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TUNNELLING IN WEAK ROCKS

BHAWANI SINGH
and RAJNISH K. GOEL

ELSEVIER GEO-ENGINEERING BOOK SERIES
VOLUME 5

Tunnelling in Weak Rocks

Dedicated to
Practicing Engineers, Scientists,
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ELSEVIER GEO-ENGINEERING BOOK SERIES
VOLUME 5

Tunnelling in Weak Rocks

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Series Preface

The objective of the Elsevier Geo-Engineering Book Series is to provide high quality books on subjects within the broad geo-engineering subject area – e.g. on engineering geology, soil mechanics, rock mechanics, civil/mining/environmental/petroleum engineering, etc. The first four books in the Series have already been published:

- "Stability Analysis and Modelling of Underground Excavations in Fractured Rocks" by Weishen Zhu and Jian Zhao;
- "Coupled Thermo-Hydro-Mechanical-Chemical Processes in Geo-systems" edited by Ove Stephansson, John A Hudson and Lanru Jing;
- "Ground Improvement – Case Histories" edited by Buddhima Indraratna and Jian Chu; and
- "Engineering Properties of Rocks" by Lianyang Zhang.

Now, I am pleased to introduce "Tunnelling in Weak Rocks" by Bhawani Singh and R.K. Goel. The authors have placed their emphasis in exactly the right area because it is much more difficult to tunnel in a soft, weak rock mass than in a stiff, strong rock mass. Also, they have set their stage in the Himalayas which is an exciting setting, not only on the surface but often even more so underground!

Readers will recall the 1999 Elsevier book written by the same authors: "Rock Mass Classification: A Practical Approach in Civil Engineering". This earlier book has proved to be a most useful reference source because all the key information relating to rock mass classification is contained in the book and so one automatically takes it off the shelf whenever there is a question about the rock mass classification approach or the associated details. The authors have adopted the same approach with "Tunnelling in Weak Rocks": they provide 29 chapters covering all aspects of the subject, including theory, reviews of rock mass classification approaches, the different types of tunnelling methods, excavation and support, hazards, instrumentation, swelling and squeezing rock conditions and many other practical aspects of tunnelling.

We hope that you enjoy the book and we welcome proposals for new books. Please send these to me at the email address below.

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Preface

"A book is a man's best friend."

Groucho Marx

The basic approach in the design of underground support system has been an empirical approach based on rock mass classification. This approach was the subject of the authors' first book, *Rock Mass Classification – A Practical Approach in Civil Engineering (1999)*, which has been enjoyed by the experts all over the world. Lately, however, a growing need for reliable software packages to aid engineering control of landslide and tunnelling hazards has inspired the writing of the next book on *Software for Engineering Control of Landslide and Tunnelling Hazards* based on the use of a rational approach to check the empirical predictions to be sure of the solution.

The instant liking and success of these two books further boosted our morale and we have written this book on *Tunnelling in Weak Rocks*, which is based on intensive field-oriented research work and experience. It is expected that the book will generate more confidence and interest among civil and mining design and construction engineers, geologists, geophysicists, managers, planners, researchers and students. The set of three complementary books that we have produced has been possible due to God's grace, team-work and worldwide acceptance and moral support.

Emphasis is given to the practical-construction solution of tunnelling hazard control rather than any rigorous analytical/numerical methods. Practical knowledge of the engineering behavior of rock masses, discontinuities, the time-tested classification approach, tunnelling hazards, and simple analytical methods are also offered to add to the understanding of realistic actual construction approach.

We have been blessed by modern tunnelling machines and shielded TBM with automatic support system to bore rapidly through soils, boulders and weak rocks, etc. By the grace of God, the modern tunnel engineers have tremendous confidence now. This book also tries to integrate the happy experience of tunnel engineers, managers, reputed field researchers and famous site engineering geologists from all over the world. This book may help in on-spot-decisions during tunnelling.

Himalaya is a vast region, an amazingly beautiful creation which possesses extensive rejuvenating life support system. It is also one of the best field laboratories for learning rock mechanics, tunnelling, engineering geology and geohazards. The research experience gained in Himalaya is precious to the whole world.

The authors are deeply grateful to Professor J. A. Hudson, Imperial College of Science and Technology, London, and President-elect, International Society for Rock Mechanics (ISRM) for continuous encouragement and for including this book in the Elsevier Geo-Engineering Series. The authors are also thankful to Elsevier Limited for publishing the book.

The authors' foremost wish is to express their deep gratitude to: Professor Charles Fairhurst, University of Minnesota; Professor E. Hoek, International Consulting Engineer; Dr. N. Barton, Norway; Professor J.J.K. Daemen, University of Nevada; Dr. E. Grimstad, NGI, Professor G.N. Pandey, University of Swansea; Professor J. Nedoma, Academy of Sciences of Czech Republic; Professor Zhao Jian, Nanyang Technological University, Singapore; Professor V.D. Choubey; Professor T. Ramamurthy, IITD; Dr. R.K. Bhandari, CSIR; Mr. B.B. Deoja, Nepal; Mr. A. Wagner, Switzerland; Professor R.N. Chowdhary, Australia; Professor S. Sakurai, Japan; Dr. R. Anbalagan, IITR; Professor M. Kwasniewski, Poland; Dr. B. Singh; Professor B.B. Dhar, Dr. N.M. Raju, Dr. A.K. Dube, Dr. J.L. Jethwa, Dr. V.M. Sharma, ATES; Late Professor L.S. Srivastava; Professor Gopal Ranjan, COER; Professor P.K. Jain, IITR; Professor M.N. Viladkar, IITR; Dr. A.K. Dhawan, CSMRS; Dr. V.K. Mehrotra; Dr. H.S. Badrinath; Dr. Prabhat Kumar, CBRI; Dr. P.P. Bahuguna, ISM; Dr. Subhash Mitra, Uttaranchal Irrigation Department; Dr. R.B. Singh, Tala Hydroelectric Project, Bhutan; Dr. Mahendra Singh, IITR; Dr. N.K. Samadhiya, IITR; Mr. H.S. Niranjan, HBTI and Dr. Rajesh K. Goel, ONGC for their constant moral support and vital suggestions and for freely sharing precious field data. The authors are also grateful to the scientists and engineers of CMRI, CSMRS, UPIRI, IIT Roorkee, IIT Delhi and ATES, AIMIL, HEICO, VS Engineering Services, New Delhi and to all project authorities for supporting the field researches. The authors are also grateful to Mr. N.P. Atterkar and Mr. Sandesh Atterkar, Soilex Ltd., Roorkee for kind support. Special thanks to Dr. Daya Shankar, IITR and Dr. A.K. Chakraborty, CMRI for sharing their research work and contributing chapters on "Application of Geophysics..." and "Blasting for Tunnels and Roadways", respectively. Thanks to Professor Yuzuru Ashdia, Kyoto University, Japan for allowing us to use his work in Chapter 2.

The authors are also very grateful to their families and friends for their sacrificing spirit. Without their support the writing of this book would have been very difficult.

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All engineers and geologists are requested to kindly send their precious suggestions for improving the book to the authors for the future editions.

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1
Introduction

"College is where you learn how to learn."

Socrates (470–399 B.C.)

Tunnelling is a joy. Tunnelling is an art. Tunnelling in weak rock masses is an adventure. Tunnels are liked by people all over the world. Tunnels are expected to become future craze of people in view of high level of curiosity of people and its uses to the society. Tunnels attract tourists, specially along hill roads and hill rail lines. Underground metros are popular and safer than surface transport, as escape routes are minimum. Tunnels are safe even during earthquakes of high intensity. Moreover the underground space technology has improved the ecosystem and environmental conditions.

Classical books of Szechy (1967), Bienisawski (1984), Bickel et al. (1997) and Hoek et al. (1995) deal with the subject of tunnelling generally in hard rocks. Bieniawski (1984) have given the history of tunnelling which is very interesting. Fascinating undersea tunnels (immersed tube road and rail tunnels all over the world) have been described by Culverwell (1990). Himalayan region is the best field laboratory to learn Rock Mechanics and Tunnelling Technology for weak rocks. Thus, the experiences of tunnelling in the tectonically disturbed, young and fragile Himalaya are precious for the tunnel engineers all over the world. The Himalaya provides the acid tests for the theories and tunnelling technologies. Therefore, Himalaya is a boon for all of us.

Prof. Charles Fairhurst once said that only a strategy of tunnelling can be designed. The design of support system may not be possible in complex geological and geohydrological conditions. Geologically complex and high mountains have big EGO (Extraordinary Geological Occurrences) problems. Geological surprises are common along deep and long tunnels (>1 km long) in young and tectonically disturbed high mountainous terrains. Geological surprises (faults/shear zones) may be discovered even after the completion of a tunnel. Thus, the designed strategy should be flexible enough to strengthen the tunnel locally near unexpected geological weaknesses, whenever discovered. Thus planning should be flexible and not rigid unlike in other civil engineering projects. In hard rocks, the art of tunnelling has evolved into a science of tunnelling with the Grace of God.

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Exploration is the weakest link in a deep long tunnelling project. There are practical difficulties in making drill-holes along a long and deep tunnel alignment in mountainous terrain as neither drilling machine can be transported on mountain top nor water is available. Generally exploration pits are made to get some idea of geological cross section. But errors of extrapolation of rock layers on the basis of observed dips at the top may be serious in a folded and faulted strata. Adits are generally made for geological exploration. Civil engineers need the engineering geological cross section in addition to a reliable geological cross section. Civil engineers should, therefore, drill a probe hole behind the tunnel face for an advance knowledge of the tunnelling ground conditions. This probe hole may also act as drainage hole in unknown water-charged strata. Engineering judgment plays a very important role during tunnelling in the weak rocks.

The properly designed tunnel boring machine (TBM) is a good choice in the homogeneous rock masses in the non-squeezing ground condition ($H < 350\ Q^{1/3}$ m) without shear zones and non-flowing rock conditions. Engineers should not use TBM where engineering geological investigations have not been done in detail and the rock masses are very heterogeneous. Contractors can design TBM according to the given rock mass conditions which are nearly homogeneous (Bhasin, 2004).

Tunnel mechanics plays an important role in planning and construction of tunnels. A deep and long tunnel should be carefully planned to avoid too high overburden causing squeezing ground condition or rock bursts; water charged or active faults and flowing ground conditions. It should be realized that the same strata which is safe may pose severe tunnelling problems when met again along a tunnel alignment but under a very high overburden. Tunnelling was done before tunnel mechanics was developed around 1970. Tunnelling hazards are better understood now and are tackled more effectively. Rock engineers and engineering geologists should be employed at major tunnelling projects for safety of workers and tackling the tunnelling hazards, etc. Good support will reduce the cost over-runs and delays in completion of tunnels. Section 4.6 and Figs 11.2–11.6 underline the importance of geological investigations in the deep and long tunnels within the young mountains.

The combined New Austrian Tunnelling Method (NATM) and Norwegian Method of Tunnelling (NMT) have been used extensively in the conventional method of tunnelling by drilling and blasting. Tunnelling machines are very helpful in rapid excavation and supporting. The NATM (Chapter 9) gives strategy of tunnelling through various ground conditions. The NMT (Chapter 10) offers a design chart for support system. The steel fiber reinforced shotcrete (SFRS) is fortunately found to be generally successful in supporting weak rocks and mild to moderate squeezing grounds. The full-column grouted rock bolts (grouted anchors) are better choice than pre-tensioned rock bolts in supporting weak rocks. Naturally SFRS with grouted rock bolts is the ideal choice in case of weak rocks where feasible. It should be understood that high support pressures be reduced significantly by allowing certain amount of tunnel closures in the case of squeezing grounds. Engineers and geoscientists should be congratulated for safe tunnelling in the modern times. The fear of tunnelling at great depths (>1000 m) is no more there. This book tries to offer a modern integrated strategy of tunnelling through weak rocks for twenty-first century (Chapter 28).

Management conditions affect the rate of tunnelling surprisingly sometimes more than the geological conditions. Therefore, improvement in the management condition is very important. The managers must make efforts to make the contractors successful and efficient in challenging jobs like tunnelling in weak and disturbed rocks. *The spirit of co-operation and commitment (i.e., mutual trust and benefit) should be created by executives*. Risk management is specially important in tunnelling. Contractors should insure lives of workers and TBM and tunnelling machines. All the willpower of civil engineers is concentrated on the fast completion of a project. Their psychology is not to tolerate any hindrance in the enthusiasm of construction activity. A tunnel instrumentation is, therefore, disliked by civil engineers at the tunnelling project. In fact, all that we have learnt today about Rock mechanics is due to tunnel instrumentation. The reliable and continuous monitoring by modern tunnel instruments is the key to success in tackling unexpected tunnelling hazards. This is shown by extensive experiences of Central Mining Research Institute in India.

The later half of twentieth century has been called as the dawn of the golden era of tunnelling all over the world. Nothing succeeds like success. In about 50 years, many deep and long tunnels were built through the Alps and the Rocky mountains. The 34 km long Loetschberg tunnel under Swiss Alps, set to open to trains in 2007 is now the longest over land tunnel. Another tunnel – 58 km long Gotthard tunnel parallel to the Loetschberg tunnel will be the world's longest tunnel when it is completed by 2020.

The tunnel engineers, geologists and managers should be trained for the challenges of the future. Bieniawski (1984) suggested that the following lessons should be learnt from the precious past field experiences.

1. Tunnelling in hard rocks has now become a science from the empirical art of tunnelling.
2. The great advances in tunnelling technology were due to team efforts but often depended upon the leadership of a single man.
3. The engineer's ingenuity has been amply proved in the past but full potential is yet to be discovered and many new inventions and breakthroughs are awaiting us in the future. A Rock engineer should be in charge of a tunnelling project for its efficient management.
4. Modern shielded tunnel boring machine may be successful in all homogeneous rocks, soils, boulders and fault zones, etc.

