3 m and at the sharp curve with distance of 58.4 m respectively. Here, note that the shield periphery is unfolded as a flat plate, i.e., the vertical axis shows the length of the shield and the horizontal axis represents the circumference of the shield.

In the case of excavation at straight line, the following are found from Fig. 7: (1) the contour lines of  $U_n$  become dense around 45°, 80°, 260°, 335° and  $U_n$ is about 30 mm around 60° and from 270° to 300°, because the copy cutter is applied from 300° to 90° shown in Fig. 4 and the effective rate of over cutting decreased between 30° and 330° by the assumption that the mucking at the invert is not sufficient; (2) the distribution of  $U_n$  at the cross section of the shield is almost same along the longitudinal direction, which is natural at straight line; (3) The contour lines of  $\sigma_{ns}$  appear around the invert and the crown of shield, since  $U_n$  around both spring lines is smaller than  $U_n$  at other area and the horizontal effective earth pressure is smaller than the vertical one due to  $K_0$  of  $D_{s3} = 0.293$ shown in Table 1.

When shield excavated at sharp curve, the following are found from Fig. 8: (1)  $U_n$  becomes positive at the end of the front body and the rear body around the right spring line of shield. At the same time,  $U_n$ becomes positive at the middle length of the both bodies along the left spring line. The shield skin plate at these locations pushes the ground, whereas  $U_n$  at the opposite side becomes negative, where the earth pressure is in extension state. These characteristics result from the equilibrium condition and correspond to the leftward curve of the tunnel alignment; (2) the contour

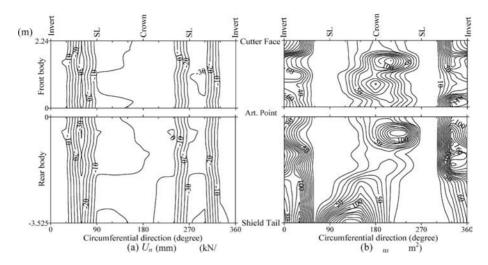


Figure 7.  $U_n$  and  $\sigma_{ns}$  around shield at the straight line with distance of 39.3 m.

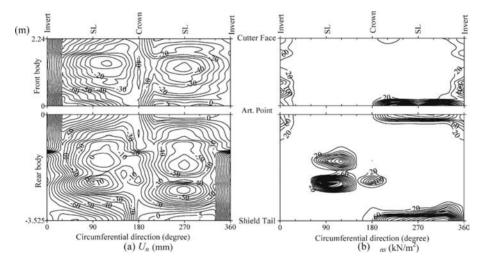


Figure 8.  $U_n$  and  $\sigma_{ns}$  around shield at the sharp curve with distance of 58.4 m.

lines of  $U_n$  become dense at the right bottom of the rear part and at the left bottom of the front part, since the applying range of copy cutter shifts at the boundary position due to the change of cutter face rotation direction shown in Fig. 4; (3)  $U_n$  along both spring lines has a fluctuation due to the wriggle motion of the shield during excavation; (4) the intensity of  $\sigma_{ns}$  appears at the area where  $U_n$  is positive, which is reasonable from the view point of ground-skin plate interaction.

## 5 CONCLUSIONS

The articulated shield behavior at sharp curve was simulated and the calculated shield behaviour was compared with the observed one. Furthermore, ground-shield interaction was discussed using the distribution of  $U_n$  and  $\sigma_{ns}$  around the shield. As a result, the following conclusions can be made:

- The kinematic shield model for articulated shield simulated the shield position within 3 cm difference and the shield pitching angle within 20 minutes difference at sharp curve. This indicates that the proposed model can simulate shield behavior at sharp curve reasonably.
- The articulated angle and the copy cutter area and length are the predominant factors affecting the shield behavior especially at sharp curve.

#### REFERENCES

- Kasper, T. & Meschke, G. 2004. A 3D finite element simulation model for TBM tunneling in soft ground. *International Journal for Numerical and Analytical Methods in Geomechanics* 28(14): 1441–1460.
- Komiya, K., Soga, K., Akagi, H., Hagiwara, T. & Bolton, M. D. 1999. Finite element modelling of excavation and advancement processes of a shield tunnelling machine. *Soils and Foundations* 39(3): 37–52.
- Melis, M. J. & Medina, L. E. 2005. Discrete numerical model for analysis of earth pressure balance tunnel excavation. *Journal of Geotechnical and Geoenvironmental Engineering* 131(10): 1234–1242.
- Rowe, R. K. & Lee, K. M. 1992. Subsidence owing to tunnelling: II. Evaluation of prediction technique. *Canadian Geotechnical Journal* 29: 941–954.
- Shimizu, Y. & Suzuki, M. 1994. Movement characteristics and control method of shield tunnelling machine

of articulate type. *Transactions of the Japan Society of Mechanical Engineers* (Series C) 60(571): 141–148. (in Japanese)

- Sramoon, A., Sugimoto, M. & Kayukawa, K. 2002. Theoretical model of shield behavior during excavation II: Application. *Journal of Geotechnical and Geoenvironmental Engineering* 128(2): 156–165.
- Sugimoto, M. & Sramoon, A. 2002. Theoretical model of shield behavior during excavation I: Theory. *Journal of Geotechnical and Geoenvironmental Engineering* 128(2): 138–155.
- Sugimoto, M., Sramoon, A., Konishi, S. & Sato, Y. 2007. Simulation of shield tunnelling behavior along a curved alignment in a multilayered ground. *Journal of Geotechnical* and Geoenvironmental Engineering 133(6): 684–694.
- Sugimoto, M., Sramoon, A., Shimizu, T., Dan, A. & Kobayashi, T. 2002. Simulation of articulated shield behavior by in-situ data based on kinematic shield model. *Journal of Tunnel Engineering* 12: 471–476. (in Japanese)

# Deformation and pore pressure model of the saturated silty clay around a subway tunnel

## Z.D. Cui & Y.Q. Tang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

## X. Zhang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China

ABSTRACT: Shanghai subway Line No. 2 passes through the center of Shanghai from Songhong Road station in the west to Zhangjianggaoke station in the east. The total length of Shanghai subway Line No. 2 is 25 km. Continuous dynamic monitoring is conducted by means of embedded earth pressure piezometers and pore piezometers around the tunnel. Based on continuous field monitored data and using the laboratory GDS (Global Digital System) test apparatus, the developing law of the pore water pressure of the saturated silty clay around the tunnel is explored with the distance. By the GDS test, the model of the increasing pore water pressure of the saturated silty clay is put forward under the vibration loading. It is also amended by field monitored data. The pore pressure model and characteristics of deformation of the saturated silty clay of Shanghai under the subway train are analyzed. The result offers a valuable reference to the design, construction and the safe operation of the subway tunnel.

## 1 INTRODUCTION

The law of increasing pore water pressure under vibration loading is an important factor for the deformation and strength of soil and is also the key to use effective stress theory for the dynamic analysis. It is of primary importance to predict correctly the changing law of pore water pressure of soil. Researchers have studied the increase and dissipation law of pore water pressure in sandy soil under vibration loading. Zeng et al. (2005) and Guan et al. (2004) studied the characteristic of pore water pressure in silt and silty sand under the cyclic loading and put forward the changing law of pore water pressure. Zhou et al. (2002) and Pradhan et al. (1998) tried to find the relation between the pore water pressure and the path of strain and stress, and summed up the effect on the development of the pore water pressure in the liquefaction of the sandy soil. Lee et al. (1975) and Shao et al. (2006) studied the test parameters of deformation capability of the saturated sandy soil under the cyclic pore water pressure. Li et al. (2005) analyzed the mechanism of liquefaction of silt, influence factors and the developing law of pore water pressure of silt in the course of vibration. Guo et al. (2005) studied the effect of different directions of the principal stress on the characteristic of undrained circulation of saturated loose sand in the

course of vibration. Zeng et al. (2001) studied the increase and dissipation law of saturated clay under shock loading. Meng et al. (2004) studied the characteristic of dynamic response of pore water pressure in saturated silty clay under shock loading. But there are few researches on the developing law of pore water pressure of saturated clay under the long term subway vibration loading. The grain composition, mechanical characteristics and moisture migration, etc. of the saturated silty clay are different from sandy soil. The speed of the increase and dissipation of pore water pressure under long-term subway vibration loading is relatively slow and the developing law of pore water pressure is also different from sandy soil. Based on continuous field data and with the use of GDS in the laboratory, this paper studies the law of the increasing pore water pressure and deformation of the saturated silty clay around the subway tunnel at different depths.

### 2 DYNAMIC MONITORING

In order to study the influence caused by the subway vibration loading on the saturated silty clay around the tunnel, the field test and monitoring are conducted in this research. The site is selected between Jingansi Station and Jiangsu Road Station. The dynamic monitoring system is adopted for field monitoring and its sampling frequency can reach 200 Hz and its precision is 0.1 kPa. It can fully reflect the soil response around the tunnel due to the subway vibration loading. The dynamic monitoring system consists of a resistance sensor, a dynamic strain amplifier, a data selector and a computer. The system can record all the sampling data collected by the computer in real time.

Figure 1 shows the layout of boreholes at the site. In the plane, there are five boreholes, each 110 mm in diameter, parallel and vertical to the subway tunnel, respectively. The distance between the site and Jingansi Station is 210 m. Boreholes No. 3, No. 4 and No. 5 are parallel to the tunnel axis only 1.8 m away from the outside of the segment of the subway tunnel and the distance between them is 15.0 m. Boreholes No. 1, No. 2, and No. 3 are vertical to the tunnel axis. In order to study the attenuation of the effect on the soil around the tunnel with the increasing distance under the subway vibration loading, the distance between No. 1 and No. 2 is 3 m and that between No. 2 and No. 3 is 2 m, so that there is a step-up course. Figure 2 shows the distribution of strata and instruments. In the section, the subway tunnel lies in gray silty clay of layer No. 4. The earth pressure piezometers and pore piezometers are located at the depths of 8.5 m, 11.5 m and 13.5 m, respectively, in layer No. 4, to monitor the response characteristic of the vibration for the subway running.

#### 3 GDS TEST

Under the natural stress, the stress state of the undisturbed saturated silty clay is in the  $k_0$  consolidation. The response amplitudes and response frequency of soil have not been studied under the subway vibration loading. In order to study the characteristics of soil under the subway vibration loading, this paper uses field monitored data and field investigation data

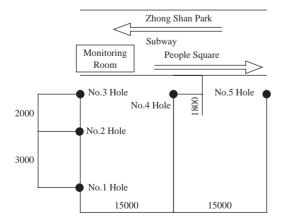


Figure 1. Layout of boreholes (unit: mm).

to design the laboratory test. GDS (Global Digital Systems) apparatus is used in the test, as shown in Figure 3. It can monitor the test process at the real time and collect data at high speed and store them. It has merit of high precision, easy operation, reliable results, etc. Its working principle is shown in Figure 4.

In the cyclic test, the soil samples are first saturated under back pressure. In order to simulate field conditions to the utmost, the samples are consolidated under  $k_0$  condition. The confining pressure  $\sigma_h$  is obtained by calculation according to the natural soil strata, that is,  $\sigma_h = k_0 \sigma_v (\sigma_v = \sum \gamma_i h_i)$ . After consolidation, the cyclic triaxial test begins and the cyclic stress  $\sigma_d$  should simulate the bearing dynamic loading to the utmost.

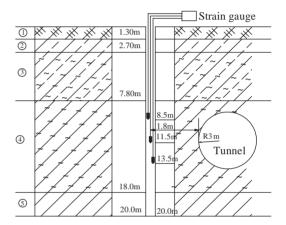


Figure 2. Soil profile and embedded instruments. Note: No. 1 layer is mixed soils; No. 2 layer is brown yellow silty clay; No. 3 layer is gray mucky silty clay; No. 4 layer is gray mucky clay; No. 5 layer is gray silty clay.

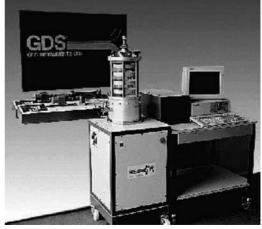


Figure 3. GDS system for dynamic multi-function triaxial test.

By the continuous dynamic field monitoring, plenty of data of boreholes are obtained. By collecting, analyzing and arranging them, two kinds of response frequency of soil are obtained when the subway train runs across the site. The high frequency is  $2.4 \sim 2.6$  Hz and the low frequency is  $0.4 \sim 0.6$  Hz. The test uses the frequency of 2.5 Hz.

Moreover, the stress response amplitudes of soil at different depths under the subway loading are obtained by field monitoring. The maximum changing amplitudes of soil stress response are 0.23 kPa at 8.5 m, 0.70 kPa at 11.5 m and 1.15 kPa at 13.5 m. The change of soil stress amplitude is approximately linear with depth. Moreover, the stress amplitude in the rush hour in the morning is larger than that in the evening which is also larger than that at noon. The test adopts the maximum stress amplitudes as reference for the worst case in construction.

The field monitored data indicate that the soil dynamic response around the tunnel under the railway vibration loading is the cyclic response. Therefore, the test uses the stress-controlled, cyclic loading module. The scheme is shown in Table 1.

#### 4 DEVELOPING LAW OF PORE WATER PRESSURE

Through the test data of GDS, the curve of pore water pressure with increasing time is obtained, as shown in Figure 5.

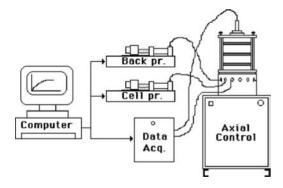


Figure 4. Working principle of GDS.

Table 1. Scheme of undrained dynamic cyclic triaxial test.

Under the different consolidated pressures, the pore water pressures increases quickly along the straight line under the original loading, then the curve bends. The increase becomes slow. After a long-time, the increase of pore water pressure becomes smooth. Finally, the pore water pressure reaches the limit value. The limit values of the three groups of pore water pressure are 115 kPa, 160 kPa and 185 kPa, respectively. They are 80% of their effective confining pressures.

From the above analysis, the increase of pore water pressure can be divided into three obvious stages: the sharply increasing stage, the slowly increasing stage and the smooth stage, as illustrated in Figure 5(c).

#### 1 Sharp increasing stage (AB)

After loading, the excess pore water pressure increases rapidly in a straight line, and in a short time (about 1300s, 3250 times of vibration), it reaches 50% of the limit value. Then the increasing speed attenuates rapidly with the increasing number of vibration and at the end of the stage, it reaches a stable value.

2 Slowly increasing stage (BC)

As a transition stage, the excess pore water pressure comes to a smooth stage from the sharply increasing stage. The pore water pressure still increases, but the increasing speed obviously becomes slow.

3 Smooth stage (CD)

In this stage, the pore water pressure hardly increases with the increase of the number of vibration. It reaches a stable value called the limit value, which is about 80% of its effective confining pressure.

When the characteristics of silty clay are combined with engineering conditions in Shanghai, Logistic model of pore water pressure is put forward (the correlation coefficient reaching over 0.99):

$$u = u_t - \frac{u_t - u_0}{1 + (N/N_0)^p}$$
(1)

where *N* is the number of vibration;  $u_0$  is the annual value of hydrostatic pressure; *u* is the limit value of excess pore water pressure;  $N_0$  and *p* are regression

Test number	Sampling depth(m)	Simulating depth(m)	Axial pressure (kPa)	Confining pressure (kPa)	Back pressure (kPa)	Dynamic load amplitude $\Delta \sigma$ (kPa)	Circular stress ratio CSR*	Frequency (Hz)
D1	$8.0 \sim 9.0$	8.5	155	130	80	0.23	0.001	2.5
D2	11.0 ~ 12.0	11.5	220	180	110	0.7	0.003	2.5
D3	13.0 ~ 14.0	13.5	250	200	130	1.15	0.006	2.5

\* The formula of circulars stress ratio:  $CSR = \frac{\Delta\sigma}{2\sigma'}$  (Tang, 2003), with  $\sigma' = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} = \frac{\sigma_1 + 2\sigma_3}{3} (\sigma_2 = \sigma_3)$ .

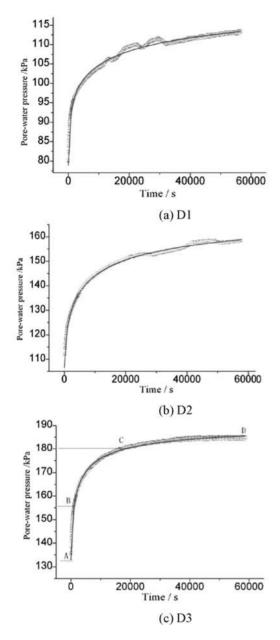


Figure 5. The increasing pore water pressure with time.

parameters, which are obtained by regression analysis for test data, as shown in Table 2.

#### 5 FITTING OF FIELD MONITORING DATA

The depth of 11.5 m is chosen as an example. The changing curve of the field monitored pore water

Table 2. Parameter table of the model.

Number	$u_0$	<i>u</i> <sub>t</sub>	$N_0$	р	Corresponding coefficient
D1	78.96	128.22	8125.4	0.43	0.9933
D2	106.44	173.00	6383.2	0.59	0.9979
D3	132.94	191.73	2130.0	0.65	0.9978

pressure under the subway loading is shown in Figure 6. y Axis (unit: kPa) is the response value of pore water pressure and x axis (unit: s) is time. According to the calibration coefficients, it is transformed into the curve of excess pore water pressure with the number of vibration, as shown in Figure 7.

Compared Figure 7 with Figure 5 (b), the increasing speed of the field monitored pore water pressure is slower than that of the test, and the increasing amplitude is also slower. The main cause is the size effect and the difference between the field test and the laboratory one. The coefficient c is used to amend the developing model of pore water pressure in Formula (1) and the amending model is obtained:

$$u = u_t - \frac{u_t - u_0}{1 + C \cdot (N/N_0)^n}$$
(2)

where C is the amending coefficient.

ž

Substituting the field monitored pore water pressures at 8.5 m, 11.5 m and 13.5 m and their number of vibration into Formula (2), we have the value of *c*, 0.0350 for D1, 0.2112 for D2 and 0.2975 for D3.

Similarly, the depth of 11.5 m is chosen as an example. The amending parameter is substituted in Formula (2) and the fitting curve is obtained. The fitting curve is compared with the monitored curve and the result is shown in Figure 8. From the figure, the value of excess pore water pressure in Formula (2) is close to the monitored value and they are fitting well.

The field monitored data indicate that the excess pore water pressure is not big when the subway train is running across. Then it dissipates quickly. The interval of the subway train is  $3 \sim 5$  minutes, when the excess pore water pressure can almost be dissipated. That is, basically, the excess pore water pressures produced by the two adjacent trains can not be superposed. Therefore, the developing law is at the initial stage of Formula (2) only, that is, the sharply increasing stage.

## 6 DEFORMATION OF THE SATURATED SILTY CLAY

The model can be used to predict the excess pore water pressure when the subway is running across and the deformation and the change of stress of soft clay around the tunnel are analyzed by the effective stress

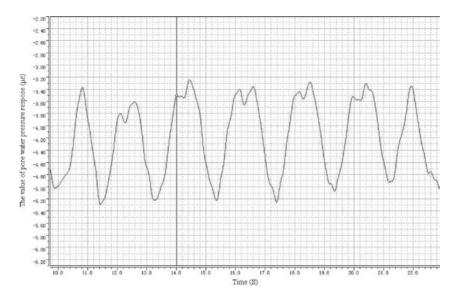


Figure 6. The oscillogram of field pore water pressure at 11.5 m.

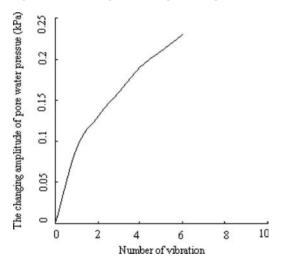


Figure 7. The curve of excess pore water pressure and number of vibration.

theory. They offer a theoretical reference to the subway operation.

The energy of the subway loading is delivered to the soil by the side wall of the tunnel and the lining. The sensitivity of pore water is more than that of the grains skeleton. At the start of the subway loading, pore water absorbs all the energy of the subway loading, which results in the rapid decline of the effective principal stress of the saturated silty clay and the elastic discharge of the texture cell of the soil. With the increasing number of vibration of the running way, the effective principal stress of the saturated silty clay

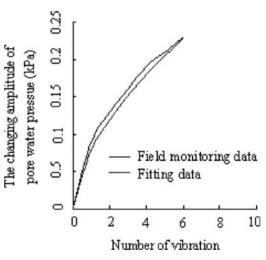


Figure 8. The compared curve of field monitoring data and fitting data.

reaches a steady value gradually and the texture cell of soil starts to bear the energy of the subway loading. The weak connection of the soil cell starts to become loose and breaks and slight cracks occur, but the soil cell is still integrated. With accumulating energy, the crack of the soil cell expands gradually and the big soil cell splits into small soil cell and crumb. The shear zone occurs in a place of serious breaking. The small soil cell and crumb in the shear zone are crushed and deformation occurs. The accumulation of microcosmic deformation results in the deformation of the axis of the subway and the ground settlement. The data monitored show that the axial settlement of the subway Line No. 1 exceeds 20 cm in some tunnel sections, which greatly affects the running of Line No. 1 and causes some old houses cracking. With the lapse of time, plastic deformation and large asymmetrical settlement occur on the bottom of the subgrade, which will affect the subway running and will possibly result in ground settlement, instability of the building foundation near the subway, cracking and inclining of the building, etc. Although no large deformation of soil occurs ate the side wall of the tunnel in a period of time, but a perceptible deformation occurs possibly at the side wall of tunnel with the passing of time of the subway working.

## 7 CONCLUSIONS

This paper uses field monitored data as a parameter for the GDS test. This guarantees the reality of test conditions. Logistic model of pore water pressure at different depths is put forward. The increase of pore water pressure can be divided into three obvious stages: the sharply increasing stage, the slowly increasing stage and the smooth stage. The model is compared with field monitored data, finding that the increasing speed of the field monitored pore water pressure is slower than that of the test and that the increasing amplitude is also slower. The main cause is the size effect and the difference between the field test and the laboratory one. Then, the parameter c is used to amend the developing model of pore water pressure and the amending model is obtained. It can be used to predict the increase of pore water pressure when the subway train is running across. The effective stress theory can be used to analyse the deformation and the change of stress of silty clay around the tunnel. The result offers a theoretical reference to the failure mechanism of the saturated silty clay, the axial deformation of the subway tunnel and the ground settlement.

#### ACKNOWLEDGEMENTS

This work are supported by the research grant (40372124) from National Natural Science Foundation of China, Shanghai Key Subject (Geotechnical Engineering) Foundation and Shanghai Leading Academic Discipline Project (Project Number: B308).

## REFERENCES

- Guan, Q.M., Zhou, S.H. & Wang, B.L. 2004. Variation of pore pressure and liquefaction of soil in metro. *Chinese Journal of Geotechnical Engineering* 26(2): 290–292 (in Chinese).
- Guo, Y., Luan, M.T., He, Y. & Xu, C.X. 2005. Effect of variation of principal stress orientation during cyclic loading on undrained dynamic behavior of saturated loose sands. *Chinese Journal of Geotechnical Engineering* 27(4): 403–409 (in Chinese).
- Lee, K.L. & Focht, J.A. 1975. Strength of clay subjected to cyclic loading. *Marine Geotechonolgy* 3(2):165–168.
- Li, L.Y., Cui, J., Jing, L.P. & Du, X.L. 2005. Study on liquefaction of saturated silty soil under cyclic loading. *Rock* and Soil Mechanics 26(10): 1663–1666 (in Chinese).
- Meng, Q.S. & Wang, R. et al. 2004. Pore water pressure mode of oozy silty clay under impact loading. *Rock and Soil Mechanics* 25(7): 1017–1022 (in Chinese).
- Pradhan, T.B.S., Tatuoka, F & Sato, Y. 1989. Stress dilatation of sand subjected to cyclic loading. *Soil and Foundation* 29(1):35–46.
- Shao, L.T., Hong, S. & Zheng, W.F. 2006. Experimental study on deformation of saturated sand under cyclic pore water pressure. *Chinese Journal of Geotechnical Engineering* 28(4): 428–431 (in Chinese).
- Zeng, C.N., Liu, H.L., Feng, T.G. & Gao, Y.F. 2005. Test study on pore water pressure mode of saturated silt. *Rock and Soil Mechanics* 26(12): 1963–1966 (in Chinese).
- Zeng, Q.J., Zhou, B., Gong, X.N. & Bai, N.F. 2001. Growth and dissipation of pore water pressure in saturated silty clay under impact load. *Chinese Journal of Rock Mechanics and Engineering* 20(1): 1137–1141 (in Chinese).
- Zhou, H.L. & Wang, X.H. 2002. Study on the pore water pressure of saturated sand in dynamic triaxial test. *Journal* of the China railway Society 24(6): 93–98 (in Chinese).

## Analytical solution of longitudinal behaviour of tunnel lining

## F.J.M. Hoefsloot

Fugro Ingenieursbureau, Leidschendam, The Netherlands

ABSTRACT: Staged construction of segmented tunnels result in permanent and constant bending moment in the longitudinal direction. This fact has been confirmed empirically, and analytical solutions for the longitudinal behaviour of a bored tunnel lining have been presented. This paper summarizes published analytical solutions for simple loading conditions, and includes corrections for the solutions where necessary. Solutions for additional loading conditions relevant to TBM tunnel construction are also presented. The analytical solutions have been built in a powerful Excel spreadsheet for rapid analyses. The results have been validated with staged construction FEM calculations in PLAXIS 2D. For final validation of the staged construction behaviour, tunnelling data from the Groene Hart tunnel in the Netherlands were analysed. Segments were instrumented with axial strain gauges, and the results have been analysed and converted to longitudinal bending moments. It has been demonstrated that the measured behaviour is reproducible with the analytical model, although selection of input parameters is complex. The presented model is well suited for quick analyses of TBM back-up train lay out, grouting conditions and moments from jacking forces with respect to longitudinal lining behaviour.

#### 1 INTRODUCTION

TBM tunnelling is characterised by phased construction of the segmented lining. Excavation by the tunnel boring machine is followed by erection of a single ring of lining segments. The complete structure can be regarded as a beam on elastic foundation according to classic structural engineering theory. In each phase of construction, a load free member is introduced, which contributes to the structural system. In the meanwhile, loading progresses simultaneously with the TBM. Examples are jacking forces on the first ring, buoyancy forces within the grouting zone and self weight of the back-up train.

Previous theoretical analyses of the structural system have shown that the distribution of bending moment and shear forces is fundamentally different from a wished-in-place beam on an elastic foundation. Measurements from the Groene Hart tunnel clearly confirm this difference.

## 2 ANALYTICAL SOLUTIONS

#### 2.1 General procedure

The progressive loading conditions are:

- 1 uniform loading
- 2 shear force at front end of the beam
- 3 bending moment at front end of the beam
- 4 local uniformly distributed load

Bogaards & Bakker (1999) have given analytical solutions for the section forces of a beam on an elastic foundation with progressive extension of the beam under a uniform load. Bakker (2000) published solutions for items 1 to 3.

Section forces can be derived in several ways. Bakker's (2000) derivation is explained and the corrected results are given below.

#### 2.2 Uniform load

A staged extension of a uniformly loaded beam can be solved as the sum of the analytical solutions of a partially loaded beam (Figure 1, left side). The mechanical

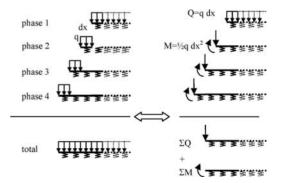


Figure 1. Staged construction uniform load.

scheme is equivalent to the scheme given on the right side of Figure 1.

The section forces  $M_q(x)$  and  $D_q(x)$  (Equation 1 to 4) can easily be summed from the analytical solution of a beam loaded with a shear force and the bending moment at the beginning of the beam. These simple solutions have been given by Hetényi (1946), Bouma (1993) and Young et al. (2002).

$$M_q(x) = \frac{-q\,dx}{\beta} \sum_{n=0}^{n=x/dx} e^{-\beta \,ndx} \cdot \sin(\beta \,n\,dx) + \dots$$

...+ 
$$\frac{1}{2}\sqrt{2} q dx^2 \sum_{n=0}^{n=x/dx} e^{-\beta n dx} \cdot \sin(\beta n dx + \frac{\pi}{4})$$
 for  $x > 0$  (2)

$$D_q(x) = 0 \quad \text{for} \quad x = 0 \tag{3}$$

$$D_q(x) = \frac{-q}{\beta} e^{-\beta x} \cdot \sin(\beta x) + \dots$$

$$\dots + \frac{1}{2} q \, dx \sqrt{2} \cdot e^{-\beta x} \cdot \sin(\beta x + \frac{\pi}{4}) \quad \text{for } x > 0 \tag{4}$$

$$\beta = \left(\frac{k}{4 EI}\right)^{0.25} \tag{5}$$

with x = coordinate starting on the left side; dx = length of beam increment; k = modulus of subgrade reaction; E = Young's modulus; I = moment of inertia.

#### 2.3 Shear force at front end of the beam

Figure 2 shows the mechanical scheme of a shear force at the front end of a stage-constructed beam. As in Figure 1, the analytical solution of a staged (beam) extension with shear force at the front end can be found as the sum of the analytical solution of a partially loaded beam as explained on the left side of Figure 2. The mechanical scheme is equivalent to the scheme given on the right side of Figure 2. The resulting section forces  $M_Q(x)$  and  $D_Q(x)$  (Equation 6 to 8) have been derived from the right side of Figure 2.

$$M_{Q}(x) = -Q \, d \, x \sqrt{2} \, \sum_{n=1}^{n - x/dx} e^{-\beta \, n \, dx} \cdot \sin(\beta \, n \, dx + \frac{\pi}{4})$$
  
for  $x > 0$ 

$$D_Q(x) = -\sqrt{2} Q e^{-\beta x} \cdot \sin(\beta x + \frac{\pi}{4})$$
(8)

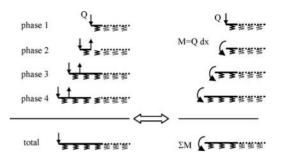


Figure 2. Staged construction shear force.

#### 2.4 Bending moment at front end of the beam

Progressive extension of the bending moment at the front end of the beam results in the simple solutions  $M_M(x)$  and  $D_M(x)$  in Equations 9 and 10.

$$M \underbrace{ \underbrace{ } \\ M_{M}(x) = -M \\ D_{M}(x) = 0 \end{aligned}$$
(9) (10)

#### 2.5 Local uniformly distributed load

The analytical solution for a progressive local uniform load can be derived as shown in the mechanical scheme in Figure 3. The resulting section forces are complex and are not presented here.

#### 3 SPREADSHEET-MODEL

The mechanical scheme for the design of a tunnel lining as a beam on an elastic foundation is given in Figure 4. An unsupported part of tunnel lining is present within the TBM  $(l_i)$  and also between the TBM and stable grout  $(l_u)$ . The following loads are present:

- Bending moment from jack forces (M<sub>jack</sub>)
- Shear force from jacking forces (D<sub>jack</sub>)
- Shear force from steel brushes (D<sub>br</sub>)
- Weight of lining segments (q<sub>w</sub>)
- Uniformly distributed load (q) starting behind the TBM at distance (l) with a length (l<sub>q</sub>).

An Excel-spreadsheet model was built for this mechanical scheme with the basic solutions given in paragraph 2. The model is equipped with 6 local, uniformly distributed loads to account for buoyancy forces and load configurations of the back-up train, backfill and permanent structure within the tunnel. Therefore the model is a powerful tool to calculate section forces for a wide variety of load conditions simply by entering the necessary parameters.

(7)

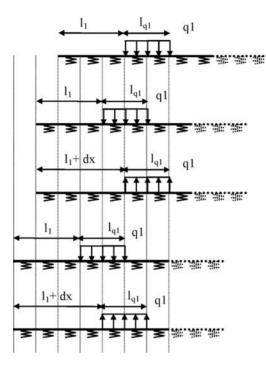


Figure 3. Staged construction local uniform load.

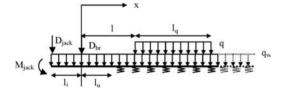


Figure 4. Load scheme and subgrade reaction tunnel beam.

Determining the angular and vertical deformation of the tunnel is not straight-forward. According to classical structural engineering theory, the angular and vertical deformation can be found by integration of the bending moment.

$$\frac{d}{dx}\phi(x) = \frac{-M(x)}{EI} \tag{11}$$

$$\frac{d}{dx}w(x) = \varphi(x) \tag{12}$$

The solution of a stage-constructed tunnel results in a constant bending moment at great distance from the TBM. Direct integration of this constant value results in a linear increase of angular deformation, and a quadratic increase of vertical displacement. Indeed, finite element calculations of a stage-constructed tunnel with strain-less extension of the beam show this type of deformation. Since the final tunnel must be

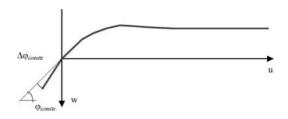


Figure 5. Angular offset installation new ring.

positioned according to its design alignment, tunnel rings must be installed with an inclination onto the previous ring. Figure 5 shows the basic idea of ring installation.

The spreadsheet-model is equipped with the calculation of angular and vertical deformation based on this principle. Part of the result is the required installation offset angle relative to the vertical and relative to the previous ring.

The model has been verified using finite element modeling with PLAXIS 2D version 8.4. For each of the basic loading conditions given in paragraph 2.1, a verification has been performed. In PLAXIS, a structural beam on an elastic soil mass was modeled under the applicable loading conditions. Phased calculations have been performed with each phase extending the beam and soil support, and moving the load one step ahead. Results form the analytical spreadsheet-model and finite element model show good to excellent agreement.

# 4 GROENE HART TUNNEL

For the High Speed Railway link between Amsterdam and Brussels, a 7.2 km long TBM tunnel was recently constructed. The tunnel's outer diameter is 14.5 m and the lining thickness is 0.6 m. The Dutch research committee COB (Center for Underground Construction) organized an extensive measuring campaign to study the longitudinal behavior of the lining. Strain gauges, tilt sensors and two systems to measure vertical deformation were placed on some of the tunnel rings.

#### 4.1 Strain gauges

Strain gauges were used to measure axial and tangential strain during construction. Pairs of gauges were placed in axial direction, both on the inside and on the outside of the segments.

After 4 days, the signals reach general equilibrium, with minor fluctuations due to jacking forces. Complete results from axial gauges for ring number 2117 are shown in Figure 6. The average of the strain measurements from the inside and the outside of the segment are given in Figure 7.

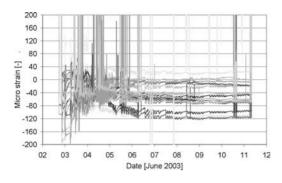


Figure 6. Results of all strain gauges ring 2117.

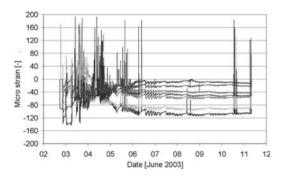


Figure 7. Average results strain gauges ring 2117.

In Figure 8 typical results of the strain distribution have been plotted against the vertical position, shortly after having passed through the TBM. The same has been done at the end of the measuring campaign (Figure 9). The slope of the trend line through these measurements changes of sign between the beginning and end of the measurement program. This axial gradient is determined by the bending moment in the cross section.

#### 4.2 Bending moment and normal force

Using the axial strain at tunnel axis and the gradient of the trend line, the average normal force and bending moment can be determined with Equations 13 and 14.

$$N = \sigma_{av} A = EA \cdot \varepsilon_{av} \tag{13}$$

$$M = EI \cdot \kappa \tag{14}$$

with N = average normal force;  $\sigma_{av}$  = average axial stress;  $\varepsilon_{av}$  = average axial strain;  $\kappa$  = axial strain gradient; E = Young's modulus = 38500 MPa; A = section area = 26.2 m<sup>2</sup>; I = moment of inertia = 634 m<sup>4</sup>.

In Figure 10, the derived axial normal force and bending moment from strain gauge measurements for

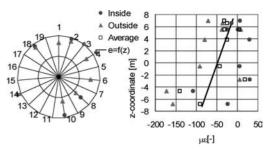


Figure 8. Axial strain ring 2117, just after having passed through the TBM, June 3 2003, 1:00 hrs.

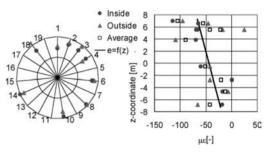


Figure 9. Axial strain ring 2117, at the end of measuring campaign, June 11 2003, 0:00 hrs.

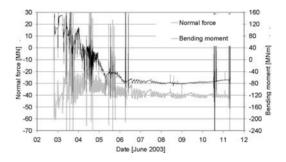


Figure 10. Normal force and bending moment from axial strain measurement ring 2117.

the entire logging period are shown. Results of normal force and bending moment at time of leaving the TBM have been compared with TBM data of jacking forces. A reduction factor has been included in the results of Figure 10. When a reduction factor of 0.9 is applied to the axial strain, the back-calculated jack forces match the directly measured jack forces. The reduction factor takes into account the influence of non-uniform strain distribution within the segments.

Also the section forces as functions of distance from the tail end of the TBM are presented in Figure 11. These lines can only be regarded as normal force and

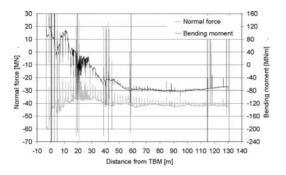


Figure 11. Normal force and bending moment ring 2117.

Table 1. Input parameter.

Description	Identification Fig. 4	Value unit
Outside diameter		14.50 m
Wall thickness		0.60 m
Young's modulus		3.85E+07 kN/m2
Reduction factor stiffness		0.650
Modulus of subgrade reaction		$367,000  \text{kN/m}^2$
Length segments in tunnel	li	6 m
Unsupported length	l <sub>u</sub>	2 m
Moment jack forces	M <sub>jack</sub>	79,000 kNm
Shear force jack	Diack	0 kN
Shear force brushes	D <sub>br</sub>	0 kN

bending moment distribution along the tunnel axis during constant tunnelling. From Figure 11 it is evident that the bending moment reaches a constant value at approximately 60 m behind the TBM. This remaining bending moment is a consequence of the staged construction of tunnelling.

#### 4.3 Back analyses

The analytical model has been applied to calculate the distribution of the bending moment within the lining behind the TBM. The input parameters have been given in Tables 1 and 2. They consist of simple geometric and material parameters and loading conditions. Two parameters need further explanation. First, a reduction factor for the bending stiffness has been applied to account for the structural behaviour of joints between segments. Secondly, the external grout load requires further analyses. Refer to the paper on this subject by Talmon et al. (2008).

The calculated and measured bending moments are in Figure 12. There is good agreement between measurements and calculation results. It is noted that

Table 2. Input parameter; uniformly distributed load.

Load identification	Load	Position behind TBM
Fig. 4	[kN/m]	[m]
qw           q1           q2           q3           q4           q5	629 -2064 437.5 70 300 187	$\begin{array}{c} -6 \text{ to } 1000 \\ 0 \text{ to } 1000 \\ 2 \text{ to } 26 \\ 30 \text{ to } 1000 \\ 52 \text{ to } 1000 \\ 82 \text{ to } 108 \end{array}$

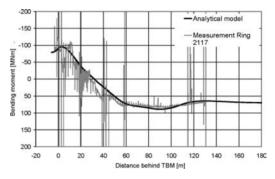


Figure 12. Analytical model and measurement results ring 2117.

alternate combinations of input parameters may also result in fair agreement. However, the analytical model clearly shows the sensitivity of different loading conditions. Therefore the model is well suited to analysing loading conditions like position of back-up train, backfill, jacking forces and grouting conditions.

#### 5 CONCLUSIONS

The mechanical behaviour of a stage-constructed TBM tunnel is fundamentally different from a wishedin-place tunnel. Analytical solutions for bending moment, shear force, angular deflection and vertical displacement have been used to create a spreadsheetmodel. Measurements from the Groene Hart Tunnel clearly show a residual bending moment far behind the TBM, a result which is in good agreement with results of the analytical model. The longitudinal behaviour of the tunnel as a consequence of the staged construction needs to be accounted for in tunnel design.

#### ACKNOWLEDGEMENT

The work described in this paper has been performed for the research committee F512 of the Centre for Underground Construction (COB). The author wishes to thank the committee for the opportunity to publish this work.

#### REFERENCES

- Bakker, K.J. 2000. Soil Retaining Structures. Rotterdam: Balkema.
- Bogaards, P.J. & Bakker, K.J. 1999. Longitudinal bending moments in the tube of a bored tunnel. *Numerical Models* in Geomechanics Proc. NUMOG VII: 317–321.
- Bouma, A.L. 1993. *Mechanica van constructies*. Delft: Delftse Uitgevers Maatschappij (in Dutch).
- Hetényi, M. 1946. *Beams on Elastic Foundations*. Michigan: The University of Michigan Press.
- Talmon, A.M., Bezuijen, A. & Hoefsloot, F.J.M. 2008. Longitudinal tube bending due to grout pressures. Shanghai: TC28.
- Young, W.C., Budynas, R.G. & Roarke, R.J. 2002. Roarke's Formulas for Stress and Strain (seventh ed.). New York: McGraw Hill.

# Design of tunnel supporting system using geostatistical methods

#### S. Jeon & C. Hong

School of Civil, Urban & Geosystem Engineering, Seoul National University, Seoul, Korea

# K. You

Department of Civil Engineering, University of Suwon, Gyungki-do, Korea

ABSTRACT: Rock mass classification provides a guideline for a tunnel excavation and reinforcement design. The borehole data and geophysical site investigation results have been popularly used for rock mass classification, but the locality and limited information from the borehole data and qualitative characteristics of geophysical data have been problematic. A geostatistical method such as kriging can be an alternative to solve these problems. This paper describes a design of tunnel supporting system based on geostatistical tools. Korean tunnel supporting system is typically composed of six different types of combination of shotcrete, rockbolts, and concrete lining based on rock mass rating (RMR). Ordinary kriging (OK), indicated kriging (IK), and sequential indicator simulation (SIS) were used to estimate RMR around the tunnel. Kriging methods could estimate RMR with the best linear unbiased estimator. Using SIS, RMR was presented in the probabilistic term such as mean, variation, and confidence interval. Reliability of the estimated values was verified by split-sample validation and compared with the real RMR obtained from the side wall of the tunnel while excavating carried out. Based on 100 equally probable simulations, RMR could be presented in the form of a probability distribution function and the uncertainty of estimation could be successfully quantified.