Due to the Grace of God, the future of tunnelling and underground space technology appears to be good. Under city bypass tunnels along highways is becoming popular as in Australia. Demand for multiple level underground metros with many lanes is increasing rapidly. The expectation is that automation in tunnelling even in weak rocks will advance rapidly. Drinking water tunnel network is an immediate necessity in over-populated nations. Interlinking of rivers is also feasible on large scale. Under sea tunnels are going to catch up imagination of planners. A Tunnel between London and New York is being planned in Atlantic Ocean. Underground cities may be feasible economically by the end of twenty-first century. United States of America invented rock melting drilling machine in 1972. It is learnt that a TBM is developed on this principle. It uses a tiny nuclear reactor to heat its head to about 2000°C. It thereby burns the rock mass into a gas. Thus a tunnel of glassy rock is created. But it is a very costly method of tunnelling. Norway is experimenting upon floating tunnels on the lakes and oceans. Vision is very good. Future engineers and scientists are going to be very bold and most efficient in using energies.

The underground structures are permanent property of the people, protected from all kinds of natural disasters, unlike surface structures in the disaster prone regions which are temporary property of the people.

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2

Application of geophysics in tunnelling and site survey activities*

"And so geology, once considered mostly a descriptive and historical science, has in recent years taken on the aspect of an applied science. Instead of being largely speculative as perhaps it used to be, geology has become factual, quantitative, and immensely practical. It became so first in mining as an aid in the search for metals; then in the recovery of fuels and the search for oil; and now in engineering in the search for more perfect adjustment of man's structures to nature's limitations and for greater safety in public works."

Charles P. Berkey, Pioneer Engineering Geologist, 1939

A modern technique in underground construction needs to use modern knowledge, which is state-of-the-art. At turn of the millennium, and with the ever increasing number of underground excavations, it has become all the more important that excavations are made economically and are safe. The modern geophysical techniques, the concept and methodology, and its application in underground construction especially in tunnels have been discussed in this chapter.

Initial development of geophysical techniques to determine the geological structure of the sub-surface was stimulated primarily by the search for potential reservoirs of petroleum and natural gas. Today, geophysical techniques are being developed for application not only to the search for deeper reservoirs of petroleum and natural gas, i.e., depths of the order of several kilometers, but also underground openings i.e., depths of the order of 100 m or less below surface, driven by the need to make effective use of underground space. This trend has been promoted by recently established special regulations governing public use of underground space in many countries.

2.1 GEOPHYSICAL EXPLORATION

Geophysical exploration may be defined as the application of non-invasive (i.e., no excavations) techniques for identification of sub-surface structures and the associated physical

*Contributed by Dr. Daya Shankar, Department of Earthquake Engineering, IIT Roorkee, India.

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properties of the rocks. The location and distribution of faults and fractured zones are now routinely identified by geophysical techniques.

Table 2.1 shows the geophysical techniques currently used and the geophysical phenomenon on which they are based. Fig. 2.1 summarizes the various techniques in terms of their application in forward and inverse analysis procedures.

Forward analysis defines techniques that are designed to acquire geophysical data with a high S/N (signal to noise amplitude) ratio, whereas inverse analysis concerns techniques designed to reconstruct the structure of the sub-surface with high resolution by interpretation of this data. The S/N ratio is defined by equation (2.1) (Ashida, 2001).

$$\text{dB (decibel)} = 20\ \log_{10} \text{S/N} \tag{2.1}$$

Where dB is the ratio expressed in decibels.

Resolution defines the ability to separate two features. In terms of geological structure, this is measured by the ability to distinguish thin layers. In the case of a reflection seismic survey, the resolution in the vertical direction is limited to one-quarter of the wavelength. For example, if the seismic velocity (V) of a wave is 2000 m/s and the frequency (f)

Table 2.1 Geophysical exploration techniques and geophysical phenomenon.

Seismic exploration	Reflection and refraction of seismic waves
Electric sounding	Resistivity and induced polarization phenomenon
Electromagnetic method	Induction phenomenon
Gravity survey	Density
Magnetic survey	Susceptibility
Radiometric survey	Scattering phenomenon of γ-ray, radon
Geothermal prospecting	Geothermal phenomenon of geothermal gradient, heat flux
Well log	Geophysical prospecting using borehole
Geotomography	Geophysical prospecting between boreholes

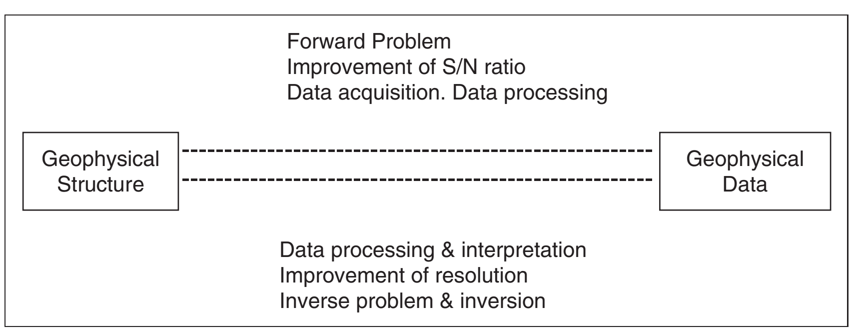

Fig. 2.1 Forward and inverse problem in geophysical techniques.

of the reflected wave is 50 Hz then, from the relationship $V = f \cdot \lambda$; one-quarter of the wavelength (λ) will be 10 m. Consequently, it is not possible in this case to resolve a layer less than 10 m thick. Since one cannot control the seismic velocity of layer, increased resolution depends on improving the ability to detect the high frequency waves emitted from the seismic sources. Considerable progress is being made in this area, aided by the recent technical innovations in electronics. The principal developments in geophysical techniques include:

- Universal application of three-dimensional geophysical surveys.
- Introduction of S wave seismic reflection surveys using three-component receivers.
- Data processing and interpretation using man–machine interaction.
- More comprehensive use of various prospecting techniques.
- Interpretation of physical properties and integrated interpretation.
- Introduction of four-dimensional surveys, i.e., periodic surveys for monitoring.
- Increased application of virtual reality techniques.
- Introduction of new mathematical and physical concepts.

Factors to be considered in applying geophysical techniques to tunnelling technology and geotechnical engineering are as follows:

- Knowledge of the physical properties of target objects to aid in selecting the geophysical technique to be applied.
- Awareness of the physical scale of the target objects to help define the survey parameters.
- Good understanding of each geophysical technique and its limitations.
- The value of integrated interpretation using several geophysical techniques to increase the accuracy of interpretation.
- Recognition of limitations on the accuracy of interpretation.

Geophysical methods may be reliable in sites where there is a contrast between properties of rock materials. A variety of measurement techniques and surveys are used in geotechnical engineering in order to determine the physical properties required in the design of structures in or on rocks. The accuracy and reliability with which these are determined controls the factor of safety assumed during planning of a project and consequently has an effect on the construction cost. Table 2.2 lists geophysical techniques currently used in geotechnical engineering.

2.2 EXAMPLES OF APPLICATION

The following section presents several examples of applications of geophysical techniques to tunnelling technology.

Table 2.2 Geophysical exploration techniques in the geotechnical and rock engineering fields.

Technical field	Object	Geophysical techniques
A. Civil and construction field	Looking ahead of tunnel face	Horizontal seismic profiling electromagnetic method
(1) Tunnel boring	Survey for tunnel	Reflection seismic sounding
	Survey for loosened areas by tunnel boring	Geotomography
(2) Road construction	Grasp of fractured zone	Electromagnetic method
(3) Strength improvement of thin ground-layer	Evaluation of strength improvement of thin ground-layer	Reflection seismic; PS logging
B. Energy development field		
(1) Gas pipe setting	Setting of gas pipes without excavation	Ground penetrating radar
C. Environment and disaster prevention field		
(1) Active fault survey	Determination of location of trench	Reflection seismic and electric sounding
(2) Landslide survey	Prediction of landslide	Electromagnetic micro-seismic method
(3) Cavity detection in river bank	Detection of cavity in river bank	Three-dimensional reflection seismic
(4) Underground disposal of industrial wastes	Leakage of water at wastes location	Electric sounding
D. Conservation of national land		
(1) Survey of archeology	Discovery of new ruins	Electric sounding
	Repair of statue	Thermal-infrared
E. Maintenance field		
(1) Diagnosis of concrete	Detection of fracture and cavity	Ground penetrating radar; thermal-infrared; sounding by hammer
F. Defence/Military field		
(1) Blasting of land mine	Removal of land mine	Energy sources

2.2.1 Looking ahead of a tunnel face

Geophysical surveys to investigate the geological conditions ahead of a tunnel face are conducted using both elastic and electromagnetic waves. In this application, looking ahead of tunnel face, a short turn-around time is essential in order to identify the geological conditions with high accuracy, without interrupting drilling operations at the face and without changing the sequence of field operations. A schematic diagram of field data acquisition in looking ahead of a tunnel face using elastic waves is shown in Fig. 2.2. Several energy sources for the elastic waves are available at the tunnel face, i.e., a 100–200 g charge of dynamite, delay blasting at the tunnel face, or vibrations induced by the TBM (Tunnel Boring Machine) as it cuts the rock. The seismic disturbances from the geological boundaries are received by three-component geophone placed on each side of the tunnel wall and fed to data recorders. The reflection waves of positive polarity generated from approximately 40 m on the left side, and about 60 and 90 m on the right side, ahead of the tunnel face indicate changes from hard rock to soft rock, as the polarities indicate a negative change in compressive strength of the rock. Fig. 2.3 is the inversion result of electromagnetic survey data using transmitter and receiver placed on the tunnel face. This figure is in good agreement with the resistivity distribution on the observation report.

2.2.2 Geological information required before tunnelling

The principle of the helicopter-borne electromagnetic method is shown in Fig. 2.4. The transmitter, carried on the helicopter generates the primary electromagnetic field and the receiver, also on the helicopter, observes the secondary electromagnetic field

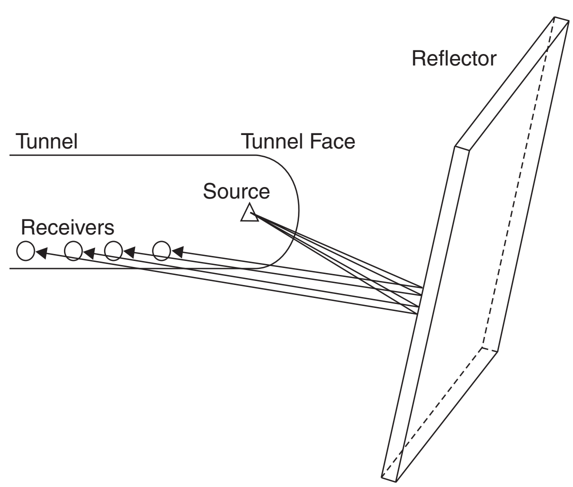

Fig. 2.2 Schematic map of data acquisition ahead of a tunnel face.

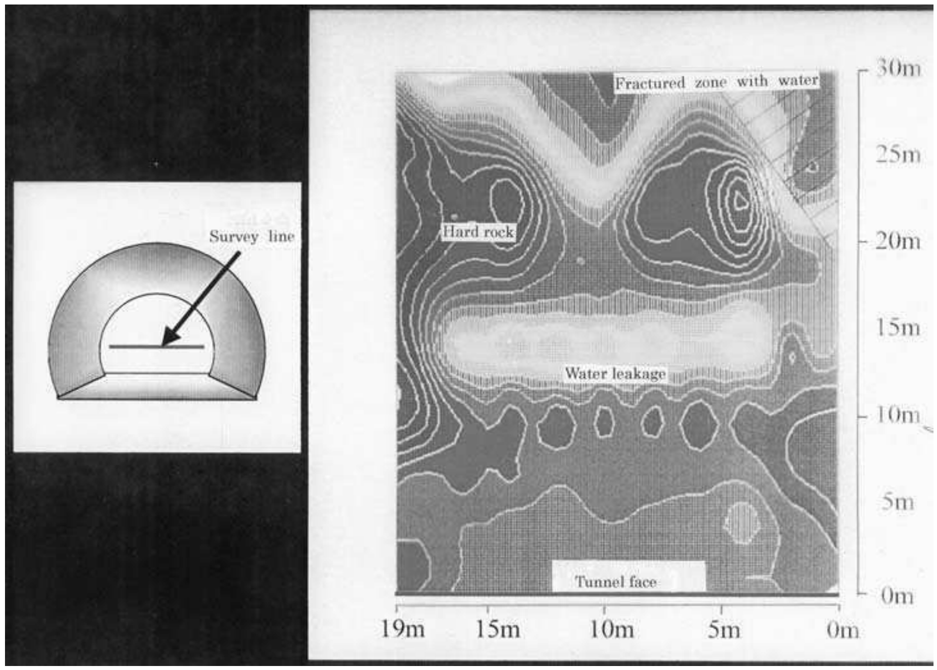

Fig. 2.3 Inversion result of electromagnetic survey for looking ahead of tunnel face.

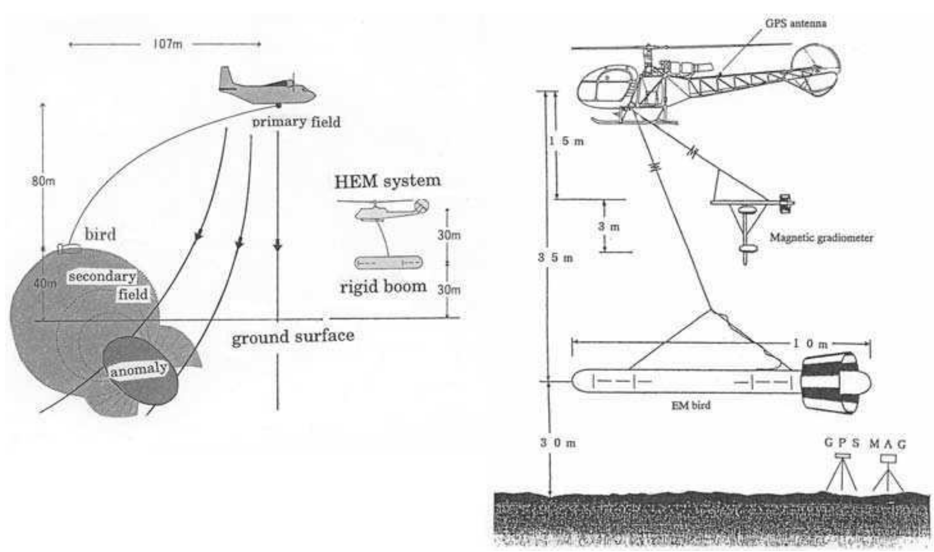

Fig. 2.4 Schematic map of helicopter-borne electromagnetic survey.

programed to cancel the principal field. The geo-magnetic field is also observed simultaneously.

Ashida (2001) also presented the results of a helicopter-borne electromagnetic survey performed to predict geological conditions ahead of the proposed route of a tunnel. This procedure allows geological information for the proposed tunnel alignment to be acquired before tunnelling is started. This is a modern trend for feasibility studies. Big firms have used this technique in unapproachable hilly terrains for submitting the tenders.

There is an approximate correlation between rock mass quality (Q) and P-wave velocity according to equation (5.15) and Fig. 5.5. The details are presented briefly in Section 5.8.3. Once the rock mass quality is determined by seismic refraction survey ahead of the tunnel face, the support design may be adopted from Fig. 10.2 for different rock conditions.

Electrical resistivity survey is good in exploring the water-charged fractured zones in rock masses. Thus, one may puncture these zones by advance drill holes ahead of tunnel face before tunnelling. Extensive water-charged zones can exist surprisingly in the hard rocks like granites, basalts, quartzites, etc. and can pose serious tunnelling problems/hazards.

2.2.3 Strength improvement of thin ground layers

The improvement of thin ground layer was carried out to eliminate adverse ground erosion effects of a river passing through a city. Weak, thin ground layers in the river bank and river bed were strengthened by injecting cement grout from a borehole almost continuously upto selected depth at intervals upto approximately 2.0 m. This was done in order to avoid differential settlement of facilities such as communication cables and drainage pipes.

The strength of weak, thin ground layers may be improved in various ways, including:

- Damming off of the river over one-half of its width to allow treatment of the river bottom layers.
- Strength improvement of selected depth intervals upto approximately 2 m.
- After removal of the soil above the strength-improved layer, installation of facilities such as communication cables and drainage pipes on the strength-improved (stabilized) layer.
- Similar treatment of the other half of the river.
- After reclamation, effective utilization of ground surface as a park.

In assessing the strength improvement of the thin ground layer, it is important to determine whether or not the compressive strength and the thickness of the strengthened layer meet the requirements.

The S waves are used in preference to P waves because the lower velocity of S waves results in higher resolution at a given frequency. In using the chart, the recorded reflection seismic signals produced by the S-wave energy sources are displayed together with the velocity profile. The width of the strength improvement of thin ground layer is obtained and the velocity of the layer is estimated from the velocity analysis. The relationship between the S-wave velocity V_s (m/s) and the uniaxial compressive strength q_u (MPa) of the layer, as determined from cores, is given by equation (2.2).

$$q_u = 6.0 \times 10^{-7} V_s^{2.3821} \tag{2.2}$$

Equation (2.2) allows the compressive strength of the layer to be determined from the velocity of the thin ground layer after strength improvement. It should be mentioned also that the shallow reflection survey method using S-wave energy sources as described above is applicable to the survey of proposed alignments prior to construction of a tunnel for underground drainage. The S-wave velocity is not affected by saturation, whereas P-wave velocity is increased by saturation due to easier passage through ground water.

2.3 PREDICTION AHEAD OF TUNNEL FACE WITH SOURCE PLACED ON FACE

The new data acquisition and analysis method has been introduced into the seismic refraction method in order to estimate the velocity ahead of the tunnel faces. The approach is the expansion of the high-resolution seismic refraction method by using sources at the tunnel faces. At first, the surface seismic refraction method using sources and receivers only on the surface is performed in the investigation phase. Next, the data acquisition and analysis using in-tunnel sources are repeated in the construction phase. The data acquired in the investigation and construction phases are analyzed simultaneously. The numerical experiments of seismic refraction method including in-tunnel sources have been carried out in order to show the efficiency of the method. The data without in-tunnel sources cannot delineate the velocity model clearly. As the number of in-tunnel sources increased, the resolution of reconstructed models also increased. Specially, the velocity just ahead of the tunnel face can be estimated precisely. The method was applied to an actual tunnel site with an in-tunnel reflection method. A velocity model was obtained from the refraction method with an in-tunnel source and reflector distribution was obtained from the in-tunnel reflection method. The prediction of rock quality ahead of a tunnel face from seismic methods agreed with the excavation records.

One of the methods applied to above objective, is the in-tunnel seismic reflection method, e.g., TSP, HSP (Sattel et al., 1992; Ashida et al., 1999 and Inazaki et al., 1999) in which sources and receivers are placed in a tunnel. The method has been applied to many construction sites although these attempts were mainly in the experimental stage. However, as the number of practical application increases, disagreements between predictions and construction results are often reported and many problems of prediction accuracy are pointed out. The surface seismic refraction method which is carried out in investigation phase is one of the other methods. The method has been applied to many tunnel investigations, as a standard method in Japan, over the years. However, more accurate investigation is required in recent years. The geoscientists have improved traditional seismic new seismographs and an automatic analysis using a computer, and named it High-resolution Seismic Refraction Method (Hayashi & Saito, 1998 and Hayashi, 1999). Although the method has greatly improved accuracy and reliability of seismic refraction results, one cannot say that the method satisfies the requirements completely.

Both the in-tunnel reflection method and the seismic refraction method predict rock quality along a tunnel route by estimating seismic velocity (or reflector) model using seismic waves generated artificially. There are two main causes for disagreement between predictions and construction results. The first is the analytical models, such as velocity models or the reflector distributions obtained from the seismic refraction method and the in-tunnel reflection method, are not accurate. The second is that the interpretation of the analytical results is not appropriate. The first one is the generic problem of methods. For example, two-dimensional seismic methods cannot obtain true velocity models in three-dimensional structures. This problem can be solved by the development of more sophisticated data acquisition and analytical approaches. The second problem is more important. The velocity models or reflector distributions are not related to rock quality directly. For example, although the same seismic velocity (e.g., 4.5 km/s) is obtained, rock quality may have large difference between igneous rocks and Mesozoic sedimentary rocks. It is obvious that other information, such as geological, hydrological in borehole, have to be considered with the results of geophysical explorations, such as the seismic refraction or in-tunnel reflection methods, in order to predict rock quality along a tunnel route with accuracy and reliability.

One of the important problems of the in-tunnel reflection method is difficulty in estimating seismic velocity ahead of the tunnel face. Geoscientists have tried to estimate the velocity ahead of the tunnel face from the precise velocity analysis of reflected waves (Hayashi & Takahashi, 1999a), and concluded that estimation of the velocity ahead of the tunnel face is difficult (Hayashi & Takahashi, 1999b). In order to solve this problem, one has to use sources and receivers surrounding the target area, that is, on the ground surface also and/or in the ground ahead of the tunnel face.

On the other hand, a generic problem in the seismic refraction method is the method using sources and receivers only on the ground surface. This source–receiver geometry limits severely the accuracy and resolution of analysis in case of a thick overburden. In order to solve this problem, one has to use sources and receivers surrounding or within the target area. The idea is to combine the seismic refraction method and in-tunnel seismic reflection method. Geoscientists have developed a new seismic refraction method in which sources are placed not only on the ground surface but also within a tunnel under construction. This method may improve simultaneously both the seismic refraction method and the in-tunnel reflection method.

2.3.1 Method

The investigation program of the new method may be summarized as follows. Before construction, the high-resolution seismic refraction survey is carried out with sources

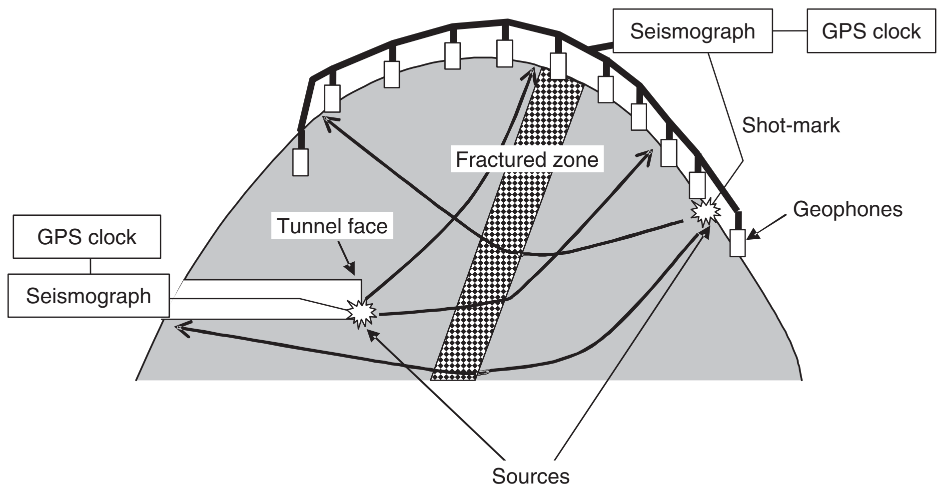

Fig. 2.5 Schematic diagram of data acquisition. Sources are placed not only on the ground surface but also within a tunnel under construction (Hayashi & Saito, 2001).

and receivers only on the ground surface. During construction, the data acquisition and analysis using sources within a tunnel are repeated periodically. As tunnel excavation proceeds, the number of geophones placed within the tunnel increases. Fig. 2.5 shows the schematic diagram of the method. GPS clocks are employed to synchronize the sources within a tunnel and receivers on the ground surface. (GPS does not work in tunnels, as open sky is needed.)

A seismic refraction analysis is repeated with the data obtained both before and during construction. The analysis is based on a non-linear least square method in which a forward modelling and first-order simultaneous equations are iteratively performed to minimize travel time error acquired both before and during the construction. The ray tracing based on the shortest-path calculation is used as the forward modelling (Moser, 1991) and SIRT (Simultaneous Iterative Reconstruction Technique) is used for solving first-order simultaneous equations.

2.3.2 Numerical example

Numerical tests have been performed in order to prove the efficiency of the method. Fig. 2.6 shows the velocity model used in the numerical tests and source locations. Theoretical travel times calculated by ray tracing for this model are considered as observed data. A velocity model that may satisfy the travel time data is obtained by a linear least square method.

Not only the travel time data but also ray paths have to be known to obtain a velocity model by linear inversion. In an actual seismic refraction method, an iterative method is used as mentioned before because both velocity model and ray paths are unknown.

True ray paths are, however, given in this study in order to simplify the problem. Consequently, a velocity model can be obtained by solving simultaneous equations once the Choresky factorization is used as a simultaneous equation solver.

Figs 2.7 to 2.9 show the examples of numerical tests. An analysis with sources only on the ground surface is shown in Fig. 2.7. Almost true velocity model is obtained in

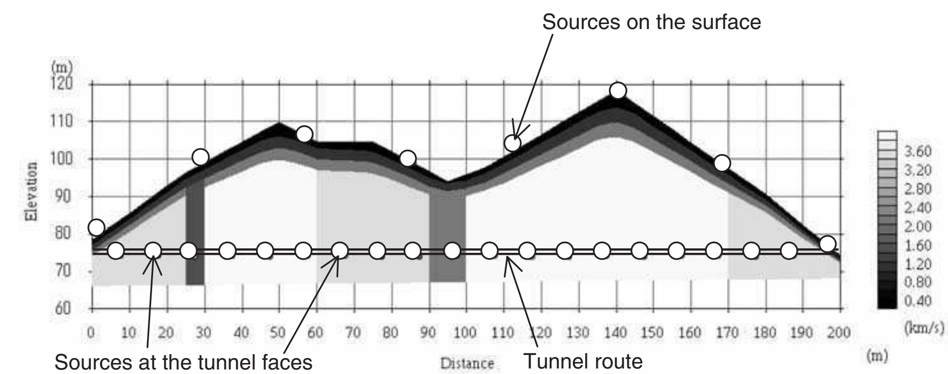

Fig. 2.6 A velocity model used in numerical tests and source location.

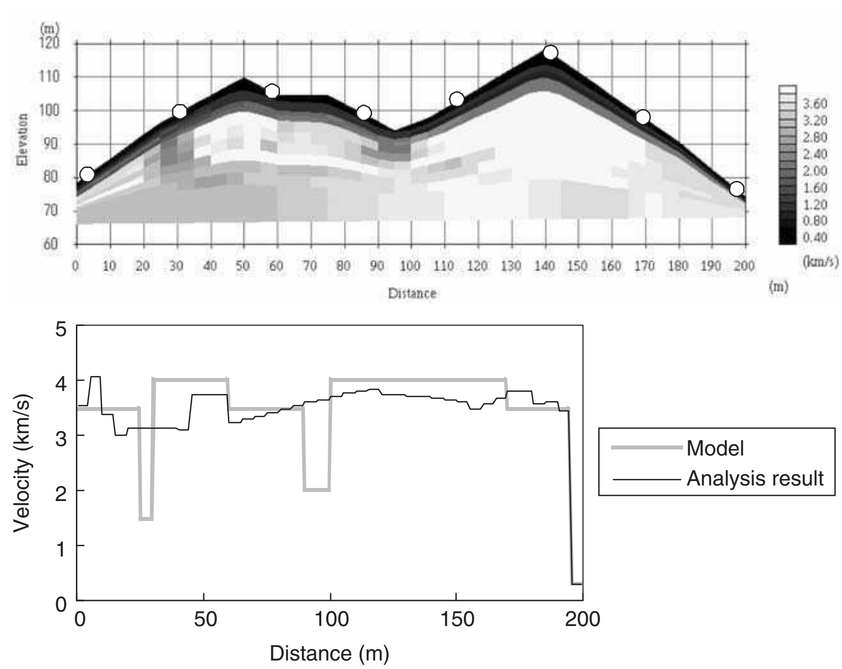

Fig. 2.7 Record of trajectory used to analyze test number 138.

near surfaces region. However, as the depth increases, the resolution of a reconstructed velocity model decreases. Specially, low velocity zones can be seen only around the top of the bedrock. Its distribution along a tunnel route is not clear.