# 1 INTRODUCTION

Tunnel excavation and reinforcement design are made according to the rock mass classification. Engineers have been using the borehole data of rock mass classification and geophysical site investigation results. Due to the locality and limited information from the borehole data and qualitative characteristics of geophysical data, geostatistical method such as kriging should be considered for the rock mass classification.

Kriging is one of the most widely used interpolation methods in geostatistics. There has been considerable researches conducted using this technique (Taboada et al., 1997; Facchinelli et al., 2001; Marinoni, 2003; Pardo-Igúzquiza and Dowd, 2005). Despite its wide use, the kriging map flattens out the local details of the spatial variation with the overestimation of small values and underestimation of large values. This type of selective bias is a serious shortcoming because of the loss of the distribution features of the original data.

Kriging is focused on the estimation of unknown points by one deterministic value, whereas sequential simulation is on the stochastic simulation by probabilistic form. Juang et al. (2003) showed the spatial distribution of soil contamination by the sequential indicator simulation, and Feng et al. (2006) proposed an improved sequential indicator simulation. This paper describes a design of tunnel supporting system based on geostatistical tools. Typical Korean tunnel supporting system was composed of six types of combination of shotcrete, rockbolts, and concrete lining based on the rock mass rating (RMR). Ordinary kriging (OK), indicated kriging (IK), and sequential indicator simulation (SIS) were used to estimate RMR around the tunnel. For IK, we estimated the threedimensional distribution of RMR with the field data of borehole logging and geophysical data. And this result was compared with the results using OK and SIS. Using SIS, an equally probable simulation was performed 100 times to quantify the uncertainty of estimation. The accuracy of estimation was checked by split-sample validation.

#### 2 ESTIMATION PROCESS

#### 2.1 Ordinary kriging

1 Construct a variogram from the scatter point set to be interpolated.

$$\gamma(h) = \frac{1}{2n} \sum_{i=1}^{n} [z(x_i) - z(x_i + h)]^2$$
(1)

where h = lag distance; z(x) = value of position x; and n = number of total data.

- 2 Define a theoretical variogram. Spherical model was used in this study.
- 3 Calculate the weights for each point and estimation value is the linear combination of weighted known values.

$$z_0^* = \sum_{i=1}^n \lambda_i z_i \tag{2}$$

with a constraint

$$1 - \sum_{i=1}^{n} \lambda_{i} = 0$$

$$\begin{pmatrix} \sigma_{11}^{2} & \sigma_{12}^{2} & \cdots & \sigma_{1n}^{2} & -1 \\ \sigma_{21}^{2} & \sigma_{22}^{2} & \cdots & \sigma_{2n}^{2} & -1 \\ \end{pmatrix} \begin{pmatrix} \lambda_{1} \\ \lambda_{2} \\ \sigma_{02}^{2} \end{pmatrix} \begin{pmatrix} \sigma_{01}^{2} \\ \sigma_{02}^{2} \\ \sigma_{02}^{2} \end{pmatrix}$$
(3)

where  $\sigma_{ab}^2$  is the covariance between a and b.

#### 2.2 Indicator kriging

Determine thresholds of borehole data and seismic data. Because borehole data is quantitative and seismic data is qualitative regarding to RMR, both borehole and seismic data changed into the indicators between 0 and 1. The thresholds were RMR 20, RMR 40, RMR 60, and RMR 80 for borehole data, and 800 km/s, 1500 km/s, 2400 km/s, and 3600 km/s for seismic velocity.

$$I(v_t, x) = \begin{cases} 1, & \text{if } V(x) \le v_t \\ 0, & \text{if } V(x) > v_t \end{cases}$$
(5)

where  $v_t =$  indicator value; V(x) = data function.

- 2 Calculate an indicator of unknown nodes with the same process of ordinary kriging.
- 3 Convert an indicator into RMR using cumulative probability distribution function of estimated four indicators for four thresholds.

#### 2.3 Sequential indicator simulation

- 1 Determine the thresholds (1st quartile, medium, and 3rd quartile) and divide the data into indicators. The indicator function is given by equation (5).
- 2 Calculate the experimental variogram and determine the theoretical variogram for each threshold.
- 3 Select an unsampled node using a random path and calculate the indicator values at the selected node by ordinary kriging.
- 4 Calculate the CDF (Cumulative probability Distribution Function) using three thresholds and sampling from the CDF.
- 5 Include the calculated value as conditioning points.
- 6 Go back to random path selection until all unknown nodes are calculated.

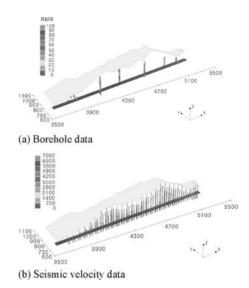


Figure 1. Borehole and seismic velocity data in the research area.

### 3 EXAMPLES AND RESULTS

(4)

#### 3.1 Estimation of three-dimensional RMR

Estimation of three-dimensional RMR distribution was performed in the highway project from 'Sosa' to 'Noksan'. It was 1700 m in length from STA 3k600 to STA 5k300, with a depth from -40 to 200 m. The grid was 86 zones in length (x-direction), 16 zones in width (y-direction) and 25 zones in height (z-direction). The dimensions of one element were 20 m in length, 20 m in width and 10 m in height. RMR estimation was performed by borehole logging data and seismic data. Borehole location and seismic data were presented in Figure 1(a) and Figure 1(b), respectively.

The results of ordinary kriging, indicator kriging, and SIS are shown in Figure 2. Ordinary kriging used a borehole logging data as input data, and indicator kriging used both borehole logging data and seismic survey data. Figure 2(c) shows the first result of 100 SIS results. The RMR distribution around the planned tunnel is presented in Figure 2(d). Korean tunnel supporting system was composed of six types based on the RMR. Five grades of RMR are matched with the five types of support system and sixth support system is for the portals of a tunnel. Therefore, the most important issue in the design of tunnel support system can be a determination of reliable RMR values.

#### 3.2 Reliability analysis of estimated RMR

Split-sample validation was performed to verify the accuracy of the GA (Genetic Algorithm) simulation. A subset that was composed of 100 data points from

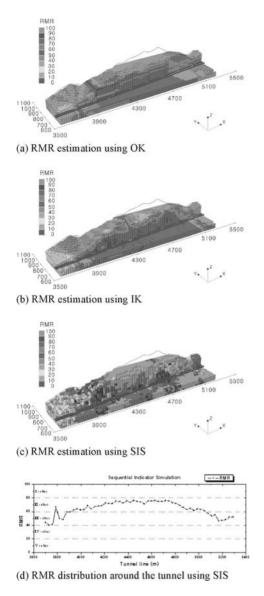


Figure 2. Estimation results and RMR distribution abound the tunnel.

the original borehole-logging data was set aside as test data; the reminder was training data. And the results of split-sample validation are presented in Figure 3. The perfectly estimated result is presented as a straight line inclined at  $45^{\circ}$ . The result of split-sample validation shows dots located around the 'perfect estimate' line. The dots in the upper and lower parts of the line are approximately random, and their numbers are almost identical.

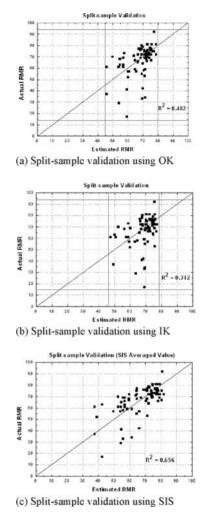


Figure 3. The results of split-sample validation.

The coefficient of variation was 0.482 and 0.342 for ordinary and indicator kriging, respectively. These values are very sensitive to local area. With the same borehole input data, SIS was performed to 100 equally probable times. Through these analyses of results, RMR could be presented in the form of a probability distribution function, and the uncertainty of estimation could be successfully quantified.

As shown in Figure 3(a) and Figure 3(b), the original input RMR ranged from 15 to 95, whereas the output RMR ranged from 45 to 80 by both ordinary and indicator kriging. Distribution features of original geological data were disappeared by kriging in the process of minimizing the error variation, and this phenomenon is called as 'smoothing effect'. The coefficient of variation for averaged SIS result was 0.656 as shown in Figure 3(c).

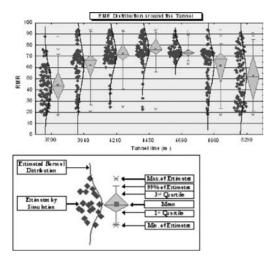


Figure 4. Box chart and probability distribution function of RMR distribution in the planned tunnel area using SIS.

Table 1. Variation of RMR in the planned tunnel area.

Station	Mean	Standard deviation		
3k700	44	16.3		
3k940	62	18.7		
4k210	72	13.4		
4k450	76	12.5		
4k690	75	6.5		
4k960	62	17.6		
5k300	52	21.1		

In order to investigate the reliability of the estimation, an equally probable simulation was performed 100 times in order to quantify the estimation uncertainty. The RMR distribution around the planned tunnel is presented in Figure 4. The left dots and distribution curve represent the RMR realizations, and the right box chart presents their normal distribution. In the figure, X, the vertical line, the diamond shape box chart, and the center dot represent the maximum and minimum values of 1% and 99% respectively, the box range from 25% and 75% of the CDF, and the mean value, respectively. The mean and standard deviation of the 100 simulations are presented in Table 1. As the reliability of the estimation increases, the variance decreases.

#### 4 CONCLUSIONS

The objective of this study was to estimate reliable RMR values and correctly design a tunnel support system. The results may be summarized as follows:

- Kriging and sequential indicator simulation have its special characteristics and sequential indicator simulation could estimate RMR effectively.
- 2 Estimation values could be shown in the form of a probability using the 100 stochastic simulations that were simulated on the condition of equi-probability. The estimation uncertainty could be quantified by a variance of RMR.
- 3 Reliability analysis was performed by split-sample validation. The differences between true values and estimation values could check the precision of the estimation.

#### REFERENCES

- Facchinelli, A. Sacchi, E. & Mallen, L. 2001. Multivariate statistical and GIS-based approach to identify heavy metal sources in soils. *Environmental Pollution* 114(3): 313–324.
- Feng, Y. Tang, S. & Li, Z. 2006. Application of improved sequential indicator simulation to spatial distribution of forest type. *Forest Ecology and Management* 222: 391–398.
- Juang, K. Chen, Y. & Lee, D. 2003. Using sequential indicator simulation to assess the uncertainty of delineating heavymetal contaminated soils. *Environmental Pollution* 127: 229–238.
- Marinoni, O. 2003. Improving geological models using a combined ordinary–indicator kriging approach. *Engineer*ing Geology 69(1–2): 37–45.
- Pardo-Igúzquiza, E. & Dowd, P.A. 2005. Multiple indicator cokriging with application to optimal sampling for environmental monitoring. *Computers & Geosciences* 31(1): 1–13.
- Taboada, J. Vaamonde, A. Saavedra, A. & Alejano, L. 1997. Application of geostatistical techniques to exploitation planning in slate quarries. *Engineering Geology* 47(3): 269–277.

# Comparative study of software tools on the effects of surface loads on tunnels

# D.K. Koungelis & C.E. Augarde

School of Engineering, Durham University, Durham, UK

ABSTRACT: In this paper results are present from parametric studies of twin-tunnelling schemes carried out using various finite element packages. The aim is not to make comparisons with field data but to show the differences obtained using different FE models, as might be used in a design office. The effects of varying surface loading on the tunnels themselves are investigated and the changing effects are studied as tunnel layout is altered.

# 1 INTRODUCTION

The ongoing development in the world's urban areas inevitably leads to the construction of structures in close proximity to already driven tunnels. Care should be taken to ensure that construction is carried out without damaging the tunnels or any other adjacent or overlying infrastructure. Considerable research has been undertaken for the case of a single tunnel where empirical methods for predicting tunnel induced deformations are applicable. For more complex geometries, however, empirical methods fail to make accurate predictions since they do not account for the soil-tunnelstructure interaction mechanism. The finite element (FE) method appears to be a solution to this prediction problem, however many difficulties in its use remain. The aim of this paper is to validate tools for numerical modelling of tunnelling related interactions in soft ground.

Plenty of publications exist which study the interaction mechanism of soil, lining and a pre-existing structure (i.e non-greenfield site) response during tunnelling operations. However the literature on the effect of surface loading on an existing tunnel is sparse to the authors' knowledge. Most refer to the case of surface loading above pipes, or pile construction and pile loading, and their effects on tunnels. The main reason preventing engineers from dealing with the subject of surface loading is the difficulty they face in accurately measuring the change of stresses acting on the lining due to the applied load. The lack of field data results in performing merely theoretical analysis.

Peck (1969) states three conditions for successful tunnelling. The first refers to safe operation of tunnelling works. The second requirement is the protection of adjacent structures. The final condition refers to the tunnel's ability to withstand all external

loads which act upon it during its service life. These loads and their influence on tunnel lining will be considered here. According to O'Rourke (1984) linings do not carry the total overburden weight of the overlying ground. The vertical  $(\sigma_v)$  and horizontal  $(\sigma_h)$ stresses instead, are re-distributed around the face due to mobilisation of the soil shear strength and continuity. This effect is often termed "arching". The tunnel therefore has to withstand only the stresses which are not "arched". Mair and Taylor (1997) presented field data from 12 different tunnel cases driven in London clay. The lining load is expressed as a percentage of full overburden weight at tunnel centre line (CL). The data collection refers to points at one week and one year after lining installation. These indicate that the measured lining load even after a year is below 70% of full overburden. In most cases it varies between 40% and 60%.

Moore (1987) described a semi-analytical solution that makes use of the Boussinesq method and other closed form solutions to estimate the deformation of a buried pipe (rather than a tunnel) in an infinite elastic medium due to surface loading. 2-D FE analysis was also employed. Provided that realistic elastic ground properties are selected the semi-analytical method compares well with the numerical results. This procedure can be used for estimations of hoop forces, bending moments and ring deformations. Clearly there are problems however with the assumption of elastic ground.

This paper investigates the effect of surface loading on pre-existing tunnels in soft ground assuming plane strain conditions (2-D FE analysis). The following two commercial FE packages are used for this purpose, Strand7 and Plaxis. The purpose of this comparison is to try and identify the differences in the FE predictions by using various codes, which might be significant to industrial users of these programs. The reasons for not using any analytical method (e.g. Boussinesq method) to estimate the tunnel deformations due to surface loading were that these methods are only applicable to elastic medium. They do not take into account the properties of the medium or the interaction between the medium and any pre-existing structure in it. According to Moore (1987) the Boussinesq method fails to take into account the effect of shear stresses and strains developing in the overlying strata.

# 2 ANALYSIS GEOMETRY

In the current plane strain analyses three different tunnel geometric configurations are considered. In the first case a single tunnel analysis is carried out (ST case). In the second a twin tunnel analysis is carried out, where both tunnels are horizontally aligned (TH case). Finally in the third case twin tunnels are vertically and diagonally aligned (TVD case). A parametric study was performed for the above three cases varying the position of the tunnel axis  $(z_0)$ , pillar width (P), pillar depth ( $P_D$ ) as well as the position of the surface loaded area (W). Figure 1 shows the parameters varied in this study. For the ST case (shown as a solid circle) zo varies. For the twin tunnel configuration (where the second tunnel is presented as a dotted circle) P and P<sub>D</sub> vary. In all cases the loaded area shifts from W1 to W6. Throughout this parametric study the dimensions of the domain (x, y), tunnel diameter (D), magnitude (400 kN/m) and area (W) of the applied load were constant. Surface load was applied directly to the surface of the finite elements hence modelling a flexible footing. No interface elements were used to model the existence of any type of foundations or treatment of the ground prior to its loading.

Tunnel diameter (D) was chosen to be 4 m which is comparable to the diameter of running tunnels for the Underground in London (Attewell, 1978). The dimensions of the modelled domain were chosen to be 70 m long (or 17.5D) in the x direction and 50 m deep (12.5D) in the y direction. For  $z_0 = 15$  m and 20 m the chosen values lie within limits proposed by Potts et al. (2002). They suggested that for tunnels in clay the depth of the mesh should be approximately 2Dto 3D below tunnel invert. As for the optimal width of the domain two factors have to be considered. The mesh has to be sufficiently wide to ensure minimal displacements along the vertical boundaries. However, the larger the domain the larger the number of the degrees of freedom (d.o.f). This immediately affects the solution in terms of computational time. Hence a compromise has to be made between these two crucial factors. It was decided that the above dimensions were appropriate for this study.

The surface load is constant at 400 kN/m. This magnitude was chosen to resemble the uniform stress from

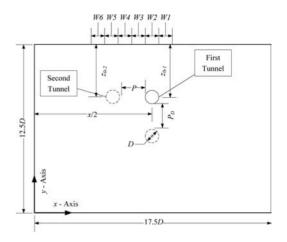


Figure 1. Geometric parameters of the domain.

a 10-storey building, assuming a stress of  $10 \text{ kN/m}^2$  per storey for a 4 m wide loaded area. The latter value over a full building width is probably unrealistic. However, it was chosen as a worst case value (perhaps including the effect of an accidental concentrated load) to accentuate the differences in the parametric study. The value of  $10 \text{ kN/m}^2$  per storey was chosen after BS 8002 (British Standards Institution, 1994) recommendations.

The mesh in Strand7 is created and imported from Gmsh (a freeware FE mesh generator) since Strand7 can neither produce an unstructured mesh nor can it be as flexible as Gmsh in the pre-processing stage of the analyses. Six-noded triangular elements are used to model the soil, and two-noded beam elements are used to model the tunnel lining. In Plaxis the autogenerated mesh consists of fifteen-noded triangular elements. Curved beam elements are used to model the tunnel lining. The reason for using different type of elements (fifteen-noded triangles rather than six-noded triangles) compared to Strand7 is to achieve greater accuracy. In Plaxis the user cannot import a mesh from another software package. Thus it was entirely created in Plaxis's pre-processing stage. As a result differences in the refinement are evident (Fig. 2).

The realistic determination of the initial stress conditions is of great importance in FE modelling in geotechnics. Several approaches exist for this purpose. The most common of which is the  $K_0$  procedure where stresses prior to any construction are initialised. This method is only applicable for horizontal ground surfaces and greenfield sites. This is not the case in this study. Consequently a different approach is adopted to simulate initial ground conditions. Tunnel excavation is not modelled. Instead tunnels with their permanent lining appear in the mesh as if wished in place. Then, gravity loading is uniformly applied to the whole domain (*gravity loading* method). The resulting

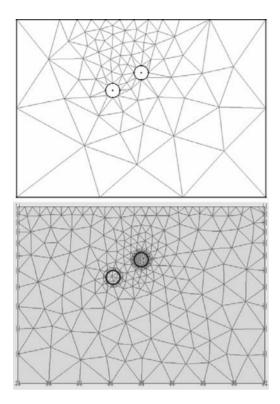


Figure 2. Generated mesh for the *TVD* case when  $z_{o,1} = 15$  m for the upper tunnel,  $z_{o,2} = 20$  m for the lower tunnel and P = 1D. The upper figure refers to the Strand7 mesh while the lower to the Plaxis mesh.

displacements are then set as the zero datum for the subsequent steps of the analysis. The stratigraphy is the same throughout the analysis. This consists of one clay layer, the characteristics of which are presented in Table 1. Undrained analyses are performed throughout using effective stiffness parameters. As for the boundary conditions, in plane strain analysis no horizontal or vertical movements are permitted along the horizontal boundary at the base of the mesh. On the two vertical mesh boundaries, only vertical movements are allowed. The top mesh boundary is free to move.

The beam elements used to model the lining are assumed to behave in a simple linear elastic way. Thus two parameters (Young's modulus E and Poisson's ratio v) are required for this model. Table 2 shows the full characteristics of the lining, including the geometrical properties. Soil does not behave in a linear nor an elastic way. Thus a more realistic and advanced constitutive model should be adopted. A simple elasto-plastic constitutive model is therefore used. For the plastic region Mohr-Coulomb yield criterion with associated flow is used in Strand7 while non-associated flow is used in Plaxis. A valid criticism here is that this elasto-plastic model is still too Table 1. Material properties of the soil.

Туре	of behaviour:	Elastic Region, Linear Elastic
Туре	of behaviour:	Yield Surface, Mohr-Coulomb

Parameters	Name	Value	Unit
Young's modulus	Е	$6.207 \times 10^{3}$	kPa
Poisson's ratio	v	0.33	_
Unit weight	γ	20	kN/m <sup>3</sup>
Cohesion	c	5	kPa
Angle of friction	$\varphi$	25°	-

Table 2. Material properties of the tunnel lining.

Parameters	Name	Value	Unit
Young's modulus Cross Sectional Area Second Moment of Area Unit weight Poisson's ratio	$\begin{array}{c} E\\ A\\ I\\ \gamma_s\\ v_s \end{array}$	$ \begin{array}{c} 10^8 \\ 0.168 \\ 3.95136 \times 10^{-4} \\ 24 \\ 0.3 \end{array} $	kPa m <sup>2</sup> /m m <sup>4</sup> /m kN/m <sup>3</sup>

crude for accurate analysis of this problem, which is true. However the use of this model matches much of the routine analysis carried out for tunnelling problems in design offices. The purpose of the paper is not to provide a link to field data but to demonstrate the different predictions which follow from the use of different FE models.

In all calculations carried out the analysis procedure began with the tunnels driven and the permanent lining installed. Displacements from this stage are not measured. Two load stages are then defined. During the first *gravity load* is applied to the mesh. In the following stage the surface load (400 kN/m) is vertically applied to the pre-defined surface areas (W1 to W6). The displacements due to the first load stage are considered as the zero datum. Thus only those predicted by the FE analysis due to the second stage (surface loading) are examined.

#### 3 RESULTS

#### 3.1 Strand7 FE predictions

In this section findings are presented for the case of surface loading above pre-existing twin tunnels which are horizontally aligned (*TH* case). Figure 3 presents the lining deformations (scaled up) of the first tunnel (Fig. 1) when  $z_{o,1} = 15$  m and P = 1D due to the effect of surface loading only (dotted circles). This is then compared to the original tunnel shape prior to any loading stage (thick circle). The first obvious outcome is that the whole tunnel seems to squat. In other words there is an elongation of the horizontal

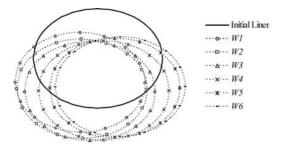


Figure 3. Deformed shape of the first tunnel (*TH* case) due to the surface loading for six different loaded areas. The tunnel axis is driven at  $z_{o,1} = 15$  m and P = 1D.

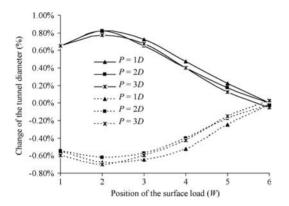


Figure 4. Changes of horizontal and vertical tunnel diameters of the first tunnel (*TH* case) expressed as a percentage of the initial tunnel diameter due to the surface loading against the position of the applied load when  $z_{0,1} = 15$  m.

diameter with a simultaneous decrease of the vertical. Further to this obvious vertical translation a secondary type of movement seems to occur coincidentally. The deformed lining seems to slightly rotate anti-clockwise opposing the position of the applied surface load as this shifts towards W4. Similar predictions were identified for the ST and the TVD cases.

Figure 4 shows plots of the change of tunnel diameter (as a percentage of the initial tunnel diameter) against the relative position of the surface loaded area for the first tunnel (TH case). The thin lines refer to the horizontal tunnel diameter (along springlines). The dotted lines on the other hand refer to the vertical (crown to invert). An increase of the horizontal diameter with a simultaneous decrease of the vertical is observed. The maximum increase of the horizontal diameter as well as the maximum decrease of the vertical (0.8% of the tunnel diameter) occurs when the surface load is directly applied above for  $z_{0,1} = 15$  m. Thus, a clear trend for the magnitudes of these changes can be identified. These changes appear to fade as the load is applied further away from the tunnel's centre line.

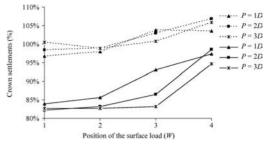


Figure 5. Plots of the crown settlements due to loading of the first tunnel in the *TH* case as a percentage of the *ST* case for various surface loading areas and two different tunnel depths.

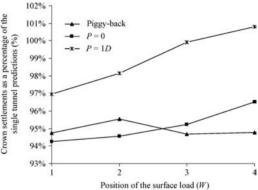


Figure 6. Plots of the crown settlements due to loading of the first tunnel (*TVD* case) as a percentage of the *ST* case for various surface loading areas when  $z_{o,2} = 20$  m.

For the *TH* case crown settlement predictions from the first tunnel are plotted (Fig. 5) as a percentage of the single tunnel case against the position of the surface loaded area (*W*) for two different depths ( $z_{o,1} = 15$  m and 20 m) and for three different pillar widths (P = 1D, 2*D* and 3*D*). Thin lines refer to the shallow case while dotted to the deeper. It can be seen that greater interaction occurs for the shallow tunnel case since there is a difference of 5% to 15% compared to the single case. For the deeper case results are almost identical to the single. This implies less or even no interaction between the two parallel driven tunnels.

For the *TVD* case crown settlement predictions from the first tunnel are plotted (Fig. 6) as percentage of the *ST* case against the position of the surface loaded area (*W*) for different pillar widths (P = piggy-back, 0 and 1*D*) when  $z_{o,1} = 15$  m and  $z_{o,2} = 20$  m. It can be observed that a small amount (less than the *TH* shallow case) of interaction exist (none to 6%) compared to the *ST* results. Greater interaction appears between the tunnels when they are closely spaced (P = piggy-backand 0) of approximately 5%. When the lower tunnel is

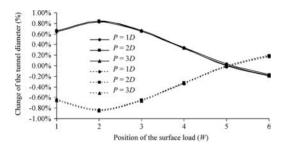


Figure 7. Changes of horizontal and vertical tunnel diameters due to the surface loading expressed as a percentage of the tunnel diameter against the position of the applied load when  $z_{o,1} = 15$  m.

driven further away (P = 1D) less interaction between the two vertically aligned tunnels occurs (none to 3%).

Comparison of the different cases in Strand7 3.1.1 In this section the FE predictions regarding the first tunnel (in the TH and the TVD cases) are compared with the predictions regarding the single tunnel (ST case) in order to investigate the interaction mechanism of soil-tunnel-structure in 2-D. The above mentioned comparison is made in terms of lining deformations. The FE predictions regarding the first tunnel (TH and TVD cases) are smaller than those regarding the ST case. This is a first indication of the existence of interaction. In both of the compared cases the shape of the tunnels seems to squat while the reduction of the vertical tunnel diameter equals to the increase of the horizontal. The maximum lining deformation occurs when the surface load is applied at W2. No lining deformation is predicted when the load is applied at W6 (Fig. 4) which indicates that at that distance the interaction ceases.

In total it seems that interaction occurs within the region of W1 to W4 ( $P \le 2D$ ) and then for ( $P \ge 3D$ ) it starts to reduce (Fig. 4). Greater interaction is predicted in the *TH* case compared to the *TVD* case. This implies that the existence of the lower tunnel (*TVD* case) does not contribute to the complex interaction mechanism in the same way as it does the second tunnel in the *TH* case (Fig. 1).

#### 3.2 Plaxis FE predictions

Figure 7 is akin to Figure 4. This graph indicates that the maximum change of D (Roughly 0.8% of the tunnel diameter. These predictions are in agreement with Strand7) regarding the first tunnel (*TH* case) occurs when the load is applied directly above. As the load shifts further away these changes reduce towards zero (no change of D). However when the surface load is applied at its furthest possible distance, the vertical diameter is seen to increase while the horizontal reduces.

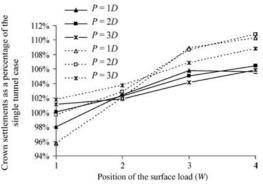


Figure 8. Plots of the crown settlements due to loading of the first tunnel in the *TH* case as a percentage of the *ST* predictions for various surface loading areas and two different tunnel depths.

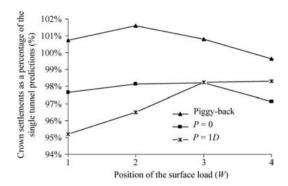


Figure 9. Plots of the crown settlements due to loading of the first tunnel in the *TVD* case as a percentage of the *ST* case for various surface loading areas when  $z_{0,2} = 20$  m.

Figure 8 shows the interaction between the first tunnel (*TH* case) and the surface load compared to the single tunnel case (*ST*) for two different depths ( $z_{o,1} = 15$  m and 20 m) and three different pillar widths (P = 1D, 2D and 3D) in terms of crown settlements (similar to Fig. 5). Thin lines refer to the shallow case while dotted refer to the deeper case. It can be seen that Plaxis predictions for the *TH* case are almost identical to the *ST* case results (i.e. almost no interaction between the two parallel tunnels). This picture is different to the Strand7 results especially for the shallow case.

Figure 9 shows the interaction between the two tunnels and the surface load in the *TVD* case compared to the (*ST*) case in terms of crown settlements for different pillar widths (P = piggy-back, 0 and 1*D*) when  $z_{0,1} = 15$  m and  $z_{0,2} = 20$  m (akin to Fig. 6). It can be observed that a small amount of interaction exists (none to 5%) compared to the *ST* case predictions. Results are similar compared to the *TH* case from *W*1

to *W*3. As the surface load shifts towards *W*4 though a difference in the prediction of interaction is identified.

3.2.1 Comparison of the different cases in Plaxis FE predictions regarding the first tunnel (in the *TH* and the *TVD* cases) are compared with those in the *ST* case. The maximum deformation of the tunnel lining occurs when the load is applied at W2 forcing the tunnel to squat. These deformations reduce as the load shifts towards W5. At W6 though the load seems to produce an ovalisation of the lining with the vertical tunnel diameter greater than the horizontal (in contrast to the previous load cases). In general Plaxis predicts similar amount of interaction between the two different tunnel geometric configurations (*TH* and *TVD*).

#### 4 DISCUSSION

In the current paper 2-D FE predictions were presented investigating the effect of surface loading above preexisting tunnels driven in soft ground. Two different FE packages were used for this purpose to compare and validate the produced results. Several parametric studies were carried out varying the excavation depth, the pillar width, the pillar depth and the position of the surface loaded area. The general trend between the two FE packages regarding the lining distortions and the crown settlements was similar. Both packages predict that when the surface load is applied within the region of W1 to W4 [i.e. a horizontal distance of  $P \le 2D$  from the first tunnel (TH case and TVD cases)] the existence of the interaction mechanism was evident regardless of the tunnel geometric configuration. Further from that distance no interaction occurred.

Small differences in the predictions between Strand7 and Plaxis occurred. Strand7 in particular predicted the existence of stronger interaction mechanism for the *TH* case compared to the *TVD* case. Plaxis on the other hand predicted a similar amount of interaction between the two different tunnel geometric configurations and smaller compared to Strand7 predictions. These differences are attributed to the following three factors:

- Different types of finite elements were used to model soil in the domain. Six-noded triangles were used in Strand7. Even though this type of finite element was available in Plaxis as well, it was decided that the fifteen-noded triangle should be used instead for greater accuracy.
- Different meshes were generated between the two FE packages. The reason was that in Plaxis the user cannot import a mesh as in Strand7.
- Finally, even though the same elasto-plastic soil model with the Mohr-Coulomb failure criterion was used between the two FE packages, the plastic potential function was different. Associated

flow was used in Strand7 while Plaxis used nonassociated flow. According to Potts and Zdravkovic (1999) the latter way of modelling real soil behaviour is more realistic than the first, although given the loading in this problem it is perhaps not significant.

From the above it is clear that differing predictions can be found from routine use of two FE modelling packages. Greater detail of this and an extended study of this problem can be found in Koungelis (2007). Such differences in predictions may not be important but without undertaking such studies that fact will be difficult to assert in most cases.

#### ACKNOWLEDGEMENTS

This work was funded by a studentship from the UK Engineering and Physical Sciences Research Council (EPSRC), the School of Engineering at Durham University and Halcrow Ltd.

#### REFERENCES

- Attewell, P.B. 1978. Ground movements caused by tunnelling in soil. In: Procs. Of Conf. on large ground movements and structures, UWIST/July 1977: 812–948.
- British Standards Institution 1994. BS 8002: Code of Practice for Earth retaining Structures.
- Koungelis, D.K. and Augarde, C.E. 2004. Interaction between multiple tunnels in soft ground. In: *Developments in Mechanics of Structures and Materials: procs. 18th Australasian Conference, Perth, Australia.* Deeks, and Hao, (eds), London: Taylor and Francis. Vol. 2: 1031–1036.
- Koungelis, D.K. 2007. Tools for numerical modelling of tunneling interactions. PhD Thesis, Durham University.
- Mair, R.J. and Taylor, R.N. 1997. Bored tunnelling in the urban environment. In: Procs 14th Int. Conf. Soil Mech. And Found. Engng., Balkema, 1997: 2353–2385.
- Moore, I.D. 1987. Response of buried cylinders to surface loads. J. Geotech. Engng. 113(7): 758–773.
- Moore, I.D. and Brachman, R.W. 1994. Three-dimensional analysis of flexible circular culverts. *J. Geotech. Engng.* 120(10): 1829–1844.
- O'Rourke, T.D. 1984. Guidelines for tunnel lining design. Published by ASCE, Technical committee on tunnel lining design of the Underground Technology Research Council.
- Peck, R.B. 1969. Deep excavations and tunnelling in soft ground. In: 7th Int. Conf. Soil Mech. and Found. Engng, Mexico, 1969: 225–290.
- Potts, D.M., Axelsson, K., Grande, L., Schweiger, H. and Long, M. eds. 2002. *Guidelines for the use of the advanced numerical analysis*. London: Thomas Telford.
- Potts, D.M. and Zdravkovic, L. 1999. Finite element analysis in geotechnical engineering: Theory. London: Thomas Telford.
- Spasojevic, A.D., Mair, R.J. and Gumbel J.E. 2007. Centrifuge modelling of the effects of soil loading on flexible sewer liners. *Géotechnique* 57(4): 331–341.

# Geologic Model Transforming Method (GMTM) for numerical analysis modeling in geotechnical engineering

# X.X. Li & H.H. Zhu

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P. R. China Department of Geotechnical Engineering, School of Civil Engineering, Tongji University, Shanghai, P. R. China

# Y.L. Lin

Geotechnical Research Institute, Hohai University, Nanjing, P. R. China

ABSTRACT: During numerical simulation of geotechnical engineering, due to the complex engineering geological condition, the simplified geologic model is usually adopted. However, the accuracy and reliability of calculating result are directly influenced by characteristic of model. The 3D geologic model can better reflect the actual geological condition. The paper presents a new and practical modeling method (Geologic Model Transforming Method) of numerical analysis by integrating geologic model and numerical model. This method is performed with the following procedures: (1) cutting the 3D geologic model according to numerical calculation region; (2) extracting control data from cutting model to reconstruct the surface model; (3) meshing and forming the numerical model automatically by the stratum attribute; (4) importing the model into numerical analysis system. An example is given to illustrate the application of the method. The implementation of the method results in high efficiency and automaticity of modeling.

# 1 INTRODUCTION

With the development of the computer technology, finite element method (FEM) is widely used in geotechnical engineering and other research domains. During the solution of FEM, preprocessing, solving and the post-processing are included. The preprocessor, which involves collecting data, inputting information, meshing and defining material property, etc., is the fussiest process and spends  $40 \sim 50$  percent of the total time in the analysis course of FEM. That is, about half effort of FEM analysis is expended on the preprocessor (Liu et al. 2002). Therefore, preprocessing system is one of the core parts of the finite element analysis and a good preprocessor plays an important role in the success of the method.

For a practical problem of geotechnical engineering, numerous data is demanded during the modeling process because of complex geological environment, structure, construction procedure and so on. If the fussy data is inputted by manual handling, it is inefficient, and is difficult to check and modify data. Thus, the simplified geologic model is generally adopted. However, a poor model may give unreliable results that are usually misleading and may lead to incorrect conclusions. Therefore, it is necessary to develop the research on a simplified good FEM preprocessor, and the problem has received a lot of interests (Yu et al. 1999, Zhou et al. 2002, Liang et al. 2004).

The information visualization of engineering geology is an important research subject in geosciences. At present, the geology simulation has the powerful function of 3D geological modeling. 3D strata model can be established by borehole data, and part spatial analysis can also be operated, such as sectioning and cutting, etc. There are the certain similarities in modeling and spatial analysis between geologic model and numerical model. If geologic model can be transformed into numerical analysis model, its data can also be directly introduced. Thus, the preprocessor of FEM is largely simplified (Hou et al. 2002, Xia et al. 2005).

In full consideration of the characteristics of the geologic model and the numerical model, this paper presents a new modeling method of numerical analysis called the Geologic Model Transforming Method (GMTM). This method is suitable for FEM preprocessor in geotechnical engineering and simpler and more efficient than the traditional operations approach. The GMTM can be used to form finite element mesh directly from 3D geologic model. The proposed technology will give some useful references to the continuous research on this subject.

The remainder of this paper is organized as follows: in the next section the 3D geologic model is described. Section 3 introduces the implementation and the algorithm of the new method. Section 4 gives an example to illustrate the results of this method.

# 2 3D GEOLOGIC MODEL

#### 2.1 Choosing data model

At present, there are more than twenty data models presented for 3D geosciences modeling. According to the form of data structure, these geologic models presented or discussed could be classified into three types as facial models, volumetric models and mixed models. The facial model emphasizes on the surface representation for terrain, strata interface, outlines of the constructions, buildings and underground engineeering. The triangulated irregular network (TIN) is widely applied for the surface modeling (Baker 1989, Huang et al. 2002). The volumetric model is based on the spatial partition and the real 3D object construction. The three dimension space can be filled with regular or irregular 3D volumes, and the representative volume is irregular tetrahedron network (TEN) (Chen et al. 1994, Pilouk et al. 1994). The mixed model is the combination of facial model and volumetric model.

The small storage space of data and its fast visualization are the characteristic of the facial model, and the information of strata interface and fault could be represented in the interior of geologic surface model. The volumetric model is convenient for the description of the attributes of each volume and the storage of its related spatial location. However, due to the enormous data amount of the solid model, the efficiency of Boolean operation to strata is lower. Although the mixed model takes the advantage of facial model for fast visualization and volumetric model for spatial analysis in theory, the modeling technology is quite complicated. By contrast, the paper chooses the 3D geologic surface model as basic model.

#### 2.2 3D strata surface modeling

The paper is concerned with the 3D geologic surface model based on TIN representation (Zhang et al. 2006). To the geological modeling, the practical implementation just requires:

- 1. Transaction of borehole data. The information of drill hole is extracted from database. Then every borehole is numbered with the dichotomic topologic data structure, and is blocked with the blocking rules of data. Thus, the stratum integrity can be opened out.
- 2. Construction of the hollow surface model. The reference TIN is constructed with the surface data of the borehole. In term of the topological relations of the formed TIN, each stratum interface is obtained

by mapping the altitude and attribute of under layer borehole. For the phantom, the modeling can be established by the sliced interface rules and the interpolation of the space attribute. So the whole hollow surface model is finished.

3. Construction of the surrounding surface model. With the nearest neighbor first algorithm and Delaunay method to the scattered points in external outline of every interface, the strata surrounding surface model is constructed. Finally, the 3D strata surface model is composed of the hollow interface model and the surrounding model.

# 3 TRANSFORM GEOLOGIC MODEL INTO NUMERICAL ANALYSIS MODEL

The finite element mesh needs to coincide with the consistency principle and the geometrical property principle. In consideration of the difference in data structure, the geologic model can not be directly taken as the numerical model (Wang et al. 2004). To satisfy the regulation of the finite element mesh, the new modeling method presented in this paper emphasis on the process of transforming geologic model into numerical model.

# 3.1 Flow process

The realized procedures from geologic model to numerical model are listed in the following:

- 1. Calculation region cutting (section 3.2). Based on the 3D strata surface model established, the region cutting is performed along the partial research region in conformity to the request of numerical analysis.
- 2. Reconstruction of surface model (section 3.3). By virtue of the intersection points and the strata control points extracted from the cutting model, the strata surface model of the calculate region is reconstructed. Thus, the initial surface mesh is generated.
- 3. FEM surface meshing (section 3.4.1). Evaluate whether the quality of the initial surface mesh is satisfied with the request of the finite element mesh or not. If the judgment is true, the initial mesh can be acted as the calculation mesh. Otherwise, the surface mesh needs to be regenerated automatically by a new approach.
- 4. FEM solid meshing (section 3.4.2). Based on the generated surface mesh which satisfied with the FEM request, the relevant solid mesh is generated automatically by an efficient approach.
- 5. Importing into numerical analysis system (section 3.5). The topological data of the FEM mesh model is imported into the numerical analysis system in accordance with the requested data format. In the end, the modeling work is finished gradually.

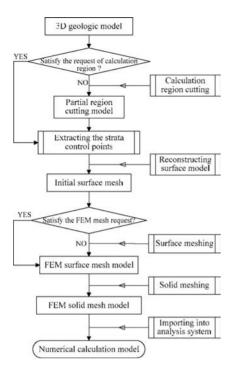


Figure 1. Flowchart of model transformation.

In the Geologic Model Transforming Method, the key technologies include the cutting, the reconstruction and the meshing. Figure 1 shows a flowchart of the transforming process. The following sections describe the key steps in detail.