Fig. 2.8 shows result of analysis in which two sources are deployed at the vicinity of both the tunnel entrances, respectively (four in-tunnel sources are used). This numerical test supposes that both the tunnel faces have proceeded 20 m from the entrances. One may see that the low velocity zone placed between 25 and 30 m and velocity boundary at distance of 170 m are more clearly imaged than Fig. 2.7. In this test, the low velocity zone and the velocity boundary can be considered as placed ahead of the tunnel face. This result shows that the use of the sources placed within a tunnel improved the resolution of a velocity model ahead of the tunnel face.

Fig. 2.9 shows an analysis in which eight sources are deployed within both the tunnel respectively (sixteen in-tunnel sources are used). Almost true velocity distribution through the tunnel route has been obtained; and the low-velocity zone placed at the middle of the tunnel, which can be considered as head of the tunnel face, has been clearly imaged.

The numerical tests show that the analysis with the data obtained during constructions using sources with a tunnel as well as the data obtained before the constructions may improve the accuracy and resolution of analyzed velocity models. The method can also improve the in-tunnel reflection method. The seismic refraction method with in-tunnel sources can supply reliable velocity distribution ahead of the tunnel face to the in-tunnel

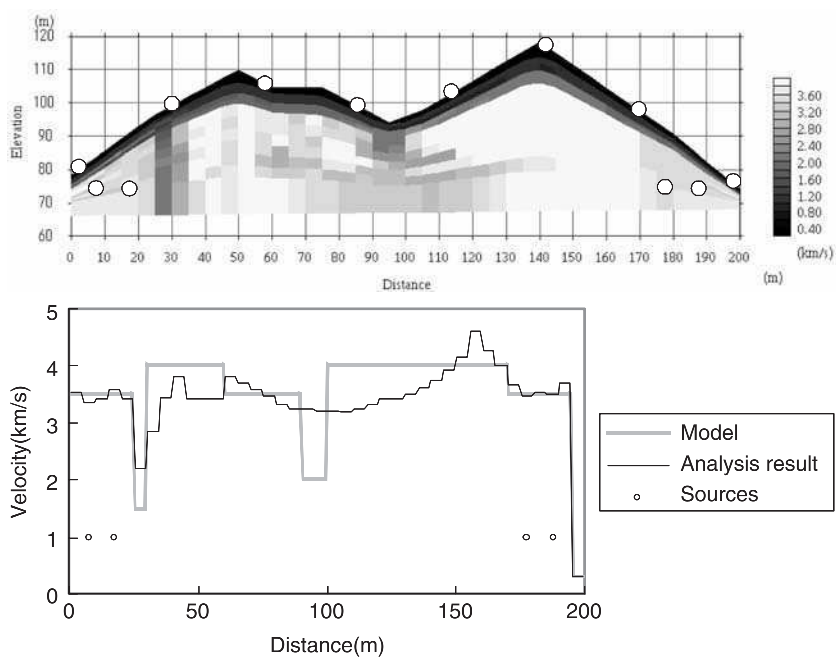

Fig. 2.8 Masses, velocities and energies of rocks crossing the plane defined by the fence post.

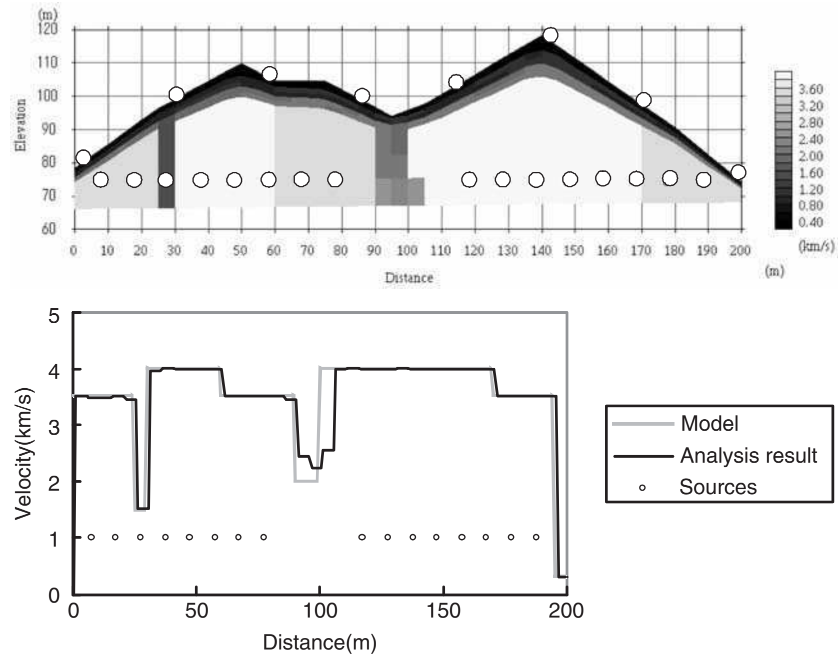

Fig. 2.9 Velocities, forces and energy dissipation in the protective system.

reflection method so that the accuracy of reflector position can be increased. Furthermore, rock classification from tunnel faces to reflectors as well as behind the reflectors is possible with precise velocity distribution obtained from the new seismic refraction method.

2.4 APPLICATION TO CONSTRUCTION SITE

The new method has been applied to an actual tunnel site. The tunnel length is about 2 km and is located within Mesozoic sedimentary rocks with slate, sandstone and chert. The seismic refraction method carried out before construction shows that velocity along the tunnel route is mainly 4.0–4.2 km/s. An accurate velocity model is required during the construction phase because of a thick overburden (maximum depth to the tunnel route is about 400 m at the middle of the tunnel). The data acquisition using a source within the tunnel under construction and receivers on the ground surface was carried out and the data were analyzed with the data acquired before the construction. In addition to this, the in-tunnel reflection method was carried out in order to evaluate the fractured zone predicted from the seismic refraction method and geological reconnaissance. Fig. 2.10 shows the velocity model obtained from the refraction method carried out before construction. Fig. 2.10 also shows the position of source and receiver for data acquisition during the construction. On the ground surface, receivers were deployed from a distance of 600 to 1050 m for the seismic refraction analysis.

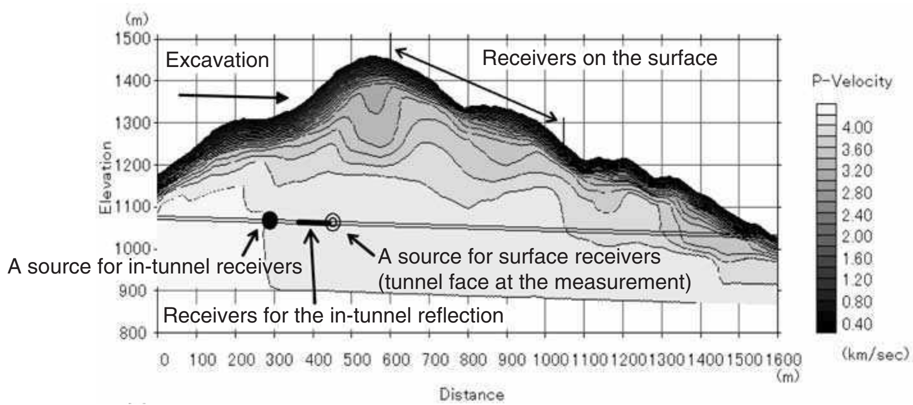

Fig. 2.10 A velocity model obtained from a refraction method carried out before construction and source and receiver location for data acquisition during the construction. A black thick line indicates receivers for the in-tunnel reflection method. ● Indicates a source for the in-tunnel reflection method. ◎ Indicates a source for surface receivers.

2.4.1 In-tunnel seismic reflection method

Fig. 2.11 shows a raw shot gather data of the in-tunnel seismic reflection method (source location is shown as ● in Fig. 2.10). The first arrival with apparent velocity of about 3.6 km/s is direct P-wave. Clear after-phase with apparent velocity of 1.2 km/s seems to be a kind of surface waves propagating along the tunnel. One may dimly see after-phase with negative apparent velocity in the distance of 20 to 50 m and in the time of 60 to 80 ms. This after-phase seems to be reflected waves from a reflector ahead of the tunnel face.

The apparent velocity of this reflected arrival is clearly faster than the direct P-waves and it suggests that the reflector is not perpendicular to the tunnel route. It is unusual that reflected waves could be seen on a raw shot gather. The data suggests that a clear reflector exists ahead of the tunnel face. Fig. 2.12 shows the shot gather after applying a band-pass and a F-K filter. Waves with negative apparent velocity are extracted and the reflected waves can be seen clearly.

2.4.2 Seismic refraction method with a source placed at tunnel face

Fig. 2.10 shows source ◎ and receiver locations for the data acquisition. A source was placed at a tunnel face and receivers were placed on the ground surface. The data were analyzed with the data obtained before the tunnel construction. Fig. 2.13 shows result of analysis. The whole velocity model does not change significantly. However, the velocity ahead of the tunnel face at distances of 500 to 900 m and elevation of 1000 to 1200 m decreases clearly. Fig. 2.14 shows the velocity along the tunnel route obtained from the seismic refraction analysis. The analysis with a source at the tunnel face suggested that

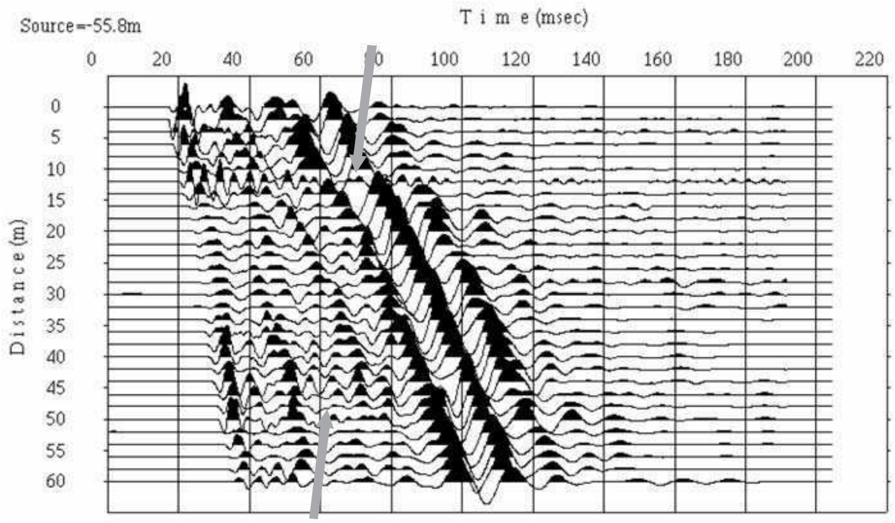

Fig. 2.11 A raw shot gather data of the in-tunnel seismic reflection method. A source was placed 55.8 m away from a receiver array (◎ in Fig. 2.10). Big arrows indicate reflected waves from a reflector ahead of the tunnel face.

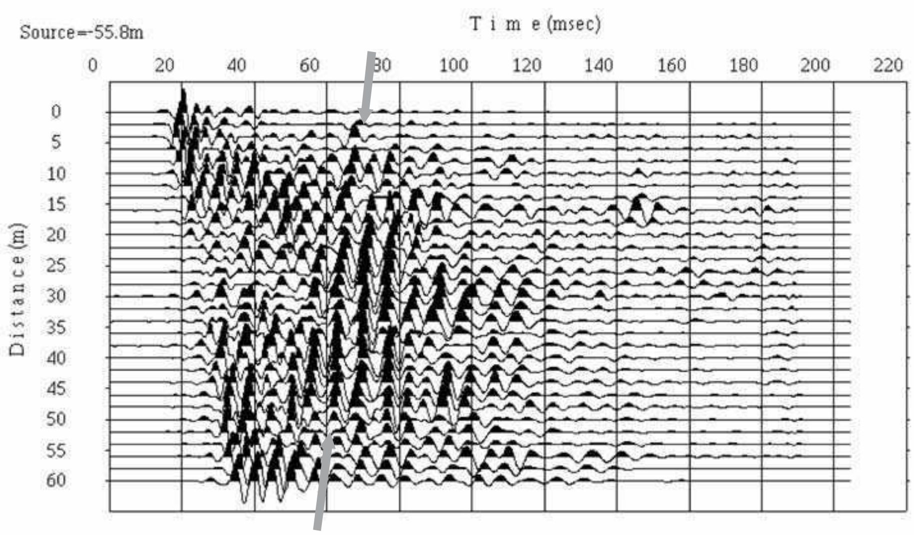

Fig. 2.12 A shot gather after applying a band-pass and F-K filter. Big arrows indicate reflected waves from a reflector ahead of the tunnel face.

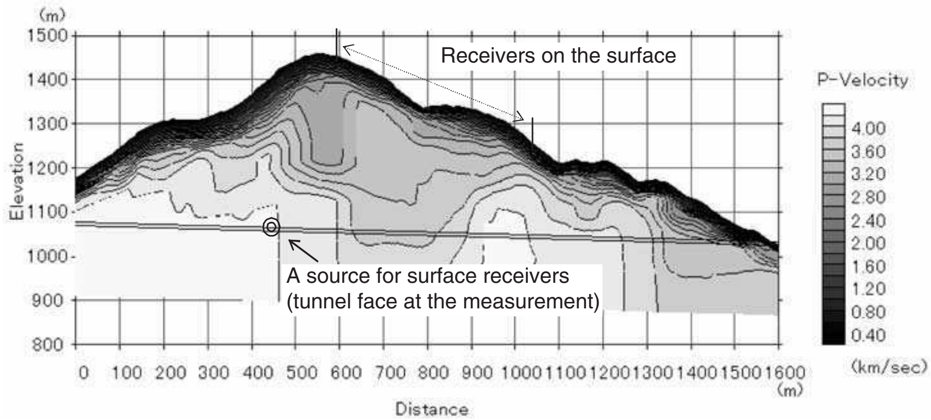

Fig. 2.13 Result of an analysis of seismic refraction data with an in-tunnel source ◎. The velocity ahead of tunnel face at distances of 500 to 900 m and elevation of 1000 to 1200 m decreases clearly, compared to results of analysis before construction (Fig. 2.10).

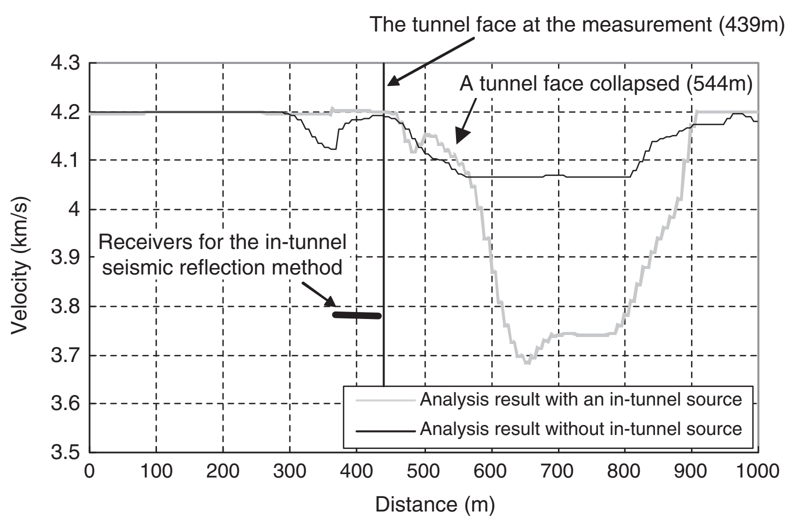

Fig. 2.14 Seismic velocity along a tunnel route. A thick line indicates result of an analysis with an in-tunnel source. A thin line indicates result of an analysis without in-tunnel sources.

the velocity ahead of the tunnel face is lower than the velocity behind the tunnel face, and that the velocity from the tunnel face to 100 m ahead of it is about 4.15 km/s.

2.4.3 Construction result

It was predicted that the condition of the tunnel face is going to be worse than the current condition because the in-tunnel reflection method imaged a clear reflector ahead of the tunnel face and the seismic refraction method suggested that the velocity ahead of the tunnel face is lower than the velocity behind the tunnel face. A shot record of the tunnel reflection method has been transformed into a reflector image by a pre-stack depth migration. Fig. 2.10 shows a reflector image by the migration. Fig. 2.15 shows a reflector image by the migration with the migration velocity of 4.15 km/s obtained from the seismic refraction analysis with a source at the tunnel face.

During excavation, the tunnel face condition got worse from 90 m ahead of the tunnel face where the measurements had been performed, and a tunnel face had collapsed at 104.7 m ahead of the tunnel face (see Figs 2.8 and 2.10). In this example, only one in-tunnel source was used. However, it is ideal to perform a data acquisition with in-tunnel sources periodically, so that as many in-tunnel sources may be used as possible.

The new data acquisition and analysis method in which sources are placed not only on the ground surface but also within a tunnel has been introduced into the seismic refraction

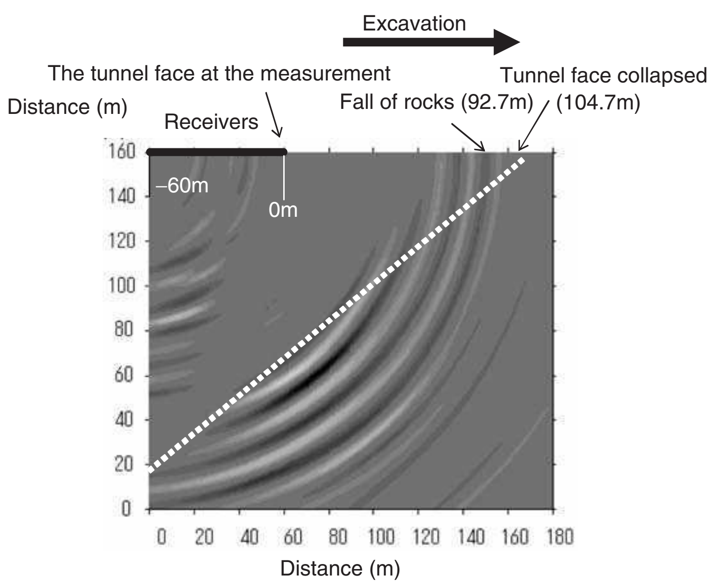

Fig. 2.15 A reflector image by the migration with the migration velocity of 4.5 km/s obtained from the seismic refraction analysis with a source at the tunnel face. A white broken line indicates the reflector that a stacking performance is maximum if velocity is 4.1 km/s. A tunnel face collapsed on the extension of clear reflector.

method for the construction of a tunnel. Numerical tests have been carried out and the results have shown the efficiency of the method. The method was applied to an actual tunnel site. Although only one in-tunnel source was used, a weak rock zone ahead of the tunnel face was successfully predicted by seismic methods. The point to be emphasized in the actual example can be summarized as follows. One could predict that tunnel face condition was getting worse by obtaining velocity ahead of the tunnel face from the seismic refraction method with a source at the tunnel face. The conventional in-tunnel reflection method in which sources and receivers are placed only within a tunnel can image reflector distribution. The method cannot, however, predict rock quality and tunnel face condition corresponding to the reflector. For example, it is difficult to determine whether the rock quality is getting better or worse from the in-tunnel seismic reflection method. It is possible to estimate the rock quality ahead of the tunnel face from the new seismic refraction method in which sources and receivers are placed not only on the surface but also within a tunnel. Applying a new seismic refraction method with the in-tunnel reflection method, valuable information may be supplied for tunnel construction.

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3
Terzaghi's rock load theory

"The geotechnical engineer should apply theory and experimentation but temper them by putting them into the context of the uncertainty of nature. Judgement enters through engineering geology."

Karl Terzaghi

3.1 INTRODUCTION

This was probably the first successful attempt in classifying the rock masses for the engineering purposes. Terzaghi (1946) proposed that the rock load factor H_p is the height of loosening zone over tunnel roof which is likely to load the steel arches. These rock load factors were estimated by Terzaghi from a 5.5 m wide steel-arch supported rail/road tunnel in the Alps during the late twenties. In these investigations, wooden blocks of known strengths were used for blocking the steel arches to the surrounding rock masses. Rock loads were estimated from the known strength of the failed wooden blocks. Terzaghi used these observations to back-analyze rock loads acting on the supports. Subsequently, he conducted "Trap-door" experiments on the sand and found that the height of loosened arch above the roof increased directly with the opening width in the sand.

3.2 ROCK CLASSES

Terzaghi (1946) considered the structural discontinuities of the rock masses and classified them qualitatively into nine categories as described in Table 3.1. Extensive experience from tunnels in the lower Himalaya has shown that the term squeezing rock is really squeezing ground condition; because a jointed and weak rock mass fails at high overburden stress and squeezes into the tunnels.

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Table 3.1 Definitions of rock classes of Terzaghi's rock load theory (Sinha, 1989).

Rock class	Type of rocks	Definition
I.	Hard and intact	The rock is unweathered. It contains neither joints nor hair cracks. If fractured, it breaks across intact rock. After excavation, the rock may have some popping and spalling failures from roof. At high stresses spontaneous and violent spalling of rock slabs may occur from the side or the roof. The unconfined compressive strength is equal to or more than 100 MPa.
II.	Hard stratified and schistose	The rock is hard and layered. The layers are usually widely separated. The rock may or may not have planes of weakness. In such rocks, spalling is quite common.
III.	Massive, moderately jointed	A jointed rock, the joints are widely spaced. The joints may or may not be cemented. It may also contain hair cracks but the huge blocks between the joints are intimately interlocked so that vertical walls do not require lateral support. Spalling may occur.
IV.	Moderately blocky and seamy	Joints are less spaced. Blocks are about 1 m in size. The rock may or may not be hard. The joints may or may not be healed but the interlocking is so intimate that no side pressure is exerted or expected.
V.	Very blocky and seamy	Closely spaced joints. Block size is less than 1 m. It consists of almost chemically intact rock fragments which are entirely separated from each other and imperfectly interlocked. Some side pressure of low magnitude is expected. Vertical walls may require supports.
VI.	Completely crushed but chemically intact	Comprises chemically intact rock having the character of a crusher-run aggregate. There is no interlocking. Considerable side pressure is expected on tunnel supports. The block size could be few centimeters to 30 cm.
VII.	Squeezing rock – moderate depth	Squeezing is a mechanical process in which the rock advances into the tunnel opening without perceptible increase in volume. Moderate depth is a relative term and could be from 150 to 1000 m.
VIII.	Squeezing rock – great depth	The depth may be more than 150 m. The maximum recommended tunnel depth is 1000 m.
IX.	Swelling rock	Swelling is associated with volume change and is due to chemical change of the rock, usually in presence of moisture or water. Some shales absorb moisture from air and swell. Rocks containing swelling minerals such as montmorillonite, illite, kaolinite and others can swell and exert heavy pressure on rock supports.

3.3 ROCK LOAD FACTOR

Terzaghi (1946) combined the results of his trap-door experiments and the estimated rock loads from Alpine tunnels to compute rock load factors H_p in terms of tunnel width B and tunnel height H_t of the loosened rock mass above the tunnel crown (Fig. 3.1) which loads the steel arches. Such rock load factors for all the nine rock classes are listed in Table 3.2.

For obtaining the vertical support pressure from the rock load factor H_p, Terzaghi suggested the following equation (Fig. 3.1).

$$p_v = \gamma \cdot H_p \tag{3.1}$$

where p_v is the support pressure, γ is the unit weight of the rock mass and H_p is the height of loose overburden above tunnel roof (Fig. 3.1). A limitation of Terzaghi's theory is that it may not be applicable for tunnels wider than 6 m.

The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for classes IV to VI in Table 3.2 can be reduced by 50 percent (Rose, 1982).

Deere et al. (1970) modified Terzaghi's classification system by introducing the RQD as the lone measure of rock quality (Table 3.3). They have distinguished between blasted and machine excavated tunnels and proposed guidelines for selection of steel set,

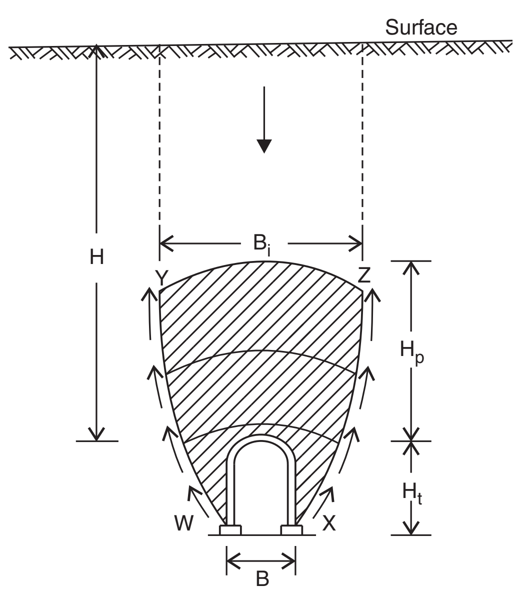

Fig. 3.1 Terzaghi's (1946) rock-load concept in tunnels.

Table 3.2 Rock load in tunnels within various rock classes (Terzaghi, 1946).

Rock class	Rock condition	Rock load factor H_p	Remarks
I.	Hard and intact	Zero	Light lining required only if spalling or popping occurs.
II.	Hard stratified or schistose	0 to $0.5B$	Light support mainly for protection against spalling. Load may change erratically from point to point.
III.	Massive, moderately jointed	0 to $0.25B$	
IV.	Moderately blocky and seamy	$0.25B$ to $0.35\ (B+H_t)$	No side pressure.
V.	Very blocky and seamy	0.35 to $1.10\ (B+H_t)$	Little or no side pressure.
VI.	Completely crushed but chemically intact	$1.10\ (B+H_t)$	Considerable side pressure. Softening effects of seepage toward bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs.
VII.	Squeezing rock – moderate depth	1.10 to $2.10\ (B+H_t)$	Heavy side pressure, invert struts required. Circular ribs are recommended.
VIII.	Squeezing rock – great depth	2.10 to $4.50\ (B+H_t)$	
IX.	Swelling rock	Upto 250 ft (80 m), irrespective of the value of $(B+H_t)$	Circular ribs are required. In extreme cases, use of yielding support recommended.

Notations: B = Tunnel span in meters; H_t = height of the opening in meters and H_p = height of the loosened rock mass above tunnel crown developing load (Fig. 3.1).

rock bolts and shotcrete supports for 6 and 12 m diameter tunnels in rock. These guidelines are presented in Table 3.4.

Deere et al. (1970) also considered the rock mass as an integral part of the support system, meaning that Table 3.4 is only applicable if the rock mass is not allowed to loosen and disintegrate extensively. Deere et al. (1970) assumed that machine excavation had the beneficial effect of reducing rock loads by about 20 to 25 percent.

Table 3.3 Terzaghi's rock load concept as modified by Deere et al. (1970).

Rock class and condition		RQD %	Rock load H_p	Remarks
I.	Hard and intact	95–100	Zero	Same as Table 3.2
II.	Hard stratified or schistose	90–99	0–0.5B	Same as Table 3.2
III.	Massive moderately jointed	85–95	0–0.25B	Same as Table 3.2
IV.	Moderately blocky and seamy	75–85	0.25B–0.35 $(B + H_t)$	Types IV, V and VI reduced by about 50% from Terzaghi values because water table has little effect on rock load (Terzaghi, 1946; Brekke, 1968)
V.	Very blocky and seamy	30–75	(0.2–0.6) $(B + H_t)$	Same as above
VI.	Completely crushed	3–30	(0.6–1.10) $(B + H_t)$	Same as above
VIa.	Sand and gravel	0–3	(1.1–1.4) $(B + H_t)$	Same as above
VII.	Squeezing rock at moderate depth	NA	(1.10–2.10) $(B + H_t)$	Same as Table 3.2
VIII.	Squeezing rock at great depth	NA	(2.10–4.50) $(B + H_t)$	Same as Table 3.2
IX.	Swelling rock	NA	Upto 80 m irrespective of the value of $(B + H_t)$	Same as Table 3.2

Notes: B = Tunnel span; H_t = height of the opening and H_p = height of the loosened rock mass above the tunnel crown developing load (Fig. 3.1).