#### 3.2 Calculation region cutting

According to the engineering geological condition and the survey data, the 3D strata surface model is established with the geological modeling technology. It has the advantages of better accuracy for strata attribute and larger overlying region. To a practical calculation model, the partial research region just is the region of numerical model. The conditions which determine the calculation region usually include removing the boundary effect and satisfying the calculation accuracy. So it is unnecessary to take the overlying region of the geologic model as the research region of the calculation model. To reduce the computing time and scale of the numerical analysis, the region cutting is performed in the method. By the means that choose the calculation region of the numerical analysis, the reasonable cutting boundary could also be determined, as shown in Figure 2.

To benefit the calculation, the cutting region generally is a rectangular solid. Of course, it can also be composed with many planes or polygons. The algorithm of the region cutting is a procedure that

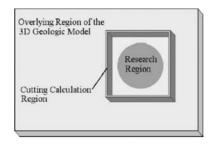


Figure 2. Calculation region cutting.

the intersection lines and the intersection points are obtained with the intersection calculation of the space planes each other. By the region cutting, the partial research region can be arbitrarily acquired on the 3D geologic model. Thus, it is unnecessary to build the calculation model again and again. At the same time, the construction process in geotechnical engineering can be simulated, such as the excavation of the foundation and the tunnel.

#### 3.3 Reconstruction of surface model

The topological relations are changed when the region cutting is performed on the original strata surface model. In order to keep the same topological structure, the surface cutting model is reconstructed by the intersection lines, the intersection points and the original strata control points. The reconstruction algorithm includes two sections: the reconstruction of strata interface and the reconstruction of strata surrounding surface.

Reconstruction of strata interface:

- 1. The intersection lines, the intersection points and the original strata control points in the cutting region are classified and restored by the strata attribute.
- To each stratum, the intersection lines are taken as the constraint boundary, and then the constraint Delaunay triangularization is applied to the scattered points in the plane region.
- 3. The topological structure of the formed mesh is kept invariability. The altitude of each point is mapped. Thus, the stratum interface model is obtained again.
- 4. Repeat step (2) and (3). Each stratum interface mesh could be reconstructed. Finally, the whole interface modeling is finished.

The illustration of the reconstruction to the strata interface is shown in Figure 3.

Reconstruction of strata surrounding surface:

- 5. The surrounding outline loop of the cutting model is constructed by the intersection points in the interface and is oriented counterclock wise.
- 6. To each cutting plane, the intersection lines of the plane are taken as the interior constraint lines, and

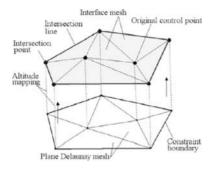


Figure 3. Strata interface reconstruction.

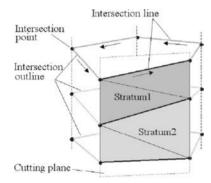


Figure 4. Surrounding surface strata reconstruction.

then the constraint Delaunay train-gularization is applied to the intersection points of the cutting plane.

- 7. The attribute of the formed mesh in the cutting planes is determined with the attribute of the intersection points.
- Repeat step (6) and (7). The surface mesh of the closed surrounding cutting planes could be generated. Finally, the whole surrounding surface modeling is finished.

The illustration of the reconstruction to the strata surrounding surface is shown in Figure 4.

The surface model of the cutting region is the combination of the strata interface model and the surrounding surface model. The cutting model has the same geometric relation and the topological data structure. Its mesh is also taken as the initial surface mesh with the strata attribute.

#### 3.4 Finite element meshing

At present, the algorithms of the finite element mesh generation are rather sophisticated and efficient. Most relevant researches are focused on the algorithm itself (George 1991; Shephard et al. 1991; Lau et al. 1996). However, there are many preconditions of the mesh

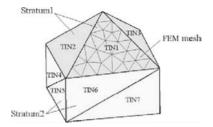


Figure 5. Multiple TIN Regions (MTR) method.

generation need to be judged by the man-machine interaction way during the process of numerical modeling, such as the preconditions whether the meshing region is closed or not and whether the total redundant edge length is zero or not, etc. Thus, the automatic meshing is often not able to be performed normally. The reason of the problem does not lie in the algorithms but in the preconditions. These factors cause the modeling difficulty to be enlarged and the automaticity to be reduced. Therefore, it is necessary to research how to improve the whole modeling efficiency entirely.

#### 3.4.1 Surface meshing

Any single irregular triangle is closed. Such the triangle region is called the Single TIN Region (STR) in the paper. The set of many STR which do not pass through each other is called the Multiple TIN Region (MTR). The surface model based on TIN representation is taken as the MTR model. So the initial surface mesh of the cutting model also is the MTR model. By performing circularly the algorithm of the surface meshing in 3D space plane to each STR, the finite element surface mesh of MTR model could be generated automatically, as shown in Figure 5. In order to coincide with topological consistency of the nodes in shared edge, the region boundaries are discretized with fixed length partition or integer division way. The method of the surface meshing is called the MTR method.

Based on the MTR method, some man-machine interaction operations can be avoided, such as seeking manually the closed region and discretizing the boundaries one by one. The MTR method is realized easily by programming, and has the advantages of better element quality and high efficiency and automaticity.

#### 3.4.2 Solid meshing

A single closed 3D space composed of the FEM surface mesh is called the Single Space Region (SSR). The set of many SSR which do not pass through each other is called the Multiple Space Region (MSR). The algorithm of the solid meshing needs to be performed in SSR. Generally, there are many different space regions in a calculation model. Thus, almost each SSR need to be formed with man-machine interaction. It is very inconvenient to the numerical modeling.

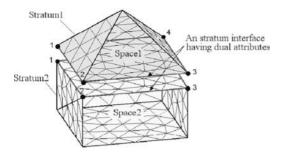


Figure 6. Multiple Space Regions (MSR) method.

So, under the direction of the block thought that the surface mesh having the same stratum attribute can constitute the closed space region, a method is presented to build automatically the SSR of the different stratum. By performing circularly the algorithm of solid meshing to each SSR, the finite element solid mesh could be generated automatically, as shown in Figure 6. In consideration of the dual attributes of the interface mesh, it needs to be used two times in forming strata space region. The method is called the MSR method.

The thought of the blocking by strata attribute is applied in the MSR method. The construction for the SSR and the geometry inspection can be avoided before the solid meshing. The efficiency of the whole modeling is improved thoroughly. It is an innovation to traditional modeling method.

#### 3.5 Importing into numerical analysis system

The topological data of the taken FEM surface mesh is imported into the numerical analysis system in accordance with the requested data format. The numerical analysis model is formed gradually and the whole modeling is finished. The finite element model generated with the GMTM is not depended on the numerical analysis system. Only changing into the demanded data format, it can be used in the different systems of the universal FEM software, such as ANSYS and Marc.

# 4 EXAMPLE

To illustrate the application of the GMTM, an example is presented in this section. The detailed modeling procedures are the following.

1. The information of boreholes is extracted from database. The number of boreholes and strata are 16 and 3, respectively. The total number of the strata control points is 64. The geologic surface model is constructed by the algorithm of the geologic modeling (see Fig.7). The total number of the TIN is

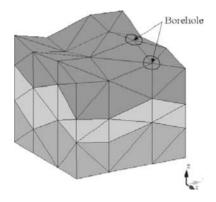


Figure 7. 3D geologic surface model.

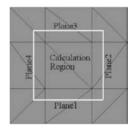


Figure 8. Cutting region.

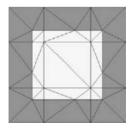


Figure 9. Topological relations.

144, and the different stratum attribute is indicated with different colors.

- 2. The calculation region is determined by importing the information of the cutting surface. The paper assumes the cutting surface is composed of four vertical planes. The plane graph of the cutting region is shown in Figure 8, and the new topological relations are shown in Figure 9.
- The cutting surface model is reconstructed according to the strata control points in cutting region. The strata interface model and the initial surface mesh are shown in Figure 10.
- 4. Based on the initial surface mesh, the surface meshing and the solid meshing are automatically performed by the MTR method and the MSR method respectively. The strata interface FEM mesh and the solid mesh are shown in Figure 11.

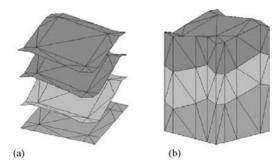


Figure 10. Surface model: (a) strata interface model; (b) initial surface mesh.

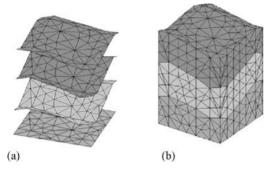


Figure 11. FEM mesh: (a) strata interface mesh; (b) solid mesh.

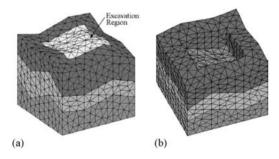


Figure 12. Mesh of the excavation model: (a) whole mesh; (b) part mesh.

5. The excavation process also can be simulated by the region cutting. In this example, the cutting region is taken as the foundation pit. The whole and part FEM mesh are shown in Figure 12. In the surface model, the total number of the point and the triangle element generated are 1140 and 3646 respectively. In the solid model, the total number of the point and the tetrahedron element generated are 1821 and 8838, respectively. The total time spent in the whole modeling process is about 10 minutes.

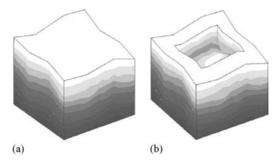


Figure 13. Stress distribution: (a)before excavation; (b) after excavation.

6. The topological data of the formed mesh is imported into the FEM software Marc. In order to check the element mesh quality, a simple numerical calculation is implemented. Figure 13 shows the self-weight stresses of the model before excavation and after excavation in the action of gravity.

# 5 CONCLUSIONS

In order to simplify the preprocessor of the numerical analysis, the paper presents a new modeling method based on the geologic model. It differs from the traditional modeling method. In our method, the numerical model is generated by the following procedures: cutting the calculation region, reconstructing the surface model, finite element automatic meshing, importing the analysis system. The characteristic of the method is generalized as:

- The element mesh is marked by the stratum attribute, so the model further nears to the real geological condition. The partial region can be randomly extracted from the 3D strata model, and must not be established repeatedly.
- Taking the advantage of the characteristic of the geologic model, the complex meshing algorithm is avoided. The programming of the meshing is simple and efficient. The whole modeling is highly automatic and rapid.
- 3. The excavation of the foundation and the tunnel can be conveniently simulated. The boundary of the excavation must not be set in advance.
- The finite element mesh can be imported into the different analysis system to calculate. The model is applicable and flexible.
- 5. In a word, the GMTM realizes the dream to establish numerical model directly based on 3D geologic model. It provides a nice possibility and prospect for FEM numerical simulation. The transforming method based on other geologic model is consistent with the method presented in the paper, but the material algorithm needs to be studied further.

#### REFERENCES

- Baker, T.J. 1989. Developments and trends in three dimensional mesh generation. Applied Numerical Mathematics.
- Chen, X.Y. & Kozo, I. 1994. Three dimensional modeling of GIS based on delaunay tetrahedral tessellations. In: *International Archives of Photogrammetry and Remote Sensing*, Munich, Germany, 30: 132–139.
- George, P.L. 1991. Automatic mesh Generation: Application to Finite Element Methods. Witey, New York.
- Hou, E.K, Wu, L.X. & Li, J.M. 2002. Study on the coupling of 3D geoscience modeling with numerical simulation. *Journal of China coal Society*, 27(4): 388–392.
- Huang, Y.Y. & Chen, S.Q. 2002. Geologic modeling by marching cube. *Journal of Engineering Graphics*, 23(2): 65–69.
- Lau, T.S. & Lo, S.H. 1996. Finite element mesh generation over analytical curved surface. *Computers & Structures*, 59(2): 301–309.
- Liang, S.W, Yang X.H. & Yang, W.B. 2004. System integration of ANYSIS preprocessor and AutoCAD. Journal of Huazhong University of Science and Technology (Urban Science Edition), 21(1): 81–84.
- Liu, L.M, Liu, H.L. & Zhu, Z.D. 2002. Integration method of geoengineering finite element analysis system based on GIS. *Chinese Journal of Rock Mechanics and Engineering*, 21(Supp.1): 1995–1998.

- Pilouk, M, Klaus, T. & Martien, M. 1994. A tetrahedronbased 3D vector data model for geoinformation. In: Advanced Geographic Data Modeling, Netherlands Geodetic Commission, Publications on Geodesy, 1994(40): 129–140.
- Shephard, M.S. & Georges, M.K. 1991. Automatic three dimensional mesh generation by the finite octree technique. *International Journal for Numerical methods in Engineering*, 32: 709–749.
- Wang, C.X. & Bai, S.W. 2004. Study on integration of 3D strata information system and FEM. *Chinese Journal of Rock Mechanics and Engineering*, 23(21): 3695–3699.
- Xia, Y.H, Bai, S.W. & Ni, C.S. 2005. Study on coupling of 3D visualization with numerical simulation for powerhouse excavation of a certain hydrojunction. *Rock and Soil Mechanics*, 26(6): 969–972.
- Yu, C, Zhou, X.H. & Zhang, Y.Q. 1999. The integration of the feature-based modeling and finite element method. *Journal of computer-aided design and computer graphics*.
- Zhang, F, Zhu, H.H. & Ning, M.X. 2006. Modeling method of 3D strata suitable for massive data. *Chinese Journal of Rock Mechanics and Engineering*, 25(Supp.1): 3306–3310.
- Zhou, E.H, Zhu, Y.W. & Wang, T. 2002. Transitionalzone method in hexahedral finite element meshing in geotechnical engineering. *Engineering Journal of Wuhan University*, 35(3): 24–29.

# Review and interpretation of intersection stability in deep underground based on numerical analysis

# T.K. Lu, B.H. Guo, & L.C. Cheng

School of Energy Science and Engineering, Henan Polytechnic University, Jiaozu, Henan, P.R. China

#### J. Wang

Shenhua Ningxia Coal Group, Yinchuan, Ningxia, P.R. China

ABSTRACT: This paper conducted a preliminary study on the stability of roadway intersection in deep underground conditions on the basis of the underground observations and numerical modeling. Firstly, according to the intersections used in current coal mines, five different geometrical shapes of two-dimensional intersections are selected and modeled under deep ground condition. It is noted that the cross-intersection is the most unstable two dimensional intersection currently used in coal mines. Also, the depth of cover is one of important factors for this investigation, which clearly indicated that the stability of the intersection is threatened seriously when the depth of cover reached at 1000 m or more below the surface. In addition, the construction sequence and direction for the cross-intersection is studied. It is believed that the construction sequence and direction are sensitive factors for the stability of two dimensional cross-intersection. It is suggested that the formation of the cross-intersection should be conducted one side by another individually, even this kind of construction sequence will cause twice stress concentration and redistribution, it is still less influence on the stability of intersection comparing with other construction methods selected.

# 1 INTRODUCTIOIN

From the statistical data, it is noted that the roof failure mainly occurred around the large span of underground opening, and the situation is getting worth, when the complex geometrical structures and deep mining conditions are encountered. This paper is describes the stability and failure characteristics of intersection on the basis of literature review and study of the stability and failure behaviour of different geometrical structure of intersection, which commonly used in underground of coal mine in China, associated with various depths of cover using numerical simulation.

It is noted that the geometrical shapes used in China coal mining industry is much complex, comparing with major coal mining countries in overseas, such as Australia and USA. With increasing the depth of cover, it is susceptible to ground control problems due to the complex geometries of underground structure.

#### 2 PREVIOUS INVESTIGATION ON UNDERGROUND INTERSECTION IN COAL MINES

Stability of underground intersection had been paid attention previously, even in the shallow condition. The investigations had been conducted to study the structural characteristics, stability and failure bahaviour in the shallow condition by former US Bureau of Mine (Hanna, 1991).

In 1976, Balachandra studied the stability of intersection using underground monitoring and computer simulation.

The department of mining engineering from West Virginia University studied the stability of three-way intersection with different geometrical parameters, and it found that the tensile stress is the main reason to cause the failure of the intersection. Comparatively, the three-way intersection is more likely having shear failure than the cross intersection. It is concluded that the tensile and shear failure on the roof strata increased with the reducing of angle between two roadways (Peng, 1978).

The department of mining engineering from University of Wollongong also studied the stability of the T-intersection (Singh, 2001). Based on the numerical modeling and underground monitoring, it indicated that the depth of cover, horizontal stress and geometrical parameter are three major factors which significantly influence the stability of T-intersection. It also indicated that the stresses induced during intersection formation may result in high incidence of roof and rib failures.

In China, the depth of cover in coal mine is increasing 8–12 m annually, and there are many collieries are going to face deep mining situation, with depth of cover around 1000–1500 m. With increasing of the depth of cover, the stress regime and deformation characteristics of roadway is changed significantly, it resulted in failure and difficult to ground control. Also, construction condition is getting worse than shallow conditions (Jin and Sun, 2001). Under such situation, the stability of intersection is facing more serious problem during the deep mining.

The depth of cover is a sensitive factor to the ground control. For example, the ground problem was occurred when the depth of cover was changed from 550 m to over 600 m below the surface in Xiezhuang colliery, Shandong. Previously, the stability of intersection can be maintained by using steel frame when the depth of cover less than 600 m, but when the depth of cover reached over 800 m, ground control around intersection became a major problem, due to: 1) the soft rock characteristics of surrounding rock mass was encountered and the mechanical properties was getting worse; 2) the capacity of reinforcing element was reduced significantly; 3) the plastic zone around intersection increased significantly, 4) roof separation was formed significantly and it is considered as a major factor to cause the stability of intersection (Wang, 2001).

The further study on two way intersection indicated that, in deep underground (700–900 m), the angle of intersection is not a major factor to influence the plastic zone around intersection, but when the angle is less than 35 degree, the reinforcing area is increased significantly with decreasing of the angle. It is, therefore, suggested that the angle between two roadways should not be less than 35 degree for the designation of intersection (Zhu and Cao, 2005).

On the other hand, many ground control problem related to intersection are not affected by single factor, but combination phenomenon, which involved roof stress, physical properties of mudstone, reinforcement technique and parameters used are not suitable for such underground environment (He, et al, 2005).

#### 3 MODELING OF INTERSECTION STABILITY

The modeling was conducted by using FLAC<sup>3D</sup> with three dimensional and non-linear simulations. The numerical model used for the analysis is presented in Figure 1, hard rock to soft rock, the mechanical properties are given in Table 1.

# 3.1 Effect of geometries on stability of intersection

There are large number of roadway intersections are constructed annually in China coal mines industry with

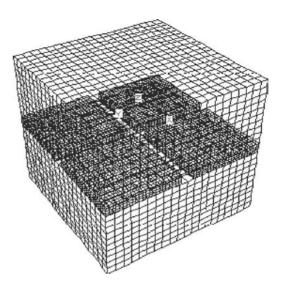


Figure 1. Model used in the simulation.

various geometries, including three-way intersection, four-way intersection, two dimensional roadway intersection (Figure 2a,b,c) and three dimensional roadway intersection (Figure 2d,e) and geological conditions, e.g. strong rock mass and soft rock mass. With increasing of the mining depth, the stability of these roadway intersections is susceptible to ground control problems due to inherently wide roof spans and complicated intersection geometry used.

The study conducted in this section is to clarify the effect of the geometrical shapes on the stability of intersection. To do so, the depth of cover is determined as 800 m, and the two dimensional intersections with five different geometrical shapes, plus one normal single roadway are compared as:

- Cross intersection, with 90° angle between two roadways;
- X intersection, similar with cross, but with 45° angle between two roadways;
- T intersection;
- L-intersection;
- Y intersection;
- Single roadway

The modeling was conducted to evaluate the stability behaviour of the intersection in terms of the roof and rib deformation and the failure depth into the rock mass with different shapes, on the basis of mechanical properties given in Table 1. The deformation and failure of the intersection have been compared with the normal single roadway, it is indicated that the simpler the geometry, the more stable the underground structures. The results also shown that the most unstable two-dimensional intersection is the cross-intersection (Figures 3 and 4).

Table 1. Mechanical parameters of rock mass.

Intersection(3D)

No	E (GPa)	μ	Modular of volume (GPa)	Modular of share (GPa)	Density (kg/m <sup>3</sup> )	Cohesion (Mpa)	Angle of inter- friction (o)	Tensile Strength (MPa)	Residual Cohesion (MPa)	Residual angle of inter- friction(o)
1	12	0.25	8	4.8	2500	3.7	26	1.4	0	22
	Cross	≷   ≷	b) Y - Intersection		F- Intersection			-3-+ +2- (C)	(d)	2

Figure 5. Sequence of intersection construction.

the stability of intersection. As the indicated above, the cross-intersection is the most unstable two dimensional intersection, thus, the effect of the excavation sequence on stability of cross-intersection is studied accordingly.

Figure 5 shows the construction sequences of the cross-intersection, the sequences are designed as five different types of a, b, c, d, and e, which represent the sequences and directions of the construction. According to the modeling results, the minimum vertical stress and deformation have been found from the excavation sequence Nos. 1 and 2, and both represented similar construction sequence, that is, the roadways formed the intersection are developed at different time. It is indicated that even the stress state around opening are interrupted twice by using these developing sequences, but the each interruption is considered to be smaller comparing with other construction sequence proposed.

The developing direction is also sensitive to the stability of intersection. If the excavation is toward the existing opening (Figure 5e and Figures 6a&b), the significant affect on the stability of intersection is detached, and if the excavation direction is moving away from intersection (Figure 5b and Figures 6a&b), the affect is comparatively small (Figure 6).

#### 3.3 Effect of depth of cover on stability of intersection

The depth of cover is always a sensitive factor, which should be taken into consideration during the stability analysis. Based on the previous work, the cross-intersection is analyzed with various levels of depth of cover from 200 m to 1100 m respectively. Figure 7 shows the stability of the intersection various with depth of cover, and it indicated that the stability is not affected directly with increasing of the depth of cover. Before the depth of cover reached a certain level, such as 1000 m, there is not significant deformation and failure occurred around the intersection, thus, the stability can be properly maintained. After

30 The second s

Figure 2. Main intersections used in Chinese coal industry.

Intersection(3D

Figure 3. Comparison of roof/rib deformation with different geometrical shapes.

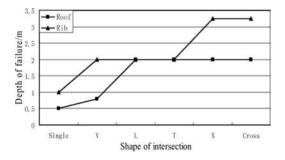


Figure 4. Comparison of roof/rib failure with different geometrical shapes.

# 3.2 Effect of excavation sequence on the stability of intersection

Stresses induced during the intersection formation may result in high incidence of roof and rib failure (Singh, 2001), thus the construction sequence (procedure) may be another sensitive factor that influences

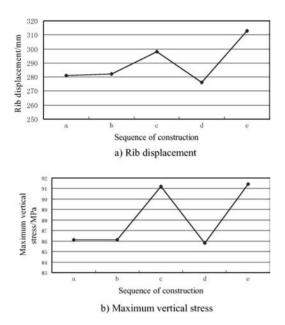
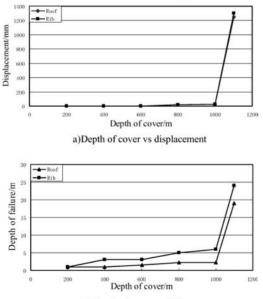


Figure 6. Effect of construction sequences on stability of intersection.



b) Depth of cover vs failure

Figure 7. Effect of depth of cover on stability of intersection.

the depth of cover reached 1000 m, the deformation and failure is developed significantly, it implies that reinforcement strategies used for the intersection in shallow conditions may not be suitably used in the deep condition.

# 4 SUMMARIES AND CONCLUSIONS

Due to inherently wide roof spans used, the stability of the roadway intersection was always paid attention by the researchers and engineers. Now, with continual increasing of the mining depth of cover, more and more serious stability problems, particularly related to the roadway intersection, will be faced in Chinese coal mines.

The study confirmed that the stability of the intersection could be maintained in the shallow condition, but when the depth of cover reached a certain level, such as over 1000 m below the surface, the stability of intersection is under the threat.

The geometry of the intersection is concerned, as comparatively the geometrical shapes used in China coal mining industry are relatively complicated, which results in much more wide roof span than normal heading roof. Comparing with six different types of the underground opening, it is noted that, under the similar conditions, the normal roadway has minimum displacement and failure than other intersections. It implies that the large roof span will cause more stability problem than small roof span. Also, comparing different geometrical shapes of the two-dimensional intersections, the most unstable one is the crossintersection, which is the most comment intersection used in coal mines.

The construction sequences also influence the stability of the intersection during the excavation. To minimize the stress interruption to existing roadway during the intersection development, particularly for the cross-intersection, it is suggested that all the additional roadway construction should always move away from rather than forward to the intersection. In addition, the construction for additional roadway around the intersection should be conducted one side by another individually (Figure 5b), even this kind of construction sequence will cause twice stress concentration and redistribution, but it still is the less influence on the stability of intersection comparing other methods of the construction.

The depth of cover is recognized as the most important factor for the stability of underground openings. According to the modeling results, the stability of the intersection is not linearly related to the depth of cover, the significant effect only occurred when the depth of cover reached to 1000 m under typical surrounding rock mass conditions selected.

#### ACKNOWLEDGMENTS

Authors wish to acknowledge The Innovation Fund for Outstanding Scholar of Henan Province (Project No.:0621000400) for providing the financial assistance.

#### REFERENCES

- Balachandra, M.B. 1976. The three-dimensional structural analysis of double-entry and single-entry coal mines. Volum I: three-dimensional finite element analysis of crosscut and entry intersection of a double-entry coal mine, *research report*, report number(s), PB-80-150345.
- Hanna, K. et al. 1991. Coal-mine-entry intersection behavior study, *Report of Investigations*.
- He, M.C. et al. 2005. Research on Stability and Support Measures of Crossing Point in Mine Soft Rock Roadway of Deep Third System, Mine *Construction Technology*, Vol.26, No.3–4, pp. 32–35 (in Chinese).
- Jin, H.H. & Sun, Q.G. 2001. High stress and soft rock roadway reinforcement problems and strategy, *Coal Technology*, Vol.3, No.3, pp. 34–35(in Chinese).

- Peng, S.S. 1978. Roof bolting patterns at three-way entry intersections, *research report*, report number(s) NP-23983.
- Singh, R.N., Porter, I. & Hematian, J. 2001. Finite element analysis of three-way roadway junctions in longwall mining, *International Journal of Coal Geology*, Vol.45, Issues 2–3, January, Pages 115–125.
- Wang, P.L. 2001. Discuss of supporting types in roadway cross area in deep seam, Coal mining, Vol.4, pp. 57–59 (in Chinese).
- Zhu, Y.S. & Cao, S.H. 2005. Research on Roof Loosed Loop and Support Reinforced Extent in Roadway Cross Point, Mine Construction Technology, Vol.26, No.2, pp. 28–32 (in Chinese).

# Analysis of surface settlement due to the construction of a shield tunnel in soft clay in Shanghai

Z.P. Lu & G.B. Liu

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, P.R.China

ABSTRACT: By comparison of methods of settlement calculations which have been applied into computing surface settlement due to shield tunneling, the paper analyzed the relativity between Peck's equation and stochastic medium method, and researched the application problem in practical engineering. Associated with the measured data of a constructing tunnel, some characters of surface deformation have been analyzed to appraise current level of construction technology in soft ground in Shanghai. At the basis of established normal ground loss, the quality of construction can be determined and job practice can be adjusted to meet the requirement of environmental protection.

# 1 INTRODUCTION

Ground movement, which is caused by a shield tunneling in soft clay, will give birth to surface settlement of different level. When surface settlement arrives at certain level, the regular service of building and underground installation around will be influenced. Therefore, we must find out the regularity of ground movement, and more exactly predict the value, range of settlement and maximum gradient of settlement curve. Besides, in order to adopt some measures reducing ground movement during construction and design, we need to analyze all kinds of factors influencing surface settlement.

Presently, the research about prediction of surface settlement due to the construction of shield tunnel can be generalized as follows:

- Empirical method;
- Theoretical method;
- Model experiment method.

This paper presents it most important to predict the settlement value in allowable error variation by the most economical and convenient method, which is used to direct construction process and control peripheral environmental safety. The final settlement is mainly caused by ground loss and soil consolidation. And in the period of construction, surface settlement is mainly caused by soil piling into shield gap in undrained condition. So based on the comparison of several computing method, the paper firstly discusses the relationship and applicability of them, and secondly analyzes the issue of parameter selecting. In the end, some characters of ground movement and the calculation of ground loss are presented. To sum up, less surface settlement indicates less ground disturbance, and the goal is to present reasonable prediction method and control measures of surface settlement.

# 2 REASON AND REGULARITY OF SURFACE SETTLEMENT

# 2.1 Reason of surface settlement

The reason of surface settlement caused by shield tunneling can be generalized as follows:

- 1. Ground loss
  - Ground movement of working;
  - Shield setback;
  - Soil piling into shield gap;
  - Changing direction of shield driving;
  - Ground friction and shearing because of shell movement of shield machine;
  - Deformation and settlement of tunnel lining.
- 2. Consolidation of disturbed soil.

# 2.2 Universal regularity of surface settlement

When shield method is adopted in saturated soft clay, the universal regularity of ground deformation along longitudinal axis of shield tunnel is presented in

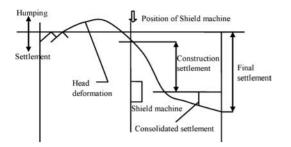


Figure 1. Universal regularity of surface deformation.

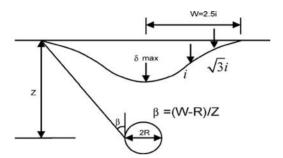


Figure 2. Figure of surface settler above tunnel.

Figure 1. Usually ground surface will give birth to little humping because of uprising and heading of squeezed soil body in front of shield machine. When shield machine passes, bilateral soil body will move outward. When tunnel lining disengages from the end part of shield machine, ground surface will occur rather large settlement and the speed rate of settlement is rather large.

# 3 ANALYSIS OF SETTLEMENT EQUATION

#### 3.1 Peck's equation

Peck(1969) adopted the following expression to describe the settlement component due to the construction of a shield tunneling. Peck's equation is expressed in Figure 2.

$$S(x) = \frac{V_i}{\sqrt{2\pi i}} \exp\left(-\frac{x^2}{2i^2}\right) \tag{1}$$

where S(x) = value of settlement;  $V_l$  = value of ground loss in unit length of shield tunnel; x = distance to center line of tunnel; i = spread factor of settler.

Peck's equation expresses the conception that surface settler is approximate normal distribution. And this estimated equation considers that ground movement is caused by ground loss. Because it is based on engineering experience, Peck's equation exists

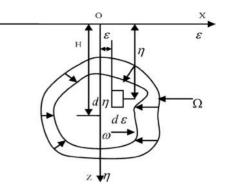


Figure 3. Schematic plan of tunnel driving.

some shortages (e.g. considering fewer factor and lack of theoretical foundation). If value of parameters exists more difference in common condition, result of calculation may be influenced obviously.

#### 3.2 Stochastic medium model

Schematic plan of tunnel driving of stochastic medium model is shown in Figure 3. In this method, after a long time  $(t \rightarrow \infty)$ , the final value of element subsidence is presented in plane strain state as follows:

$$S_{e}(x) = \frac{1}{r(z)} \exp\left[-\frac{\pi}{r^{2}(z)}x^{2}\right] d\varepsilon d\eta$$
<sup>(2)</sup>

$$r(z) = \frac{z}{\tan \beta}$$
(3)

where  $S_e(x)$  = value of element subsidence in Z level plane; r(z) = main range of influence due to element driving in Z level plane;  $\beta$  = main angle of influence.

Applying superposition principle, value of crosssection surface settlement (z = 0) due to tunnel driving may be calculated by

$$S(x) = \iint_{\Omega \to \omega} \frac{\tan \beta}{\eta} \exp\left[-\frac{\pi \tan^2 \beta}{\eta^2} (x - \varepsilon)^2\right] d\varepsilon d\eta \tag{4}$$

where S(x) = value of cross-section surface settlement;  $\Omega$ ,  $\omega$  = area of tunnel driving and tunnel in unit length;  $\Omega - \omega$  = area of ground loss.

The solution of equation (4) depends on double integral. However, the integrands of these integrals can not be integrated, so the method of numerical integration must be applied. Here, value of integral can be gained by Legendre-Gauss method,

$$f(\varepsilon,\eta) = \frac{\tan\beta}{\eta} \exp\left[-\frac{\pi \tan^2\beta}{\eta^2} (x-\varepsilon)^2\right]$$
(5)

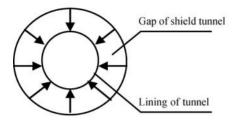


Figure 4. Model of uniform radial displacement.

$$s(x) = \sum_{i=1}^{n} \sum_{j=1}^{n} A_i A_j f\left(\lambda_{1i}, \lambda_{2j}\right)$$
(6)

where *n* = number of integral point; *A<sub>i</sub>*, *A<sub>j</sub>* = weight number of Gauss;  $\lambda_{1i}, \lambda_{2j}$  = interpolation points of integrated variable.

#### 3.3 Analysis of computing settlement

Based on theory of ground deformation presented by Sagaseta (1987), Verruijt & Booker (1996) extend the theory and present new analytical equation of ground deformation due to tunnel driving in elastic half-space, which is also based on equivalent ground loss model (Lee et al. 1992). Verruijt & Booker adopted the model of uniform radial displacement (Fig. 4).

Parameter of equivalent ground loss model is defined as

$$\varepsilon = \frac{\pi \left(d+g\right)^2 - \pi d^2}{\pi d^2} \times 100\% \tag{7}$$

where d = external diameter of tunnel; g = parameter of gap.

Verruijt & Booker's equation is expressed as follows(not considering deformation of tunnel lining in long-time effects and considering undrained state):

$$U_{z} = -\varepsilon \frac{d^{2}}{4} \left( \frac{z_{1}}{r_{1}^{2}} + \frac{z_{2}}{r_{2}^{2}} \right) + \frac{\varepsilon d^{2}}{2m} \left[ \frac{(m+1)z_{2}}{r_{2}^{2}} + \frac{mz(x^{2} - z_{2}^{2})}{r_{2}^{4}} \right]$$
(8)

where *H* = embedded depth of tunnel; v = Poisson'sratio;  $z_1 = z - H$ ;  $z_2 = z + H$ ;  $r_1^2 = x^2 + z_1^2$ ;  $r_2^2 = x^2 + z_2^2$ ; m = 1/(1 - 2v).

When z = 0, surface settlement is calculated by

$$s(x) = (1 - \nu)\varepsilon d^2 \frac{H}{x^2 + H^2}$$
(9)

# 4 COMPARISON OF METHODS OF SETTLEMENT COMPUTATION

In fact, the gap of tunnel driving (ground loss) must be defined for several universally adopted method of surface settlement above. And it corresponds to separately:

- $-V_l$  in Peck's equation;
- $\Omega \omega$  in stochastic medium model;
- $-\varepsilon$  in Verruijt and Booker's equation

Assuming that ground loss of tunnel is a infinitesimal element  $- d\xi d\eta$ .

Calculating by Peck's equation:

$$S(x) = \frac{1}{\sqrt{2\pi i}} \exp\left[-\frac{1}{2i^2}x^2\right] d\xi d\eta \tag{10}$$

Calculating by stochastic medium method:

$$S(x) = \frac{1}{r(z)} \exp\left[-\frac{\pi}{r^2(z)}x^2\right] d\xi d\eta \tag{11}$$

Contrasting equations (10) and (11), if

 $r(z) = \sqrt{2\pi i} = 2.5i,$ 

the two equations are completely identical.

On the one hand, the approximate solution of surface settlement corresponding to different value of x can be obtained, by the numerical integration of equation (4). Then the curve of surface settlement due to tunnel driving will be gained by curve fitting of the result above. On the other hand, regression analysis can be applied into equation (4) according to the format of Gauss curve:

$$S(x) = A \exp\left(-\frac{x^2}{B}\right)$$
(12)

The model of ground loss is usually obtained by assumption, and here surface settlement will be calculated separately by three methods above, adopting model of uniform radial displacement.

Assuming R = d/2 = 3 m, g = 0.1 m,  $\beta = 50^{\circ}$ , and surface settlement will be calculated according to two conditions ( $z_0/R = 1.3$  or 6.7,  $z_0 =$  distance between ground surface and center of tunnel). By conversion:

$$V_l = 1.85m^2, i = 0.336z_0, \varepsilon = 6.78\%$$

By regression analysis, equation (12) is expressed: If  $z_0/R = 1.3$ ,

$$s(x) = 280.51 \exp\left(-\frac{x^2}{11.6}\right)$$

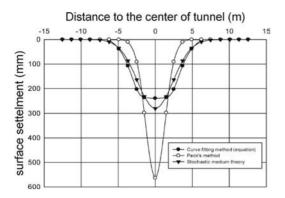


Figure 5. Comparison of computing methods.

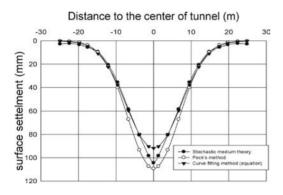


Figure 6. Comparison of computing methods.

If 
$$z_0/R = 6.7$$
,

$$s(x) = 91.82 \exp\left(-\frac{x^2}{103.09}\right)$$

Figure 5-6 indicate that the computing results exist less difference between Peck's equation and stochastic medium model if embedded depth of tunnel is high (e.g.  $z_0/R > 6$ ). If embedded depth of tunnel is low (e.g.  $z_0/R < 2$ ), calculation error is relatively large between two methods. Besides, the curve of surface settlement by stochastic medium model corresponds with the superimposition of Gauss curves of surface settlement due to series of infinitesimal elements driving. Therefore, it is related with geometric character of working face, convergence style after driving, not corresponding to Gauss distribution completely in strict meaning and especially generating relatively large error being close to maximum settlement. On the other hand, Peck's equation directly agrees to Gauss distribution. In a word, stochastic medium theory should be the theoretical basis of Peck's equation, and Peck's equation is the simplified method of stochastic medium

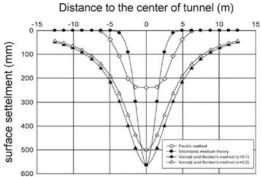


Figure 7. Comparison of computing methods.

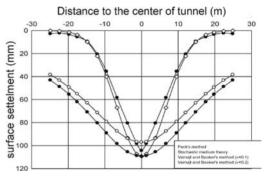


Figure 8. Comparison of computing methods.

method, but adopting different methods for parameter selecting.

Figure 7–8 indicate that Poisson's ratio of Verruijt and Booker's method locates between 0.1 and 0.2, adopting relative method of parameters. And its result is close to Peck's equation when computing maximum settlement. If embedded depth is relatively high, the maximum value of settlement is very close.

# 5 PRINCIPLE OF APPLICATION OF COMPUTING METHODS

At present, methods of settlement computation above have been applied into practical engineering. According to analysis above, it is the key to correctly selecting parameters for different methods. In other words, whether method is applied, the separate method of parameter selecting should be established by experience or theory.

Peck's equation has been applied in the district of soft soil in Shanghai, because of its characters of simplicity and practicality. And the principle of parameter

	• SZ05-1 , SZ05-2	SZ10-1	• SZ15-1 SZ15-2		SZ171-1 SZ171-2	• SZ176-1 SZ176-2	• SZ181-1 SZ181-2
SZ01	*SZ05-3 SZ05	• SZ10-3 SZ10	* SZ15-3 SZ15	Direction of	• SZ171-3 SZ171	*SZ176-3 SZ176 S	*SZ181-3 Z181 SZ185
	• SZ05-4	• SZ10-4	• SZ15-4	shield griving	•SZ171-4	• SZ176-4	• SZ181-4
	SZ05-5	SZ10-5	SZ15-5		SZ171-5	SZ176-5	SZ181-5
	* SZ05-6	* SZ10-6	* SZ15-6		• SZ171-6	*SZ176-6	• SZ181-6

Figure 9. The placement of monitoring points.

selecting has been generalized. Spread factor of settler is expressed as follows (Clough & Schmidt 1981):

$$\frac{i}{R} = \left(\frac{z_0}{2R}\right)^{0.8} \tag{13}$$

Stochastic medium theory can consider multiform factors (e.g. different job practices of tunnel and sectional styles, etc.), and can calculate multiform conditions of ground movement (e.g. horizontal displacement of ground surface and surface sloping, etc.). About parameter selecting, integral domain ( $\Omega - \omega$ ) and angel of influence ( $\beta$ ) need to be obtained by back analysis of measured monitoring data.

Verruijt & Booker's method was modified later (Loganathan & Poulos 1998, Bobet 2001). The result relatively corresponds with practice, but these methods still exist some problem (e.g. uncertainty of parameters of ground loss, curve shape, etc.).

In a word, surface settlement can be predicted by Peck's equation or analytical equation when solving problem of parameter selecting according a mass of statistic of practical engineering. To important and complicated conditions, stochastic medium method associated with back analysis is able to gain more information and more precise resolution.

#### 6 ANALYSIS BASED ON MEASURED MOINTORING DATA

The paper chooses a constructing tunnel at Shanghai Metro line 7 as subject investigated. The tunnel adopts shield method of earth-pressure balance through two main layers:

- Muddy clay belonging to typical soft soil;
- Silty clay belonging to hard soil.

The longitudinal direction of tunnel is "V" in shape, and the tunnel passes through the crossover facet of two soil layers. The placement of monitoring points on ground surface is shown in Figure 9, and the spacing interval of monitoring points is 5 m. The surface settlement will be separately researched by mainly considering two factors (ground movement of working face and soil piling into gap of end part of shield machine).

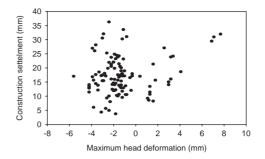


Figure 10. The relationship between maximum surface deformation before shield arriving and maximum surface settlement after shield heading at the same points.

#### 6.1 Surface settlement caused by ground movement of working face

Figure 10 generalizes the relationship between maximum surface deformation before shield arriving and maximum surface settlement after shield heading at the same points. In fact, they appear to dispersive distribution: surface deformation before shield arriving is either humping or settlement and corresponding surface settlement after shield heading is either large or small. Therefore the whole process of surface deformation at certain point is influenced by multiform factors (e.g. soil layer of tunnel driving, embedded depth, construction procedure, etc.).