3.3.1 Limitations

Terzaghi's approach was successfully used earlier when conventional drill and blast method of excavation and steel-arch supports were employed in the tunnels of comparable size. This practice lowered the strength of the rock mass and permitted significant roof convergence which mobilized a zone of loosened rock mass above the tunnel roof. The height of this loosened rock mass, called "coffin cover", acted as dead load on the supports. Cecil (1970) concluded that Terzaghi's classification provided no quantitative information regarding the rock mass properties. Despite all these limitations, the immense practical values of Terzaghi's approach cannot be denied and this method still finds application under conditions similar to those for which it was developed.

Table 3.4 Guidelines for selection of steel sets for 6 to 12 m diameter tunnels in rock (Deere et al., 1970).

		Steel sets		Rock bolt		Conventional shotcrete		
						Total thickness (cm)		
Rock quality	Construction method	Weight of steel sets	Spacing	Spacing of pattern bolt	Additional requirements	Crown	Sides	Additional supports
Excellent RQD > 90	Tunnel boring machine	Light	None to occasional	None to occasional	Rare	None to occasional	None	None
	Drilling and blasting	Light	None to occasional	None to occasional	Rare	None to occasional	None	None
Good RQD 75 to 90	Boring machine	Light	Occasional or 1.5 to 1.8 m	Occasional or 1.5 to 1.8 m	Occasional mesh and straps	Local application 5 to 7.5 cm	None	None
	Drilling and blasting	Light	1.5 to 1.8 m	1.5 to 1.8 m	Occasional mesh or straps	Local application 5 to 7.5 cm	None	None
Fair RQD 50 to 75	Boring machine	Light to medium	1.5 to 1.8 m	1.2 to 1.8 m	Mesh and straps as required	5 to 10 cm	None	Rock bolts
	Drilling and blasting	Light to medium	1.2 to 1.5 m	0.9 to 1.5 m	Mesh and straps as required	10 cm or more	10 cm or more	Rock bolts
Poor RQD 25 to 50	Boring machine	Medium circular	0.6 to 1.2 m	0.9 to 1.5 m	Anchorage may be hard to obtain. Considerable mesh and straps required	10 to 15 cm	10 to 15 cm	Rock bolt as required (1.2 to 1.8 m center to center)

	Drilling and blasting	Medium to heavy circular	0.2 to 1.2 m	0.6 to 1.2 m	As above	15 cm or more	15 cm or more	As above
Very poor RQD < 25	Boring machine	Medium to heavy circular	0.6 m	0.6 to 1.2 m	Anchorage may be impossible. 100 percent mesh and straps required	15 cm or more on whole section		Medium sets as required
	Drilling and blasting	Heavy circular	0.6 m	0.9 m	As above	15 cm or more on whole section		Medium to heavy sets as required
Very poor squeezing and swelling ground	Both methods	Very heavy circular	0.6 m	0.6 to 0.9 m	Anchorage may be impossible. 100 percent mesh and straps required	15 cm or more on whole section		Heavy sets as required

With the advent of the New Austrian Tunnelling Method (NATM) (Chapter 9) and Norwegian Method of Tunnelling (NMT) (Chapter 10), increasing use is made of controlled blasting and machine excavation techniques and support system employing steel fiber reinforced shotcrete and rock bolts. Even in steel-arch supported tunnels, wooden struts have been replaced by pneumatically filled lean concrete. These improvements in the tunnelling technology preserve the pre-excavation strength of the rock mass and use it as a load carrying structure in order to minimize roof convergence and restrict the height of the loosening zone above the tunnel crown.

Consequently, the support pressure does not increase directly with the opening width. Based on this argument, Barton et al. (1974) advocated that the support pressure is independent of opening width in rock tunnels. Rock mass – tunnel support interaction analysis of Verman (1993) also suggests that the support pressure is practically independent of the tunnel width, provided support stiffness is not lowered. Goel et al. (1996) also studied this aspect of effect of tunnel size on support pressure and found that there is a negligible effect of tunnel size on support pressure in non-squeezing ground conditions, but the tunnel size could have considerable influence on the support pressure in squeezing ground condition. This aspect has been covered in detail in Chapter 6.

The estimated support pressures from Table 3.2 have been compared with the measured values and the following conclusions emerge:

(i) Terzaghi's method provides reasonable support pressure for small tunnels ($B < 6$ m).
(ii) It provides over-safe estimates for large tunnels and caverns (Diam. 6 to 14 m) and
(iii) The estimated support pressure values fall in a very large range for squeezing and swelling ground conditions for a meaningful application.

3.4 MODIFIED TERZAGHI'S THEORY FOR TUNNELS AND CAVERNS

Singh et al. (1995) have compared support pressure measured from tunnels and caverns with estimates from Terzaghi's rock load theory and found that the support pressure in rock tunnels and caverns does not increase directly with excavation size as assumed by Terzaghi (1946) and others mainly due to dilatant behavior of rock masses, joint roughness and prevention of loosening of rock mass by improved tunnelling technology. They have subsequently recommended ranges of support pressures as given in Table 3.5 for both tunnels and caverns for the benefit of those who still want to use Terzaghi's rock load approach. They observed that the support pressures are nearly independent of the size of opening.

It is interesting to note that the recommended roof support pressures turn out to be the same as those obtained from Terzaghi's rock load factors when B and H_t are substituted

Table 3.5 Recommendations of Singh et al. (1995) on support pressure for rock tunnels and caverns.

Terzaghi's classification			Classification of Singh et al. (1995)				
					Recommended support pressure (MPa)		
Category	Rock condition	Rock load factor H_p	Category	Rock condition	p_v	p_h	Remarks
I.	Hard and intact	0	I.	Hard and intact	0	0	–
II.	Hard stratified or schistose	0 to $0.5B$	II.	Hard stratified or schistose	0.04–0.07	0	–
III.	Massive, moderately jointed	0 to $0.25B$	III.	Massive, moderately jointed	0.0–0.04	0	–
IV.	Moderately blocky seamy and jointed	$0.25B$ to $0.35\ (B+H_t)$	IV.	Moderately blocky seamy very jointed	0.04–0.1	0–0.2 p_v	Inverts may be required
V.	Very blocky and seamy, shattered arched	0.35 to $1.1\ (B+H_t)$	V.	Very blocky and seamy, shattered highly jointed, thin shear zone or fault	0.1–0.2	0–0.5 p_v	Inverts may be required, arched roof preferred
VI.	Completely crushed but chemically intact	$1.1\ (B+\mathrm{H}_t)$	VI.	Completely crushed but chemically unaltered, thick shear and fault zone	0.2–0.3	0.3–1.0 p_v	Inverts essential, arched roof essential

continued

Table 3.5—Continued

Terzaghi's classification			Classification of Singh et al. (1995)				
					Recommended support pressure (MPa)		
Category	Rock condition	Rock load factor H_p	Category	Rock condition	p_v	p_h	Remarks
VII.	Squeezing rock at moderate depth	1.1 to 2.1 $(B+H_t)$	VII.	Squeezing rock condition			
			VIIA.	Mild squeezing (u_a/a upto 3%)	0.3–0.4	Depends on primary stress values, p_h may exceed p_v	Inverts essential. In excavation flexible support preferred. Circular section with struts recommended.
			VIIB.	Moderate squeezing ($u_a/a = 3$ to 5%)	0.4–0.6	As above	As above
VIII.	Squeezing rock at great depth	2.1 to 4.5 $(B+H_t)$	VIIC.	High squeezing (u_a/a >5%)	6.0–1.4	As above	As above
IX.	Swelling rock	upto 80 m	VIII.	Swelling rock			
			VIIIA.	Mild swelling	0.3–0.8	Depends on type and content of swelling clays, p_h may exceed p_v	Inverts essential in excavation, arched roof essential.
			VIIIB.	Moderate swelling	0.8–1.4	As above	As above
			VIIIC.	High swelling	1.4–2.0	As above	As above

Notations: p_v = Vertical support pressure; p_h = horizontal support pressure; B = width or span of opening; H_t = height of opening; u_a = radial tunnel closure; $a = B/2$; thin shear zone = upto 2 m thick.

by 5.5 m. The estimated roof support pressures from Table 3.5 were found comparable with the measured values irrespective of the opening size and the rock conditions (Singh et al., 1995). They have further cautioned that the support pressure is likely to increase directly with the excavation width for tunnel sections through slickensided shear zones, thick clay-filled fault gouges, weak clay shales and running or flowing ground conditions where interlocking of blocks is likely to be missing or where joint strength is lost and rock wedges are allowed to fall due to excessive roof convergence on account of delayed supports beyond stand-up time. It may be noted that wider tunnels require reduced spacing of bolts or steel-arches and thicker linings since rock loads increase directly with the excavation width even if the support pressure does not increase with the tunnel size.

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4

Rock mass rating (RMR)

"Effectiveness of knowledge through research (E) is $E = mc^2$; where m is mass of knowledge and c is communication of knowledge by publications."

Z. T. Bieniawski

4.1 INTRODUCTION

The geomechanics classification or the rock mass rating (RMR) system was initially developed at the South African Council of Scientific and Industrial Research (CSIR) by Bieniawski (1973) on the basis of his experiences in shallow tunnels in sedimentary rocks (Kaiser et al., 1986). Since then the classification has undergone several significant evolutions: in 1974 – reduction of classification parameters from 8 to 6; in 1975 – adjustment of ratings and reduction of recommended support requirements; in 1976 – modification of class boundaries to even multiples of 20; in 1979 – adoption of ISRM (1978) rock mass description, etc. It is, therefore, important to state which version is used when RMR-values are quoted. The geomechanics classification reported by Bieniawski (1984) is referred in this book.

To apply the geomechanics classification system, a given site should be divided into a number of geological structural units in such a way that each type of rock mass is represented by a separate geological structural unit. The following six parameters (representing causative factors) are determined for each of the structural unit:

(i) Uniaxial compressive strength of intact rock material,
(ii) Rock quality designation RQD,
(iii) Joint or discontinuity spacing,
(iv) Joint condition,
(v) Ground water condition and
(vi) Joint orientation.

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4.2 COLLECTION OF FIELD DATA

The rating of six parameters of the RMR system are given in Tables 4.1 to 4.6. For reducing doubts due to subjective judgments, the rating for different parameters should be given a range in preference to a single value. These six parameters are discussed in the following paragraphs. The beginners do not get the feeling of the value of RMR or Q, etc. at a location and they get confused on transition from one category to another (Tables 4.4 and 4.5). In fact, approximate average RMR is good enough.

4.2.1 *Uniaxial compressive strength of intact rock material (q_c)*

The strength of the intact rock material should be obtained from rock cores in accordance with site conditions. The ratings based on both uniaxial compressive strength (UCS) (which is preferred) and point load strength are given in Table 4.1. UCS may also be obtained from the point load strength index tests on rock lumps at the natural moisture content. Please see Table 5.12 also for UCS.

4.2.2 *Rock quality designation (RQD)*

Rock quality designation (RQD) should be determined from rock cores or volumetric joint count (Singh & Goel, 1999). RQD is percentage of rock cores (equal to or more than 10 cm) in one meter of drill run. The details of rating are given in Table 4.2 (see also Section 5.1.1). The fresh broken cores are fitted together and counted as one piece.

Table 4.1 Strength of intact rock material (Bieniawski, 1979, 1984).

Qualitative description	Compressive strength (MPa)	Point load strength (MPa)	Rating
Exceptionally strong	>250	8	15
Very strong	100–250	4–8	12
Strong	50–100	2–4	7
Average	25–50	1–2	4
Weak	5–25	Use of uniaxial compressive strength is preferred	2
Very weak	1–5	As above	1
Extremely weak	<1	As above	0

Note: At compressive strength less than 0.6 MPa, many rock materials would be regarded as soil.

Table 4.2 Rock quality designation, RQD (Bieniawski, 1979).

Qualitative description	RQD	Rating
Excellent	90–100	20
Good	75–90	17
Fair	50–75	13
Poor	25–50	8
Very poor	< 25	3

4.2.3 Spacing of discontinuities

The term discontinuity covers joints, beddings or foliations, shear zones, minor faults or other surfaces of weakness. The linear distance between two adjacent discontinuities should be measured for all sets of discontinuities and the rating should be obtained from Table 4.3 for the most critically oriented discontinuity.

4.2.4 Condition of discontinuities

This parameter includes roughness of discontinuity surfaces, their separation, length or continuity, weathering of the wall rock or the planes of weakness and infilling (gouge) material. The details of rating are given in Table 4.4. The joint set which is oriented unfavorably with respect to a structure (tunnel or cavern) should be considered as in Section 4.2.3.

4.2.5 Ground water condition

In the case of tunnels, the rate of inflow of ground water in liters per minute per 10 m length of the tunnel should be determined, or a general condition may be described as completely

Table 4.3 Spacing of discontinuities (Bieniawski, 1979).

Description	Spacing (m)	Rating
Very wide	> 2	20
Wide	0.6–2	15
Moderate	0.2–0.6	10
Close	0.06–0.2	8
Very close	< 0.06	5

Note: If more than one discontinuity sets are present and the spacing of discontinuities of each set varies, consider the unfavorably oriented set with lowest rating.

Table 4.4 Condition of discontinuities (Bieniawski, 1979).

Description	Joint separation (mm)	Rating
Very rough and unweathered, wall rock tight and discontinuous, no separation	0	30
Rough and slightly weathered, wall rock surface separation <1 mm	< 1	25
Slightly rough and moderately to highly weathered, wall rock surface separation <1 mm	< 1	20
Slickensided wall rock surface or 1–5 mm thick gouge or 1–5 mm wide continuous discontinuity	1–5	10
5 mm thick soft gouge, 5 mm wide continuous discontitnuity	> 5	0

dry, damp, wet, dripping and flowing. If actual water pressure data are available, these should be stated and expressed in terms of the ratio of the seepage water pressure to the major principal stress. The ratings as per the water condition are shown in Table 4.5.