#### 6.2 Final settlement during construction

 $V_l$  in equation (1) is calculated by

$$V_t = \frac{\pi}{4} \times D_y^2 \times V_t \% + V_h \tag{14}$$

where  $D_y =$  external diameter of tunnel;  $V_l \% =$  ratio of ground loss;  $V_h =$  ground loss due to ground movement of working face.

Table 1–2 and Figure 11 indicate:

- In order to control surface settlement in the district of complicated surrounding, the method of lowspeed driving and synchronal injecting as soon as possible can be adopted to gain good result (e.g. S77,S117).
- 2. The ratio of ground loss caused by ground humping before shield arriving in ground loss caused by soil piling into gap of end part of shield machine is very little. So ground disturbance before shield arriving can be controlled at reasonable range at present in Shanghai.
- The amount of humping has an effect on the constriction of maximum surface settlement during construction, but larger humping will give birth to larger ground disturbance and influence subsequent value of consolidation settlement.

Table 1. Parameters calculation of monitoring points based on Peck's equation.

Monitoring points	Z <sub>0</sub> m	i m	S <sub>hmax</sub> mm	$V_h$ mm <sup>2</sup>	S <sub>max</sub> m	$V_l$ mm <sup>2</sup>
SZ47	14.89	6.25	-1.0	-0.02	17.1	0.27
SZ77	16.80	6.88	-1.2	-0.02	8.50	0.15
SZ107	16.90	6.91	-1.5	-0.03	16.7	0.29
SZ117	16.61	6.82	-2.7	-0.05	5.60	0.10
SZ127	15.85	6.57	-1.8	-0.03	14.5	0.24
SZ137	14.70	6.18	-3.6	-0.06	14.4	0.22
SZ157	13.13	5.65	-1.4	-0.02	13.4	0.19
SZ171	11.95	5.24	-1.7	-0.02	24.3	0.32
SZ176	11.95	5.24	-3.4	-0.04	17.6	0.23

\* $S_{hmax}$  is the maximum value of humping at monitoring points, and  $S_{max}$  is the maximum value of settlement at monitoring points.

Table 2. Controlled maximum settlement and measured maximum settlement.

Monitoring points	S <sub>cmax</sub> mm	S <sub>max</sub> mm	
SZ47	14.0	17.1	
SZ107	12.5	16.7	
SZ127	12.0	14.5	
SZ137	14.0	14.4	
SZ157	16.5	13.4	
SZ171	15.3	24.3	
SZ176	15.3	17.6	

\*S<sub>cmax</sub> is the maximum allowable value of settlement at monitoring points.

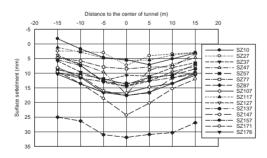


Figure 11. Final settlement during construction period.

4. Constructing shield-driven tunnel in Shanghai, 30 mm is regarded as controlling standard for maximum settlement (considering that the depth of earth covering is 4 m). With many metro lines constructed and larger embedded depth, the new standard of controlling surface settlement is established according to  $V_l \% \le 1\%$  in Peck's equation. By analyzing monitoring result, the construction technology is satisfied with the requirement of controlling surface settlement when the depth of earth covering is 15 m or so.

# 7 CONCLUSIONS

By comparison of methods of settlement calculations which have been applied into computing surface settlement due to shield tunneling, the paper analyzed the relativity between Peck's equation and stochastic medium method, and researched the application problem in practical engineering. Associated with the measured data of a constructing tunnel, some characters of surface deformation have been analyzed to appraise current level of construction technology in soft ground in Shanghai. At the basis of established normal ground loss, the quality of construction can be determined and job practice can be adjusted to meet the requirement of environmental protection.

# REFERENCES

- Bobet, A. 2001. Analytical solutions for shallow tunnels in saturated ground. *Journal of Engineering Mechanics* 127(12):1258–1266.
- Clough, G.W. & Schmidt, B. 1981. Design and performance of excavations and tunnels in soft clay. New York: Elsevier Science Publishing Company.
- Lee, K.M. & Rowe R.K. 1992. Subsidence owing to tunneling I:Estimating the gap parameter. *Canadian Geotechnical Journal* 29(6): 929–940.
- Liu, J.H. & Hou X.Y. 1991. Shield-driven tunnel. Beijing: China Railway Publishing House.
- Loganathan, N. & Poulos, H.G. 1998. Analytical prediction for tunneling-induced ground movement in clays. *Journal of Geotechnical and Geoenvironmental Eengineering* 124(9): 846–856.
- Peck, R.B. 1969. Deep excavations and tunneling in soft ground. Proceeding of 7th international conference on soil mechanics and foundation engineering. Mexico City: State of the Art Report.
- Sagaseta, C. 1987. Analysis of undrained soil deformation due to ground loss. *Geotechnique* 37(3):301–320.
- Verruijt, A. & Booker, J.R. 1996. Surface settlements due to deformation of a tunnel in an elastic half plane. *Geotechnique* 46(4):753–756.
- Zhu, Z.L. & Zhang, Q.H. 2001. Stochastic theory for predicting longitudinal settlement in soft-soil tunnel. *Rock and Soil Mechanics* 22(1):56–59.

# Urban tunnels in soil: Review of current design practice in Brazil

# A. Negro

Bureau de Projetos, São Paulo, Brazil

ABSTRACT: The current design practice of urban tunnels in soil as perceived in Brazil is reviewed and discussed. The review is based on answers to a questionnaire sent in 2006 to practitioners involved in the design or in the design supervision of tunnel projects. The results of this investigation are carefully considered to identify trends and needs for development.

#### 1 INTRODUCTION

The first assessment of the design practice of urban tunnels driven in soil performed in Brazil was carried out in 1993 through a survey whose results were published in the 1st. International Symposium on Underground Constructions in Soft Ground in New Delhi (Negro and Leite, 1995).

This time cut-and-cover structures are not included and the review covers underground openings of any shape with more than 0.5 m diameter, directional drillings also not included. The survey is not limited to tunnels built in Brazil and covers any tunnel project conducted with criteria and procedures presently adopted in Brazil. In all cases tunnels were designed and built in areas with buildings on the surface and utilities on the subsurface, both liable to damages induced by ground movements associated to tunnel excavation.

Multiple choice questions to which one or more answers could be selected were sent to professionals involved in the design or in the design supervision of tunnel projects over the last few years. Considering the multidisciplinary nature of tunnel design, the questionnaire was sent to practitioners from a variety of expertise and included topics beyond strict geotechnical context. Accordingly, questions were sent to geotechnical engineers, geologists, structural engineers, experts in numerical modelling.

These professionals could opt out questions transcending their specialties. Most questions asked for answers expressing the practitioner current preference on each technical aspect of tunnel design, in such a way that they translated the respondents routine design practice. The questions were sent to 30 tunnel experts and 20 replied the survey.

The results presented below are grouped withinrelated topics. One should note that respondents offered sometimes more than one answer to a question. Therefore, frequencies shown refer to percentage of total answers offered or to percentage of respondents as applicable.

# 2 THE SCENARIO OF THE PRACTICE

To make simpler the analysis of the answers, few questions were formulated to define the scenario of the design practice to which it refers.

More than two thirds of the answers refer to tunnels built for trains, metro systems and vehicles in general.

Accordingly, more than four fifths of the answers (from 26 given) refer to tunnels with more than 4 to 6 m diameter (almost two thirds with diameters in excess of 6 m).

The ground condition more frequently encountered, is that of tunnelling with mixed face. Tunnelling through cohesionless ground is not frequent. On the other hand, more frequently than not, tunnelling takes place below water table.

#### 3 LINING DESIGN AND WATER PROOFING

The type of secondary lining more frequently used over the recent years is the wet mix sprayed concrete (45%), followed in equal proportions by the dry mix and by the cast in place concrete (23% each). The answers are consistent with the larger tunnel sizes focused. Steel liner plates as final lining are not in use, possibly for being liable to electric or chemical corrosion in adverse urban scenarios.

The survey revealed that wire meshes are the most popular type of reinforcement for secondary linings. It is somewhat surprising to see that lattice girders and steel sets are still in use, though they are not usually taken into account in the design of secondary linings. Steel fibers are not much used and it is almost non existent the old practice of unreinforced concrete secondary linings: it looks like that there is a general perception that it is necessary to ensure an adequate ductility rendered by steel reinforcement to final tunnel linings, assuring that brittle failure of the lining do not take place under unexpected catastrophic conditions. This is in line with the requirement of minimum steel reinforcement accepted by 85% of the respondents (the remaining do not adopt it).

Forty percent of the respondents explained that they use only the minimum steel section for bending whereas 60% adopt the minimum both for bending and for shrinkage control.

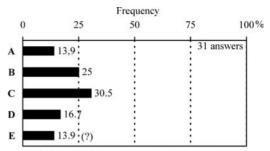
60% of the respondents either never design tunnels to be waterproof (15%) or they did it only few times (45%). The most favorite technique to water proof tunnel linings are impervious geomembranes, system (39% of the answers) that has been just introduced in the country. On the other hand this is seconded by a group of more traditional procedures in Brazil that aims the reduction of concrete permeability, by special mix design, special admixtures and controlled curing of the concrete (summing 32,3% of the answers). This group combined with the design of concrete with limited crack widths total 45% of the answers, while 5 to 10 years ago they would count for 100% of the answers.

#### 4 GEOTECHNICAL DESIGN

All respondents declared they normally perform stability analysis of the tunnel heading. Figure 1 shows that almost one third of the answers indicate the use of the safe Lower Bound solutions from the Theory of Plasticity. Some prefer the use of limit equilibrium methods or the sole use of Upper Bound solutions from Plasticity, which are normally unsafe approaches. Empirical methods (that can either be safe or unsafe) and finite element analysis with elastic plastic models ranked equally, with almost 14% of the answers both. It should be noted that within the latter, none of them were really Numerical Limit Analysis (NLA), such as that presented by Durand el al (2000). In fact they refer to 2D FE analysis in which a full ground stress unloading is applied to check if the 2D opening withstands it, without any support.

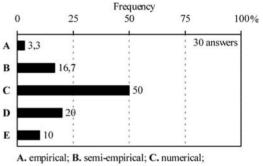
It is worth noting that respondents adopt, in equal proportion (40%), distinct approaches for stability analysis of headings of tunnels built with a shield and without it (mined tunnel, NATM). No formal reason justifies such practice.

All respondent confirm that they normally predict the settlements induced by tunnel construction in their routine practice and that this is done through numerical analysis (both finite elements and finite differences) – see Figure 2. Comparing these results, with those of the 1993 survey, a considerable decrease in the use of

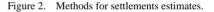


A. limit equilibrium; B. empirical methods; C. Lower Bound solution from Plasticity; D. Upper Bound solution from Plasticity; E. finite element methods with plastic models.

Figure 1. Methods of analysis for assessment of tunnel heading stability.

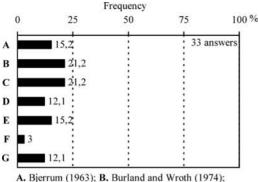


D. numerically derived; E. no indication of preference.



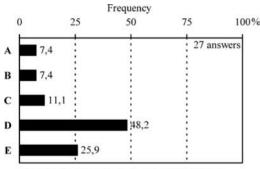
empirical and semi-empirical methods is noted (from 60% to 20% now). With respect to the assessment of induced damages on buildings (Figure 3), no clear preference to a single method or procedure is noted: the methods by Burland and Wroth (1974) and by Boscardin and Cording (1989) were those with more indications (21% of the answers both). The most used plane static systems to calculate normal forces and bending moments in the tunnel lining, accounting the interaction between it and the ground (see Figure 4) are numerical solutions with 74% of the answers (48% for FE and 26% for FD). A considerable increase in the use of these types of solutions was seen in the country since 1993, when only 43% of the answers favored them.

It was noted also that in the design of the primary lining, using 2D static systems, 85% of the cases consider a reduction of the geostatic stresses and the remaining 15% of answers consider or not this reduction, depending of the specific conditions of the case being studied. The procedure more frequently used to estimate the reduction on the ground stresses (Figure 5)



C. Boscardin and Cording (1989);
 D. Mair, Taylor and Burland (1996);
 E. Namba et al. (1999);
 F. other methods: G. no answer.

Figure 3. Methods for assessment of damages induced by tunnelling on buildings at ground surface.



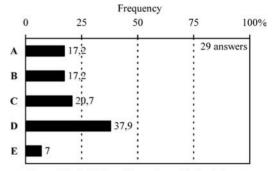
A. closed form solutions; B. analytical ring and spring solutions; C. numerical solutions with lining as bar elements and soil as discrete springs;
D. finite element analysis; E. finite difference analysis.

Figure 4. 2D static systems to assess lining loads.

are those derived numerically (Negro and Eisenstein, 1997, for instance), followed by Terzaghi's arching theory, notwithstanding the fact that this corresponds to large ground stress releases associated to large induced displacements, which are not compatible to an urban scenario.

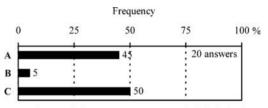
When designing the secondary lining, only 59% of the practitioners assume a reduction on the ground stresses, possibly reflecting an old and debatable believe of long term creep of geological materials, leading to fully decayed shear strength.

Practitioners were asked how to account for groundwater loads on the sprayed concrete primary lining of a tunnel driven below water table (see Figure 6). 50% of the answers indicate that no account is taken of water pressures, on the assumption that the soil is



A. empirical relations; B. semi-empirical relations;
C. Terzaghi's arching theory; D. numerically derived methods; E. other procedures.

Figure 5. Procedures to estimate reduction of ground stresses.

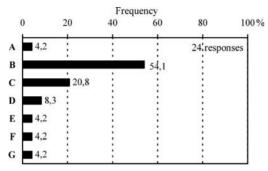


A. do not take into account: assumes soil fully drained;B. assume drained ground and consider seepage forces;C. consider pore water pressures and effective ground stresses (reduced or not).

Figure 6. Account of groundwater loading over a primary lining of a tunnel below water table.

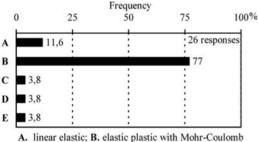
fully drained (45% of answers) by compulsory dewatering; few (5%) take into account the body forces resulting from dewatering the ground. The other half of responses indicate that they take into account the acting pore water pressures and the effective stresses onto the lining (either reduced or not).

It is worth noting (see Figure 7) that more than half of answers offered indicate that practitioners either do not take into account the effects of pore water pressures in the soil behaviour (4%) or limit the account to the water pressure onto the lining at most (54%). Just over 20% perform numerical analysis coupling soil stresses and pore water pressures using finite elements. Also worth noting (Figure 8) that almost 80% of the answers indicate the use of elastic-plastic models associated to the Mohr-Coulomb failure criterion. The problem with this popular option is that considering that we are dealing with an urban tunnel, in which considerable efforts are spent in reducing the degree of ground stress relaxation and soil deformation, plastic straining is usually prevented and ground response



A. do not account; B. consider only the water pressure over the lining; C. perform coupled analysis with stresses and pore pressures; D. perform uncoupled analysis; E. use flow nets to asses drainage effects and dewatering on pore pressures; F. assess pore pressure changes and equalizations using analytical or numerically derived solutions; G. no response.

Figure 7. Account of pore water pressure on soil behaviour.

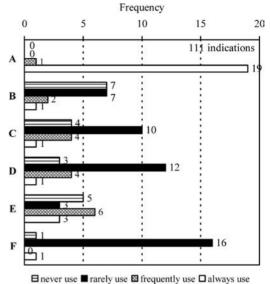


A. linear elastic; B. elastic plastic with Mohr-Coulomb failure criterion; C. non-linear elastic; D. Cam-Clay family; E. others.

Figure 8. Type of constitutive models used for soil in numerical analysis.

is reduced to a linear elastic behaviour, an oversimplified portray of a material known to exhibit non linear behaviour even at small strain range.

Despite the 3D stress redistribution around a tunnel heading, rarely or never (55% of respondents) 3D numerical analysis are performed in routine design. When they are performed, they refer to tunnels intersections or start up shafts of tunnels. When 3D analysis are performed, the most frequently used type of analysis (in 32% of the answers) involve shell elements for the lining and bars or springs to represent the soil. 3D finite element programs normally used are SAP, Ansys, Plaxis and FLAC 3D. The more intense use of 3D analysis already noted in the academy (see the 5th Proceedings of TC28 Symposium in Amsterdam, 2005), resulting from the availability of more powerful PCs and more efficient numerical codes, will also be note in industry in the next few years.



A. SPT and SPT-T; B. CPT, deep sounding; C. DMT,

dilatometer, and presuremeter; **D**. plate bearing tests; **E**. disturbed sampling and soil classification tests; **F**. undisturbed sampling and triaxial tests.

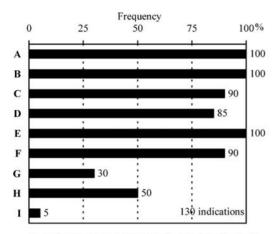
Figure 9. Choices for geotechnical investigations.

#### 5 GEOTECHNICAL INVESTIGATIONS AND MONITORING

One of the questions prepared referred to the preference of practitioners with respect to different geotechnical investigations performed in the field and in the laboratory.

They were asked to indicate, in a scale from 0 (never use) to 3 (always use), their choices regarding each class of investigation. Results of this survey are shown in Figure 9, from which it can be noted that: a) standard penetration tests are always requested; b) cone penetration testing and undisturbed sampling followed by laboratory special testing are never or rarely requested; c) disturbed sampling followed by characterization tests in the laboratory are frequently used; d) dilatometers, DMT, pressuremeters and plate bearing tests are rarely used.

With respect to field monitoring, 81% of the answers confirmed that this is always used in practice or that it is frequently requested (19%). The answers indicated also the frequency in using different instruments (Figure 10). Leveling surface settlement points, building leveling and convergence measurements are performed in 100% of the cases. Deep settlement points and lining levelings and the use of piezometers or of water level indicators are observed in 85% to



A. surface monuments leveling; B. buildings leveling;
C. deep settlement points leveling; D. piezometers and water level indicators; E. convergence measurements;
F. tunnel lining leveling; G. load cells, stress or strain meters in the lining; H. clinometers and slope indicators;
I. other instruments.

Figure 10. Frequency in using field monitoring.

90% of the cases. Inclinometers, load cells and strain or stress meters are less frequently used.

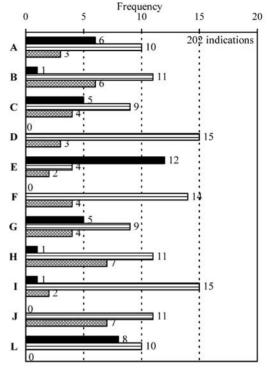
#### 6 SATISFACTION WITH THE PRACTICE

Finally, the survey investigated the areas where practitioners believe that there are needs for technical developments to be introduced in their practice and areas they are satisfied with the current practice and new developments are less needed (Figure 11). The highest levels of satisfaction refer to available construction methods and lining types. The area with least satisfaction refers to the prediction of pore water pressure and water flow. Practitioners are also unhappy with the available water proofing techniques and with the available methods for stability analysis.

#### 7 CONCLUSIONS

The conclusions found refer to the scenario of the practice in Brazil, defined in broad terms by the respondents: large size tunnels, with equivalent diameter larger than 6 m, designed for rails, metros, highways, driven under mixed face condition, in cohesive soils, below water table using sprayed concrete as lining (NATM).

Steel liner plates are not much in use, and are being replaced by wet mix sprayed concrete or by cast in place concrete as secondary lining. This may be explained by the higher risks of corrosion of the former in urban environments, but attention should be



■ Not satisfied ⊟ marginally satisfied ⊠ fully satisfied A. stability analysis; B. soil-structure interaction; C. prediction of settlements and associated damage; D. prediction of lining loads and lining structural design; E. prediction of pore pressures and ground water flow; F. field monitoring; G. laboratory and field investigations; H. available construction methods; I. available methods for ground conditioning; J. available lining types; K. available water proofing systems; L. other.

Figure 11. Satisfaction with the current practice.

paid to problems related to alkali-silica reaction (ASR) frequently overlooked in concrete lining design.

It was also noted that the old practice of using unreinforced concrete secondary lining is declining, possibly revealing a general perception that the design should ensure an adequate ductility to the concrete lining, rendered by the steel reinforcement. On the other hand, it should be argued what is the impact of the steel reinforcement in the lining safety when subjected to the high temperatures of a fire.

A considerable increase in the use of water proof membranes has been observed whereas until recently, the control of water ingress through the lining was exerted only by using low permeability concrete. One wonders if this change may not be followed by lessening of the concrete quality in the future. It is still high the frequency in the use of limit equilibrium methods and of empirical methods to assess the stability of the tunnel heading and face, though it is reckoned that these approaches may provide unsafe estimates.

Further, an unjustified posture and criterion regarding the assessment of stability of a NATM and a TBM driven tunnels were noted. For the latter, it looks like that the tunnel designer delegates to the contractor or to the machinery manufacturer, the responsibility to ensure the tunnel face stability and, in general, they accept it, inasmuch they may not have the technical abilities to judge what is the best operational procedure of the equipment at hand, for a certain geotechnical scene. This subtle omission of the geotechnical engineer is technically and professionally noxious. The study of cases of frequent instabilities observed in EPB shields may put some light on this issue.

The use of numerical methods (FE or FD) is widespread in the local practice for settlements estimates or for lining design. Equally widespread is the assumption of reduced ground stresses in 2D tunnel models. However, it is unjustified the persisting practice of using Terzaghi's arching theory to estimate that reduction on geostatic stresses, which is doomed to be put aside in the practice of urban tunnels.

It is intriguing to observe that practitioners either do not take into consideration the effect of pore water changes in the soil behaviour or account only the water pressure over the lining. FE numerical analysis coupling generation of pore pressure and stresses are infrequent.

The most used constitutive models in numerical analysis are those with the elastic plastic behaviour with Mohr-Coulomb failure criterion. These models when applied to tunnels built with limited relaxation of ground stresses, resulting from restrictive construction methods, used to inhibit ground movements and associated damages in a urban scene, result in linear elastic soil responses, with inhibited or limited plastic zones. This is not in line with the reckoned non-linear behaviour of soils even at small straining: there is a clear need of using more adequate soil modeling in the local practice.

The use of three-dimensional modeling in practice is yet limited, but it is believed that the increased use of this type of modeling, already noted in the academy, will be seen in local practice soon.

A poor practice in geotechnical investigation for tunnel design was noted. It tends to be limited to soundings with SPT blow counts and simple lab testing in deformed soil samples. This may be so because we are focusing urban tunnels in environments which are, in general, well known both in geological and geotechnical terms, for which sizeable data bank is available. Perhaps a way to compensate such deficiency is to stimulate the use of in situ testing such as the pressuremeters, the dilatometers and others. A favourable situation is seen in the practice, in terms of field monitoring. Field instrumentation is always present. The noted deficiency is the lack of measurements of lining loads. There are a number of robust and reliable procedures to assess loads in concrete linings, using stress release techniques (see, for instance, Negro, 1994) that could be used at low costs.

The respondents believe that the area in need for further technical developments is that of pore water pressure estimates and ground water flow. Moreover, they are not satisfied with available techniques for water proofing tunnels and for assessment of stability analysis of tunnel heading and face. The survey indicates areas in the practice where the academy may contribute significantly for technology advancement.

#### ACKNOWLEDGEMENTS

This investigation has been possible only thanks to the kind attention of those that replied the questionnaire sent. The author is grateful to all of them for the efforts and time spent in replying.

#### REFERENCES

- Bjerrum, L. 1963. Discussion to European Conf. On Soil Mech. Found. Eng. (Wiesbaden), Vol. II, p. 135.
- Boscardin, M.D. & Cording, E.J. 1989. Building response to excavation induced settlement. *Journal of Geotechnical Engineering*, ASCE 115(1): 1–21.
- Burland, J.B. & Wroth, C.P. 1974. Settlement of buildings and associated damage. *Conf. on Settlement of Structures*, London: 611–654.
- Durand, A.F., Vargas, E.A. & Vaz, L.E. 2000. Some Experiments in 3D Numerical Limit Analysis (NLA). 3rd Intl. Conf. On Advances of Computer Methods in Geotech and Geoenvironmental Engineering Geoecology and Computers, Vol. 1: 223–227.
- Mair, R.J., Taylor, R.N. & Burland, J.B. 1996. Prediction of Ground Movements and Assessment of Risk of Building Damage due to Bored Tunneling. Proceed. Intl. Symp. Geotechnical Aspects of Undeground Construction in Soft Ground, London, UK: 713–718.
- Namba, M., Ruiz, A.P.T., Queiroz, P.I.B., Negro, A. & Vasconcellos, C.A. 1999. Assessment of Building Damages Due to Urban Tunnelling. *Proc. 11th Panam. Conf. Soil Mech. And Geotech. Eng.*, Vol. 2: 549–555.
- Negro, A. 1994. Soil Tunnels and Their Supports. Special Conference to the 10th Brazilian Congress of Soil Mech. Found. Engineering, Iguaçu Falls, Brazil, ABMS, Proc. Vol. 5: 33–60.
- Negro, A. & Eisenstein, Z. 1997. Delayed Lining Activation and Ground Stress Relaxation in Shallow Tunnels. *Proc. of The 4th Intl. Conference on Soil Mechanics and Foundation Eng*, Hamburg, Germany: 2391–2396.
- Negro, A. & Leite. R.L.L. 1995. Design of Underground Structures in Brazil – National Report on Tunnelling and Braced Wall Excavation in Soft Ground. *Proc. Intl. Symposium on Underground Construction in Soft Ground*, New Delhi, India: 49–56.

### A study on loads from complex support system using simple 2D models

Z. Shi, W. Bao, J. Li, W. Guo & J. Zhu

China State Construction Engineering Corporation (SH), Shanghai, P.R. China

ABSTRACT: In deep pit engineering, spatial displacements are readily available, commonly along horizontal or vertical lines. It's traditionally difficult to get strut loads from complex concrete strut system due to limited scale of instrumentation and uncertainties in field measurements. This results in problem in predicting wall deformation where not instrumented. This paper proposes to utilize spatial wall displacements measured to back-analyze supporting loads on the wall from concrete struts through modeling wall-soil system. The analysis can be applied to each strut layer to obtain loads between wailing and wall in horizontal plane. The obtained support loads at different levels are then used as input in a vertical section model, from which deformation profile of wall can be predicted. The application is verified in project case and shows close correlation to field measurements.

#### 1 INTRODUCTION

Reinforced concrete strut system is widely adopted with diaphragm wall for support of deep foundation pits in soft soil ground. During excavation, the diaphragm wall is supported by concrete struts from inside and subject to soil pressure on the other side. It is typical for deep foundation pits to have a few layers of concrete struts at different excavation depths. The interaction between diaphragm wall and concrete struts (through wailing), which is one of the most important factors in analyzing deep foundation pits, presents a very complicated scenario under loads from soil.

Because wailing is in continuous direct contact with diaphragm wall, it is difficult to measure the internal loads between them. By direct method, the internal force of struts is commonly measured by embedded load cells. This measurement is usually insufficient to estimate support loads on diaphragm wall because:

- 1 The scale of field measurements of strut force is limited; and the geometry of strut system is complex. Therefore, it is difficult to derive reliable support loads on diaphragm wall or through complex strut system by mechanical analysis with limited strut force measurements.
- 2 Field measurement of strut force could be deviated by other factors such as creep, contraction and thermal stress of concrete material (Xia & Li, 1999). Zhao (1996) reported significant thermal stress measured in concrete struts.

Therefore, indirect methods are widely used for analyzing the support loads. Back-analysis from displacement is one of the commonly used. By indirect method, there have been numerous 2D and 3D studies on the analysis of such support system (Li & Hu, 1995; Zhao, et. al., 1996).

In contrary to the difficulty of direct measurement of support force, spatial displacement information of diaphragm wall is more readily available and more reliable by field measurement. Therefore, wall displacement measurements, for example in plan section at a strut level, can be used to back-analyze loads on diaphragm wall by a simple wall-soil model.

During excavation, the diaphragm wall deforms subject to the soil pressure on one side and support loads on the other side. The soil pressure can be estimated by empirical method. Therefore, for a system consisting of supports, diaphragm wall and soil, the measured displacement pattern of diaphragm wall on the plan can be used to back-analyze supporting loads on the wall from concrete struts. It is recommended to carry out the back-analysis in plan section since the vertical sections normally involve much more complexity associated with construction sequence etc. In plan of each strut layer, a simple 2D model can easily represent such case scenario. This can be repeated for each layer of strut. Therefore, support loads at a section can be back-analyzed from spatial displacement information from field.

Based on a case history, this paper presents a simple method of estimating support loads on diaphragm wall by wailings. The estimated support loads are then verified in models for vertical cross section.

#### 2 ANALYSIS OF SUPPORT LOADS IN PLAN

#### 2.1 Assumptions and simplifications

On the plan section of each strut layer, the system of diaphragm wall and soil resembles a plain strain scenario at the center elevation of wailing. The diaphragm wall deforms under the support loads and soil pressure after excavation. The sidelines of pit, which in most cases are straight, can be simplified as an elastic continuous wall, i.e. a continuous beam on plan. The deflection pattern of the beam can be easily defined by displacement measurement along the side and is taken as target deformation in back-analysis. The two ends of the sideline can normally be deemed as pinned ends, as shown in Figure 1.

For simplicity, following assumptions are made in the analysis:

- On plan section, the diaphragm wall is modeled as elastic continuous beam.
- The loads between the wailing and diaphragm wall are evenly distributed along vertical direction but vary along horizontal direction for each strut layer.
- On plan of each strut layer, the support loads from wailing are simplified as point loads along the pit side.
- In vertical plan, the support loads of each strut layer are invariable during construction.
- The initial stress level of soil in the model is calculated from its depth following Rankin's active earth pressure formula.

In the plan model, point loads equivalent to the initial stress level prior to excavation are applied onto the beam (diaphragm wall) so that the model is in initial balance with zero displacement in diaphragm wall. In back-analysis, the applied point loads are adjusted from initial values in a controlled manner till that the diaphragm wall deforms approximately to the target displacement pattern, which is defined from field measurements. The result point loads can be converted to internal stress between the diaphragm wall and wailing (Fig. 1 (d)) along the length of sideline. This can be repeated for each strut layer without major changes to the model. In this way, the support loads of each strut layer can be found.

In the model for a vertical cross section, analysis can be carried out to simulate the construction cases corresponding to plan models. The predicted deformation pattern of diaphragm wall can checked against field measurements and the quality of estimated support loads can be verified.

A history case of a deep foundation pit in soft clay in Shanghai area is selected for the application of the

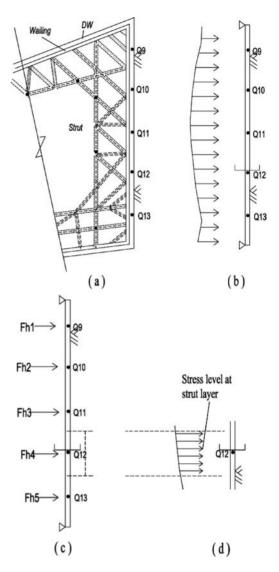


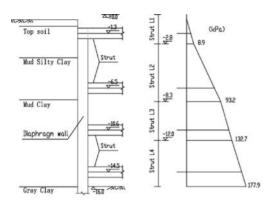
Figure 1. Plan section: (a) Plan of strut layout; (b) Line loads on diaphragm wall from wailing; (c) Converted point loads; (d) Calculated support stress on diaphragm wall.

method. The program used in the study is the FLAC by Itasca CG (Itasca, 1997).

#### 2.2 Model conditions and material properties

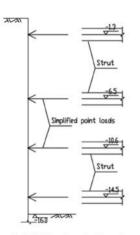
In the case history, one sideline of the pit with a length of about 100m is selected for application (Fig. 1 (a)).

The initial stress level is calculated according to Rankin's active pressure with a surcharge of 20 kPa, as shown in Figure 2 (b). The excavation process is divided into four steps equaling to the number of strut layers (Fig. 2 (b)).





( b ) Exc depth for each strut layer and stress levels



(c) Model of vertical section

Figure 2. Vertical section and simulation conditions.

On each plan section, the loads from wailing on diaphragm wall are simplified as five point loads ( $F_{h1} \sim F_{h5}$ ) at the elevation of each strut layer (Fig. 1 (c)). The target displacements for each support layer ( $S_{1\sim} \sim S_i$ ) are taken at the end of excavation at time  $t_k$ . The back-analyzed support forces ( $F_{h1\sim} \sim F_{h5}$ ) represent the site condition at the end of excavation.

The geometry of the plan model is shown in Figure 3. The soil layers are listed in Table 1 with material properties. The soil is modeled as elasto-plastic material with Mohr-Coulomb strength criteria.

The reinforced concrete diaphragm wall reaches 41 m from ground surface, modeled as elastic continuous beam.

#### 2.3 Analysis of support loads at strut layers

There are five monitoring points available for each layer with field measurement except for the first layer as listed in Table 2. Due to availability of field data,

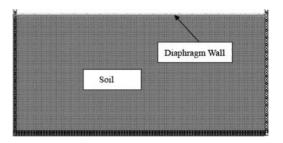


Figure 3. Model mesh and set up for plan sections.

there are measurements for three monitoring points for the first strut layer.

According to excavation history, the corresponding excavation steps for each strut layer are defined as: up to 2.8 m for the first layer;  $6.3 \text{ m} (2.8 \text{ m} \sim 8.3 \text{ m})$  for the second layer;  $3.7 \text{ m} (8.3 \text{ m} \sim 12.0 \text{ m})$  for the third layer; and  $4.0 \text{ m} (12.0 \text{ m} \sim 16.0 \text{ m})$  for the fourth layer. The initial stress condition for each strut layer is determined in the same way as above (Fig. 2 (b)).

To reduce noise during back-analysis, only the field measurements of the center three points (Q10 $\sim$ Q12) are set as target deformation because the other two monitoring points (Q9 and Q13) could be easily affected by fixed ends of diaphragm wall. The matching criteria for model prediction is set as within 5% of field measurement, i.e. the model reaches its target condition when the model predictions at each selected points are within 5% difference from field measurements. Under this condition, the point loads on wall are measured from the model.

The result deformations of back-analysis are shown in Table 3 with corresponding calculated point loads of support in Table 4.

#### 3 VERIFICATION OF CALCULATED SUPPORT LOADS IN VERTICAL SECTION

#### 3.1 Models of vertical section

Two vertical cross sections intersecting the diaphragm wall at Q10 and Q12 are selected to verify the calculated support loads.

The vertical section model has a total width of 120 m with 40 m on the side of the pit as shown in Figure 4. The height of model is 80 m from the ground surface.

The strut levels and excavation steps are shown in Figure 2. The model is prescribed with initial lateral stress conditions as calculated above. The two sides are fixed only in horizontal direction. The bottom is fixed in both directions.

Considering the high stiffness and limited height of wailing, the loads of wailing are assumed evenly distributed along its height at each strut layer. Therefore in plan model, the support loads can be represented

No	Soil Layer	Thickness (m)	Unit Weight (kN/m3)	Modulus (MPa)	Poisson's ratio	Cohesion (kPa)	Friction (°)
1~2	Top Soil	3.5	18.3	11.25	0.35	24	15.5
3	Mud Silty Clay	3	17.4	5	0.35	15	20
4	Mud Clay	9.9	16.6	2.75	0.4	12	11.0
5 <sub>1a</sub>	Grey Clay	4.2	17.5	5	0.35	18	11.5
5 <sub>1b</sub>	Grey Silty Clay	7.2	17.9	12.5	0.35	17	20.5
5 <sub>1c</sub>	Sanded Silty Clay	10.4	17.9	25	0.3	16	23.5
6	Silty Clay	2.4	19.9	36	0.35	51	24.5
	Diaphragm Wall	1.0	20.0	30000	0.17	-	-

Table 1. Material properties of soil and diaphragm wall.

Table 2. Field measurements of deformation at strut layers (mm).

	Q9	Q10	Q11	Q12	Q13
Layer 1	_	0.1	0.3	0.2	- 13.2
Layer 2	15.9	24.3	23.7	19.6	
Layer 3	32.3	52.4	49.0	43.5	23.3
Layer 4	42.0	62.7	59.9	48.0	33.2

Table 3. Calculated deformations at strut layers (mm).

	Q9	Q10	Q11	Q12	Q13
Layer 1		0.1	0.3	0.1	
Layer 2		24.3	24.0	19.6	
Layer 3		52.7	50.1	44.1	
Layer 4		63.0	59.1	48.8	

Table 4. Calculated support point loads at strut layers (kN/m).

	Q9	Q10	Q11	Q12	Q13
Layer 1	150	106	120	136	108
Layer 2	89	34	80	39	170
Layer 3 Layer 4	220 310	50 150	220 320	70 200	382 540

by point loads at the center elevation of wailings as illustrated in Figure 2 (c).

The calculated point loads from plan models are converted to line loads along horizontal direction at all strut layers, where the values at Q10 and Q12 can be obtained (Fig. 1 (d)) for use in vertical model. The values of point load are listed in Table 5.

In correspondence to the calculated support loads, the following construction cases are simulated in the plan model:

Case 1: Initial condition before excavation; installation of diaphragm wall;

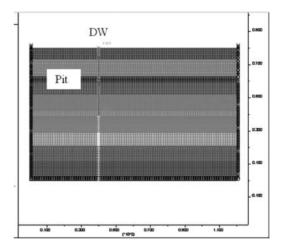


Figure 4. Overview of model for vertical sections.

Table 5. Converted support loads for vertical model (kN/m).

	Layer 1	Layer 2	Layer 3	Layer 4
Section (Q10)	318	136	200	300
Section (Q12)	408	156	280	400

Case 2: Excavate to -2.8 m; apply support load at first strut layer;

Case 3: Excavate to -8.3 m; apply support load at second strut layer;

Case 4: Excavate to -12.0 m; apply support load at third strut layer;

Case 5: Excavate to -16.0 m; apply support load at fourth strut layer.

#### 3.2 Results of vertical section models

The deformation patterns of diaphragm wall at target points are very similar as Q12 shown in are shown in Figure 5.

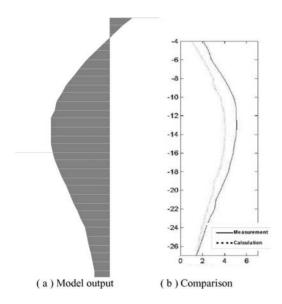


Figure 5. Horizontal deformation pattern (mm) of diaphragm wall along depth (m) in vertical section through Q12.

The calculated deformations of diaphragm wall are compared with field measurements in Figure 5. The maximum calculated displacement is 40.5 mm while 51.1 mm in field measurement on vertical section through Q12. It is noted that all cases show good correlation of deformation pattern between model calculation and field measurement as shown in Figure 5(b).

#### 4 DISCUSSIONS AND CONCLUSIONS

By using spatial displacement measurements in simple 2D models, the difficulties associated with complex support system and uncertainties in load measurement are avoided with reasonable correlation to field data in vertical sections.

However, the displacement in vertical section shows notable discrepancies on magnitude, which is partly a result of simplifications made in the analysis. In the system of diaphragm wall and soil, the calculated support loads are considerably affected by the stress conditions in the model. The influence from stress can be minimized if soil stress measurements are incorporated in the back analysis.

The discrepancies in prediction of vertical model could also result from the assumption that the stress release due to an excavation step is undertaken entirely by a specific strut layer. Therefore the support loads can only be calculated for limited cases in the same number of excavation steps as strut layers.

However, the method produces reasonable estimation of support loads that lead to acceptable prediction of deformation pattern of diaphragm wall. This would be helpful in many engineering cases.

#### ACKNOWLEDGEMENT

The Author would like to acknowledge great assistance and suggestion in preparation of this paper from Dr Dongmei Zhang from Tongji University.

#### REFERENCES

- Bolton, M. & Powrie, W. 1988. Behavior of Diaphragm Walls in Clay Prior to Collapse. *Geotechnique*, 1988, 38(2): 167–189.
- Li, Y. & Hu, Z. 1995, Deformation and Loads of Diaphragm Wall for Deep Foundation Pits. Architectural Construction, Vol 17, 1995(3).
- Itasca, C.G. 1993. Fast Language Analysis of Continua. Itasca Consulting Group Inc.
- SIDA, 1997. Shanghai Standard Code for Design of Excavation Engineering (DBJ08 – 61–97).
- Xia, C. & Li, Y. 1999. Monitoring Theory and Technology in Underground Engineering. Tongji University Press.
- Zhao et al. 1996. Practice and Study of Support System for Deep Foundation Pit. Tongji University Press.

### Ground reaction due to tunnelling below groundwater table

Y.J. Shin Purdue University, West Lafavette, USA

J.H. Shin Kon-Kuk University, Seoul, Korea

I.M. Lee Korea University, Seoul, Korea

ABSTRACT: Tunnelling below the groundwater table influences the hydraulic regime in the surrounding ground. This will, in turn, cause seepage into the tunnel through the pores and discontinuities and often produces a long-term interaction between the tunnel and the ground. In this paper an attempt is made to identify the behavior of surrounding ground due to seepage, and ground reaction curves (GRC) considering seepage forces are presented for a tunnel under drainage conditions using an analytical method. It is found that the flow of groundwater has a significant effect on the radial displacement of a tunnel wall. While the effective overburden pressure is reduced by the arching effect during tunnel excavation, seepage forces still remain. Therefore, the presence of groundwater induces large radial displacement of the tunnel wall compared to dry conditions.

#### 1 INTRODUCTION

Tunnelling below the groundwater table affects the hydraulic equilibrium. This will, in turn, cause seepage into the tunnel through the pores and discontinuities in the ground and often produce long-term interaction between the tunnel and the ground. The effect of seepage on the tunnel is initially reflected in the ground loading, transmitted through the ground and then, eventually applied to the tunnel resulting in additional stresses in the linings. It would be appropriate, therefore, to include the influence of seepage force in estimating ground behavior due to tunnelling.

The ground behavior due to tunnelling can be indicated theoretically by the ground reaction curve which shows the increasing trends of radial displacement as the internal pressure of the tunnel decreases.

In this paper, the theoretical solution of ground reaction curve considering seepage forces due to groundwater flow under steady-state flow was derived. The studies were performed for a non-supported condition as well as a supported condition with shotcrete lining.

## 2 GROUND REACTION CURVE WITH SEEPAGE FORCES

#### 2.1 Theoretical solutions

It is assumed that a soil-mass behaves as an isotropic, homogeneous and permeable medium. Also, an

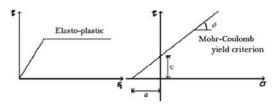


Figure 1. Elasto-plastic model based on Mohr-Coulomb yield criterion.

elasto-plastic model based on a linear Mohr-Coulomb yield criterion is adopted in this study, as indicated in Figure 1:

$$\sigma_1' = k\sigma_3' + (k-1)a \tag{1}$$

where,  $\sigma'_1$  indicates the major principal stress,  $\sigma'_3$  is the minor principal stress,

$$k = \tan^2(45 + \frac{\phi}{2})$$
 and  $a = \frac{2c}{\tan \phi}$ ,

k and a are the Mohr-Coulomb constants, c is the cohesion, and  $\phi$  is the friction angle.

Figure 2 shows a circular opening of radius  $r_0$  in an infinite soil-mass subject to a hydrostatic in situ stress,  $\sigma'_0$ . The opening surface is subject to the outward radial pressure to the tunnel surface,  $p_i$ . Considering

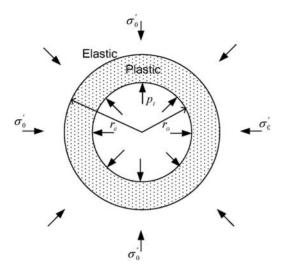


Figure 2. Circular opening in an infinite medium.

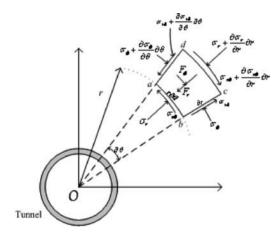


Figure 3. Body forces under the groundwater table.

all the stresses on an infinitesimal element *abcd* of unit thickness during excavation of a circular tunnel in Figure 3, equilibrium equation can be expressed as:

$$\frac{\partial \sigma'_r}{\partial r} + \frac{1}{r} \frac{\partial \sigma_{r\theta}}{\partial \theta} + \frac{\sigma'_r - \sigma'_\theta}{r} + F_r = 0$$
<sup>(2)</sup>

$$\frac{1}{r}\frac{\partial\sigma_{\theta}}{\partial\theta} + \frac{\partial\sigma_{r\theta}}{\partial r} + \frac{2\sigma_{r\theta}}{r} + F_{\theta} = 0$$
(3)

If the tunnel is excavated under the groundwater table, then it acts as a drain in unsupported condition. The body force is the seepage stress, as illustrated in Figure 3:

$$F_r = i_r \gamma_w \tag{4}$$

$$F_{\theta} = i_{\theta} \gamma_{w} \tag{5}$$

In this state,  $i_r$  and  $i_{\theta}$  are the hydraulic gradient in r and  $\theta$  directions respectively, and  $\gamma_w$  is the unit weight of groundwater.

If the stress distribution is symmetrical with respect to the origin ("O") in Figure 3, then the stress components are not varied with angular orientation,  $\theta$ , and therefore they are functions of the radial distance *r* only. By putting Equation (4) into Equation (2) the equilibrium equation reduces to the single equation of equilibrium as follows:

$$\frac{d\sigma'_r}{dr} + \frac{\sigma'_r - \sigma'_{\theta}}{r} + i_r \gamma_w = 0$$
(6)

By substituting  $\sigma'_{\theta} = \sigma'_1$  and  $\sigma'_r = \sigma'_3$  in Equation (1) and by putting Equation (1) into (6), Equation (6) can be given as follows:

$$\frac{d\sigma'_r}{dr} + \frac{1}{r} \{ (1-k)\sigma'_r + (1-k)a \} + i_r \gamma_w = 0$$
<sup>(7)</sup>

The above partial differential equation can be solved by using the boundary conditions  $\sigma'_r = p_i$  at  $r = r_0$ . Then, the radial and circumferential effective stresses in the plastic region are as follows:

$$\sigma_{rp}' = \left(\frac{r_0}{r}\right)^{1-k} (p_i + a) - a - \frac{\gamma_w}{r^{1-k}}$$

$$\left[\int_{R_0}^{r} \xi^{1-k} i_r(\zeta) d\xi - \int_{R_0}^{w} \xi^{1-k} i_r(\zeta) d\xi\right]$$

$$\sigma_{dp}' = k \left(\frac{r_0}{r}\right)^{1-k} (p_i + a) - a - k \frac{\gamma_w}{r^{1-k}}$$

$$\left[\int_{R_0}^{r} \xi^{1-k} i_r(\xi) d\xi - \int_{R_0}^{r_0} \xi^{1-k} i_r(\xi) d\xi\right]$$
(9)

In this equation,  $p_i$  is all the support pressure developed by in situ stress and seepage. Subscripts rp and  $\theta p$ indicate the radial and tangential effective stresses in the plastic region respectively.

In order to estimate the effective stress in the elastic region, the superposition concept is used. As shown in Figure 4, the effective stress considering the seepage force can be assumed as a combination of the solution of equilibrium equation in dry condition and the effective stress only considering seepage. The Kirsch solutions are applied to solve the effective stresses in the elastic region under dry condition (Timoshenko and Goodier, 1969). And the solution proposed by Stern (1969) is adopted to obtain effective stresses in elastic region with consideration of seepage forces.

The following Equation (10) is derived by combining the Kirsh solutions with the Stern's solution in the elastic region as follows:

$$\sigma'_{r_{e}} = \frac{1}{1+k} \left( 2\sigma'_{0} \right) + \frac{1-k}{1+k} a + \frac{1}{1+k}$$

$$\left( -A \left[ \log(r_{e}) \right] - B - \frac{1}{(\nu-1)} I(r_{e}) \right)$$
(10)

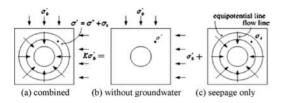


Figure 4. Concept of superposition in elastic region. where

$$A = \frac{Z}{\frac{-1}{r_0^2} + \frac{2\log R_0 - 2\log r_0 + 1}{R_0^2}} \frac{4}{R_0^2}$$
$$B = \frac{-Z}{\frac{-1}{r_0^2} + \frac{2\log R_0 - 2\log r_0 + 1}{R_0^2}} \frac{4\log R_0}{R_0^2}$$

v = Poisson ratio, and  $R_0 =$  infinite radius

Finally, at the interface between the plastic and elastic regions,  $r = r_e$  as shown in Figure 2, the radial stress calculated in the plastic region must be identical to that in the elastic region. The following Equation (11) of the radius of the plastic zone,  $r_e$  can be derived as follows:

$$r_{e} = r_{0} \left[ \frac{1}{p_{i} + a} \left\{ \frac{1}{1+k} \left( 2\sigma_{0}^{\prime} \right) + \frac{1-k}{1+k} a + a + \frac{1}{1+k} \left( \frac{-1}{(\nu-1)} \gamma_{w} \int_{c_{0}}^{c_{0}} i_{r}(\xi) d\xi \right] \right\} \right]^{\frac{1}{k-1}} \\ + \frac{\gamma_{w}}{r_{e}^{1-k}} \left[ \int_{R_{0}}^{c_{0}} \xi^{1-k} i_{r}(\xi) d\xi - \int_{R_{0}}^{c_{0}} \xi^{1-k} i_{r}(\xi) d\xi \right] \right]^{\frac{1}{k-1}}$$

$$(111)$$

The radial displacement for a circular tunnel can be worked out based on the elasto-plastic theory. By following the same procedure proposed by Sharon (2003), the expression for the radial displacement in the plastic region can be obtained as follows:

$$u_{r} = \frac{1}{2G} r^{-k_{r}} [D_{1}(1-2\nu)(r_{e}^{k_{r}+1}-r^{k_{r}+1}) - D_{2}(r_{e}^{k_{r}-1}-r^{k_{r}-1})] + u_{r(r=r_{r})}(\frac{r_{e}}{r})^{k_{r}}$$
(12)

where,

$$D_{1} = \frac{(\sigma_{r(r=r_{e})}^{\prime} - \sigma_{0}^{\prime})r_{e}^{2} - (p_{i} - \sigma_{0}^{\prime})r_{0}^{2}}{r_{e}^{2} - r_{0}^{2}},$$
  
$$D_{2} = \frac{(p_{i} - \sigma_{r(r=r_{e})}^{\prime})r_{0}^{2}r_{e}^{2}}{r_{e}^{2} - r_{0}^{2}}$$

The radial displacement  $u_{r(r=r_0)}$  at the opening surface  $r = r_0$  is given by Equation (13).

$$u_{r(r=r_0)} = \frac{1}{2G} r_0^{-k_{\varphi}} [D_1(1-2\nu)(r_e^{k_{\varphi}+1} - r_0^{k_{\varphi}+1}) - D_2(r_e^{k_{\varphi}-1} - r_0^{k_{\varphi}-1})] + u_{r(r=r_e)}(\frac{r_e}{r_0})^{k_{\varphi}}$$
(13)

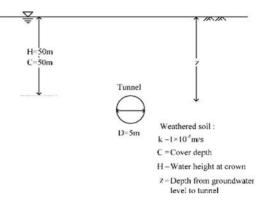


Figure 5. The example tunnel for seepage analysis.

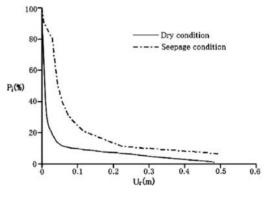


Figure 6. The ground reaction curve (C/D = 10, H/D = 10).

The theoretical solution shown above was verified by performing numerical analysis and comparing the two analysis results (Shin, 2007).

#### 2.2 Example problem

As defined in previous section, the ground reaction curve, GRC is the relationship between the decreasing internal pressure,  $p_i$  and the increasing radial displacement,  $u_r$ . Given a value of  $p_i$ ,  $r_e$  can be calculated using Equation (11), then, the  $u_r$  value using Equation (13).

As shown in Figure 5, the ground reaction curve is calculated for a sample circular tunnel with diameter of D = 5 m, the cover depth of C = 50 m, under the ground surface, and the groundwater table of H = 50 m, above the tunnel crown. As shown in Figure 6, the ground reaction curve with consideration of seepage force is bigger than the ground reaction curve in dry condition, which means that there is no ground water while the cover depth of tunnel, *C* is 10 times diameter of tunnel, *D*. This is due to the fact that even if the effective overburden pressure

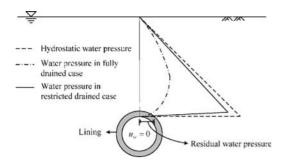


Figure 7. The distribution of water pressure in fully and restricted drained cases.

can be decreased by the arching effect during tunnel excavation, seepage forces still remain.

#### 2.3 Theoretical solution for lined tunnel when including consideration into seepage forces

For a real tunnel, the first lining (such as shotcrete), is installed during tunnelling, and the permeability of this material is relatively lower than that of ground. The theoretical solution when considering seepage is obtained by assuming that the tunnel acts like a drain. However, the installation of lining makes the groundwater flow change from a fully drained condition to a partly drained condition.

When the installation of the lining is considered, the distribution of water pressure in the ground is shown in Figure 7. The water pressure caused by tunnelling can be different according to the ratio of permeabilities of the soil and the tunnel lining. In case of the fully drained condition, the water pressure increases with depth to a certain depth but it converges smoothly to zero,  $u_w = 0$  at tunnel wall. On the other hand, if the drained condition is changed to a partially drained condition due to installation of the lining, then the water pressure increases with depth like the hydrostatic pressure, but it converges rapidly to zero at the inside of the tunnel wall. It develops a residual stress on the surface of lining, therefore eventually induces a bigger seepage force adjacent to tunnel wall than that of fully drained condition. For the development of residual water pressure, a large total head loss is developed within relatively short distance (i.e., thickness of lining).

When considering the installation of a lining with a relatively lower permeability than ground, the ground water flow is changed as shown in Figure 7. This induces the development of a residual water pressure on the surface of lining and makes the hydraulic gradient vary. This means that  $i_r$  in Equation (4) is changed to  $i'_r$  considering the lining installation. Figure 8 shows the variation of hydraulic gradient as the result of the installation of the lining. Therefore, Equation (11) and

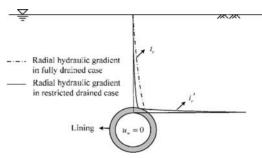


Figure 8. The variation of hydraulic gradient due to lining set up.

(13) can be rewritten with the consideration into lining installation as follows, where  $r_{el}$  is the elasto-plastic interface considering lining installation under groundwater table, assuming the stiffness of the shotcrete is the same as that of the ground.

$$r_{a} = r_{0} \left[ \frac{1}{\frac{1}{p_{i} + a}} \left\{ \frac{\frac{1}{1 + k} (2\sigma_{0}^{\prime}) + \frac{1 - k}{1 + k} a + a + \frac{1}{1 + k} (\frac{-1}{(\nu - 1)} \gamma_{u} \int_{0}^{c} i_{r}^{\prime}(\xi) d\xi}{-4[\log(r_{a})] - B} + \frac{\gamma_{u}}{r_{a}^{-1 + 1}} \left[ \int_{0}^{c} \xi^{1 - k} i_{r}^{\prime}(\xi) d\xi - \int_{0}^{c} \xi^{1 - k} i_{r}^{\prime}(\xi) d\xi} \right] \right]^{\frac{1}{k + 1}} \left( 14 \right)$$

$$u_{r(r = r_{0})} = \frac{1}{2G} r_{0}^{-k_{\nu}} [D_{1}(1 - 2\upsilon)(r_{el}^{k_{\nu} + 1} - r_{0}^{k_{\nu} + 1}) - D_{2}(r_{el}^{k_{\nu} - 1} - r_{0}^{k_{\nu} - 1})] + u_{r(r = r_{el})} (\frac{r_{el}}{r_{0}})^{k_{\nu}}$$

$$(15)$$

The ground reaction curve is estimated through theoretical solution when considering a lining in which the permeability is one thousandth of the soil permeability,  $k_l/k_s = 0.001$ , under the groundwater table. The geometry is same as previous one. Before the lining is installed, the ground reaction curve is the same as the fully drained curve. Then, after installation of the lining, a residual water pressure develops on the surface of lining. It induces an increase of seepage forces near the tunnel wall, thus the ground reaction curve increases as shown in Figure 9. Accordingly, it is known that the permeability and the installation time of the lining is the most important factor in the behavior of the ground.

#### 3 THE SIMPLE METHOD OF GROUND REACTION CURVE

#### 3.1 Estimation of equation of hydraulic gradient

In this study, to evaluate the term related to the seepage in the theoretical solution, the hydraulic gradient should be estimated in advance under conditions corresponding to the existing geometry. The seepage analysis is conducted for a sample circular tunnel

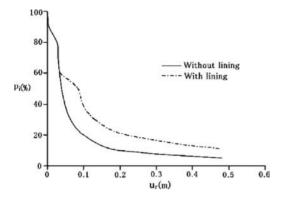


Figure 9. The ground reaction curve considering lining set up under groundwater table (C/D = 10, H/D = 10).

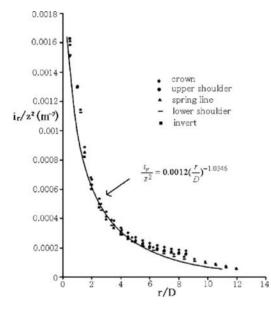


Figure 10. Generalized hydraulic gradient distribution in example tunnel.

shown in Figure 5. The radial component of hydraulic gradient,  $i_r$  as estimated by the seepage analysis, is normalized by square of the depth from the ground-water table to the tunnel,  $z^2$  as the sample tunnel shows in Figure 10, where z is unit of [m], thus unit of  $i_r/z^2$  becomes [m<sup>-2</sup>]. If the distribution of the hydraulic gradient is expressed as a hyperbolic function, then it can be easily integrated into the theoretical solution. The equation of distribution of hydraulic gradient shown in Figure 10 is as follows:

$$\frac{i_r}{z^2} = 0.0012 \left(\frac{r}{D}\right)^{-1.0346} \tag{16}$$

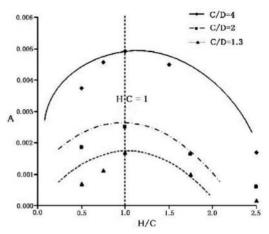


Figure 11. The variation of parameter A for water height and tunnel diameter.

where,  $i_r$  = the radial component of the hydraulic gradient, z = the depth from the groundwater table to the tunnel (ex. crown, shoulder, invert), r = the distance from the center of the tunnel, and D = the diameter of the tunnel. In general form, Equation (16) can be expressed as

$$\frac{l_r}{z^2} = A(\frac{r}{D})^{-B} \tag{17}$$

where, *A* and *B* are parameters depending on C/D and H/C. Shin (2007) performed parametric and sensitivity analysis to assess the two parameters. The results of his analysis are as follows. Firstly, the parameter *B* can be assumed as a constant of one through the sensitivity analysis. In other words, the estimation of the hydraulic gradient can be reduced to the problem of the only parameter *A*. Thirdly, when the groundwater level is higher than cover depth (H/C > 1), the parameter A decreases as the groundwater level increases. On the contrary, when the groundwater level is lower than the cover depth, the parameter *A* increases as water level increases.

Also, when the diameter of tunnel increases, the parameter A tends to decrease. Thus, if the diameter of tunnel, groundwater level and cover depth are given in any geometric condition, the normalized equation of hydraulic gradient can be estimated by using the tendency established above. This can be used to estimate the hydraulic gradient necessary to calculate the ground reaction curve without a seepage analysis in a preliminary design. Figure 11 is the diagram of the parameter A including various conditions; it can be used to estimate the equation of hydraulic gradient in any geometric condition.

### 3.2 Simplified theoretical solution with consideration of seepage forces

As mentioned before, the equation of normalized radial hydraulic gradient can be predicted. The equation of normalized radial hydraulic gradient can be expressed as follows:

$$i_r = Cr^{-1}$$
 (18)

where,  $C = A \frac{z^2}{d}$ 

Therefore, the elasto-plastic interface and radial displacement can be written as follows, where  $r_{ecs}$  is the simplified elasto-plastic interface considering seepage forces.

$$r_{exs} = r_0 \left[ \frac{1}{p_i + a} \left\{ \frac{1}{1 + k} (2\sigma'_0) + \frac{1 - k}{1 + k} a + a + \frac{1}{1 + k} (\frac{-1}{(v - 1)} \gamma_w C \ln \frac{r_{ev}}{R_0} \right] - \mathcal{A}[\log(r_{exs})] - B) + \frac{\gamma_w}{r_{exs}^{-1 + k}} \frac{C}{1 - K} [r_{exs}^{1 - k} - r_0^{1 - k}] \right]^{\frac{1}{k - 1}}$$
(19)

$$u_{r(r=r_{0})} = \frac{1}{2G} r_{0}^{-k_{v}} [C(1-2\upsilon)(r_{e_{x}}^{k_{v}+1} - r_{0}^{k_{v}+1}) - D(r_{e_{x}}^{k_{v}-1} - r_{0}^{k_{v}-1})] + u_{r(r=r_{e_{x}})} (\frac{r_{e_{x}}}{r_{0}})^{k_{v}}$$
(20)

#### 4 CONCLUSIONS

The groundwater has a significant effect on the behavior of tunnel. This is due to the fact that even if the effective overburden pressure can be decreased by the arching effect during tunnel excavation, seepage forces still remain. The results obtained from this study can be summarized as follows:

1 The flow of groundwater has a significant effect on the radial displacement of a tunnel wall. While the effective overburden pressure is reduced by the arching effect during tunnel excavation, seepage forces still remain. Therefore, the presence of groundwater induces the large radial displacement of tunnel wall compared to dry condition.

- 2 When the shotcrete lining (with a relatively lower permeability than that of ground) was installed, the residual water pressure occurs on the surface of the lining. The seepage forces near tunnel wall increase and consequently the radial displacement of tunnel increases, too under the assumption that the shotcrete has the same mechanical properties as the surrounding soil.
- 3 The hyperbolic curve  $(i_r = Cr^{-1})$  that can estimate the distribution of hydraulic gradient due to tunnelling under groundwater table is suggested through parametric studies and sensitivity analysis. Using it, the simplified ground reaction curve can be achieved without performing seepage analysis.

#### ACKNOWLEDGEMENTS

This paper was supported by the Korea Institute of Construction and Transportation Technology Evaluation and Planning under the Ministry of Construction and Transportation in Korea (Grant C04-01).

#### REFERENCES

- Sharan, S.K. 2003. Elastic-brittle-plastic analysis of circular openings in Hoek-Brown media, *International Journal of Rock Mechanics and Mining Sciences* 40: 817–824.
- Shin, Y.J. 2007. Elasto-plastic ground response of underwater tunnels considering seepage forces, Ph.D. Thesis, Korea University, Seoul
- Stern, M. 1965. Rotationally symmetric plane stress distribution, Zeitschrift fur Angewandte Mathematik und Mechanik 45(No. 6): 446–447.
- Timoshenko, S.P. & Goodier, J.N. 1969. Theory of elasticity, McGraw-Hill, New York.

# Basal stability of braced excavations in $K_0$ -consolidated soft clay by upper bound method

X.Y. Song & M.S. Huang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: Design of braced excavations in soft clays is usually controlled by the short-term undrained stability. The paper takes into consideration the principal stress axial rotation induced by excavation and undrained anisotropic soil strength. Assuming the Prandtl soil slip failure modes and considering the anisotropic soil strength recommended by Casagrande & Carillo and the non-homogeneous feature of soft soils, a method for evaluating the basal stability is proposed based on the upper bound analysis. Results obtained from the proposed method indicate that the basal stability is significantly influenced by the anisotropy ratio of soil, as well as the plane geometry of the excavation and the thickness of soft soil layer between excavation base and hard stratum. For field case studies, the proposed method is testified by field observations as well as finite element methods.

#### 1 INTRODUCTION

For deep excavations in soft clay, design of the lateral earth support system is often controlled by stability requirements. In current practice, basically there are three methods available for performing stability calculations of braced excavations: (1) limit equilibrium methods; (2) displacement-based elastoplastic finiteelement methods; (3) upper and lower bound limit analysis. The limit equilibrium methods are widely used in design practice and include separate calculations of basal stability (based on failure mechanisms proposed by Terzaghi 1948; Bjerrum and Eide 1956) or overall slope stability (using circular or noncircular arc mechanisms) based on well established methods (Morgenstern and Price 1965; Bishop 1966; Spencer 1967). It is often difficult to assess the accuracy of these solutions due to *ad hoc* assumptions: (1) in selecting the shape of the failure surface; (2) in the search procedures used to locate the critical surface; and (3) in the approximations used to solve the equilibrium calculations (Ukritchon et al., 2003). Further complications arise in analyzing soil structure interactions for embedded supported walls, tieback anchors, etc.

Displacement-based elastoplastic finite-element methods provides a comprehensive framework that can evaluate multiple facets of excavation performance ranging from the design of the wall and support system, to the prediction of ground movements, and the effects of construction activities such as dewatering, ground improvement, etc. They are indispensable for predicting the distribution of ground movement caused by excavations, and for simulating process where there is partial drainage within the soil. Excavation stability is usually assessed by factoring the strength parameters of the soil.

A powerful method of calculating stability is based on the upper and lower bound theory. Chang (2000) analyzed the safety factor of basal stability based on the Prandtl slide failure mode using upper bound theory. Ukritchon *et al.*(2003) formulated the numerical limit analysis by assuming the anisotropic yield criterion proposed by Davis and Christian (1971). Zou (2004) assumed a kind of complex velocity field and analyzed the basal stability without considering the anisotropic feature of  $K_0$  consolidated clay.

However, the undrained shear strength of natural  $K_0$ -consolidated clay is anisotropic and has a close relation with the vertical effective stress. The classical rotation of principal stress direction of clays induced by excavation is illustrated in Figure 1 (Clough and Hansen, 1981) and it is well-known that the undrianed shear strength of clay varies with the angle of the major principal stress reorientation during the loading. Some researchers (Jiang *et al.*, 1997; Su *et al.*, 1998; Ukritchon *et al.*, 2003) pointed out that without considering the anisotropic and non-homogeneous feature of clays the result of basal stability is not realistic and the safety factor may be much smaller than the real one.

In this paper, an upper bound analysis method for evaluating the basal stability of deep excavations in

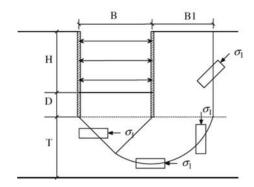


Figure 1. Typical rotation of principal stress.

soft clay is introduced by taking into consideration of anisotropic feature of  $K_0$  consolidated soft clays in the world, especially in Shanghai, as well as the undrained shear strength variation with vertical effective consolidation stress. Practical application of the proposed method is demonstrated through case studies using data published in literature.

#### 2 UNDRAINED ANISOTROPIC SHEAR STRENGTH OF K<sub>0</sub>-CONSOLIDATED SOFT CLAYS

#### 2.1 Undrained shear strength obtained from triaxial tests

Wei and Huang (2006) presented the detailed formulation of a constitutive model for soft clays which can consider the anisotropic feature of clays based on the bounding surface plasticity. In the case of the shape parameter R = 2, the volumetric strain rate can be formulated from the yield function and hardening rule which can be expressed as follows.

$$\dot{\varepsilon}_{v} = \dot{\varepsilon}_{v}^{e} + \dot{\varepsilon}_{v}^{p} = \frac{\lambda}{1+e_{0}}\frac{\dot{p}}{p} + \frac{\lambda-\kappa}{1+e_{0}}\frac{2(\eta-\alpha)}{M^{2}-\alpha^{2}+(\eta-\alpha)^{2}}\dot{\eta} (1)$$

where  $\eta = q/p = \text{stress ratio}$ ;  $e_0 = \text{initial void ratio}$ ;  $\lambda$ ,  $\kappa = \text{the slopes of virgin consolidation line and swelling line respectively in the <math>e - \ln p$  space; M = critical stress ratio; and  $\alpha = \text{slope of yield surface in } p - q$  space.

Under the triaxial undrained condition, i.e.  $\dot{\varepsilon}_v = 0$ , equation (1) can be rewritten as

$$\frac{\lambda - \kappa}{\lambda} \frac{2(\eta - \alpha)}{M^2 - \alpha^2 + (\eta - \alpha)^2} \dot{\eta} = -\frac{\dot{p}}{p}$$
(2)

By integrating both sides of equation (2), the undrained stress path can be expressed as

$$\frac{p}{p_c} = \left[\frac{M^2 - \alpha^2}{M^2 - \alpha^2 + (\eta - \alpha)^2}\right]^{\frac{\lambda - \kappa}{\lambda}}$$
(3)

Table 1. Comparison between author's and measured results.

Test method	Undrained shear strength expression	Test value (Ladd, 1973)	Theoretical value
K <sub>0</sub> UC	eq. (7)	0.330	0.367
K <sub>0</sub> UE	eq. (8)	0.155	0.140

The undrained limit shear strength is the intersection of undrained stress path and critical state stress line. If let  $\eta = M$ , the undrained limit shear strength obtained from compressive test can be expressed as

$$q_{uhc} = Mp_c \left[\frac{M^2 - \alpha^2}{M^2 - \alpha^2 + (M - \alpha)^2}\right]^{\frac{2-\kappa}{\lambda}} = Mp_c \left[\frac{M + \alpha}{2M}\right]^{\frac{2-\kappa}{\lambda}}$$
(4)

Similarly, if let  $\eta = -M$ , the undrained limit shear strength obtained from triaxial extensive test can be expressed as

$$q_{uhe} = Mp_{e} \left[ \frac{M^{2} - \alpha^{2}}{M^{2} - \alpha^{2} + (M + \alpha)^{2}} \right]^{\frac{2-\kappa}{\lambda}} = Mp_{e} \left[ \frac{M - \alpha}{2M} \right]^{\frac{2-\kappa}{\lambda}}$$
(5)

where  $p_c$  = mean effective stress;  $\alpha$  = the second invariant of anisotropic tensor:  $\alpha_{ij}$ . In the analysis, the initial value of  $\alpha_{ij}$  is determined by the initial consolidation state of the soil. For the K<sub>0</sub>-consolidated clay,  $p_c$  can be expressed as

$$p_c = (1 + 2K_0)\sigma'_{v0}/3 \tag{6}$$

where  $\sigma'_{v0}$  = vertical effective consolidation stress.

Substituting equation (6) and  $S_u = q_{ult}/2$  to equations (4) and (5) respectively leads to the following expressions as

$$\frac{S_{w}}{\sigma_{v0}} = \frac{1+2K_0}{6} M \left(\frac{M+\alpha}{2M}\right)^{1-\frac{\pi}{4}}$$
(7)

and

$$\frac{S_{wh}}{\sigma_{v0}'} = \frac{1 + 2K_0}{6} M \left(\frac{M - \alpha}{2M}\right)^{1 - \frac{2}{\lambda}}$$
(8)

where  $S_{uv}$  = undrained shear strength obtained from K<sub>0</sub>UE triaxial test; and  $S_{uh}$  = undrained shear strength obtained from K<sub>0</sub>UC triaxial test.

Table 1 shows the comparison between the aforementioned formulas and experimental data from Ladd(1973), which testifies that the current expression of undrained strength for soft clays is suitable. According to Jiang *et al.* (1997),  $1 - \lambda/\kappa = 0.76$ . Other input parameters are as follows:

 $K_0 = 0.5, \ \phi' = 33^\circ, \ \alpha = 3(1 - K_0)/(1 + 2K_0) = 0.75,$  $M = 6\sin \phi'/(3 - \sin \phi') = 1.331.$ 

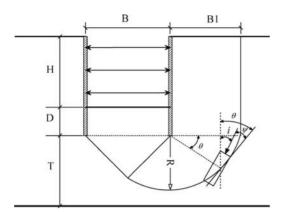


Figure 2. Definition of geometric parameters.

# 2.2 Undrained shear strength considering the rotation of principal stress direction caused by excavations

In order to consider the rotation of principal stress direction caused by excavation in clays, the classical anisotropic strength formula for soft clays recommended by Casagrande & Carillo (1944) is adopted here. In the vertical plane, the undrained shear strength under any principal stress direction is expressed as

$$S_{ui} = S_{uh} + (S_{uv} - S_{uh})\cos^2 i$$
(9)

where  $S_{ui}$  = undrained shear strength of clays when the angle between principal stress direction and vertical direction is *i*.  $S_{uh}$  and  $S_{uv}$  can be obtained from general undrained triaxial tests.

According to the geometric relationship in Figure 2, i can be express as

$$i = \theta - \psi$$
 (10)

where  $\theta$  = angle between the direction of soil failure surface and vertical direction;  $\psi$  = angle between the direction of soil failure surface and the principal stress direction.

According to the test result from Lo (1965)  $\psi$  is a constant which does not vary with the rotation of the principal stress direction. For an undrained analysis,  $\psi = \pi/4$  is preferred. In addition, the anisotropic strength ratio is defined as  $k = S_{uh}/S_{uv}$ . For an isotropic clay  $S_{uv} = S_{uh}$ , k = 1. Substituting the expressions of k and  $\psi$  results in the following expression,

$$S_{u\theta} = S_{w}[k + (1 - k)\cos^2(\theta - \pi/4)]$$
(11)

By substituting equation (4) to equation (11), the anisotropic strength considering both the vertical effective stress and the rotation of principal stress direction caused by excavation can then be described as follows:

$$S_{\omega\theta} = [k + (1-k)\cos^2(\theta - \pi/4)] \frac{1+2K_0}{6} M \left(\frac{M+\alpha}{2M}\right)^{1-\frac{1}{2}} \sigma'_{v_0}$$
(12)

Taking advantage of the definition of safety factor by strength reduction finite element method, we can define the basal stability factor as

$$F_s = S_u(z) / S_u(z)_{critical}$$
(13)

where  $S_u(z)$  = real undrained shear strength of clays; and  $S_u(z)_{critical}$  = critical undrained shear strength.

#### 3 UPPER BOUND ANALYSIS OF BASAL STABILITY OF EXCAVATION CONSIDERING UNDRAINED ANISOTROPIC SHEAR STRENGTH OF CLAYS

#### 3.1 Soil slide failure mechanism

Chen (1975) strengthened that the assumed soil slide failure mode is crucially important to the final result of the limit analysis. Chang (2000) and Faheem *et al.* (2003) suggested that the Prandtl slide failure is close to the real basal failure mode of excavations. In the paper, the Prandtl's slide failure mode is adopted.

According to the distance between the hard stratum under the base of excavation and the excavation base, the slide failure mode can be classified into two typical modes as shown in Figure 3. Based on the strength reduction finite element method, Goh (1990) and Faheem *et al.* (2003) found that the shear strength of clay-wall interface has slight influence on the base stability. Here, we simply assume that the adhesion of clay-wall interface is neglectable, which leads to the simplicity in the upper bound analysis. The sliding surface consists of a 90° circular *arc* sandwiched between two 45° isosceles wedges, an elastic wedge *gjh* and plastic *jik*. The soil column *efji* acts as a surcharge.

According to the upper bound limit theorem, the rate of external work should be equal to the rate of internal energy dissipation in the system in a stability problem. If a realistic, kinematically admissible sliding mechanism is assumed, a reasonable collapse load can be evaluated.

#### 3.2 Calculation of basal stability

Assume the clay has a unit weight  $\gamma$  and a uniform surcharge *q* is present adjacent to the excavation.

#### Case One: $T \ge T_c = B/\sqrt{2}$

From Figure 3(a), the rate of external work done by (1) the weight of the soil columns *efji* and *mnjg*, (2) the

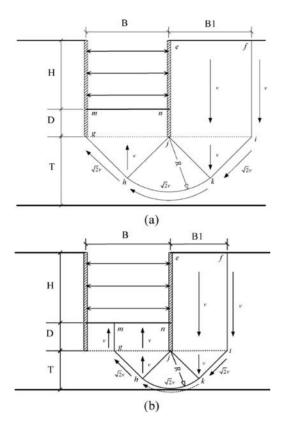


Figure 3. Velocity field based on assumed Prandtl sliding mechanism.

weight of soil in the two isosceles wedges *jik* and *gjh*, and (3) the weight of soil in the radial zone *jhk* is

$$dw = rHBv + qHv \tag{14}$$

The total rate of internal energy dissipation from (1) sliding along fi (2) sliding along ik and gh, and (3) the radial shear combined with arc sliding in the radial shear zone jhk is

$$dE_i = \sum_{n=1}^5 dE_n \tag{15}$$

where

$$dE_{1} = \int_{0}^{(H+D)} v S_{w}(z) [k + (1-k)\cos^{2}(\pi/4)] dz$$
$$= \int_{0}^{(H+D)} v S_{w}(z) (1+k)/2 dz$$
(16)

$$dE_{2} = \int_{(H+D)}^{(H+D+B/2)} \sqrt{2}v S_{uv}(z) [k + (1-k)\cos^{2} 0] \sqrt{2}dz$$
$$= \int_{(H+D)}^{(H+D+B/2)} 2v S_{uv}(z) dz$$
(17)

$$dE_{3} = \int_{\frac{\pi}{4}}^{\frac{3\pi}{4}} \int_{H+D}^{H+D+R\sin\theta} S_{uv}(z) [k + (1-k)\cos^{2}(\theta - \pi/4)] \sqrt{2}v \frac{dz}{\sin\theta} d\theta$$
(18)

$$dE_4 = \int_{\frac{\pi}{4}}^{\frac{3\pi}{4}} S_{uv(H+D+R\sin\theta)} [k + (1-k)\cos^2(\theta - \pi/4)] \sqrt{2}\nu R d\theta$$
(19)  
$$dE_5 = \int_{(H+D)}^{(H+D+B/2)} \sqrt{2}\nu S_{uv}(z) [k + (1-k)\cos^2(\pi/2)] \sqrt{2} dz$$

$$= \int_{(H+D)}^{(H+D+B/2)} 2vkS_{uv}(z)dz$$
(20)

By equating dW to dE, the basal stability of excavation is

$$F_{s} = \frac{\int_{0}^{(H+D)} S_{wv}(z)(\frac{1+k}{2})dz + 2(1+k)\int_{(H+D)}^{(H+D+B/2)} S_{wv}(z)dz}{(rH+q)B} + \frac{\sqrt{2}\left[\frac{1}{\frac{\pi}{4}}\int_{(H+D)}^{(H+D+Rsin\theta)} S_{wv}(z)[k+(1-k)\cos^{2}(\theta-\pi/4)]\frac{dz}{\sin\theta}d\theta}{(rH+q)B} + \frac{\sqrt{2}\left[\frac{1}{\frac{\pi}{4}}S_{wv(H+D+Rsin\theta)}[k+(1-k)\cos^{2}(\theta-\pi/4)]Rd\theta}{(rH+q)B}\right]}{(rH+q)B}$$

When the clay is homogeneous and the undrained shear doesn't vary with the soil profile.

$$F_s = \frac{(1+k)(H+D+2B) + [\pi - 1 + k(\pi + 1)]B}{2(rH+q)B} S_{uv}$$
(22)

**Case Two:**  $T < T_c$ Similarly,

$$F_{s} = \frac{\int_{0}^{(H+D)} S_{uv}(z)(\frac{1+k}{2})dz + 2(1+k)\int_{(H+D)}^{(H+D+\sqrt{2}T/2)} S_{uv}(z)dz}{(rH+q)\sqrt{2}T} + \frac{\sqrt{2}\int_{\frac{\pi}{4}}^{\frac{3\pi}{4}}\int_{(H+D)}^{(H+D+R\sin\theta)} S_{uv}(z)[k+(1-k)\cos^{2}(\theta-\pi/4)]\frac{dz}{\sin\theta}d\theta}{(rH+q)\sqrt{2}T} + \frac{\sqrt{2}\int_{\frac{\pi}{4}}^{\frac{3\pi}{4}} S_{uv(H+D+R\sin\theta)}[k+(1-k)\cos^{2}(\theta-\pi/4)]Rd\theta}{(rH+q)\sqrt{2}T} + \frac{\frac{(1+k)}{2}\int_{H}^{H+D} S_{uv}(z)dz}{(rH+q)\sqrt{2}T}$$

When the clay is homogeneous and the undrained shear doesn't vary with the soil profile.

$$F_s = \frac{(1+k)(H+2D+2\sqrt{2T}) + [\pi-1+k(\pi+1)]\sqrt{2T}}{2\sqrt{2}(rH+q)T} S_{sv}$$
(24)

#### 4 PARAMETRIC STUDIES

The current analysis of basal stability can consider the rotation of principal stress direction and anisotropic

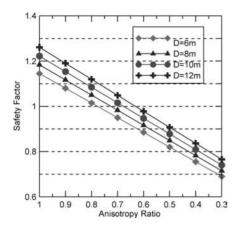


Figure 4. Influence D/H on the factor of safety.

feature of clays. To study the effect of the strength anisotropy, *D/H* and *T/T<sub>c</sub>* to the basal stability of deep excavation in clays, an infinitely long vertical deep excavation in soft clays is analyzed using the aforementioned method. The unit weight of the clay is  $\gamma = 18 \text{ kN/m}^3$ , the undrained shear strength is  $S_{uv}(z) = 0.33\sigma'_v$ , the width of the excavation is B = 15 m, the depth of the excavation is H = 12 m, the penetration of the diaphragm is 6 m, 8 m, 10 m and 12 m, respectively.

#### 4.1 Effects of strength anisotropy

Figure 4 shows the factor of safety of the basal stability of deep excavation varies linearly with the degree of strength anisotropy of clay. In addition, the larger degree of anisotropy, the smaller of safety factor. With the same anisotropic ratio, increasing the penetration of the diaphragm can not increase the basal safety factor greatly.

#### 4.2 Effects of D/H

Figure 5 indicates the safety factor of basal stability increases with the increasing of D/H, which shows the contribution of D/H to  $F_s$ . However, with the same D/H, the anisotropic ratio is more influential than D/H to the safety factor.

#### 4.3 Effects of $T/T_c$

Figure 6 indicates that presence of the bedrock close to the base of the excavation increases  $F_s$ . This can be explained that the size of the yielding zone is affected since the displacement of the soil beneath and around the excavation is restrained when the rigid stratum is close to the base of excavation. However, with the same  $T/T_c$  the factor of basal safety is decreased clearly with the decreasing of anisotropic ratio.

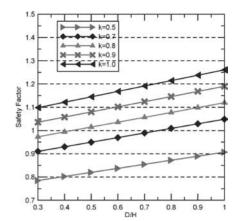


Figure 5. Influence D/H on factor of safety.

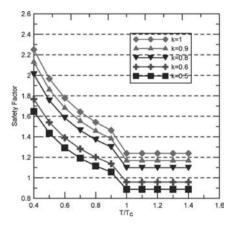


Figure 6. Influence  $T/T_c$  on the factor of safety.

#### 5 FIELD CASE STUDIES

#### 5.1 Boston case

Hashash and Whittle (1996) analyzed an deep excavation in Boston Blue Clays through finite element method incorporating an advanced effective stress soil model, MIT-E3 (Whittle and Kavvadas, 1994). The study focus on an idealized (symmetric) plain strain excavation geometry with half-width, B/2 = 20 m; the depth of excavation is 10 m, 15 m, 22.5 m and 30 m respectively; the penetration of the diaphragm is 2.5 m, 5 m, 17.5 m and 30 m respectively. The vertical effective stress is expressed as  $\sigma'_{\nu 0} = 8.19z + 24.5$ , the gravity of the clay is  $\gamma = 18.0 \, kPa$ . When the OCR is 1.0, 2.0 and 4.0, the strength anisotropy ratio is 0.5, 0.48 and 0.43 respectively. In the current analysis the author assumes the strength anisotropy ratio is about k = 0.5 for BBC. According to the proposed method, the factor of basal stability is 1.16 when the depth of excavation is 10 m and penetration of diaphragm is 2.5 which quite agrees with the result (Hashash *et al.* 1996) based on the finite element method incorporating with the complicated MIT-E3 model.