Ratings of the above five parameters (Tables 4.1 to 4.5) are added to obtain what is called the basic rock mass rating, RMR_{basic}.

4.2.6 Orientation of discontinuities

Orientation of discontinuities means the strike and dip of discontinuities. The strike should be recorded with reference to magnetic north. The dip angle is the angle between the horizontal and the discontinuity plane taken in a direction in which the plane dips. The value of the dip and the strike should be recorded as shown in Table 4.6. In addition, the orientation of tunnel axis or slope face or foundation alignment should also be recorded.

The influence of the strike and the dip of the discontinuities is considered with respect to the direction of tunnel drivage or slope face orientation or foundation alignment. To facilitate a decision whether or not the strike and the dip are favorable, reference should be made

Table 4.5 Ground water condition (Bieniawski, 1979).

Inflow per 10 m tunnel length (liter/min.)	None	<10	10–25	25–125	>125
Ratio of Joint water pressure to major principal stress	0	0–0.1	0.1–0.2	0.2–0.5	>0.5
General description	Completely dry	Damp	Wet	Dripping	Flowing
Rating	15	10	7	4	0

Table 4.6 Orientation of discontinuities.

A.	Orientation of tunnel/slope/foundation axis	
B.	Orientation of discontinuities:	
Set - 1	Average strike........(from.......to.......)	Dip/Dip direction..........
Set - 2	Average strike........(from.......to.......)	Dip/Dip direction..........
Set - 3	Average strike........(from.......to.......)	Dip/Dip direction..........

to Tables 4.7 and 4.8 which provide a quantitative assessment of critical joint orientation effect with respect to tunnel and dam foundations, respectively. Once the ratings for the effect of the critical discontinuity is known, as shown in Table 4.9 an arithmetic sum of the joint adjustment rating and the RMR_{basic} is obtained. This number is called the final rock mass rating (RMR).

It should be kept in mind that the effect of orientation in rough-dilatant joint is not so important in the case of tunnels according to Table 4.9. That is why orientation of joints is ignored in the Q-system of Norwegian Geotechnical Institute (NGI) (Chapter 5). The effect of orientation of joints is more important for rafts. It is most important obviously in rock slopes for which slope mass rating (SMR) is recommended.

4.3 ESTIMATION OF ROCK MASS RATING

The rock mass rating should be determined as an algebraic sum of ratings for all the parameters given in Tables 4.1 to 4.5 and 4.9 after adjustments for orientation of

Table 4.7 Assessment of joint orientation effect on tunnels (dips are apparent dips along tunnel axis) (Bieniawski, 1984).

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Irrespective of strike
Drive with dip		Drive against dip				
Dip 45°–90°	Dip 20°–45°	Dip 45°–90°	Dip 20°–45°	Dip 20°–45°	Dip 45°–90°	Dip 0°–20°
Very favorable	Favorable	Fair	Unfavorable	Fair	Very unfavorable	Fair

Table 4.8 Assessment of joint orientation effect on stability of dam foundation.

	Dip 10°–30°			
	Dip direction			
Dip 0°–10°	Upstream	Downstream	Dip 30°–60°	Dip 60°–90°
Very favorable	Unfavorable	Fair	Favorable	Very unfavorable

Table 4.9 Adjustment for joint orientation (Bieniawski, 1979).

Joint orientation assessment for	Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Tunnels	0	−2	−5	−10	−12
Raft foundation	0	−2	−7	−15	−25
Slopes*	0	−5	−25	−50	−60

** It is recommended to use slope mass rating (SMR) (Singh & Goel, 1999).*

discontinuities given in Tables 4.7 and 4.8. The sum of ratings for four parameters (Tables 4.2 to 4.5) is called rock condition rating (RCR) which discounts the effect of compressive strength of intact rock material and orientation of joints (Goel et al., 1996). Heavy blasting creates new fractures. Experience suggests that 10 points should be added to get RMR for undisturbed rock masses in situations where TBMs or road headers are used for tunnel excavation and 3 to 5 points may be added depending upon the quality of the controlled blasting.

On the basis of RMR values for a given engineering structure, the rock mass is classified into five classes, namely very good (RMR 100–81), good (80–61), fair (60–41), poor (40–21) and very poor (<20) as shown in Table 4.10.

In case of wider tunnels and caverns, RMR may be somewhat less than obtained from drifts. As in drifts, one may miss intrusions of weaker rocks and joint sets having

Table 4.10 Design parameters and engineering properties of rock mass (Bieniawski, 1979).

S. No	Parameter/properties of rock mass	Rock mass rating (Rock class)				
		100–81 (I)	80–61 (II)	60–41 (III)	40–21 (IV)	<20 (V)
1.	Classification of rock mass	Very good	Good	Fair	Poor	Very poor
2.	Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 h for 2.5 m span	30 min. for 1 m span
3.	Cohesion of rock mass (MPa)*	> 0.4	0.3–0.4	0.2–0.3	0.1–0.2	< 0.1
4.	Angle of internal friction of rock mass	> 45°	35°–45°	25°–35°	15°–25°	< 15°
5.	Allowable bearing pressure (T/m^2)	600–440	440–280	280–135	135–45	45–30

** These values are applicable to slopes only in saturated and weathered rock mass.*

Note: During earthquake loading, the above values of allowable bearing pressure may be increased by 50 percent in view of rheological behavior of rock masses.

lower joint condition ratings. Separate RMR should be obtained for tunnels of different orientations after taking into account the orientation of tunnel axis with respect to the critical joint set (Table 4.6).

The classification may be used for estimating many useful parameters, such as the unsupported span, the stand-up time or the bridge action period and the support pressure for an underground opening as shown in the following paragraphs under Section 4.4. It can also be used for selecting a method of excavation and the permanent support system. Further, cohesion, angle of internal friction, modulus of deformation of the rock mass and allowable bearing pressure for foundations can also be estimated for analysis of stability of rock slopes. Back-analysis of rock slopes in distress is more reliable approach for assessment of shear strength parameters. It is emphasized that the correlations suggested in Section 4.4 should be used for feasibility studies and preliminary designs only. In situ tests, supported with numerical modelling could be essential, particularly for a large opening, such as a cavern.

4.4 APPLICATIONS OF RMR

The following engineering properties of rock masses can be obtained using RMR. If the RMR lies within a given range, the value of engineering properties can be interpolated between the recommended range of properties.

4.4.1 Average stand-up time for arched roof

The stand-up time depends upon effective span of the opening which is defined as the width of the opening or the distance between the tunnel face and the last support, whichever is smaller. For arched openings, the stand-up time would be significantly higher than that for a flat roof. Controlled blasting will further increase the stand-up time as damage to the rock mass is decreased. For the tunnels with arched roof, the stand-up time is related with the rock mass class in Table 4.10 (Fig. 4.1). It is important that one should not unnecessarily delay supporting the roof in the case of a rock mass with high stand-up time as this may lead to deterioration in the rock mass which ultimately reduces the stand-up time. Lauffer (1988) observed that the stand-up time improves by one class of RMR value in case of excavations by TBM.

4.4.2 Cohesion and angle of internal friction

Assuming that a rock mass behaves as a Coulomb material, its shear strength will depend upon cohesion and angle of internal friction. RMR is used to estimate the cohesion and angle of internal friction (Table 4.10). Usually the strength parameters are different for peak failure and residual failure conditions. In Table 4.10, only peak failure values are given. It is experienced that these values are applicable to slopes only in saturated and

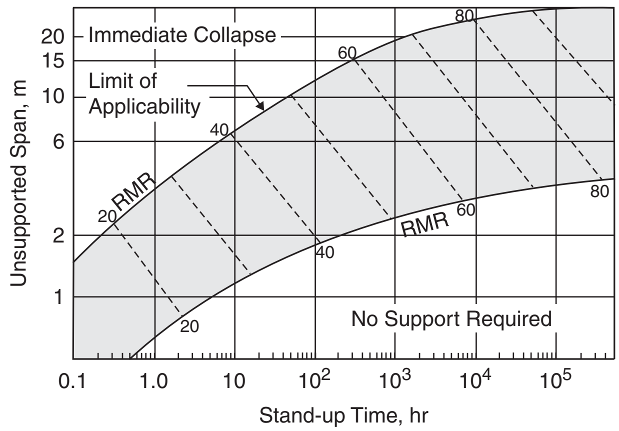

Fig. 4.1 Stand-up time vs. unsupported span for various rock mass classes as per RMR.

weathered rock masses. Cohesion is small under low normal stresses due to rotation of rock blocks. The cohesion is one order of magnitude higher in the case of tunnels because joints are relatively discontinuous, tight and widely spaced. Joints may have smaller lengths than those near rock slopes. See Section 4.4.5.

4.4.3 Modulus of deformation

Following correlations are suggested for determining modulus of deformation of rock masses.

Modulus reduction factor

Fig. 4.2 gives a correlation between RMR and modulus reduction factor (MRF), which is defined as a ratio of modulus of deformation of a rock mass to the elastic modulus of the rock material obtained from core. Thus, modulus of deformation of a rock mass (E_d) can be determined as a product of the modulus reduction factor corresponding to a given RMR (Fig. 4.2) and the elastic modulus of the rock material (E_r) from the following equation (Singh, 1979),

$$E_d = E_r \cdot \text{MRF} \tag{4.1}$$

There is an approximate correlation between modulus of deformation and RMR suggested by Bieniawski (1978) for hard rock masses (q_c > 100 MPa).

$$E_d = 2\,\text{RMR} - 100, \quad \text{GPa (applicable for RMR > 50)} \tag{4.2}$$

Serafim and Pereira (1983) suggested the following correlation

$$E_d = 10^{(\text{RMR}-10)/40}, \quad \text{GPa (applicable for RMR < 50 also)} \tag{4.3}$$

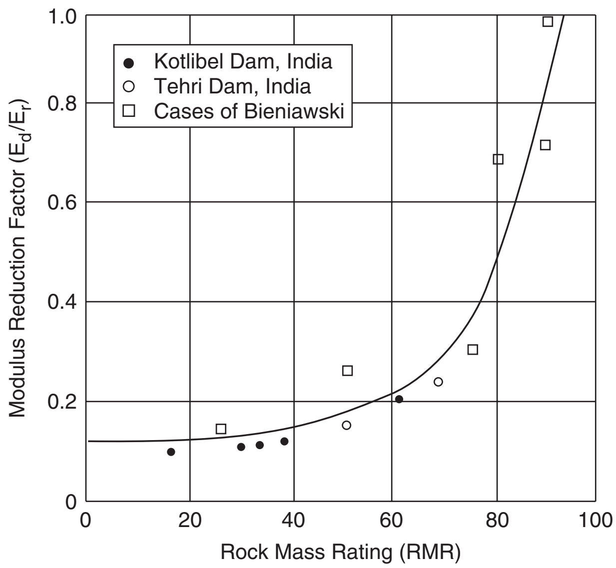

Fig. 4.2 Relationship between RMR and modulus reduction factor (Singh, 1979).

These correlations are shown in Fig. 4.3. Here q_c means average uniaxial crushing strength of the intact rock material in MPa.

The modulus of deformation of a dry and weak rock mass ($q_c < 100$ MPa) around underground openings located at depths exceeding 50 m is dependent upon confining

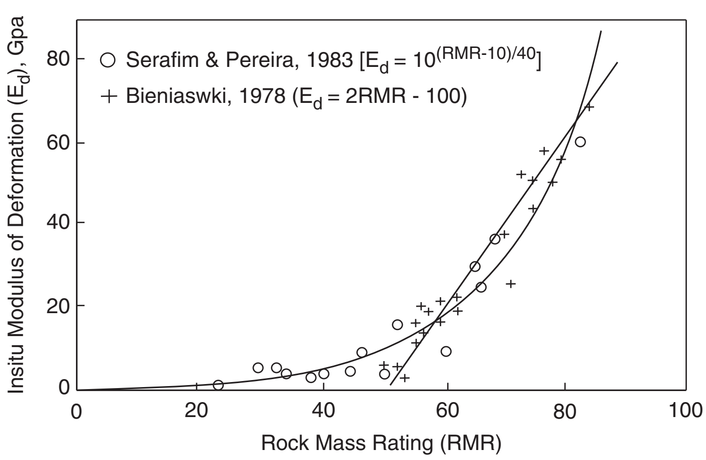

Fig. 4.3 Correlation between modulus of deformation of rock masses and RMR (Bieniawski, 1984).

pressure due to overburden and may be determined by the following correlation (Verman, 1993),

$$E_d = 0.3H^{\alpha} \cdot 10^{(RMR-20)/38}, \quad GPa \tag{4.4}$$

where

α = 0.16 to 0.30 (higher for poor rocks) and
H = depth of location under consideration below ground surface in meters.
$\geq$ 50 m

Table 5.13 summarizes various other correlations for assessment of modulus of deformation.

The modulus of deformation of poor rock masses with water sensitive minerals decreases significantly after saturation and with passage of time after excavation. For design of dam foundations, it is recommended that uniaxial jacking tests should be conducted very carefully soon after the excavation of drifts, particularly for poor rock masses in saturated condition.

4.4.4 Allowable bearing pressure

Allowable bearing pressure for 12 mm settlement of foundation is also related to RMR and can be estimated from the last row of Table 4.10 (Mehrotra, 1992).

4.4.5 Shear strength of rock masses

Singh and Goel (2002) summarized the non-linear shear strength equations for various RMR, degree of saturation and rock types. The recommended criteria is based on 43 block shear tests by Mehrotra (1992). It has been realized that for highly jointed rock masses, the shear strength (τ) may not be governed by the strength of the rock material as suggested by Hoek and Brown (1980). The results show that saturation does affect shear strength of rock mass significantly (see Fig. 29.5).

For hard and massive rock masses (RMR > 60), the shear strength is proportional to the UCS. It follows that block shear tests on saturated rock blocks should be conducted for design of concrete dams and stability of abutments.