#### 5.2 Shanghai case

One failure case of excavation for a railway station in Shanghai was reported by Jiang *et al.* (1997). The width of the excavation is 23 m, the depth and the length is 16.7 m and 600 m. The penetration of the diaphragm is 13.3 m and 0.8 m thick. Soil parameters concerned are  $K_0 = 0.65$ ,  $1 - k/\lambda = 0.91$ , M = 0.818. According to the formula for calculating the basal stability recommended by the Shanghai Foundation Design Code and the formula recommended by the Shanghai Tunnel Engineering Design and Research Institute, the safety factors are 2.4 and 1.4, respectively. However, the safety factor will reduce to 0.97 when the undrained shear strength is considered as anisotropic and non-homogeneous.

#### 5.3 Taipei case

Su et al. (1998) analyzed the base failure of an excavation in Taipei using an anisotropic strength formula introduced by themselves. The dimension of this excavation is about 100 m long, 17.5 m to 25.8 m wide, and 13.45 m deep. The diaphragm wall retaining the excavation is 24.0 m deep and 0.7 m thick. Failure occurred about two and a half hours after completion of the last stage excavation and only two minutes were needed before the entire internal bracing system collapsed. The excavation site was located in a reclaimed land along the Keelung River, which meanders through the Taipei Basin. The top 8.7 m of the subsoil profile was backfill and hydraulic fill materials. Underlying the fills are a 2.0 m thick silty sand layer, a thick soft clay layer ranging from Ground Level -10.7 to -44.7 m, and a silty sand layer ranging from GL - 44.7 m to the bed rock located at  $GL - 55.0 \,\mathrm{m}$ . The distribution of  $S_{uc}$  and  $S_{ue}$  with depth are  $S_{uc} = 0.271\sigma'_{vc}$  and  $S_{ue} = 0.189\sigma'_{vc}$ , respectively, i.e. k = 0.7. The calculated safety factor is 1.10 which shows a possibility of basal failure.

#### 6 CONCLUSIONS

A method for analyzing the base stability of deep excavations in anisotropic soft clay is presented in this paper. It is derived from the upper bound theory and strength reduction theory assuming the Prandtl's soil slide failure mode suitable for a critical basal stability which can consider the anisotropy and non-linear feature of clays. The undrained shear strength and anisotropic ratio required for the method can be determined from the conventional  $CK_0UC$  and

 $CK_0UE$  triaxial tests. Comparisons between the result of the current method and the numerical results presented by Hashash and Whittle (1996) for an excavation in normally consolidated Boston blue clay demonstrate the ability of the proposed method to simulate the strength anisotropy and nonhomogeneity of soils on the basal stability of deep excavations. Furthermore, this method also shows good accuracy in back-analyzing the safety factor for two basal failure cases in Shanghai and Taipei respectively. Conclusions of the paper can be summarized as follows.

- 1 The necessities of considering the anisotropic feature of clays for the calculation of basal stability of deep excavations in soft clays have been demonstrated.
- 2 Without taking into consideration of strength anisotropy of clays, the safety factor against base heave from the traditional methods is higher than the real one.
- 3 The parameters used in the current method are easily obtained by conventional lab tests (e.g. triaxial compression test and extension test or plain compression and extension test). Additionally, the method can consider the relationship between soil strength and vertical effective consolidation stress.

#### REFERENCES

- Bishop, A.W. 1966. The strength of soils as engineering materials. *Geotechnique* 16(2): 89–130.
- Bjerrum, L. & Eide, O. 1956. Stability of strutted excavations in clay. *Geotechnique* 6: 32–47.
- Casarande, A. & Carillo, N. 1944. Shear failure of anisotropic soil. J. of the Boston Society of Civil Engineers 31(4).
- Chang, M.F. 2000. Basal stability analysis of braced cuts in clay. Journal of Geotechnical and Geoenvironmental Enigineering, ASCE, 126(3): 276–279.
- Chen, W.F. 1975. *Limit analysis and soil plasticity*. Elsevier Scientific, Amsterdam.
- Clough, G.W. & Hansen, L.A. 1981. Clay anisotropy and braced wall behavior. J. Geotech. Eng. Div., ASCE, 107(7): 893–913.
- Davis, E.H. & Christian, J.T. 1971. Bearing capacity of anisotropic cohesive soil. J. Soil Mech. Found. Div., ASCE, 97(5): 753–769.
- Duncan, J.M. & Seed, B.H. 1966. Strength variation along failure surfaces in clay. J. Soil Mech. Found. Div., ASCE, 92(9): 81–104.
- Faheem, H., Cai, F., Ugai, K. & Hagiwara, T. 2003. Twodimensional base stability of excavations in soft soils using FEM. *Comp. and Geotechnics* 30(2): 141–163.
- Goh, A.T.C. 1990. Assessment of basal stability for braced excavation systems using the finite element method. *Comp. and Geotechnics* 10(4): 325–338.
- Hashash, Y.M.A. & Whittle, A.J. 1996. Ground movement prediction for deep excavations in soft clay. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 122(6): 474–486.

- Hu, Z.F., Zhou, J. & Yang, L.D. 2001. Study of subsoil stability under deep excavation. *Chinese Journal of Civil Engineering*, 34(2): 84–95.(in Chinese)
- Jiang, H.W., Zhao, X.H. & Hang, B.L. 1997. Analysis of heave-resistant stability for deep braced excavation in soft clay under anisotropic condition. *Chinese Journal of Geotechnical Engineering*, 19(1): 1–7.(in Chinese)
- Ladd, C.C. 1973. Discussion. Main Session 4. In: Proc. 8th ICSMFE, eds. Committee of 8th ICSMFE, Moscow, 4(2): 108–115.
- Lo, K. Y. 1965. Stability of slopes in anisotropic soils. Journal of the soil mechanics and foundations division, ASCE, 91(SM): 85–106.
- Morgenstern, N.R. & Price, V.E. 1965. The analysis of the stability of general slip surface. *Geotechnique* 15:79–93.
- Spencer, E. 1967. A method of analysis of the stability of embankments assuming parallel interslice forces. *Geotechnique* 17:11–26.

- Su, S.F., Liao, H.J. & Lin, Y.H. 1998. Basal stability of deep excavation in anisotropic soft clay. J. Geotechnical and Geoenvironmental Eng., ASCE, 124(9): 809–819.
- Terzaghi, K. & Peck, R.B. 1948. Soil mechanics in engineering practice. Wiley, New York.
- Ukritchon, B., Whittle, A.J. & Sloan, S.W. 2003. Undrained stability of braced excavations in clay. J. Geotechnical and Geoenvironmental Eng., ASCE, 129(8): 738–755.
- Wei, X. & Huang, M.S. 2006. Anisotropic bounding surface model for clays. *Chinese Journal of Hydraulic Engineering* 37(7): 831–837.(in Chinese)
- Whittle, A.J., DeGroot, D.J., Ladd, C.C. & Seah, T.H. 1994. Model prediction of the anisotropic behavior of Boston Blue clay. *Journal of Geotechnical Engineering*, ASCE, 120(1): 199–224.
- Zou, G.D. 2004. Analysis of stability against upheaval of deep excavation by an upper limit method. *Rock and Soil Mechanics*, 25(12):1873–1878.(in Chinese)

## Analytical two and three dimension models to assess stability and deformation magnitude of underground excavations in soil

#### L. E. Sozio

Promon Engenharia, Sao Paulo, Brazil

ABSTRACT: Two and three dimensional elastic and plastic analytical solutions are presented, envisaging the evaluation of stability and deformation of a tunnel or cavern. These solutions are based on thick wall cylinder or thick wall sphere models, where inner and outer radius are defined according to tunnel or cavern geometry and corresponding soil cover to ground surface. Radial body forces are introduced into the equations to emulate gravity forces. Cohesive and frictional materials can be considered. The resulting model permits the calculation of the plastic zone radius as a function of the support pressure acting at the tunnel or cavern inner surface and obtain the critical support pressure. By adopting linear elastic behaviour at the elastic zone and no volume change at the plastic zone it is possible to derive a complete convergence confinement curve. Radial displacement and stress resulting from these models are compared with published case histories. Comparison is also made with a known published elastic perfectly plastic plane strain model.

#### 1 INTRODUCTION

A fundamental parameter in tunnel design is the allowable distance between tunnel face and the point where support is effective. Support is generally effective upon closure of the invert arch in NATM (New Austrian Tunnelling Method) tunnels (Fig. 1), or where segmental lining is erected and grout is injected in non pressurized shield tunnels. In any case the tunnel Engineer must anticipate if the opening will remain stable within such length. If not, he should specify construction of the invert arch closer to the face of the NATM tunnel, or employ an earth pressure balanced type of shield. In both cases he would be required to estimate the magnitude of soil deformation up to the activation of the

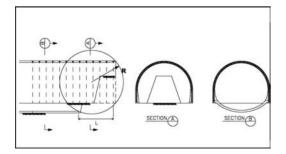


Figure 1. Tunnel length L, where support is not fully effective.

support. The load on the support will depend on this deformation.

#### 2 SOME ASPECTS OF CURRENT DESIGN PRACTICE

Notwithstanding the increasing availability of three dimensional (3D) numerical computer program codes, the predominant design practice of a tunnel lining is still based on a two dimensional (2D) model. The relief of ground stresses prior to the activation of the lining, if accounted for, is generally simulated by a progressive softening of the soil within the excavated area. The design engineer must rely either on his experience or on some published 3D analytical or numerical analyses, which generally do not match the particular characteristics of his problem, to define how much softening will be allowed prior to the lining activation. If he allows little or no softening, the lining design may be ultraconservative, particularly if the soil cover above the tunnel is large. If he allows too much softening, the stresses on the lining may be underestimated, and, if he defines a construction method allowing for an excessive softening, his tunnel is in risk of a collapse.

A comprehensive study into 3D effects at a tunnel heading was performed by Negro (1988).

The analytical models presented herein are proposed to be used as a preliminary estimate of a tunnel stress relief and corresponding soil displacements. More sophisticated (and time consuming) analysis can be performed by using 3D numerical models, at a detailed design stage.

## 3 ELASTIC-PLASTIC ANALYTICAL MODELS

These models are based on a "thick wall sphere" (3D model) and on a "thick wall cylinder" (2D model) in which part of the sphere or the cylinder is in elastic state and the remainder in plastic state. The area of the tunnel where support is considered either to be not effective, or where a supporting pressure p can be applied, is simulated as a sphere (Fig. 1) with an inner radius Ri. The outer radius Ro (Fig. 2) simulates the ground surface, where a surcharge pressure load **s** can be considered. This concept has already been used by Muhlhaus (1985) for a limit analysis where the whole sphere is assumed to be in plastic state, but no information is obtained on displacements prior to collapse.

The analytical model was derived in a way that either a 3D or a 2D analysis can be performed. The 3D analysis is the objective of this concept model. Although less interesting, the 2D model can be used to roughly evaluate average lining load vs. soil displacement, after lining installation, bearing in mind that the axi-symmetry model condition does not allow for bending of the lining.

In the plastic zone of the sphere or cylinder the stress state is found by combining the differential equation of equilibrium (in polar coordinates) and the Mohr Coulomb criterion.

In the elastic zone of the sphere or cylinder the stress state is found by combining the differential equation of equilibrium, the stress strain relationships and the compatibility strain displacement relationships. Refer to Figure 3 for notation and equations.

Radial body forces are included in the equilibrium equations of both the plastic and elastic zones to emulate gravity forces.

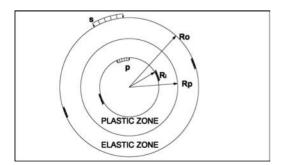


Figure 2. Thick Wall Cylinder (2D) or Sphere (3D).

The boundary between the elastic and plastic zones (Rp in Figure 2) is determined by equating the elastic stresses to the Mohr Coulomb criterion. Note that for unstable configurations no solution is found for that equation. On the other extreme, if  $Rp \le Ri$ , then the whole sphere or cylinder is in elastic state.

The radial displacement at the elastic-plastic boundary is obtained from the elastic equation (9) of Figure 3, which is derived from the differential equation of equilibrium, the stress strain relationships and the compatibility strain displacement relationships, applied to the elastic zone, and considering the previously determined radial stress at elastic-plastic boundary.

For displacements in the plastic zone it is neccessary to consider the plastic deformation behaviour. No volume change was adopted for this analysis, as imposed on equation (10). This equation should be modified if dilating or contracting behaviour is to be modelled.

If  $Ro \rightarrow \infty$ , equation (9) will reproduce the classical elastic expression for radial displacement due to a cylindrical or spherical opening in an infinite elastic medium.

Frictionless materials (undrained analysis) have a particular set of equations to define plastic radius, since for friction angle equal to zero, parameter M = 0 (equation 2 of Fig. 3) make the general equations indeterminate. A check has been made by adopting a very small value of M, for instance  $M = 1e^{-5}$ , and the general and undrained analyses give practically the same results.

The equations of Figure 3 can be inserted into a programmable pocket calculator (such as the HP32s, which contains an algorithm that solves equation (7a) or (7b) = equation (8)) to enable rapid assessment of a given case.

#### 4 LIMITATIONS

These models were developed to give a preliminary insight into stability and deformation behaviour of underground excavations, with a particular interest in tunnel headings. The main limitations that should always be considered are listed below:

#### 4.1 Radial simmetry

The models imply in radial simmetry, thus any shape of tunnel or sphere, and ground surface will have to be approximated to a cylinder or sphere. One type of approximation required when considering a tunnel heading is the change of shape of the unsupported length of tunnel into a sphere, as shown in Figure 1. Some geometrical situations may arise for which a great distortion occurs, and one of particular interest is a long heading of a shallow tunnel. If such a condition is to be analysed, is advisable to complement the evaluation by conservatively checking the results of a 2D

Notation Ri: Inner Radius Rp: Plastic Radius Ro: Outer Radius p, s: Inner and Outer surface pressure respectively t : radial stress at elastic – plastic boundary u: elastic radial displacement at elastic – plastic boundary w:radial displacement at Inner surface  $\gamma$ :Unit Weight c:Cohesion  $\phi$ :Friction Angle E: Elastic Modulus v: Poisson Ratio N = 3 (3D) N = 2 (2D) Equations Plastic Zone  $\frac{\delta \sigma r}{s_r} + (N-1)\frac{\sigma r - \sigma \theta}{r} = -\gamma \quad \sigma \theta = \sigma c + \lambda \sigma r$ (1) $\sigma c = \frac{2c \cos \phi}{1 - Sen \phi} \quad \lambda = \frac{1 + Sen \phi}{1 - Sen \phi} \quad M = \lambda - 1 \quad (2)$ Elastic Zone  $\frac{\delta \sigma r}{\delta r} + (N-1)\frac{\sigma r - \sigma \theta}{r} = -\gamma$  (3)  $\sigma r = \frac{E}{1+\nu} \left( \frac{\nu}{1-2\nu} \times \Delta + \varepsilon r \right) \quad \sigma \theta = \frac{E}{1+\nu} \left( \frac{\nu}{1-2\nu} \times \Delta + \varepsilon \theta \right) \quad (4)$  $\varepsilon\theta = \frac{u}{r}$   $\varepsilon r = \frac{\delta u}{\delta r}$   $\Delta = \varepsilon r + (N-1) \times \varepsilon\theta$  (5) Radial stress at plastic boundary:  $if \phi = 0 \quad t = \sigma c \times Loge \left(\frac{Rp}{p_i}\right)^{N-1} + p - \gamma \times \left(Rp - Ri\right) \quad (6)$ For  $\phi \neq 0$   $t = \left(\frac{Rp}{Ri}\right)^{(N-1) \times M} \left(p + \frac{\sigma c}{M} - \frac{\gamma Ri}{M(N-1)-1}\right) - \frac{\sigma c}{M} + \frac{\gamma Rp}{M(N-1)}$ (7a)For  $\phi = 0$   $t = \sigma c \times Log_e \left(\frac{Rp}{Ri}\right)^{N-1} + p - \gamma \times \left(Rp - Ri\right)$  (7b)  $=\frac{\frac{\gamma}{1-\nu}\left\{\left(\frac{(2+(N-3)\nu)}{N^2-1}\right)NRo-\frac{1-2\nu}{N+1}Rp\left(\frac{Rp}{Ro}\right)^{N}-\frac{(1+(N-2)\nu)}{N-1}Rp\right\}+\frac{N}{N-1}s-\sigma c\left(1-\left(\frac{Rp}{Ro}\right)^{N}-\frac{(1+(N-2)\nu)}{N-1}Rp\right)+\frac{N}{N-1}s-\sigma c\left(1-\left(\frac{Rp}{Ro}\right)^{N}-\frac{(1+(N-2)\nu)}{N-1}Rp\right)+$ (8)  $\frac{(N-1)M+N}{N-1} - \left(\frac{Rp}{Ro}\right)^N \times M$ Rp is obtained by solving (7a) or (7b) = (8)Elastic displacement at elastic – plastic boundary:  $\left[s+\gamma(Ro-Rp)-t\right]\times\left((1-2\nu)\times\left(\frac{Rp}{2}\right)^{N}+\frac{(N-2)\nu+1}{2}\right)$ 

$$u = (1 + (3 - N)\nu)\frac{Rp}{E} \left\{ \frac{1}{1 - \left(\frac{Rp}{Ro}\right)^{N}} - \left(\frac{Rp}{Ro}\right)^{N} \right\}$$
(9)  
Radial Displacement at the inner surface  
$$w = Ri - \left[Ri^{N} + (Rp - u)^{N} - Rp^{N}\right]^{\frac{1}{N}}$$
(10)



analysis, for an infinite cylinder of inner radius equal to the long heading radius.

#### 4.2 Constitutive model

Soil is assumed to be homogeneous, isotropic and linear elastic with Mohr Coulomb plastic criterion and no volume change. Soils stress strain behaviour is in general not linear, and this would affect mainly the displacements estimate.

#### 4.3 In situ stresses

Radial simmetry implies in radial body forces, other than vertical gravity. The effect of a horizontal to vertical in situ stress ratio different than one is not possible to be considered in these models.

Also, the effect of a flow net resulting from ground water table above the tunnel axis cannot be accounted for, except for the simplified assumption of a descending vertical flow net, with hydraulic gradient equal to one, thus implying in a saturated (in lieu of submerged) unit weight of the soil. In general a flow net towards the tunnel heading represents risk of a hydraulic piping failure, and should be avoided by vertical or horizontal drains, compressed air, or a slurry type pressure.

#### 4.4 Consolidation

Settlements due to consolidation are not considered in this model, analyses are either fully undrained or drained.

#### 4.5 Local collapse

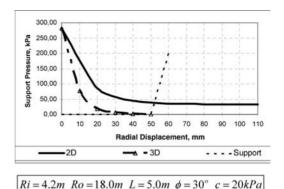
Local Collapse as defined by Davis et ali (1980), is not included in these models.

#### 5 APPLICATIONS AND COMPARISONS WITH OBSERVED DATA

Figure 4 shows an example application of the 2D and 3D models for a tunnel, considering a given length L between tunnel face and effective support activation. The settlement of the tunnel roof on activation of the lining is estimated as the displacement of the sphere inner surface, which is the intersection of the 3D model convergence curve with the abcissa (Fig. 4). At this point the lining is activated.

The final pressure on the lining and crown settlement are estimated from the intersection of the tunnel lining stress displacement characteristic curve with the 2D model convergence curve.

An analysis can be made on the sensitivity of the unsupported length; the crown settlement increases with increase of this length up to the point where the whole sphere is in plastic state, resulting in collapse conditions.



 $\gamma = 20kN/m^3 \quad E = 60MPa \quad v = 0.25 \quad s = 5kPa$ 

Figure 4. Example of convergence - confinement curve.

For the 3D analysis the inner radius was defined to equate the unsupported length tunnel volume, additionaly considering the tunnel face as a half sphere of diameter equal to the tunnel diameter,

$$Ri = \left\{ \frac{D^2}{16} \times (3L + D) \right\}^{\frac{1}{3}}$$
(11)

where D is the tunnel Diameter.

Eight tunnel case histories were analysed in terms of the models presented herein, see Table 1 for references and Table 2 for parameters and results.

The tunnel geometry and soil strength and deformability parameters were inferred from the respective case history papers. Drained parameters were used for sandy soil cases and fully undrained parameters were used for clayey soil cases. In the absence of better information, the undrained deformability moduli of clays were taken as 600 times their undrained shear strength. The measured settlement figures were taken on activation of the lining, although this was not significantly different from the final settlements on all cases. For the Washington D.C. shield case it appears that soil movements were partially restrained to move freely at the rear of the shield steel hood. For this case the unsupported length in the analysis was adopted slightly smaller than the actual distance from face to the segmental lining installation.

The stresses on the lining were measured in the Edmonton cases (case 7 and case 8, see Table 1) and their authors published the idealized soil convergence confinement curve plus the carachteristic curves of two types of linings, concrete segments and steel ribs with wood lagging. The 2D model was applied to this case and the resulting curve is shown in Figure 5. The convergence confinement curve obtained from the 3D model to simulate the unsupported length of the tunnel is also shown, for the concrete lining condition.

A reasonable approximation to the measured loads and / or settlements for this and the other cases can be observed.

A comparison was also made with the model presented by Mair & Taylor (1993), for the unloading of plane strain cylindrical cavity, for linear elastic, perfectly plastic undrained soil behaviour. Curves from both models and for a specific set of parameters are shown in the graph of Figure 6. It was found that the conformity between these models is affected by the soil cover to tunnel radius ratio. In general the ratio between radial displacements of each model increases as the tunnel becomes shallower. This could be due to the fact that the Mair & Taylor model is applicable to an infinite medium condition.

#### 6 CONCLUSIONS

Despite the significant evolution on numerical computer program codes, two dimensional tunnel analyses are still predominant in the current design practice. Consequently the 3D stress state in the zone of the tunnel heading is often either ignored or unsatisfactorily simulated.

Table 1. Case histories and references.

Case	Reference
1	Frankfurt, NATM, Baulos 25 – Cording (1976)
2	Frankfurt, Shield, Fahrgasse – Cording (1976)
3	Washington DC, F2A Line 1 – Cording (1976)
4	Heathrow Cargo – Cording (1976)
5	Green Park – Jubilee Line – Attewell (1974)
6	Sao Paulo E-W Line – Sozio (1978)
7	Edmonton, concrete – Eisenstein (1979)
8	Edmonton, steel ribs – Eisenstein (1979)

The elastic plastic 3D analytical model presented herein is meant to be used as an auxiliary tool to enable assessment of tunnel heading stability conditions and deformation magnitude prior to installation of support.

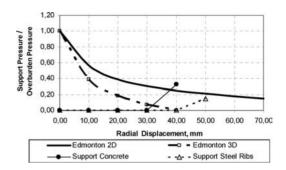


Figure 5. Convergence – confinement curves, Edmonton cases.

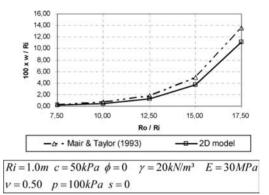


Figure 6. Comparison between Mair & Taylor and 2D model.

Table 2.	Application	of 3D	model vs.	observed data.
----------	-------------	-------	-----------	----------------

Case	1	2	3	4	5	6	7	8
Diameter m	6.5	6.5	5.5	10.9	4.1	6.1	2.6	2.6
Lm	3.0	8.0	5.0	5.0	4.0	7.0	6.0	11.0
<i>Ri</i> m	3.44	4.32	3.38	5.77	2.59	3.98	2.03	2.44
<i>Ro</i> m	14.6	12.4	20.1	13.3	29.3	20.0	26.9	26.9
$\gamma \text{ kN/m}^3$	19.0	18.5	20.0	19.0	19.0	19.0	20.0	20.0
<i>c</i> kPa	55	15	15	100	150	25	110	110
ذ	0	30	35	0	0	33	0	0
E MPa	33	35	80	60	90	60	66	66
υ	0.5	0.25	0.25	0.50	0.50	0.25	0.50	0.50
<i>Rp</i> m	6.68	9.07	5.20	6.11	4.33	5.20	4.52	5.33
w mm	42	69	23	11	20	22	38	43
calculated								
w mm	45	70	21	14	17	25	33	38
observed								

The 2D model permits an estimate on the stress level to be expected on the support.

Comparisons with some published case histories and with a known elastic perfect plastic analytical model are presented, no significant discrepancies being observed.

A series of limitations have to be considered when employing these models, as outlined above (Heading 4). Therefore these models are not to be used as a sole design tool, but as complementary elements to the accepted tunnel design procedures and standards.

#### REFERENCES

- Attewell, P. & Farmer, I. 1974. Ground deformations resulting from shield tunnelling in London clay. *Canadian Geotechnical Journal 11.*
- Cording, E. & Hansmire, W. 1976. Displacements around tunnels in soil. U.S. Department of Transportation Report TST-76T-22.

- Davis, E., Gunn, M., Mair, R. & Seneviratne, H. 1980. The stability of shallow tunnels and underground openings in cohesive material. *Geotechnique*, 30.
- Eisenstein, A., El-Nahas, F. & Thomson, S. 1979. Pressure displacement relations in two systems of tunnel linings. 6th Soil Mechanics and Foundation Engineering Conference, Lima.
- Mair, R.J. & Taylor, R.N. 1993. Prediction of clay behaviour around tunnels using plasticity solutions. *Predictive Soil Mechanics, Proc. Wroth Memorial Symp.*, Oxford, U.K.
- Muhlhaus, H.B. 1985. Lower bound solutions for circular tunnels in two and three dimensions. *Rock Mechanics & Rock Engineering*, 18.
- Negro, A. 1988. Design of shallow tunnels in soft ground. *PhD. Thesis,* University of Alberta, Canada.
- Sozio, L.E. 1978. Settlements in a Sao Paulo shield tunnel. *Tunnels & Tunnelling*, vol. 10 n° 7.

# Dynamic response of saturated silty clay around a tunnel under subway vibration loading in Shanghai

#### Y.Q. Tang & Z.D. Cui

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

#### X. Zhang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China

ABSTRACT: This paper studies the saturated soft clay around the tunnel between Jingansi Station and Jiangsu Station of Shanghai subway Line No.2. The continuous dynamic monitoring is conducted by means of embedded earth pressure piezometers and pore piezometers around the tunnel at different locations and different depths. The response frequency and stress amplitude of the saturated soft clay are studied with the distance from the tunnel due to the subway vibration loading. A formula is proposed for the attenuation of the dynamic response of the soil. The distance of influence and the amplitude of the dynamic response are calculated and the influence on the surrounding buildings under the subway vibration load is predicted, which offers valuable references for the design, the construction and the safe operation of the subway.

#### 1 INTRODUCTION

The subway is indispensable as a safe, comfortable and high speed transport vehicle, in modern cities, but in the course of its running, the vibration problem should not be ignored. Some models (Alabi, 1992; Lipen and Chigarev, 1998; Sheng, et al., 1999; Jones, et al., 2000) were used to analyze the ground vibration under the train loading on the railway. Pan (1995), Hirokazu (2001) and Xie (2002) studied the dynamic response of the railway system, but they focused only on the response of the upper structure of the foundation and did not study the dynamic response of the soil. Up to now, only a few researchers have studied the dynamic response of the saturated silty clay around the subway tunnel. Yet, the long-term vibration loading of the subway has resulted in large deformation of the saturated silty clay (Chen, et al., 2002; Wang, et al., 2003). According to monitored data, a large deformation of the axis of the subway tunnel and the ground settlement had occurred in some sections of the tunnel of Shanghai subway Line No.1 (Lin, et al., 2000) and affected the normal operation of the subway. The settlement and deformation originally began with the change of pore water pressure and soil stress (Tang, et al., 2003; Tang, et al., 2005). So it is important to study the dynamic

response of the soil around the subway tunnel for the safe operation of the subway.

As the soil around the subway tunnel in Shanghai is mainly the saturated silty clay, this paper focuses on it under the subway vibration loading. The response frequency of the saturated soft clay and the law of the amplitude of stress response changing with the depth and the distance away from the tunnel are all studied under the subway vibration loading. The result can offer a valuable reference to the design, construction and the safe operation of the subway.

#### 2 DYNAMIC MONITORING

In order to study the influence caused by the subway vibration loading on the saturated soft clay around the tunnel, field test and monitoring are conducted in this research. The site is selected between Jingansi Station and Jiangsu Road Station. The dynamic monitoring system is adopted for field monitoring and its sampling frequency can reach 200 Hz and its precision is 0.1 kPa. It can fully reflect the soil response around the tunnel due to the subway vibration loading. The dynamic monitoring system consists of a resistance sensor, a dynamic strain amplifier, a data selector

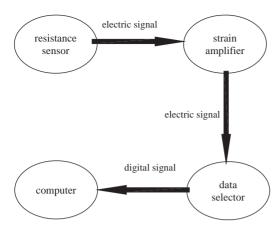


Figure 1. Scheme of dynamic monitoring system.

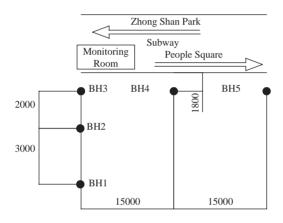


Figure 2. Layout of boreholes (unit: mm).

and a computer. The system can record all the sampling data collected by the computer in real time. The dynamic monitoring system is shown in Figure 1.

Figure 2 shows the layout of boreholes at the site. In the plane, there are five boreholes, each 110 mm in diameter, parallel and vertical to the subway tunnel, respectively. The distance between the site and Jingansi Station is 210 m. Boreholes BH3, BH4 and BH5 are parallel to the tunnel axis only 1.8 m away from the outside of the segment of the subway tunnel and the distance between them is 15.0 m. Boreholes BH1, BH2, and BH3 are vertical to the tunnel axis. In order to study the attenuation of the effect on the soil around the tunnel with the increasing distance under the subway vibration loading, the distance between boreholes BH1 and BH2 is 3 m and that between boreholes BH2 and BH3 is 2 m, so that there is a step-up course.

Figure 3 shows the distribution of strata and instruments. In the section, the subway tunnel lies in gray

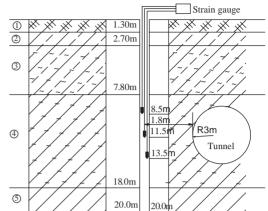


Figure 3. Soil profile and embedded instruments. Note: layer No.1 is mixed soils; layer No.2 is brown yellow silty clay; layer No.3 is gray muddy silty clay; layer No.4 is gray mucky clay; layer No.5 r is gray silty clay.

silty clay of layer No.4 and the earth pressure piezometers and pore piezometers are located at the depths of 8.5 m, 11.5 m and 13.5 m, respectively, in layer No.4, to monitor the response characteristic of the vibration for the subway running.

#### 3 RESPONSE FREQUENCY OF SOIL

The train of Shanghai subway has six carriages and its whole length is 139.46 m. Its normal running speed is 60 km/h and the break at each station is  $30{\sim}40$  s. The monitoring site is near the subway station. The train will decrease its speed when running into the station. According to statistics, the speed is generally at  $30{\sim}40$  km/h when the train passes through the monitoring site, and the time is  $12{\sim}16$  s in general. The interval of the subway train crossing the monitoring site is unequal, generally  $3{\sim}6$  min and  $3{\sim}4$  min during rush hours.

There are two groups of wheels in every carriage of the subway train, and the subway vibration loading is transferred to the soil outside the tunnel segments through the system of wheels-segments. When the subway train comes near the monitoring site, the vibration loading is produced and spread in waves, and the soil can be induced to respond. In Figure 4, the horizontal axis is time and the vertical axis is stress response, and the wave can be seen clearly when the subway train runs across the monitoring site. Owing to the different loading of each carriage (the number of passengers being always different), the response amplitude is also different. The response frequency of soil is also different because of the different interval of the groups of wheels. There are two types of response

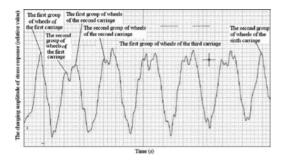


Figure 4. Wave of the soil response.

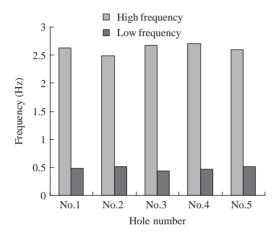


Figure 5. The response frequency of soil for the five boreholes.

frequency: the high frequency  $f_h$  and low frequency  $f_i$ . The high frequency is produced mainly by the rear wheels of the first carriage and the front wheels of the second carriage. The distance between them is short, so the response time of soil is also short. When the two groups of wheels run across the monitoring site, the high frequency is produced. The low frequency is produced mainly by the front and rear wheels of the same carriage and the distance is longer, so it is lower. The vibration response of soil can be seen clearly when every group of wheels runs across the monitoring site, as shown in the figure.

By continuous dynamic field monitoring, plenty of data are obtained. All the waves are analyzed and compared and the typical response wave of the saturated soft clay is obtained. The response frequency of soil (silty clay of layer No.4) of every borehole is obtained by collecting, analyzing and arranging its data. The statistic values are  $2.4 \sim 2.6$  Hz and  $0.4 \sim 0.6$  Hz for the high and low frequency, respectively, as shown in Figure 5.

When these two kinds of response frequency and the natural frequency of soft clay in Shanghai area are

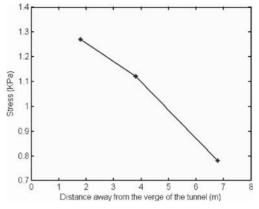


Figure 6. Attenuation of horizontal stress vertical to tunnel axis.

taken into consideration, the corresponding measure should be adopted to avoid the equality of monitoring frequencies and natural frequency in the design of subway tunnel. This can result in resonance, causing different kinds of calamity, such as the instability of the subway tunnel, the fissure of the tunnel segments, and so on.

#### 4 DYNAMIC RESPONSE OF SOIL

### 4.1 Attenuation of dynamic response of soil vertical to the axis of the subway tunnel

The test and research on the attenuation of vibration of the train on the ground with the increasing distance from origin of vibration were mainly focused on the transverse direction. From Chen's test (Chen, et al., 1998), the vibration almost attenuated to zero at 25 m away from the rail of the railway, and had no effect beyond 25 m. The same idea was proved by other tests, and only the distance of attenuation was different for different vibration loadings and conditions. The subway tunnel is embedded in the semi-infinite soil body, and the value of attenuation is larger than that of the ground. Boreholes BH1, BH2 and BH3 are horizontally ardistanced at different locations vertical to the tunnel axis and are 6.8 m, 3.8 m and 1.8 m away from the edge of the subway tunnel, respectively. In order to find the attenuation law more clearly and try to avoid the interference by other factors, the soil at 13.5 m depth is chosen for study. The monitored data are averaged and the result is given in Figure 6.

According to the statistic result of large amounts of monitored data, the relationship between the soil response and the distance vertical to the subway tunnel axis is given in Formula (1):

$$\Delta p = K_0 - K_1 x - K_2 x^2 \tag{1}$$

where  $\Delta p$  (kPa) is the value of dynamic response of soil when the subway is running;  $K_0$  (kPa) is the value of dynamic response of soil at the edge of tunnel;  $K_1$  (kPa/m) is the first order coefficient of the attenuation of dynamical response with the distance;  $K_2$  (kPa/m) is the second order coefficient of the attenuation of dynamical response with the distance; x is the distance away from the edge of the subway tunnel.

According to field monitored data by fitting Formula (1), the coefficients are obtained:  $K_0 = 1.3526$ ,  $K_1 = 0.0321$  and  $K_2 = 0.0077$ .

Substitution of coefficients into Formula (1) yields:

$$\Delta p = 1.3526 - 0.0321x - 0.0077x^2 \tag{2}$$

Formula (2) is giving the dynamic response attenuation law of horizontal soil vertical to the tunnel axis with distance in this field monitoring, where the distance of x is  $0 \sim 11.33$  m.

Using Formula (2), the distance of influence and the amplitude of dynamic response can be calculated. The influence on the surrounding buildings under the subway vibration loading can be predicted. It can offer a theoretical reference to the design and construction of the building near the subway tunnel.

It can be seen from Figure 6 that the attenuation of the subway vibrating loading takes on a certain rule with the increasing distance. The influence distance is 11.33 m obtained from Formula (2). The result is quite different from that of the dynamic railway loading on the ground. In addition to the factor of soil, the main reason is that the shear modulus and the damping of soil increase with the increasing depth. This makes the dynamic loading attenuate rapidly to vanish. Along the subway, the rails are joined with tunnel segments and the rigidity is much larger than that of the soil vertical to the subway.

### 4.2 Dynamic response law of soil at different depths

Sensors are embedded at the top, the middle and at the bottom of the tunnel. The early static monitoring data indicate that the soil pressure approximately takes on the linear increase with the increasing depth. In order to find the relationship between the stress amplitude and depth, the statistic data at the rush hour are selected for analysis, as shown in Figure 7.

It can be seen from Figure 7 that under the subway vibration loading, the maximum changing amplitudes of the stress response of soil are 0.23 kPa at 8.5 m, 0.70 kPa at 11.5 m and 1.15 kPa at 13.5 m. The change of stress amplitude is approximately linear with depth. Moreover, the stress amplitude at the rush hour in the morning is larger than that at the rush hour in the evening, which is also larger than that at noon. This indicates that the number of passengers is the largest

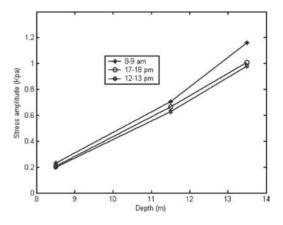


Figure 7. Amplitude of soil pressure response with depth.

in the morning. This should be treated as a reference to design and calculate the worst case in the construction.

#### 5 CONCLUSIONS

- 1 The attenuation of the subway vibration loading vertical to the axis of subway tunnel takes on a certain rule with the increasing distance. According to the statistic results of large amounts of monitored data, the formula for the attenuation of dynamic response of the horizontal soil vertical to the axis of the subway tunnel is obtained with the increasing distance. Using this formula, the influence distance and the amplitude of dynamic response can be calculated. The influence on the surrounding building under the subway vibration loading can be predicted. It offers a theoretical reference to the design and construction of buildings near the subway tunnel.
- 2 The change of stress amplitude of soil is approximately linear with depth when subway trains are running across. Moreover, the stress amplitude is the largest during the rush hour in the morning. It indicates that the flow of passengers is the largest during the rush hour in the morning when the soil is in the worst condition. This value should be adopted as reference for the design and construction of the subway tunnel.
- 3 According to the two kinds of response frequency by the field monitoring of the soil around the tunnel, which are produced by the subway vibration loading and matched with the natural frequency of soft clay in Shanghai area, the corresponding measurement should be adopted in the design of the subway tunnel in order to avoid the response frequency equal to the natural frequency. It could result in resonance causing kinds of calamity, such as the instability of the subway tunnel, the fissure of the tunnel segments, etc.

#### ACKNOWLEDGEMENTS

This work are supported by the research grant (40372124) from National Natural Science Foundation of China, Shanghai Key Subject (Geotechnical Engineering) Foundation and Shanghai Leading Academic Discipline Project(Project Number: B308).

#### REFERENCES

- Alabi, B. 1992. A parametric study on some aspects of ground-borne vibrations due to rail traffic. *Journal of Sound and Vibration* 153(1): 77~87.
- Chen, Y.M., Chen, R.P. & Lu, S. 2002. Several soil mechanics problems in metro construction and operation on soft clay foundation. *In:Proceedings of Seminar on Metro Con*struction and Environmental Geotechnical Engineering. Hangzhou: (in Chinese).
- Hirokazu, T., Shuhei, S. & Xie, W.P. 2001. Train track-ground dynamics due to high speed moving source and ground vibration transmission. *Journal of Structure Mechanics* and Earthquake Engineering 682(7): 299~309.
- Jones, C.J., Sheng, X. & Petyt, M. 2000. Simulations of ground vibration from a moving harmonic load on a railway track. *Journal of Sound and Vibration* 231(3): 739~751.
- Ling, Y.G., Liao, S.M. & Liu, G.B. Discussion of Influencing factors on axial deformation of subway tunnel. Underground Space 20(4): 264~267(in Chinese).

- Lipen, A.B. & Chigarev, A.V. 1998. The displacements in an elastic half-space when a load moves along a beam lying on its surface. *Journal of Applied Maths Mechanics* 62(5): 791~796.
- Pan, S.C., Li, D.W. & Xie, Z.G. 1995. The discussion of the influencing on environment by subway train vibration. *Journal of Vibration and Shock* 14(4): 29~34(in Chinese).
- Sheng, X., Jones, C. & Peryt, M. 1999. Ground vibration generated by a harmonic load acting on a railway track. *Journal of Sound and Vibration* 225(1): 3~28.
- Tang, Y.Q., Hang, Y. & Ye, W.M. 2003. Critical dynamic stress ratio and dynamic strain analysis of soil around the tunnel under subway traffic loading. *Journal of Rock Mechanics* and Engineering 22(9): 1566~1570(in Chinese).
- Tang, Y.Q., Zhang, X., Zhou, N.Q. & Huang, Y. 2005. Microscopic study of saturated soft clay's behavior under cyclic loading. *Journal of Tongji University* 33(5): 626~630(in Chinese).
- Wang, C.J., Ji, M.X. & Chen, Y.M. 2003. Subsequent settlement of saturated soft clay ground induced by train. In: Proceedings of the 9th Conference on Soil Mechanics and Geotechnical Engineering. Beijing: Tsinghua University Press (in Chinese).
- Xie, W.P., Hu, J.W. & Xu, J. 2002. Dynamic response of trackground systems under high speed moving load. *Journal* of Rock Mechanics and Engineering 21(7):1075~1078(in Chinese).

# Lateral responses of piles due to excavation-induced soil movements

C.R. Zhang, M.S. Huang & F.Y. Liang

Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, P.R. China Department of Geotechnical Engineering, Tongji University, Shanghai, P.R. China

ABSTRACT: Lateral soil movements induced by excavation of a deep foundation pit may adversely affect nearby pile foundations. In this paper, a simple analysis method is proposed for computing lateral responses of passive pile groups subject to excavation induced lateral soil movement. Based on a two-stage method, the Winkler model is adopted for simulating the pile-soil interaction, combined with finite difference method in the case of multi-layered soils. A specified free-field soil movement profile is used as input. Then, the governing equation for a pile group is obtained considering the shielding effect in pile groups by the simplified Mindlin's equation. Comparisons are made between the observed behavior of centrifuge model tests and those computed by the proposed method. The present method can in general give a satisfactory prediction of the lateral response of passive pile groups. However, the major limitation is the assumption of linear elastic soil springs, which provides only an upper bound estimate.

# 1 INTRODUCTION

Deep excavations for basements and other underground facilities are unavoidable in big cities. The lateral soil movements resulting from excavations will impose additional bending moments and deflections on nearby piles, which may lead to structural distress or failure (Pan *et al.* 2002, Goh *et al.* 2003). Developing reliable and simple methods to estimate the behavior of piles next to excavations is urgent in practical engineering.

Available methods of analysis can be broadly classified into complete three-dimensional method and two-stage method. The former, which is carried out by a finite element analysis, can consider complex pile-soil interaction and the whole construction process (Goh *et al.* 2002, Miao *et al.* 2005). However, it is computationally expensive and the accuracy depends on the accuracy of the constitutive soil models, which are currently under a stage of calibration with results from physical tests (Juirnarongrit & Ashford 2006). Furthermore, it is more suited to obtain a benchmark solution or to obtain solutions of detailed analysis for final design, rather than as a preliminary routine design tool (Kitiyodom & Matsumoto 2002).

Comparatively, the simplified two-stage method appears to be a more attractive choice. The known freefield soil movement is a prerequisite (Goh *et al.* 1997). As a wealth of experience accumulations on estimation of the soil deformation resulting from excavation have been obtained by engineers, the two-stage method can give a satisfactory result to guide construction and design. In the works of Poulos (Poulos & Chen 1997, Chen & Poulos 1997, 1999), a combined finite element method and boundary element method was used to analyse piles adjacent to an excavation. However, most of their works are focused on single passive piles and the surrounding soil is modelled as an elastic continuum. Issues such as group effect due to pile-soil-pile interaction and the effect of non-homogeneous soils are not well-understood. Further research is clearly required.

This paper describes a simplified two-stage numerical procedure for analyzing response of piles in group subjected to excavation-induced lateral soil movements. A numerical model, considering of nonhomogeneous soils, is achieved using the finite difference method and the concept of shielding effects is introduced to analyse the pile-soil-pile interaction. The assessment is performed by comparing the results from the presented method with those from centrifuge model tests and the predicted results of centrifuge tests by Leung (Leung *et al.* 2000, Leung *et al.* 2003).

# 2 ANALYSIS METHOD

According to the two-stage method, the analysis can be decomposed into two components (Poulos & Chen 1997). First, the free-field soil movement (without the presence of piles in the substratum) is obtained by measurement or calculation. Second, the acquired soil movement is imposed on a nearby pile to calculate its response. The flexural bending of the pile is modelled by an elastic beam while the complex phenomenon of pile-soil interaction is modeled by linear elastic soil springs based on the Winkler model. The lateral deflection equation for a single pile is formulated. Then, considering the restriction of soil movement due to pile-soil-pile interaction, the shielding effect between two piles is imposed using simplified Mindlin's elastic solution for a lateral point load in an elastic half-space. At last the response of group piles is obtained with superposition theorem.

# 2.1 Free-field soil movement induced by the construction of deep foundation pit

Analysis methods for estimating free-field soil movement include an empirical method, a finite element method and an analytical method. As a means to verify the validity of the two-stage method, supposed freefield soil movement or that from in-situ measurement can be adopted. Bigot *et al.* (1982) clearly showed that the displacement-based method of analysis provides very good predictions of bending moment profiles and pile deflections if the measured free-field soil displacement is used as input, or if an accurate prediction of soil movement can be made.

#### 2.2 Analysis of a single pile

As an approximate method, the nonlinear springs which represent the actual pile-soil interaction and tension cracks developed around piles are not taken into account. In other words, the following hypotheses are adopted:

- 1 the pile is represented by an elastic beam based on a Winkler subgrade reaction model;
- 2 the complex phenomenon of pile-soil interaction is modeled using linear elastic soil springs and no crack appears between pile and surrounding soils;
- 3 the effect of axial load on the pile is ignored.

The linear elastic soil spring is represented through a modulus of subgrade reaction, which is defined as

$$k_z = -\frac{p}{y} \tag{1}$$

where  $k_z$  has the units of force/length<sup>2</sup>, p = soil reaction per unit-length of pile, in unit of force/length, y = the relative displacement between pile and surrounding soils. Vesic (1961) analyzed an infinite horizontal beam in an elastic foundation and compared the results with those obtained by the use of subgrade-reaction theory, which related the modulus

of subgrade reaction  $k_z$  to the elastic parameter  $E_s$  and  $v_s$  of the soil mass, as follow in:

$$k_{z} = \frac{0.65E_{s}}{(1-\nu_{s}^{2})} \sqrt[12]{\frac{d^{4}E_{s}}{E_{p}I_{p}}}$$
(2)

where  $E_p I_p$  is pile rigidity.

Then, the governing differential equation of single pile is given by:

$$\frac{d^4 U_i(z)}{dz^4} + 4\lambda^4 [U_i(z) - h_x(z)] = 0$$
(3)

in which  $U_t(z)$  is the lateral deflection of pile caused by excavation;  $h_x(z)$  is the free-field soil movements due to excavation; and  $\lambda$  is written as

$$\lambda = \sqrt[4]{\frac{k_z}{4E_p I_p}} \tag{4}$$

The equation (3) can be solved either by finite difference method or by analytical method. The analytical method has been undertaken by the authors before, and can only be applied for homogeneous soils. In order to consider the influence of non-homogeneous soils, the numerical finite difference method is adopted in this paper.

The total pile length L is divided into *n* cells and the length of each cell is  $\delta$ , with a node in each cell, which is  $0,1,\ldots n-1,n$ . In addition to *n* real cells, there are four additional imaginary cells (two at each end of the pile and additional nodes of -2, -1, n+1, n+2) to implement boundary conditions.

The basic differential equation (3) can be written in the finite difference form for any real cell i, as

$$U_{t,i-2} - 4U_{t,i-1} + (6 + 4(\lambda_i \delta)^4)U_{t,i} - 4U_{t,i+1} + U_{t,i+2} = 4(\lambda_i \delta)^4 h_{x,i}(5)$$

The different boundary conditions at the top and tip of the pile provide four different additional equations (Poulos & Davis 1990).

With free-head pile exerted load H and bending moment M, equations are as follow:

$$-U_{i,-2} + 2U_{i,-1} - 2U_{i,1} + U_{i,2} = \frac{2H}{E_p I_p} \delta^3$$
(6)

$$U_{i,-1} - 2U_{i,0} + U_{i,1} = \frac{M}{E_p I_p} \delta^2$$
(7)

With fixed pile head, the equations are

$$U_{i,1} - U_{i,-1} = 0$$
,  $U_{i,0} = 0$  (8)

When deflection and rotation of pile head is possible, the equation is changed corresponding to the relevant boundary condition.

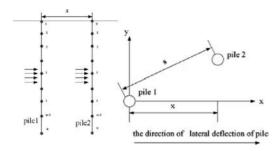


Figure 1. Computation model for two passive piles.

Similarly, the equations for cell n - 1, n can also be acquired, based on their boundary conditions.

Substituting the equations of boundary conditions into equation for nodes 0, 1, n - 1 and n, the n + 1unknowns, which represent the deflection of pile in n + 1 nodes, can be solved by the whole system of n + 1 simultaneous equations. The rotation, shear force and bending moment of the pile can be obtained from deflection based on the theory of material mechanics.

#### 2.3 Interaction between two piles

In general, piles do not follow exactly the free-field soil movement at the pile location and the soil profile is also altered by pile. The hindered free-field soil movement affects nearby piles, the amount of which depending on the relative stiffness between pile and surrounding soils. This is called shielding effect.

The problem of interaction between two piles is depicted in Figure 1, where two piles with pile spacing *s* are represented. By discretizing each pile into *n* cells with a node in each cell, the free-field soil movement in arbitrary node *i* at the position of pile 1 is  $h_{s1,i}$  and the deflection of pile 1 caused is  $U_{p1,i}$ . The corresponding lateral soil deflection is  $U_{s1,i}$ , where  $U_{p1,i}$  is identical to  $U_{s1,i}$  based on compatibility of lateral displacement in node *i* due to shielding effect of pile 1 is then expressed as

$$\Delta U_{s1,j} = h_{s1,j} - U_{s1,j} \tag{9}$$

The corresponding lateral soil shielding displacement in any node j at the position of pile 2 due to soil displacement in node i at the position of pile 1, is

$$h_{s_{21,ij}} = \xi_{ij} \cdot \Delta U_{s_{1,i}} = \xi_{ij} \cdot \left[ h_{s_{1,i}} - U_{s_{1,i}} \right]$$
(10)

where  $\xi_{ij}$  is the attenuation function of the lateral shielding movement based on the simplified Mindlin's equation as follows (Poulos & Davis 1990). Suppose the load in arbitrary node i at the position of pile 1 is

 $k_{zi}U_{s1,i}\delta$  from Winkler subgrade model in the case of active piles. Based on Mindlin's equation, the corresponding soil displacement in arbitrary node j at the position of pile 2(without the presence of pile 2) due to load at node i is

$$U_{s2,i} = \frac{U_{s1,k_{2i}}\delta_{zi}}{16\pi G(1-\nu)} \left\{ \frac{3-4\nu}{R_{1}} + \frac{1}{R_{2}} + \frac{x^{2}}{R_{1}^{3}} + \frac{(3-4\nu)x^{2}}{R_{2}^{3}} + \frac{2z_{i}z_{j}}{R_{2}^{3}} \left( 1 - \frac{3x^{2}}{R_{2}^{2}} \right) + \frac{4(1-\nu)(1-2\nu)}{R_{2}+z_{i}+z_{j}} \left( 1 - \frac{x^{2}}{R_{2}(R_{2}+z_{i}+z_{j})} \right) \right\}$$
(11)

where  $R_1^2 = s^2 + (z_j - z_i)^2$ ,  $R_2^2 = s^2 + (z_j + z_i)^2$ ,  $z_i$  and  $z_j$  is the depth of cell i, j, x is the distance between two piles in the direction of lateral soil deflection. The coefficient  $\xi_{ij}$  can be therefore written as

$$\xi_{ij} = \frac{U_{s2,j}}{U_{s1,j}}$$
(12)

With the superposition theorem, the lateral soil shielding displacement in any node j at the position of pile 2 is

$$h_{s21,j} = \sum_{i=1}^{n} h_{s21,jj} = \sum_{i=1}^{n} \xi_{ij} \cdot \Delta U_{s1,i} = \sum_{i=1}^{n} \xi_{ij} \cdot \left(h_{s1,i} - U_{s1,i}\right)$$
(13)

The lateral equilibrium equation of pile 2 due to shielding effect of pile 1 is written as

$$\frac{d^4 U_{t21}(z)}{dz^4} + 4\lambda^4 [U_{t21}(z) - h_{s21}(z)] = 0$$
(14)

where  $U_{t21}(z)$  is the corresponding lateral deflection of pile 2. The equation can also be solved by finite difference method in the same way.

# 2.4 Pile group analysis

For the usual case of a group of n piles, the total response of a single pile in the group(i.e. the total displacement and the total internal force) is written as the sum of two components:

- 1 displacement due to excavation-induced free-field soil movement.
- 2 shielding displacement resulting from pile-to-pile interaction (shielding effect) which decreases the response of single pile caused by excavation.

Considering the arbitrary pile i in the group, the total displacement of the pile head could be obtained by superposition

$$U_{ij} = \sum_{j=1}^{n} U_{ij}$$
(15)

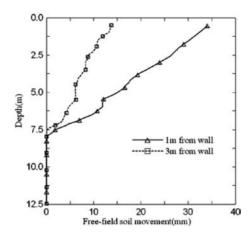


Figure 2. Lateral soil movement induced by excavation.

where  $U_{tii}$  represents the response of pile *i* due to excavation-induced free-field soil movement, and  $U_{tij}(i \neq j)$  represents the shielding displacement of pile *i* due to the existence of pile *j*.

# 3 VERIFICATION BY COMPARISON WITH CENTRIFUGE MODEL TESTS AND FEM ANAYSIS

# 3.1 Experiments

Only very limited field data are available so far, especially in the case of pile groups. Leung *et al.* (2000, 2003) have published a series of centrifuge model tests on unstrutted deep excavation in dense sand and its influence on an adjacent single pile and pile group foundation behind the retaining wall.

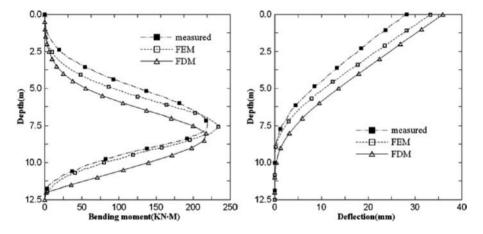


Figure 3. Comparison of free-head pile response for pile located 1 m behind wall.

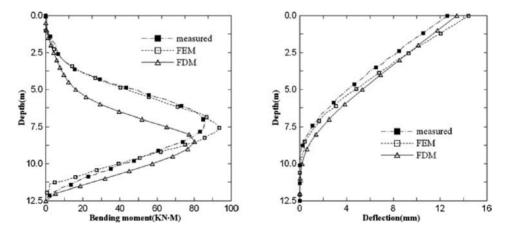


Figure 4. Comparison of free-head pile response for pile located 3 m behind wall.

Due to approximate assumption about linear elastic soil springs in the present simplified method, cases in which soil experienced failure are not taken into account. The predicted results by Leung in the case of single pile with finite element method are also shown herein.

## 3.2 Analysis of single piles

Two tests for single free-head pile located at 1 and 3 m behind the retaining wall and two tests for restraint head (fixed deflection-free rotation head and fixed deflection-fixed rotation head) at 3 m behind the retaining wall are presented here. The prototype square pile has a width of 0.63 m and a length of 12.5 m with  $E_pI_p = 220 \times 10^3$  kNm. The free field soil displacement profiles with depth at the location of the pile

are instrumented and shown in Figure 2. Es = 6z (in MPa, z is the depth below ground surface, in m) is applied for the analysis as suggested by Leung et al. (2000). Figures 3-4 show comparisons between the measured and predicted bending moments and deflection profiles along a free-head pile located at 1, 3 m from the wall. Figures 5-6 give the results for the case of head restraint. FE analysis by Leung et al. (2000) gives a relatively better estimation than the present method, especially at the position of the maximum bending moment. The reason maybe the assumption of linear elastic soil springs along the pile. The soil around the long pile up to a depth less than 4 times the pile diameter may have reached limited soil pressures, even experiencing very small displacement (Pan et al. 2002). In Leung's works, the ultimate soil pressures acting on pile were introduced to consider this effect,

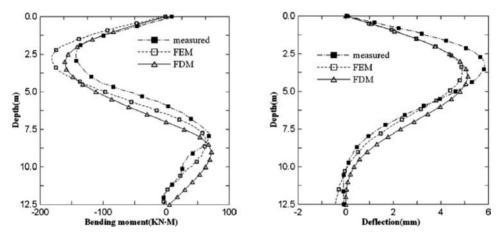


Figure 5. Comparison of fixed (deflection)-free(rotation) head pile response.

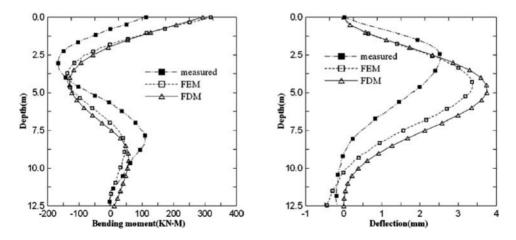


Figure 6. Comparison of fixed (deflection)-fixed(rotation) head pile response.

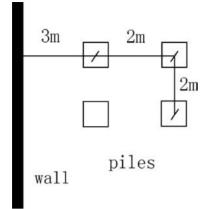


Figure 7. Configuration of pile group.

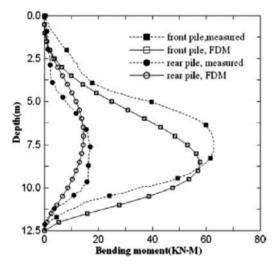


Figure 8. Comparison of free head pile group.

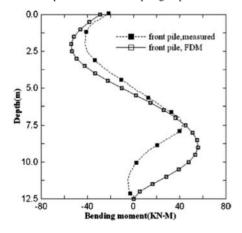


Figure 9. Comparison of front pile in capped head pile group.

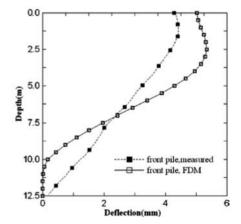
while they are not taken into account in the presented method. However, the simplified method still gives reasonable estimation with little computational effort and it can be used with some confidence in the preliminary design.

# 3.3 Analysis of pile group

17 group-pile tests with different number of piles and different configures are shown in Leung et al. (2003). Due to the limited length of this paper, only 2 typical pile groups are described here. The prototype square pile has a width of 0.48 m and a length of 12.5 m with  $E_p I_p = 240 \times 10^3 \,\mathrm{KNm^2}$ . The pile cap with a thickness of 0.55 m is placed above the ground, which can be treated as rigid cap. Pile groups with four piles in free-head and capped-head are described here, the configuration of which are shown in Figure 7. The predicted results by the present simplified method are compared with the data from centrifuge tests in Figures 8-10. The calculated results provide reasonable approximation to the centrifuge tests data for freehead and capped-head pile groups. The discrepancy between the predicted and measured bending moment profiles is seem to be small along the upper portion of the pile, while it is relatively large along the lower portion of piles. However, deflections of capped-head piles show the tendency that the front pile is dragged back by the rear file through connection of pile cap.

# 4 CONCLUSIONS

This paper presents a numerical analysis with finite difference method for studying the behavior of piles subjected to excavation-induced lateral soil movement in non-homogeneous soils. Response of a single pile is determined by imposing the known free-field soil movement profile to the passive pile. The Mindlin's



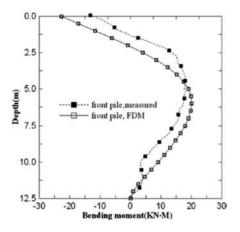


Figure 10. Comparison of rear pile in capped head pile group.

equation is employed to calculate the shielding effect of passive pile groups due to pile-soil-pile interaction. Comparisons with centrifuge model tests confirm that the method provides reliable estimates in simple way.

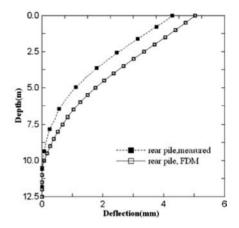
However, the major limitation of the method is the assumption of linear elastic soil springs, which provides only an upper bound estimate of the maximum bending moments and pile deflections. Hence, the analysis considering the nonlinear effect is still needed in order to decisively assess the responses of piles due to excavation-induced lateral soil movements.

# ACKNOWLEDGMENTS

This work was supported by Shanghai Leading Academic Discipline Project, Project Number: B308.

# REFERENCES

- Bigot, G., Bourges, F. & Frank, R. 1982. Etude Experimental D'un Pieu Soumis Aux Poussees Laterales Du Sol. *Revue Francaise de Geotechnique* 18: 29–47.
- Chen, L.T. & Poulos, H.G. 1997. Piles subjected to lateral soil movements. *Journal of Geotechnical and Geoenvi*ronmental Engineering, ASCE, 123(9): 802–811.
- Goh, A.T., Teh, C.I. & Wong, K S. 1997. Analysis of piles subjected to embankment induced lateral soil movements. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 123(9): 792–801.
- Goh, A.T.C., Wong, K.S., Teh, C.I. & Wen, D. 2003. Pile response adjacent to braced excavation. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 129(4):
- Juirnarongrit, T. & Ashford, S.A. 2006. Soil-pile response to blast-induced lateral spreading. II:analysis and assessment of the p-y method. *Journal of Geotechnical and Geoenvironmental Engineering* 132(2):163–172.



- Kitiyodom, P. & Matsumoto, T. 2003. A simplified analysis method for piled raft foundations in non-homogeneous soils. *Int. J. Numer. Anal. Meth. Geomech.* 27: 85–109.
- Leung, C.F., Chow, Y.K. & Shen, R.F. 2000. Behavior of pile subjected to excavation-induced soil movement. *Journal of Geotechnical and Geoenvironmental Engineering* 126(11): 947–954.
- Leung, C.F., Lim, J.K., Shen, R.F. & Chow, Y.K. 2003. Behavior of Pile Groups Subject to Excavation-Induced Soil Movement. Journal of Geotechnical and Geoenvironmental Engineering 129(1):58–65.
- Miao, L.F., Goh, A.T.C., Wong, K.S. & Teh, C.I. 2006. Three-dimensional finite element analyses of passive pile behavior. *Int. J. Numer. Anal. Meth. Geomech.* 30: 599–613.
- Pan, J.L., Goh, A.T.C., Wong, K.S. & Teh, C.I. 2002. Ultimate soil pressure for piles subjected to lateral soil movements. *Journal of Geotechnical and Geoenvironmental Engineering* 128(6):530–535.
- Pan, J.L., Goh, A.T.C., Wong, K.S. & Selby, A.R. 2002. Threedimensional analysis of single pile response to lateral soil movements. *Int. J. Numer. Anal. Meth. Geomech.* 26:747– 758.
- Poulos, H.G. & Chen, L.T. 1997. Pile response due to excavation-induced lateral soil movement. *Journal of Geotechnical and geoenvironmental engineering* 123(2): 94–99.
- Poulos, H.G. & Davis, E.H. 1990. Pile foundation analysis and design. Florida: Pobert E. Krieger Publishing Company.
- Stewart, D.P. 1999. Analysis of piles subjected to embankment induced lateral soil movements. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, May: 425–426.
- Vesic, A.S. 1961. Bending of beam resting on isotropic elastic solid. *Journal of. Engineering Mechanics Division*, ASCE, 87(2): 35–53.

# Elastic-plastic analysis for surrounding rock of pressure tunnel with lining based on material nonlinear softening

L.M. Zhang & Z.Q. Wang

College of Science, Qingdao Technological University, Qingdao, P.R. China

ABSTRACT: The elastic-nonlinear softening-residual plastic surrounding rock model is analyzed. According to the total strain theory, the relationship between equivalent stress and equivalent strain is deduced from uniaxial compression of practical rock. The relational expressions are related with triaxial stress ( $\sigma_{\theta}$ ,  $\sigma_{z}$ , and  $\sigma_{r}$ ) and triaxial strain ( $\varepsilon_{\theta}$ ,  $\varepsilon_{z}$ , and  $\varepsilon_{r}$ ) of surrounding rock. The mechanism of load bearing and acting relation between surrounding rock and support are studied. Stress distribution and displacement of broken zone, plastic zone and elastic zone of tunnel are presented out. The ultimate bearing capacity of surrounding rock is given. The critical pressure leading to yield firstly of surrounding rock. It is pointed out that there are obvious limitations in Kastener formula, which is based on ideal elastic-plastic model. Analysis shows that the ideal plastic model and the brittle model are special cases of the proposed solution.

# 1 INTRODUCTION

Kastner's solution is often used in elastic-plastic analysis for surrounding rock of circular tunnel. There are obvious limitations in Kastner's formula, which is based on ideal elastic-plastic constitutive model. This leads to the Kastner's solution is far away to coresponding actual values in surounding rock of tunnel. Following along the path of pioneered by Kastner, researchers such as Ma (1995, 1996), Jiang and Zheng (1996, 1997), Ma (1998, 1999), Yu (2002), Fan (2004), Ren and Zhang (2001) and Pan and Wang (2004) published different solutions for surrounding rocks of circular tunnel. However, these solutions are resticted to very simple material models, such as simple linear relationship between stress-strain. They are of limited practical value. This study successfully gets the stress distribution laws of surrounding rock plastic and broken zone of tunnel according to the total strain theory. The relationship between equivalent stress and equivalent strain is deduced from practical rock. The relational expressions are related with triaxial stress  $(\sigma_{\theta}, \sigma_z, \text{ and } \sigma_r)$  and triaxial strain  $(\varepsilon_{\theta}, \varepsilon_z, \text{ and } \varepsilon_r)$  of surrounding rock.

# 2 ELASTIC-PLASTIC ANALYSIS FOR SURROUNDING ROCK

Fig. 1 shows the geometric model condition of a tunnel in a plane strain state subjected to a pressure difference between its internal and external pressures. Where a and b are respectively the inner and outer radius of tunnel,  $P_0$  and Pa are pressures acting on the inner and outer surfaces of tunnel, and R is the radius of the interface of elastic and plastic zones. Surrounding rock may be generally divided into broken, plastic and elastic zones on the basis of their states, as shown in Fig. 1. The surrounding rock within the elastic zone is in an elastic state, within the plastic zone, in a strain softening state, and within the broken zone, in a residual-strength state. So the surrounding rock within the broken zone is the direct object of tunnel support.

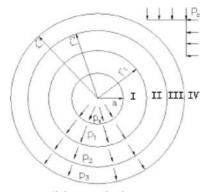
## 2.1 Constitutive model of plastic zone

The tunnel may be simplified as an axisymmetrical plane strain problem. Substituting  $r = r_3$ ,  $\varepsilon_z = 0$  and  $\varepsilon_i = \varepsilon_c$  into the interface of elastic and plastic zones, the equivalent strain is defined by (Zheng, 1988)

$$\varepsilon_{i} = \frac{\sqrt{2}}{3} \sqrt{(\varepsilon_{\theta} - \varepsilon_{z})^{2} + (\varepsilon_{z} - \varepsilon_{r})^{2} + (\varepsilon_{r} - \varepsilon_{\theta})^{2}}$$
$$= \varepsilon_{c} \frac{r_{3}^{2}}{r^{2}}$$
(1)

If the volumetric strain of softening zone equals to zero, we can obtain

$$\sigma_z^p = \frac{1}{2} (\sigma_\theta^p + \sigma_r^p) \tag{2}$$



I lining II broken zone III plastic zone IV elastic zone

Figure 1. Model of tunnel.

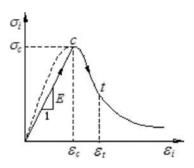


Figure 2. Equivalent stress-equivalent strain.

The equivalent stress is given by

$$\sigma_{i} = \frac{1}{\sqrt{2}} \sqrt{(\sigma_{\theta}^{p} - \sigma_{r}^{p})^{2} + (\sigma_{z}^{p} - \sigma_{r}^{p})^{2} + (\sigma_{z}^{p} - \sigma_{\theta}^{p})^{2}}$$
$$= \frac{\sqrt{3}}{2} (\sigma_{\theta}^{p} - \sigma_{r}^{p})$$
(3)

The constitutive model of uniaxial compression is calculated from (Guo, 2004)

$$\sigma = \sigma_c \frac{\varepsilon/\varepsilon_c}{\alpha_c (\varepsilon/\varepsilon_c - 1)^2 + \varepsilon/\varepsilon_c}, (\varepsilon \ge \varepsilon_c)$$
(4)

where  $\alpha_c$  is a constant,  $\sigma_i$  refers to the stress corresponded to peak strength,  $\varepsilon_i$  refers to the strain corresponded to peak strength. The constant  $\alpha_c$  may be determined by the results of a set of uniaxial tests. The softening section on equivalent  $\sigma_i - \varepsilon_i$  curve can be plotted as Fig. 2 shown. We find that the assumed rock model agrees well with practical rocks.

The ultimate bearing capacity of surrounding rock in complex stress state is analyzed in the following parts. We consider that the strain component of surrounding rock keep constant propotion, ie  $\varepsilon_z: \varepsilon_\theta: \varepsilon_r = 0:1: (-1)$ . So it may be simplified as simple loading condition. According to the total strain theory (Zheng, 1988), the relationship between equivalent stress and equivalent strain can be deduced from Eq. (4)

$$\sigma_i = \sigma_c \frac{\varepsilon_i / \varepsilon_c}{\alpha_c (\varepsilon_i / \varepsilon_c - 1)^2 + \varepsilon_i / \varepsilon_c}, \quad (\varepsilon_i \ge \varepsilon_c)$$
(5)

The relational expressions are related with triaxial stress ( $\sigma_{\theta}$ ,  $\sigma_{z}$ , and  $\sigma_{r}$ ) and triaxial strain ( $\varepsilon_{\theta}$ ,  $\varepsilon_{z}$ , and  $\varepsilon_{r}$ ) of surrounding rock.

## 2.2 Stresses in the plastic zone

Its corresponding mechanical equilibrium equation is

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \tag{6}$$

Substituting  $(\sigma_r)_{r=r_2} = p_2$  into Eq. (3) and Eq. (5) and using the Eq. (6), the stresses in the plastic zone can be obtained as follows

$$\sigma_{r}^{p} = p_{2} + \frac{2\sigma_{c}}{\sqrt{3}\sqrt{4\alpha_{c}-1}} a \tan \frac{2\alpha_{c} r_{3}^{2}/r_{2}^{2} - (2\alpha_{c}-1)}{\sqrt{4\alpha_{c}-1}}$$

$$- \frac{2\sigma_{c}}{\sqrt{3}\sqrt{4\alpha_{c}-1}} a \tan \frac{2\alpha_{c} r_{3}^{2}/r^{2} - (2\alpha_{c}-1)}{\sqrt{4\alpha_{c}-1}}$$

$$\sigma_{\theta}^{p} = \frac{2\sigma_{c}}{\sqrt{3}\sqrt{4\alpha_{c}-1}} a \tan \frac{2\alpha_{c} r_{3}^{2}/r_{2}^{2} - (2\alpha_{c}-1)}{\sqrt{4\alpha_{c}-1}}$$

$$- a \tan \frac{2\alpha_{c} r_{3}^{2}/r^{2} - (2\alpha_{c}-1)}{\sqrt{4\alpha_{c}-1}}$$

$$+ \frac{2\sigma_{c}}{\sqrt{3}} \frac{r_{3}^{2}/r^{2}}{\alpha_{c}(r_{3}^{2}/r^{2}-1) + r_{3}^{2}/r^{2}} + p_{2}$$
(7)

# 2.3 Stresses in the elastic zone

The stresses in the elastic zone may be expressed by

$$\sigma_{\theta}^{e} = p_{0}(1 + \frac{r_{3}^{2}}{r^{2}}) - p_{3}\frac{r_{3}^{2}}{r^{2}}$$

$$\sigma_{r}^{e} = p_{0}(1 - \frac{r_{3}^{2}}{r^{2}}) + p_{3}\frac{r_{3}^{2}}{r^{2}}$$
(8)

Substituting the bounding condition on the interface of elastic and plastic zones  $(\sigma_{\theta}^{e})_{r=r_{3}} + (\sigma_{r}^{e})_{r=r_{3}} = 2p_{0} = (\sigma_{\theta}^{p})_{r=r_{3}} + (\sigma_{r}^{p})_{r=r_{3}}$  into Eq. (7) and Eq. (8), we get

$$p_{0} - p_{2} = \frac{\sigma_{c}}{\sqrt{3}} - \frac{2\sigma_{c}}{\sqrt{3}\sqrt{4\alpha_{c} - 1}} a \tan \frac{1}{\sqrt{4\alpha_{c} - 1}} + \frac{2\sigma_{c}}{\sqrt{3}\sqrt{4\alpha_{c} - 1}} a \tan \frac{2\alpha_{c} r_{3}^{2}/r_{2}^{2} - (2\alpha_{c} - 1)}{\sqrt{4\alpha_{c} - 1}}$$
(9)

The plastic zone has the support effect to surrounding rock. Eq. (9) shows that for a given  $p_0$ , surrounding rock can be balanced by itself through adjusting the plastic zone. So it is also named the equilibrium equation of surrounding rock. Surrounding rock without support has ultimate bearing capacity. If  $r_3 \rightarrow \infty$ , we get the ultimate bearing capacity of surrounding rock. In practical, the surrounding rock is collaped as  $r_3 \rightarrow \infty$ . However, it gives us the theoretical result. The ultimate bearing capacity of surrounding rock in practical can not be larger than the theoretical result of surrounding rock.

# 2.4 Deformation in the plastic zone

According to the elastic-plastic theory, the toatal strain of plastic zone can be calculated by the following formula (Zheng, 1988)

$$\begin{cases} \mathcal{E}_r \\ \mathcal{E}_{\theta} \end{cases} = \begin{cases} \mathcal{E}_r^{\theta} \\ \mathcal{E}_{\theta}^{\theta} \end{cases} + \begin{cases} \mathcal{E}_r^{p} \\ \mathcal{E}_{\theta}^{p} \end{cases}$$
(10)

The elastic strain of plastic zone is defined by

$$\varepsilon_r^e = \frac{1}{E_c} \left[ \left( 1 - \frac{1}{2} \mu_c \right) \sigma_r - \frac{3}{2} \mu_c \sigma_\theta \right]$$
$$\varepsilon_\theta^e = \frac{1}{E_c} \left[ \left( 1 - \frac{1}{2} \mu_c \right) \sigma_\theta - \frac{3}{2} \mu_c \sigma_r \right]$$
(11)

The plastic strain of plastic zone is defined by

$$\varepsilon_r^p = \frac{\varphi}{4G_c} (\sigma_r - \sigma_\theta)$$

$$\varepsilon_\theta^p = \frac{\varphi}{4G_c} (\sigma_\theta - \sigma_r)$$
(12)

The total strain of plastic zone can be expressed by

$$\varepsilon_{r} = \frac{du}{dr} = \frac{1}{E_{c}} \left[ \left( 1 - \frac{1}{2} \mu_{c} \right) \sigma_{r} - \frac{3}{2} \mu_{c} \sigma_{\theta} \right] - \frac{\varphi}{4G_{c}} (\sigma_{\theta} - \sigma_{r})$$

$$\varepsilon_{\theta} = \frac{u}{r} = \frac{1}{E_{c}} \left[ \left( 1 - \frac{1}{2} \mu_{c} \right) \sigma_{\theta} - \frac{3}{2} \mu_{c} \sigma_{r} \right] + \frac{\varphi}{4G_{c}} (\sigma_{\theta} - \sigma_{r})$$
(13)

where  $E_c$  refers to elastic modulus of surrounding rock,  $G_c$  refers to shear modulus,  $\mu_c$  refers to Poission's radio,  $\varphi$  refers to plastic function. The plastic functio  $\varphi$  is zero in elastic deformation. Using Eq. (13), the deformation on the interface of elastic and plastic zones can be calculated from

$$u_{r_{0}} = \frac{r}{E_{c}} \left[ \left( 1 - \frac{1}{2} \mu_{c} \right) \sigma_{\theta} - \frac{3}{2} \mu_{c} \sigma_{r} \right] - \frac{r_{3} (1 + \mu_{c}) (1 - 2\mu_{c})}{E_{c}} p_{0}$$

$$= \frac{2\sigma_{c} r_{3} (1 - 2\mu_{c})}{\sqrt{3E_{c}} \sqrt{4\alpha - 1}} \left[ a \tan \frac{\frac{2\alpha r_{3}^{2}}{r_{2}^{2}} - 2\alpha + 1}{\sqrt{4\alpha - 1}} - a \tan \frac{1}{\sqrt{4\alpha - 1}} + \frac{2\sigma_{c} r_{3} (1 - \mu_{c}/2)}{\sqrt{3E_{c}}} + \frac{r_{3} (1 - 2\mu_{c})}{E_{c}} \left[ p_{2} - (1 + \mu_{c}) p_{0} \right]$$
(14)

Substituting Eq. (7) into Eq. (13), we have

$$\frac{du}{dr} + \frac{u}{r} = \frac{2(1-2\mu_c)}{E_c} \left[ p_2 + \frac{\sigma_c}{\sqrt{3}} \frac{r_3^2/r^2}{\alpha_c (r_3^2/r^2 - 1) + r_3^2/r^2} \right] + \frac{4\sigma_c (1-2\mu_c)}{\sqrt{3E_c}\sqrt{4\alpha_c - 1}} \left[ a \tan \frac{2\alpha_c r_3^2/r_2^2 - (2\alpha_c - 1)}{\sqrt{4\alpha_c - 1}} \right] - \frac{4\sigma_c (1-2\mu_c)}{\sqrt{3E_c}\sqrt{4\alpha_c - 1}} a \tan \frac{2\alpha_c r_3^2/r^2 - (2\alpha_c - 1)}{\sqrt{4\alpha_c - 1}} \right]$$
(15)

Combining Eq. (14), we can solve Eq. (15) and get the following formula

$$\begin{split} u_{r} &= C_{1} + \frac{gp_{2}r}{2} + \frac{gsr}{2} a \tan(\frac{2\alpha_{c}r_{3}^{2} - Ar_{2}^{2}}{r_{2}^{2}\sqrt{4\alpha_{c} - 1}}) \\ &- \frac{ghr_{3}^{2}}{2\alpha_{c}r} \ln\left[(\alpha_{c} + 1)r_{3}^{2} - \alpha_{c}r^{2}\right] + \frac{gsr}{2} a \tan(\frac{A - \frac{2\alpha_{c}r_{3}^{2}}{r^{2}}}{B}) \\ &+ \frac{2\alpha_{c}r_{3}^{2}Bgs}{r(B^{2} + A^{2})} \left[\ln(\frac{1}{r}) + \ln(B^{2} + A^{2} + \frac{4\alpha_{c}^{2}r_{3}^{4}}{r^{4}} - \frac{4A\alpha_{c}r_{3}^{2}}{r^{2}})\right] \\ &\frac{gs}{r} \left\{ \frac{A\alpha_{c}r_{3}^{2}}{B^{2} + A^{2}} a \tan(\frac{B}{A}) + \frac{A\alpha_{c}r_{3}^{2}}{B^{2} + A^{2}} a \tan\left[\frac{A}{B} - \frac{r^{2}(A^{2} + B)}{2B\alpha_{c}r_{3}^{2}}\right] \right\} \end{split}$$
(16)

where

$$\begin{split} A &= 2\alpha_c - 1, B = \sqrt{4\alpha_c - 1}, g = \frac{2(1 - 2\mu_c)}{E_c}, h = \frac{\sigma_c}{\sqrt{3}}, \\ s &= \frac{2h}{\sqrt{4\alpha - 1}}, n = \frac{gp_0(1 + \mu_c)}{2}, \quad w = \frac{2h(1 - \mu_c/2)}{E_c}, \\ C_1 &= \frac{sgr_3}{2} \left[ a \tan(\frac{2\alpha_c r_3^2/r_2^2 - A}{\sqrt{4\alpha_c - 1}}) - a \tan(\frac{1}{\sqrt{4\alpha_c - 1}}) \right] + \\ -gsr_3 \left\{ \frac{2\alpha_c B}{B^2 + A^2} \left[ \ln(\frac{1}{r_3}) + \ln(B^2 + A^2 + 4\alpha_c^2 - 4A\alpha_c) \right] \right\} \\ -gsr_3 \left\{ \frac{A\alpha_c}{B^2 + A^2} a \tan(\frac{B}{A}) + \frac{A\alpha_c}{B^2 + A^2} a \tan(\frac{A}{B} - \frac{A^2 + B}{2B\alpha_c}) \right\} \\ -\frac{gsr_3}{2} a \tan(\frac{2\alpha_c r_3^2 - Ar_2^2}{r_2^2 \sqrt{4\alpha_c - 1}}) + \frac{ghr_3}{2\alpha_c} \ln(-r_3^2) \\ -gsr_3 \left[ \frac{1}{2} a \tan(\frac{-1}{B}) \right] + wr_3 - nr_3 + \end{split}$$

Substituting  $r = r_2$  into Eq. (16), we have

$$u_{r_{2}} = \frac{gsr_{2}}{2} a \tan(\frac{2\alpha_{c}r_{3}^{2} - Ar_{2}^{2}}{r_{2}^{2}\sqrt{4\alpha_{c} - 1}}) - \frac{ghr_{3}^{2}}{2\alpha_{c}r_{2}} \ln\left[(\alpha_{c} + 1)r_{3}^{2} - \alpha_{c}r_{2}^{2}\right] + \frac{gs}{r_{2}} \left\{ \frac{2\alpha_{c}r_{3}^{2}B}{B^{2} + A^{2}} \left[\ln(\frac{1}{r_{2}}) + \ln(B^{2} + A^{2} + \frac{4\alpha_{c}^{2}r_{3}^{4}}{r_{2}^{4}} - \frac{4A\alpha_{c}r_{3}^{2}}{r_{2}^{2}})\right] \right\} + \frac{gs}{r_{2}} \left\{ \frac{A\alpha_{c}r_{3}^{2}}{B^{2} + A^{2}} a \tan(\frac{B}{A}) + \frac{A\alpha_{c}r_{3}^{2}}{B^{2} + A^{2}} a \tan(\frac{A}{B} - \frac{A^{2}r_{2}^{2} + Br_{2}^{2}}{2B\alpha_{c}r_{3}^{2}}) \right] \right\} + \frac{gs}{r_{2}} \left\{ \frac{r_{2}^{2}}{2} a \tan(\frac{-\frac{2\alpha_{c}r_{3}^{2}}{r_{2}^{2}} + A}{B}) \right\} + C_{1} + \frac{gp_{2}r_{2}}{2} + \frac{gr_{2}^{2}}{2} + \frac{gr_{2}^{2}$$

# 2.5 Stresses and deformation in the broken zone

The broken zone cannot bear the tangential stress, so the tangential stress is zero. Its corresponding mechanical equilibrium equation is

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r}{r} = 0 \tag{18}$$

$$\sigma_r = \frac{p_1 r_1}{r} \tag{19}$$

$$p_2 = \frac{p_1 r_1}{r_2}$$
(20)

The deformation in the broken zone may be expressed by

$$du_r = \varepsilon_r dr = \frac{1 - \mu_0^2}{E_0} \sigma_r dr = \frac{(1 - \mu_0^2) p_1 r_1}{E_0 r} dr$$
(21)

Combining Eq. (17), we can solve Eq. (21) and get the deformation formula

$$u_{r} = \frac{gsr_{2}}{2} a \tan(\frac{2\alpha_{e}r_{3}^{2} - Ar_{2}^{2}}{r_{2}^{2}\sqrt{4\alpha_{e}-1}}) - \frac{ghr_{3}^{2}}{2\alpha_{e}r_{2}} \ln\left[(\alpha_{e}+1)r_{3}^{2} - \alpha_{e}r_{2}^{2}\right] + \frac{gs}{r_{2}} \left\{ \frac{A\alpha_{e}r_{3}^{2}}{B^{2} + A^{2}} a \tan(\frac{B}{A}) + \frac{A\alpha_{e}r_{3}^{2}}{B^{2} + A^{2}} a \tan(\frac{A}{B} - \frac{A^{2}r_{2}^{2} + Br_{2}^{2}}{2B\alpha_{e}r_{3}^{2}}) \right\} + \frac{gs}{r_{2}} \left\{ \frac{2\alpha_{e}r_{3}^{2}B}{B^{2} + A^{2}} \left[ \ln(\frac{1}{r_{2}}) + \ln(B^{2} + A^{2} + \frac{4\alpha_{e}^{2}r_{3}^{4}}{r_{2}^{4}} - \frac{4A\alpha_{e}r_{3}^{2}}{r_{2}^{2}}) \right] \right\} + \frac{gs}{r_{2}} \left\{ \frac{r_{2}^{2}}{2} a \tan\left( \frac{-\frac{2\alpha_{e}r_{3}^{2}}{r_{2}^{2}} + A}{B} \right) \right\} + fp_{1}\ln(\frac{r}{r_{2}}) + C_{1} + \frac{gp_{2}r_{2}}{2} \right\}$$

$$(22)$$

where  $f = \frac{(1-\mu_0^2)r_1}{E_0}$ .

Combining Eq. (22) and Eq. (20), we get

$$u_{r_1} = fp_1 \ln(\frac{r_1}{r_2}) + \frac{gp_1r_1}{2} + C_2$$
(23)

$$\begin{split} C_{2} &= \frac{gsr_{2}}{2} a \tan \left( \frac{2\alpha_{c}r_{3}^{2}}{r_{2}^{2}} - A}{\sqrt{4\alpha_{c}-1}} \right) - \frac{ghr_{3}^{2}}{2\alpha_{c}r_{2}} \ln \left[ (\alpha_{c}+1)r_{3}^{2} - \alpha_{c}r_{2}^{2} \right] + \\ \frac{gs}{r_{2}} \left\{ \frac{A\alpha_{c}r_{3}^{2}}{B^{2} + A^{2}} a \tan \left( \frac{B}{A} \right) + \frac{A\alpha_{c}r_{3}^{2}}{B^{2} + A^{2}} a \tan \left( \frac{A}{B} - \frac{A^{2}r_{2}^{2} + Br_{2}^{2}}{2B\alpha_{c}r_{3}^{2}} \right) \right\} + \\ \frac{gs}{r_{2}} \left\{ \frac{2\alpha_{c}r_{3}^{2}B}{B^{2} + A^{2}} \left[ \ln \left( \frac{1}{r_{2}} \right) + \ln \left( B^{2} + A^{2} + \frac{4\alpha_{c}^{2}r_{3}^{4}}{r_{2}^{4}} - \frac{4A\alpha_{c}r_{3}^{2}}{r_{2}^{2}} \right) \right] \right\} + \\ \frac{gs}{r_{2}} \left\{ \frac{r_{2}^{2}}{2} a \tan \left( -\frac{2\alpha_{c}r_{3}^{2}}{r_{2}^{2}} + A \right) \right\} + C_{1} \end{split}$$

# 3 SUBMISSION OF MATERIAL TO THE EDITOR

The lining can be considered as thick-wall cylinder in inner pressure  $p_a$  and outer pressure  $p_1$ . The deformation of lining may be expressed by

$$u_{r1} = \frac{(1+\mu_d)r_1}{E_d(r_1^2 - a^2)} \Big[ 2(1-\mu_d)a^2 p_a \Big] - \frac{(1+\mu_d)r_1}{E_d(r_1^2 - a^2)} \Big[ (1-2\mu_d)r_1^2 + a^2 \Big] p_1$$
(24)

Combining Eq. (23) and Eq. (24), we get

$$p_{1} = \frac{kp_{a} - C_{2}}{m + f \ln(r_{1}/r_{2}) + gr_{1}/2}$$
(25)  
$$r(1 + u) \left[ (1 - 2u) r^{2} + a^{2} \right]$$

where  $m = \frac{r_1(1 + \mu_d)\lfloor (1 - 2\mu_d)r_1^- + a^- \rfloor}{E_d(r_1^2 - a^2)},$  $k = \frac{2(1 - \mu_d^2)a^2r_1}{E_d(r_1^2 - a^2)}.$ 

Combining Eq. (25), Eq. (20) and Eq. (9), we get

$$p_{a} = C_{2} + \frac{r_{2} \left[ m + \frac{gr_{1}}{2} + f \ln(\frac{r_{1}}{r_{2}}) \right]}{kr_{1}} \left( p_{0} - \frac{\sigma_{c}}{\sqrt{3}} \right) - \frac{2\sigma_{c}r_{2} \left[ m + \frac{gr_{1}}{2} + f \ln(\frac{r_{1}}{r_{2}}) \right]}{kr_{1}\sqrt{3}\sqrt{4\alpha_{c} - 1}} \left[ a \tan(\frac{2\alpha_{c}r_{3}^{2}}{\sqrt{4\alpha_{c} - 1}}) - a \tan(\frac{1}{\sqrt{4\alpha_{c} - 1}}) - a \tan(\frac{1}{\sqrt{4\alpha_{c} - 1}}) - a \tan(\frac{1}{\sqrt{4\alpha_{c} - 1}}) \right] \right]$$
(26)

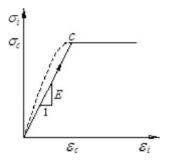


Figure 3. Ideal plastic model of rock.

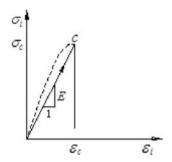


Figure 4. Brittle model of rock.

Using Eq. (26), we may get  $r_3$ . Substituting  $r_3$  into Eq. (25), we find the stress formula of lining as follows

$$\sigma_{rc} = \frac{p_1 r_1^2 - p_a a^2}{r_1^2 - a^2} + \frac{r_1^2 a^2 (p_a - p_1)}{(r_1^2 - a^2)r^2}$$
$$\sigma_{\theta c} = \frac{p_1 r_1^2 - p_a a^2}{r_1^2 - a^2} - \frac{r_1^2 a^2 (p_a - p_1)}{(r_1^2 - a^2)r^2}$$
(27)

# 4 DISCUSSION OF RESULTS

(1) According to Eq. (5), we know

If  $\alpha_c = 0$ , we get  $\sigma = \sigma_c$ . The curve of strength after yield changes into horizon line, as shown in Fig. 3. Which means the equation stands for ideal plastic material. This suggests that the ideal plastic model, such as Fenner's solution and Kastner's solution is special cases of the proposed solution.

If  $\alpha_c \rightarrow \infty$ , we get  $\sigma = 0$ . The curve of strength after yield changes into zero, as shown in Fig. 4. Which means the equation stands for brittle material. This shows that the brittle model is special case of the proposed solution.

If  $0 < \alpha_c < \infty$ , the equation stands for the behavior of post-failure strength. Different integration constant  $\alpha_c$  means different behavior of strength after yield, as shown in Fig. 2. The equation betterly describes the characteristic of the strength dropping after softening of surrounding rock's plasticity. It is suitable for analyzing the stress of plastic zone of wall rock openings.

(2) If the rock does not enter plastic state, but has broken zone, substituting  $r_3 = r_2$  into Eq. (25), we find

$$p_1 = \frac{kp_a + nr_2 - wr_2}{m + f \ln(r_1/r_2) + gr_1/2}$$
(28)

If the rock does not enter broken state, but has plastic zone, substituting  $r_2 = r_1$  into Eq. (25), we find

$$p_{1} = \frac{kp_{a} + nr_{3} - wr_{3}}{m + gr_{1}/2} - \frac{sgr_{3}}{2} \left[ a \tan(\frac{2\alpha_{c} r_{3}^{2}/r_{1}^{2} - A}{\sqrt{4\alpha_{c} - 1}}) - a \tan(\frac{1}{\sqrt{4\alpha_{c} - 1}}) \right] \qquad (29)$$

$$m + gr_{1}/2$$

If the rock does not have plastic zone and broken zone, substituting  $r_3 = r_2$ ,  $r_2 = r_1$  into Eq. (25), we find

$$p_1 = \frac{kp_a + nr_1 - wr_1}{m + gr_1/2}$$
(30)

(3) Substituting  $r_3 = r_2$  into Eq. (26), we get the formula of critical pressure leading to yield firstly for surounding rock that caused by inner pressure

$$p_a^{cr} = \frac{r_2(p_0 - \frac{\sigma_c}{\sqrt{3}}) \left[ m + \frac{gr_1}{2} + f \ln(\frac{r_1}{r_2}) \right]}{kr_1} - nr_2 + wr_2 \qquad (31)$$

# 5 CALCULATION EXAMPLE AND EXTENSION OF ITS APPLICATION

Typical cross-section of a pressure tunnel is shown in Fig. 1. The design length of tunnel is 150 m with inner diameter of a = 3.0 m, outer diameter of  $r_1 = 3.5$  m. The mechanics parameters of rock and lining can be gotten by test,  $E_0 = E_c = 30$  MPa,  $E_d = 60$  MPa,  $\mu_c = u_0 = u_d = 0.25$ ,  $\sigma_d = 50$  MPa,  $r_2 = 3.5$  m,  $p_0 = 1.3$  MPa,  $p_a = 2$  MPa.

According to Eq. (31), we get the critical pressure  $p_a^{cr} = 1.67$  MPa. As the inner pressure  $p_a$  is greater than the critical pressure  $p_a^{cr}$ , plastic zone occurs. Then we get  $r_3 = 5.01$  m from Eq. (26). The loosen range of surrounding rock is obtained by Ultrasonic tests to be 5.15 m. It is very close to the theory result to be 5.29 m from Eq. (26). Table 1 shows stresses of different positon of lining from Eq. (29).

Table 1. Stresses of different position of lining.

r/m	3.00	3.25	3.50	
$\sigma_{rc}$ (kPa)	1.96 - 1.92	1.74	1.47	
$\sigma^{\Delta}_{\theta c}$ (kPa)		-1.62	-1.37	

# 6 CONCLUSIONS

Here we may draw the following conclusions.

- 1 The elastic-nonlinear softening-residual plastic surrounding rock model is analyzed. According to the total strain theory, the relationship between equivalent stress and equivalent strain is deduced from uniaxial compression of practical rock, which is related with triaxial stress ( $\sigma_{\theta}$ ,  $\sigma_z$ , and  $\sigma_r$ ) and triaxial strain ( $\varepsilon_{\theta}$ ,  $\varepsilon_z$ , and  $\varepsilon_r$ ) of surrounding rock. Stress distribution laws of different position of surrounding rock, the mechanism of load bearing and acting relation between surounding rock and support are studied. Analysis shows that the ideal plastic model and the brittle model are special cases of the proposed solution.
- 2 Different radial stresses of the interface under different conditions, such as elstic-plastic condition, elastic-broken conditions are obtained. The ultimate bearing capacity of surrounding rock is given. The critical pressure leading to yield firstly for surounding rock caused by inner pressure is also obtained.

### REFERENCES

- Fan, H. & Yu, M.H. 2004. An analytic solution of elastoplastic pressure tunnel considering material softening and dilatancy. *Engineering Mechanics* 21(5):16–24
- Guo, Z.H. 2004. The strength and constitutive model of conctrete. China Architecture and building Press, Beijing
- Jiang, M.J. & Sheng, Z.J. 1996. On expansion of cylindrical cavity with linear softening and shear dilatation behaviour. *Journal of Rock Mechanics and Engineering* 16(6): 550–557
- Ma, N.J. 1996. A new analysis on ground pressures around openings. *Journal of Rock Mechanics and Engineering* 15(1): 84–89
- Ma, N.J. 1999. Multilinear strength attenuation model of rock body and plastic area of wall rock of openings. *Journal of Metal Mine* 9: 10–12
- Ma, G.W., Iwasaki, S. & Miyamoto, Y. 1998. Plastic limit analysis of circular plates with respect to unified yield criterion. *Int. J. Solids & Structure* 43: 1137–115
- Ma, G.W., Iwasaki, S. & Miyamoto, Y. 1999. Dynamic plastic behavior of circular plats using the unified yield criterion. *Int. J. Solids & Structure* 36: 3257–3275
- Pan, Y. & Wang, Z.Q. 2004. Research on relationshp of load-displacement for cavern surrounding rock with strain nonlinear softeningng. *Journal of Rock Mechanics and Engineering* 25(10):1515–1521
- Ren, Q.W. & Zhang, H.C. 2001. A modification of fenner formula. *Journal of Hohal University* 29(6): 109–111
- Yu, M.H. 2002. Advances in strength theory of materials under complex stress state in the 20th century. *Applied Mechanics Reviews* 55(3): 169–218
- Zheng, Y.T. 1998. Fundamentals of elastic-plastic-sticky theory of rockmechanics. Coal industry Press, Beijing

# Modification of key parameters of longitudinal equivalent model for shield tunnel

# W. Zhu & X.Q. Kou

Geotechnical Research Institute, Hohai University, Nanjing, Jiangsu, P.R. China

# X.C. Zhong

College of Civil Engineering, Hohai University, Nanjing, Jiangsu, P.R. China

# Z.G. Huang

Suzhou Traffic Design and Research Institute, Suzhou, Jiangsu, P.R. China

ABSTRACT: The shield tunnel actually is a slender construction which has many weak spots in the longitudinal direction. Because of the character, the longitudinal non-uniform deformation of the tunnel would happen easily. With increasing of shield tunnel in China, the problem of the longitudinal non-uniform deformation has become attracted. The equivalent continuous model which has clear concept and could be calculated simply has been used widely in projects to analyze the longitudinal performance of the tunnel. The shell-spring model which bases on the three-dimensional FEM was introduced to analyze and modify the error of the equivalent continuous model which of linings segment. The suggested range of the effective ratio of the longitudinal rigidity in common situations of the shield tunnel was induced.

# 1 INTRODUCTION

With the development of municipal works in china, the shield tunnels which are mainly used in soft ground have been in a widespread availability. The layer which has been traveled by the shield tunnel has become more and more sophisticated. So the problem that longitudinal non-homogeneous deformation occurs on the tunnel lining because of its traveling sophisticated layers has become attracted.

Now there are mainly two kinds of exiting longitudinal calculation models which are equivalent continuous model and longitudinal beam-spring model to analysis the longitudinal mechanical property of the shield tunnel linings. The equivalent continuous model simplifies the tunnel a continuous beam which is on the elastic foundation in the longitudinal direction and recognizes that the cross section of the tunnel lining is uniform. At the same time, it considers that the deformation of the tunnel in the longitudinal direction is continuous at the longitudinal joints. The influence of the longitudinal joints of rings is simplified by the way of equivalent rigidity. The equivalent continuous model has the clear conception and it is easy to be calculated, so the model has been used widely in the actual projects.

With the advancement in the design and construction experience, there is a trend to increase the width of the segmental linings. For example, in Japan, the width of the lining segment was  $0.8 \sim 0.9$  m in past, it was developed to 1.0 m in 1975, and the width of the tunnel linings used in Gulf of Tokyo was indeed up to 1.5 m; in China, the width was developed from 1.0 m for shanghai subway to 1.5 m for Guangzhou No.2 subway, and the width of 1.2 m has been commonly adopted in Guangzhou subway No.1 line, Nanjing subway, Shenzhen subway and Beijing subway No.5 line. The segment ring width of the tunnel which traverses the Yangzi River in Nanjing was 2.0 m. The number of joints can be reduced by 20% by increasing the width of the lining segment. The production cost can be reduced, and the waterproof quality of the tunnel can be improved because of reduction of the number of joints.

The equivalent continuous model discussed above assumes that the joints of ring have the influence on mechanical performance of rings segment along the whole width. This model considers that the delivery of the deformation and internal force of linings along the longitudinal direction is continuous and homogeneous. Actually with the increasing of the width of the segment ring of linings, the shear force produced by the bolts of the joints leads to the increases in bending stress which are concentrated on the edge of the ring. The delivery of the deformation and internal force of linings along the longitudinal direction is inhomogeneous especially at the joints of rings because of the concentration of the stress. The equivalent continuous model which is the 2D model isn't able to reflect the complexity of the deformation of the linings increasingly while the width of segment linings increasing.

Thus it is a need to modify the equivalent continuous model and make it suitable to the development of the shield tunnel.

Huang & Zang (2003) considered the influence of the prestress of the longitudinal bolts on the longitudinal rigidity of linings, and modified the equivalent continuous model.

Liao (2002) improved the equivalent continuous model, considered that the influence area of joints of rings is finite and not the whole ring length and suggested that the effective ratio of longitudinal bending rigidity of the Shanghai subway was in the range of 1/7-1/5.

Huang & Xu (2005) suggested the values of the effective ratio of longitudinal and transversal rigidity of shield linings in Shanghai area though indoor modeling tests.

In this study, the 3D numerical simulation called the shell-spring model which could reflect the actually mechanical performance of the segmental linings properly was introduced to modify the equivalent continuous model. In this model, the segments of the shield tunnel lining are simulated by the association of shell elements, the joints of the segments are simulated by the spring-elements, and effects of the soil support are realized by the soil-spring elements.

# 2 THREE-DIMENSIONAL MODELLING OF SHIELD TUNNEL LINING

Figure 1 shows a typical section of shield tunnel lining. In the shield tunnel construction, the segmental linings of each ring are jointed together by the bolts in transverse direction of tunnel. Similarly, two consecutive rings are connected together by the bolts in the longitudinal direction of tunnel. In the numerical modeling, two coordinate systems were used: global coordinates (x', y', z') as shown in Figure 1 and local coordinates (x, y, z) which were used to describe the stresses and deformations for a single element.

# 2.1 Shell element

In this study each segmental lining was modeled by a mesh of three-dimensional shell elements. Each shell element has four nodes. Each node has six degrees of freedom, which are axial and angular displacements in x, y and z direction.

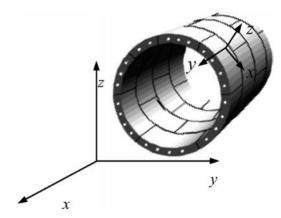


Figure 1. Typical section of tunnel lining.

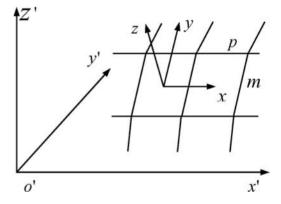


Figure 2. Local coordinates and global coordinates.

# 2.2 Segmental joint element

Each ring of tunnel lining is composed of several segmental linings, which are connected together by the bolts in the transverse direction of tunnel. The segmental joint element was used to simulate the joints between the segmental linings. Zhu and Tao (1998) have put forward the beam-joint discontinuous element to simulate the mechanical behavior of linings and joints in both transverse and longitudinal direction of tunnel. They adopted the non-linear numerical analysis through the concept of Goodman element (1968). In their model the joint element exhibits tensile resistance to simulate the bolt connection. Their numerical result shows that the beam-joint discontinuous model was suitable for simulating the non-linearity of joint and discontinuous deformation of linings.

In this study, the concept of Goodman element was used together with the three-dimensional joint element. The axial, shear and rotation of the segmental joint element in the local coordinates system are described by axial stiffness ( $K_{sx}$ ), shear stiffness in

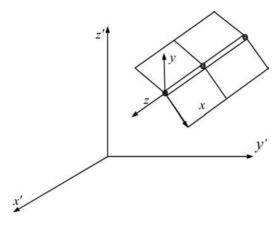


Figure 3. Segmental joint element.

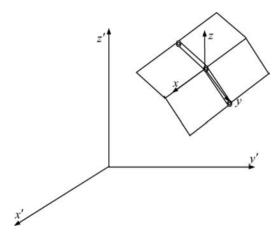


Figure 4. Longitudinal joint element.

radial direction  $(K_{sy})$ , shear stiffness in tangential direction  $(K_{sz})$  and rotational stiffness  $(K_{\theta})$ , respectively. Each segmental joint element has two coincidence nodes. The discontinuity of the joint between two segmental linings is denoted by the relative displacement of two nodes in the local coordinates system x, y and z direction. Each node of the segmental joint element has four element has four degrees of freedom, which are axial displacement in x, y, z direction and angular displacement in z direction.

## 2.3 Longitudinal joint element

The shear model was used to simulate the reinforcing effects in the longitudinal direction of tunnel induced by the staggered arrangement of linings in the beamjoint discontinuous model. The shear model include radial and tangential shear of the linings. In this study, the beam-joint discontinuous element was extended to the three-dimensional joint element. In addition to the radial and tangential shear in the linings, the tension and compression along the longitudinal direction of tunnel were also considered.

The axial, shear in radial and tangential direction of the longitudinal joint element in the local coordinates system (x, y and z) are described by axial stiffness  $(K_{rz})$ , shear stiffness in radial direction  $(K_{ry})$ , shear stiffness in tangential direction  $(K_{rx})$ , respectively. Each longitudinal joint element has two coincidence nodes. The discontinuity of joint between two rings is denoted by the relative displacement of two nodes in the local coordinates systems. Each node of the longitudinal joint element has three degrees of freedom which are axial displacement in x y and z direction. The radial and tangential shear, and axial force in the longitudinal direction of tunnel are considered in each node. Compressive stiffness is assumed to be infinite. Tensile stiffness is the same as the stiffness of bolt, k = EA/L, where E is Young's modulus of bolt, A is cross-section area, L is length.

# 2.4 Soil-spring element

The interaction between the segmental tunnel linings and the surrounding ground was modeled by the soil spring in this study. In this paper, the radial and tangential spring was used to simulate the radial and tangential force acting on the segmental linings from the ground. The springs were arranged over the whole circular section of the tunnel lining.

# 3 NUMERICAL SIMULATION AND ANALYSIS

The numerical simulation was used to calculate the longitudinal rigidity of the shield tunnel linings. In this study, the effect of the soil didn't need to be considered. So the soil-spring elements in the simulation system were neglected. The simulated model is shown in figure 5. There are four rings in the model and boundary condition is selected to be one side fixed and one side free. The outside and inside diameter is 6.2 m and 5.5 m. the linings were constructed by C50 concrete. The young's modulus of concrete is  $3.45 \times 10^4 \text{ N/mm}^2$  (see Table 1).

The arrangement of segmental linings is stagger joint. Figure 5 shows that there are six segmental linings in each section. Segment A B and K have a central angle of  $67.5^{\circ}$ ,  $68.0^{\circ}$  and  $21.5^{\circ}$ , respectively. The linings are arranged in sequence and in stagger in the longitudinal direction of tunnel, as shown in Figure 5. There are 18 bolts which connected two consecutive rings of linings in the longitudinal direction of tunnel.

For analyzing the longitudinal banding rigidity of the linings, the model of numerical simulation (shown in Figure 6) was adopted. In this study, the linings is one end fix and one end free for loading. The

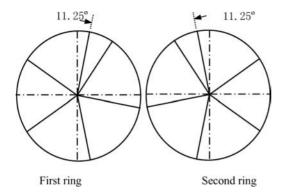


Figure 5. Arrangement of segmental linings in first and second sections.

Table 1. Influence of the lining segment width to the longitudinal equivalent bending rigidity.

Width of lining segment/m	1	1.4	1.8
Analytical solution/GPa	72.43	100.64	128.48
Numerical simulation/GPa	100.8	159.7	205.9

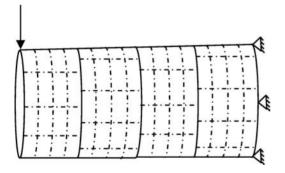


Figure 6. Simulation model.

displacement of the free end of lining would be observed.

# 3.1 Model parameters

The model parameters used in the segmental and longitudinal joint element were chosen based on the national and international shield tunneling experience and are summarized. For the stiffness of segmental joint, the values are the total stiffness of one ring. In the numerical simulation, each ring is further divided into four portions in the longitudinal direction of tunnel as shown in Figure 4.

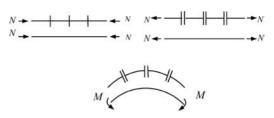


Figure 7. Equivalent continuous model for shield lining.

The mainly model parameters in numerical simulation are shown below.

Outside diameter of the tunnel: D = 6.2 m; inside diameter: d = 5.5 m; The thickness of the lining: t = 350 mm; The diameter of the bolts: d = 30 mm; The length of the bolts: l = 400 mm; The number of the bolts: n = 17; Young's modulus of the bolts:  $E_s = 2.1 \times 10^5$  N/mm<sup>2</sup>; Poisson's ratio of the bolts: v = 0.3; Yield stress of the bolts:  $[\sigma_y] = 6.4 \times 10^5$  N/mm<sup>2</sup>; Breaking stress of the bolts:  $[\sigma_f] = 6.4 \times 10^5$  N/mm<sup>2</sup> The grade of the concrete: C50 Young's modulus:  $E_c = 3.45 \times 104$  N/mm<sup>2</sup>; Poisson's ratio; v = 0.3.

# 3.2 The influence of the lining width to longitudinal rigidity of tunnel lining

The equivalent continuous model simplifies the tunnel a continuous beam which is on the elastic foundation in the longitudinal direction and recognizes that the cross section of the tunnel lining is uniform. At the same time, it considers that the deformation of the tunnel in the longitudinal direction is continuous at the longitudinal joints. The influence of the longitudinal joints is simplified by the way of equivalent rigidity (Figure 7).

The equivalent banding rigidity of the tunnel:

$$(EI)_{eq} = \frac{\cos^3 \varphi}{\cos \varphi + (\varphi + \pi/2) \sin \varphi} \cdot E_c I_c \tag{1}$$

The effective rate of equivalent banding rigidity:

$$\eta = \frac{\cos^3 \varphi}{\cos \varphi + (\varphi + \pi/2) \sin \varphi}$$
(2)

 $\varphi$  could be solved with this formulation:

$$\cot \varphi + \varphi = \pi \left(\frac{1}{2} + \frac{K_j l_s}{E_c A_c}\right) \tag{3}$$

Where  $K_j$ —the elastic stiffness rigidity of all longitudinal bolts,  $K_j = n \cdot k_j$ 

 $k_j$ —the elastic stiffness rigidity of one longitudinal bolt

 $I_c$ —inertia moment of the section

 $I_c = \frac{\pi}{64} \cdot [D^4 - (D - t)^4]$ 

 $E_c$ —Young's modulus of the linings

 $A_c$ —cross-section area of the linings

 $\varphi$ —the position angle of the neutral axis.

Figure 8 shows the relationship between the average vertical displacements of the observed points at the free end which calculated by the simulation model based on shell-spring model and the loads when the width of linings is 1 m, 1.4 m and 1.8 m. It could be concluded that the displacements which is calculated by the simulation model are generally lower than the analysis solution which is calculated by the equivalent continuous model.

# 3.3 Analysis of simulation results

As the Table 1 shown, the equivalent rigidity of the equivalent continuous model is lower than the numerical values. Actually, the influence area of the longitudinal joints to the whole lining's deformation is finite. And the simplified assumption of the model which considers the influence is at the whole ring of lining is not correct. The equivalent banding rigidity of the equivalent continuance model should be lower than the actual values. Besides that, the influence of the arrangement of segmental linings to the longitudinal performance of the tunnel isn't considered by the equivalent continuous model and the influence of that should not be ignored when the internal force delivers along the longitudinal linings. The local deformation at the joints of rings also could influence the delivery of the endogen force. This phenomenon is just so-called local arrangement effect.

With the lining width developed, the difference of longitudinal equivalent rigidity between the simulation model and the equivalent continuous model became bigger (Figure 8 & Table 2).

It was also approved that with the lining segment width increasing the influence of the joints of rings to the whole rings' performance became reduced. And this change above can not be reflected by the equivalent continuance model which generally adds the influence of the joints of rings to the whole ring's performance as the width of lining developed. And, the error of the equivalent continuous model would increase (Table 2) while the width of linings increases.

As the Table 2 shown, the effective ratio of the longitudinal rigidity has been increased when the width of

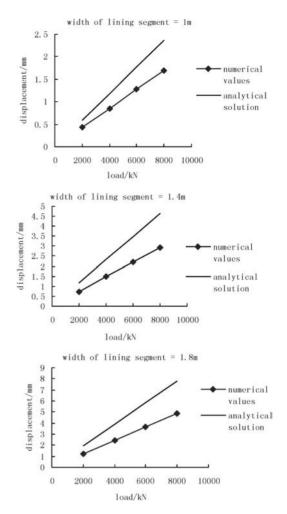


Figure 8. Relationship between the load and the average vertical displacement of the observed points at the free end.

Table 2. Influence of the lining segment width to the effective ratio of the longitudinal rigidity.

Width of lining segment/m	1	1.4	1.8
Equivalent continuous model	1/12.14	1/8.74	1/6.85
Numerical simulation Difference between the above two models (%)	1/8.73 39.1	1/5.51 58.6	1/4.27 60.4

lining increased. Actually, with the number of joints of rings and its influence decreased the longitudinal performance of the segmental lining tunnel has become more similar to the homogeneous tunnel.

# 4 CONCLUSIONS

- 1 The equivalent bending rigidity which is used in the analytical solution of the equivalent continuous model is lower than the actual values. Because of the assumption of the model, the calculated method of the equivalent bending rigidity in the equivalent continuous model couldn't reflect the influence of the segmental arrangement effect which generally causes the delivering discontinuity of the internal force of the linings at the joints of rings.
- 2 With the width of lining increasing, the error of the equivalent bending rigidity in the classical equivalent continuous model would increase.
- 3 When the width of linings is the general range of  $1.0 \text{ m} \sim 1.8 \text{ m}$  used in the shield tunnel engineering in China, the effective ratio of the longitudinal bending rigidity of the common situations of the subway could be in the range of  $1/8.73 \sim 1/4.27$ .

# REFERENCES

Goodman, R.E., Taylor, R.L., & Brekke, T.L. 1968. A model for the mechanics of jointed rock. *Journal of Soil Mechanics and Foundation Division*, ASCE, 94: 1121–1129.

- Hashimoto, T., Zhu, H. H., & Nagaya, J. 1994. A New Model for Simulating the Behavior of Segments in Shield Tunnel. *Proceeding of the 49th Annual Conference of the JSCE*, 1242–1243.
- Huang, H.W., & Xu, L. 2006. Study on Transverse Effective Rigidity Ratio of Shield Tunnels. *Chinese Journal of Geotechnical Engineering*, 28(1):11–18. [in Chinese]
- Huang, H.W., & Zang, X.L. 2002. Research and Analysis on Longitudinal Deformation Characteristics of Shield Tunnel. Underground Space, 22(3):244–250. [in Chinese]
- ITA Working Group 2, 2000, International tunneling association, guidelines for the design of shield tunnel lining. *Tunneling and Underground Space Technology*, 15(3), pp.303–331.
- Lee, K.M., Hou, X.Y., Ge, X.W., & Tang Y. 2001. An analytical solution for a jointed shield-driven tunnel lining. *International Journal for Numerical and Analytical Methods in Geomechanics*, 25(4):365–390.
- Zhu, H.H., Yang, L.D., & Chen, Q.J. 1996. Two design models for segment joint of lining system in shield tunnel. *Engineering Mechanics, Supplement*, 395–399. [in Chinese]

# Author Index

Ahmadi-adli, M. 295, 301 Ai, X.O. 499 Alhieb, M. 649 Amorosi, A. 615 Antiga, A. 365 Arslan, U. 459 Assis, A.P. 519 Augarde, C.E. 785 Avdarsolkyzy, S. 751 Baimakhan, A.R. 751 Baimakhan, R.B. 751 Bakker, K.J. 243, 249, 255 Bao, H.L. 757 Bao, W. 817 Bao, W.Y. 215 Bao, X.H. 507 Batali, L. 187 Bezuijen, A. 3, 243, 249, 255, 261, 281, 349, 357 Boldini, D. 615 Bolton, M.D. 15 Broere, W. 621 Burghignoli, A. 627 Cai, Y.M. 465 Caporaletti, P. 627 Chan, Y. 471 Chang, G.M. 133 Chao, H.C. 67 Cheah, C.K. 447 Chen, C. 513 Chen, D.C. 579 Chen, J. 761 Cheng, L.C. 799 Chi, Y. 491 Chin, C.T. 67 Chiorboli, M. 365 Chissolucombe, I. 519 Cho, G.C. 45, 717 Choi, J.H. 669 Chuay, H.Y. 393 Chung, S.S. 567 Cong, L. 235 Cui, Z.D. 769, 843 Danaev, N.T. 751 Dashdorj, S. 751 Date, K. 635 Deng, A.Z.G. 269 Dias, D. 373

Dijkstra, J. 621 Ding, G.Y. 545 Ding, W.Q. 525 Ding, Y.H. 455 Du, J.H. 643 Eclaircy-Caudron, S. 373 Emeriault, F. 77, 207, 689 Erridaoui, A. 385 Farias, M.M. 519 Fillibeck, J. 275 Finno, R.J. 87 Gafar, K. 281 Gao, W.J. 607 Geraldni, Y.D.S. 655 Gong, Q.M. 381 Guatteri, G. 287 Guiloux, A. 385 Guo, B.H. 531, 799 Guo, W. 817 Gutierrez, M. 537 Hajialilue-Bonab, M. 295, 301 Han, C. 513 Han, X. 331, 545, 725 Hashimoto, A. 173, 405 Hashimoto, T. 99, 173, 307 He, C. 313 Hoefsloot, F.J.M. 357, 775 Hong, C. 781 Hou, J. 551 Hrustinec, L. 325 Hsi, J. 141 Hsiung, B.C.B. 393 Hu, M.Q.S. 385 Hu, Q.F. 595, 601 Hu, X.D. 319 Huang, H.W. 159, 215, 507, 595, 601, 643, 729, 757 Huang, M.S. 829, 849 Huang, R. 29 Huang, Z.G. 863 Idris, J. 649 Ito, H. 173 Iwata, N. 655 Jackson, P.G. 663 Jeon, S. 781

Jiang, J. 419 Jiang, X.H. 441 Kasper, T. 663 Kastner, R. 77, 207, 373, 689 Katebi, H. 295, 301 Kikumoto, M. 709 Kim, D.H. 555 Kim, I. 677 Kim, J. 717 Kim, S.B. 485 Kim, S.M. 399 Kim, U.Y. 555 Kokubun, T. 141 Konda, T. 173, 307, 405 Kong, X.L. 153 Koshima, A. 287 Kou, X.Q. 863 Koungelis, D.K. 785 Kulmaganbetova, Zh.K. 751 Kuzma, J. 325 Lam, S.Y. 15 Lam, T.S.K. 561 Languette, F. 385 Latt. K.M. 579 Le Bissonnais, H. 385 Lee, H.Y. 555 Lee, I.M. 45, 823 Lee, J.S. 555 Lee, S.D. 669, 677 Lee, S.P. 555 Lee. W. 567 Lee, Y.J. 485, 683 Leung, L.P.P. 471 Li, J. 499, 817 Li, J.P. 573, 697 Li, L. 147 Li, X.X. 165, 791 Li, Y. 689 Li, Y.Q. 153 Li, Z.H. 159 Li, Z.X. 331, 725 Liang, F.Y. 697, 849 Liang, Q.H. 455 Lin, Y.L. 165, 791 Liu, D.P. 413, 587 Liu, G.B. 227, 413, 419, 433, 587,805 Liu, J.H. 465

Jeong, K.H. 669

Liu, S.T. 427 Liu, T. 227, 433 Liu, X. 491 Lopes, R. 287 Lu, T.D. 223 Lu, T.K. 531, 799 Lu, Z.P. 805 Lüftenegger, R. 193 Ma, X.F. 307, 477 Ma, Z.Z. 477, 587 Mair, R.J. 635, 703 Makhanova, A.A. 751 Marcu, A. 187 Marlinge, J. 385 Marshall, A.M. 703 Matsumoto, A. 761 McNamara, A.M. 735 Mei, Y.B. 441 Meissner, S. 459 Michael, J. 459 Mitew-Czajewska, M. 201 Nagaya, J. 307 Nakahara, E. 709 Nakai, T. 655, 709 Negro, A. 811 Ng, C.C. 447, 579 Ng, C.W.W. 419, 433 Ng, N.W.H. 471 Niinomi, M. 655 Osborne, N.H. 447, 579 Osman, A.S. 15 Ota, H. 173, 405 Pang, P.L.R. 109, 567 Phienwej, N. 181 Pi, A.R. 319 Pieroni, M.R. 287 Popa, H. 187 Qiao, H.C. 441 Oiu, D.W. 455 Quick, H. 459 Ravaglia, A. 287 Roberts, K.J. 567

Rudi, J. 579

Ryckaert, J. 385 Rysbaeva, G.P. 751 Sabetamal, H. 295, 301 Salgaraeva, G.I. 751 Sanders, M.P.M. 261, 281 Scarpelli, G. 627 Scharinger, F. 193 Schweiger, H.F. 193 Sfriso, A. 121 Sfriso, A.O. 335 Shahin, H.M. 655, 709 Shi, Z. 817 Shin, J.H. 823 Shin, Y.J. 823 Siemińska-Lewandowska, A. 201 Soga, K. 281, 635 Song, K.I. 717 Song, T.T. 343 Song, X.Y. 829 Sozio, L.E. 837 Sugimoto, M. 761 Tabata, Y. 709 Talmon, A.M. 3, 349, 357 Tan, G.H. 579 Tang, Y.Q. 769, 843 Taylor, R.N. 627, 735 Teng, L. 313 Thiebault, H. 385 Tu, M. 551 van Tol, A.F. 261, 281 Verdel, T. 649 Viel, G. 385 Vogt, N. 275 Wang, J. 799 Wang, J.Q. 235 Wang, K.S. 331, 725 Wang, R. 413, 587 Wang, R.L. 465, 573 Wang, X.M. 729 Wang, Z.Q. 857 Wang, Z.W. 427

Xia, C.C. 537 Xie, K.H. 153 Xie, X.Y. 729 Xu, Q.W. 477 Xu, S.F. 223 Xu, Y. 525 Yan, J.Y. 313, 573 Yan, Y.R. 595 Yanagawa, T. 173, 405 Yang, H.Y. 399 Yang, J.W. 669 Yang, L. 513 Yang, M. 147 Yang, S.L. 455 Yao, C.P. 601 Yao, G.S. 697 Yao. H. 607 Yao, J. 735 Ye, B. 99 Ye, G.L. 99, 307 Yoo, C. 485 Yoo, C.S. 683 Yoon, S.G. 399 You, G.M. 743 You, K. 781 Yuan, Y. 491 Zghondi, J. 207 Zhang, C.R. 849 Zhang, D.M. 215, 757 Zhang, F. 655, 709 Zhang, H. 141, 223 Zhang, J. 227 Zhang, L.M. 857 Zhang, M.X. 551 Zhang, Q.H. 269 Zhang, X. 769, 843 Zhang, Z.X. 689 Zhao, H.L. 491 Zhong, X.C. 863 Zhou, H.B. 607 Zhou, J. 153, 235 Zhou, K.Q. 455 Zhou, S.H. 343, 381 Zhu, H.H. 791 Zhu, J. 817 Zhu, W. 863 Zhu, Y.M. 441

Wong, K.K.W. 471

This volume comprises a collection of four special lectures, six general reports and 112 papers presented at the Sixth International Symposium of Gentechnical Aspects of Underground Construction in Soft Ground (IS-Shanghai) held between 10 and 12 April 2008 in Shanghai, China.

The Symposium was organised by Tongji University and supported by China Civil Engineering Society, Chinese Society for Chinese Society for Rock Mechanics and Engineering, Geotechnical Division, the Hong Kong Institution of Engineers, Hong Kong Geotechnical Society, Hong Kong University of Science and Technology, Science and Technology Commission of Shanghai Municipality, Shanghai Changjiang Tunnels and Bridge Development Co. Ltd and Shanghai Society of Civil Engineering. This was the most recent symposium in a series of symposia starting in New Delhi, India (1994), followed by symposia in London, UK (1996), Tokyo, Japan (1999), Toulouse, France (2002) and Amsterdam, the Netherlands (2005).

The four invited special lectures from A. Bezuijen, Huang Rong, M.D. Bolton and I.K. Lee are about "Processes around a TBM", "Overview of Shanghai Yangtze river tunnel project", "Supporting excavations in clay - from analysis to decision-making" and "Underground construction in decomposed residual soils", respectively.

The six general reports from Richard Finno, Tadashi Hashimoto, Alejo Sfriso, C.T. Chin, Richard Pang and Richard Kastner, cover the symposium themes:

- 1. Analysis and numerical modelling of deep excavations
- 2. Construction method, ground treatment, and conditioning for tunnelling
- 3. Case histories
- 4. Safety issues, risk analysis, hazard management and control
- 5. Physical and numerical modelling
- 6. Calculation, design methods, and predictive tools

This volume provides a valuable source of information on the the state-ofthe-art in geotechnical engineering associated with the design, construction and monitoring of tunnels and excavations in soft ground, and will be of interest to academics and professionals involved in these areas.



6000 Broken Sound Parkway, NW Suite 300, Boca Raton, FL 33487 Schipholweg 107C 2316 XC Leiden, NL 2 Park Square, Milton Park Abingdon, Oxon OX14 4RN, UK



an **informa** business