4.4.6 Estimation of support pressure

In 1983, Unal, on the basis of his studies in coal mines, proposed the following correlation for estimation of support pressure using RMR for openings with flat roof,

$$p_v = \left[\frac{100 - RMR}{100}\right] \cdot \gamma \cdot B \tag{4.5}$$

where

p_v = support pressure,
γ = unit weight of rock and
B = tunnel width.

Goel and Jethwa (1991) have evaluated equation (4.5) for application to rock tunnels with arched roof by comparing the measured support pressures with estimates from equation (4.5). The comparison shows that equation (4.5) is not applicable to rock tunnels. They found that the estimated support pressures were unsafe for all sizes of tunnels under squeezing ground conditions. Further, the estimates for non-squeezing ground conditions were unsafe for small tunnels (diam. upto 6 m) and oversafe for large tunnels (diam. > 9 m) which implies that the size effect is over-emphasized for arched openings. This observation is logical since bending moments in a flat roof increases geometrically with the opening unlike in an arched roof.

Subsequently, using the measured support pressure values from 30 instrumented Indian tunnels, Goel and Jethwa (1991) have proposed equation (4.6) for estimating the short-term support pressure for underground openings in both squeezing and non-squeezing ground conditions in the case of tunnelling by conventional blasting method using steel rib supports (but not in the rock burst condition).

$$p_v = \frac{7.5B^{0.1} \cdot H^{0.5} - \text{RMR}}{20\,\text{RMR}}, \quad \text{MPa} \tag{4.6}$$

where

B = span of opening in meters,
H = overburden or tunnel depth in meters (50–600 m),
p_v = short-term roof support pressure in MPa and
RMR = post-excavation rock mass rating just before supporting.

Bieniawski (1984) provided guidelines for selection of tunnel supports (Table 4.11). This is applicable to tunnels excavated with conventional drilling and blasting method. These guidelines depend upon the factors like depth below surface (to take care of overburden pressure or the in situ stress), tunnel size and shape and method of excavation. The support measures in Table 4.11 are the permanent and not the temporary or primary supports.

The interrelation between RMR and Q is presented in Section 6.2.

4.5 PRECAUTIONS

It must be ensured that double accounting for a parameter should not be done in the analysis of rock structures and estimating rating of a rock mass. For example, if pore water pressure is being considered in the analysis of rock structures, it should not be

Table 4.11 Guidelines for excavation and support of rock tunnels in accordance with the RMR system (Bieniawski, 1984).

Rock mass class	Excavation	Supports: Rock bolts (20 mm diam. fully grouted)	Supports: Conventional shotcrete	Supports: Steel sets
Very good rock RMR = 81–100	Full face 3 m advance.	Generally, no support required except for occasional spot bolting.		
Good rock RMR = 61–80	Full face. 1.0–1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m, with occasional wire mesh.	50 mm in crown where required.	None
Fair rock RMR = 41–60	Heading and bench. 1.5–3 m advance in heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5–2 m in crown and walls with wire mesh in crown.	50–100 mm in crown and 30 mm in sides.	None
Poor rock RMR = 21–40	Top heading and bench. 1.0–1.5 m advance in top heading. Install support concurrently with excavation 10 m from face.	Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and wall with wire mesh.	100–150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required.
Very poor rock RMR < 20	Multiple drifts 0.5–1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5–6 m long spaced 1–1.5 m in crown and walls with wire mesh. Bolt invert.	150–200 mm in crown 150 mm in sides and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert

Shape: Horseshoe; Width: 10 m; Vertical stress < 25 MPa; Construction: Drilling & blasting.

accounted for in RMR. Similarly, if orientation of joint sets is considered in stability analysis of rock slopes, the same should not be accounted for in RMR.

It is cautioned that the RMR system is found to be unreliable in very poor rock masses. Care should therefore be exercised to apply the RMR system in such rock mass. Q-system is more reliable for tunnelling in the weak rock masses.

4.6 TUNNEL ALIGNMENT

The following checklist can be followed for an economical, trouble-free alignment of a long tunnel.

(i) Does the tunnel passes through the young mountains?
(ii) Is there intra-thrust zone?
(iii) Are there active and inactive fault/thrust zones?
(iv) Where are thick shear zones?
(v) Is rock cover excessive?
(vi) Is pillar width between tunnels adequate?
(vii) Are there thermic zones of too high ground temperature?
(viii) What is the least rock cover or shallow tunnel beneath the gullies/river/ocean?
(ix) Are there water-charged rock masses?
(x) Are there swelling rocks?
(xi) Are joints oriented unfavorably or strike parallel to the tunnel axis (Table 4.7)? Is tunnel along anticline (favorable) or syncline (unfavorable)?
(xii) Please mark expected tunnelling conditions along all alignments according to Chapter 13.
(xiii) In which reaches, tunnel boring machines may be used?
(xiv) In which reaches, conventional method is recommended?
(xv) Is it likely that a landslide-dam is formed and lake water enters the tail race tunnel and powerhouse cavern, etc.
(xvi) What are the expected costs of tunnelling for different alignments along with their periods of completion?

Mega chaos is self-organizing.

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5

Rock mass quality Q

"Genius is 99 percent perspiration and 1 percent inspiration."

Bernard Shaw

5.1 THE Q-SYSTEM

Barton et al. (1974) at the Norwegian Geotechnical Institute (NGI) originally proposed the Q-system of rock mass classification on the basis of about 200 case histories of tunnels and caverns. They have defined the rock mass quality Q by the following causative factors

$$Q = \left[\frac{\text{RQD}}{J_n}\right] \left[\frac{J_r}{J_a}\right] \left[\frac{J_w}{\text{SRF}}\right] \tag{5.1}$$

where

RQD = Deere's Rock Quality Designation ≥ 10,

$= 115 - 3.3\, J_v \leq 100$ (5.1a)

J_n = Joint set number,

J_r = Joint roughness number for critically oriented joint set,

J_a = Joint alteration number for critically oriented joint set,

J_w = Joint water reduction factor,

SRF = Stress reduction factor to consider in situ stresses and

J_v = Volumetric joint count.

For various rock conditions, the ratings (numerical value) to these six parameters are assigned. The six parameters given in equation (5.1) are defined below. The goal of Q-system is preliminary empirical design of support system for tunnels and caverns (see Chapter 10 on NMT). There are 1260 case records to prove efficacy of this design approach. It is the best among all the classification systems for support in tunnels (Kumar, 2002).

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B. Singh and R. K. Goel

5.1.1 Rock quality designation (RQD)

RQD is discussed in Section 4.2.2 and in detail by Singh and Goel (1999). The RQD value in percentage is also the rating of RQD for the Q-system. In case of a poor rock mass where RQD is less than 10 percent, a minimum value of 10 should be used to evaluate Q (Table 5.1). In case the rock cores are not available, the RQD may be estimated by the volumetric joint count (J_v) from equation (5.1a). Experience shows that the RQD estimated from J_v is conservative. The J_v is sum of frequencies of all joint sets per meter in a pit of 1 m × 1 m × 1 m.

5.1.2 Joint set number (J_n)

The parameter J_n, representing the number of joint sets, is often affected by foliations, schistocity, slaty cleavages or beddings, etc. If strongly developed, these parallel discontinuities should be counted as a complete joint set. If there are few joints visible or only occasional breaks in rock core due to these features, then one should count them as "a random joint set" while evaluating J_n from Table 5.2. Rating of J_n is approximately equal to square of the number of joint sets.

5.1.3 Joint roughness number and joint alteration number (J_r and J_a)

The parameters J_r and J_a, given in Tables 5.3 and 5.4, respectively, represent roughness and degree of alteration of joint walls or filling materials. The parameters J_r and J_a should be obtained for the weakest critical joint set or clay-filled discontinuity in a given zone. If the joint set or the discontinuity with the minimum value of (J_r/J_a) is favorably oriented for stability, then a second less favorably oriented joint set or discontinuity may be of greater significance, and its value of (J_r/J_a) should be used when evaluating Q from equation (5.1). For the critical orientation of the joint sets, Table 4.7 in Chapter 4, may be referred.

Table 5.1 Rock quality designation RQD (Barton, 2002).

Condition	RQD
A. Very poor	0–25
B. Poor	25–50
C. Fair	50–75
D. Good	75–90
E. Excellent	90–100

Notes: (i) Where RQD is reported or measured as ≤10 (including 0), a nominal value of 10 is used to evaluate Q in equation (5.1). (ii) RQD intervals of 5, i.e., 100, 95, 90 etc. are sufficiently accurate.

Table 5.2 Joint set number J_n (Barton, 2002).

Condition	J_n
A. Massive, no or few joints	0.5–1.0
B. One joint set	2
C. One joint set plus random	3
D. Two joint sets	4
E. Two joint sets plus random	6
F. Three joint sets	9
G. Three joint sets plus random	12
H. Four or more joint sets, random, heavily jointed, "sugar cube", etc.	15
J. Crushed rock, earthlike	20

Notes: (i) For intersections use $(3.0 \cdot J_n)$. (ii) For portals use $(2.0 \cdot J_n)$.

Table 5.3 Joint roughness number J_r (Barton, 2002).

Condition	J_r
(a) Rock wall contact and	
(b) Rock wall contact before 10 cm shear	
A. Discontinuous joint	4
B. Rough or irregular, undulating	3
C. Smooth, undulating	2.0
D. Slickensided, undulating	1.5
E. Rough or irregular, planar	1.5
F. Smooth, planar	1.0
G. Slickensided, planar	0.5
(c) No rock wall contact when sheared	
H. Zone containing clay minerals thick enough to prevent rock wall contact	1.0
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0

Notes: (i) Descriptions refer to small-scale features and intermediate scale features, in that order. (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m. (iii) $J_r = 0.5$ can be used for planar, slickensided joints having lineation, provided the lineations are favorably oriented. (iv) J_r and J_a classification is applied to the joint set or discontinuity that is least favorable for stability both from the point of view of orientation and shear resistance, τ.

5.1.4 Joint water reduction factor (J_w)

The parameter J_w (Table 5.5) is a measure of water pressure, which has an adverse effect on the shear strength of joints. This is due to reduction in the effective normal stress across joints. Water in addition may cause softening and possible wash-out in the

Table 5.4 Joint alteration number J_a (Barton, 2002).

Condition	ϕ_r approx (degree)	J_a
(a) Rock wall contact (No mineral filling, only coating)		
A. Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote		0.75
B. Unaltered joint walls, surface staining only	25–35	1.0
C. Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	25–30	2.0
D. Silty or sandy clay coatings, small clay fraction (non-softening)	20–25	3.0
E. Softening or low friction clay mineral coatings, i.e., kaolinite, mica. Also chlorite, talc, gypsum and graphite, etc. and small quantities of swelling clays (Discontinuous coatings, 1–2 mm or less in thickness)	8–16	4.0
(b) Rock wall contact before 10 cm shear (Thin mineral fillings)		
F. Sandy particles, clay-free disintegrated rock, etc.	25–30	4.0
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, <5 mm in thickness)	16–24	6.0
H. Medium or low over-consolidation, softening, clay mineral fillings (continuous, <5 mm in thickness)	12–16	8.0
J. Swelling clay fillings, i.e., montmorillonite (continuous, <5 mm in thickness). Value of J_a depends on percent of swelling clay-size particles, and access to water, etc.	6–12	8–12
(c) No rock wall contact when sheared (Thick mineral fillings)		
K, L, M. Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6–24	6, 8 or 8–12
N. Zones or bands of silty or sandy clay, small clay fraction (non-softening)	–	5
O, P, R. Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	6–24	10, 13 or 13–20

case of clay-filled joints. The value of J_w should correspond to the future ground water condition where seepage erosion or leaching of chemical can alter permeability of rock mass significantly.

5.1.5 Stress reduction factor (SRF)

The parameter SRF (Table 5.6) is a measure of (i) loosening pressure in the case of an excavation through shear zones and clay bearing rock masses, (ii) rock stress q_c/σ_1 in a competent rock mass, where q_c is uniaxial compressive strength of rock material and σ_1 is

Table 5.5 Joint water reduction factor J_w (Barton, 2002).

Condition	Approx water pressure (MPa)	J_w
A. Dry excavations or minor inflow, i.e., 5 l/min locally	<0.1	1
B. Medium inflow or pressure, occasional out-wash of joint fillings	0.1–0.25	0.66
C. Large inflow or high pressure in competent rock with unfilled joints	0.25–1.0	0.5
D. Large inflow or high pressure, considerable out-wash of joint fillings	0.25–1.0	0.33
E. Exceptionally high inflow or water pressure at blasting, decaying with time	>1.0	0.2–0.1
F. Exceptionally high inflow or water pressure continuing without noticeable decay	>1.0	0.1–0.05

Notes: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed. (ii) Special problems caused by ice formation are not considered. (iii) For general characterization of rock masses distant from excavation influences, the use of $J_w = 1.0, 0.66, 0.5, 0.33$, etc. as depth increases from 0–5, 5–25, 25–250 to >250 m is recommended, assuming that RQD/J_n is low enough (e.g., 0.5–25) for good hydraulic conductivity. This will help to adjust Q for some of the effective stress and water softening effects, in combination with appropriate characterization values of SRF. Correlations with depth-dependent static modulus of deformation and seismic velocity will then follow the practice used when these were developed.

Table 5.6 Stress reduction factor SRF (Barton, 2002).

Condition	SRF
(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated	
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10.0
B. Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤50 m)	5.0
C. Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation >50 m)	2.5
D. Multiple-shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5
E. Single-shear zones in competent rock (clay-free) (depth of excavation ≤50 m)	5.0
F. Single-shear zones in competent rock (clay-free) (depth of excavation >50 m)	2.5
G. Loose, open joints, heavily jointed or "sugar cube", etc. (any depth)	5.0

Continued

Table 5.6—Continued

	q_c/σ_1	σ_θ/q_c	SRF (Old)	SRF (New)
(b) Competent rock, rock stress problems				
H. Low stress, near surface open joints	> 200	<0.01	2.5	2.5
J. Medium stress, favorable stress condition	200–10	0.01–0.3	1.0	1.0
K. High stress, very tight structure (usually favorable to stability, may be unfavorable to wall stability	10–5	0.3–0.4	0.5–2	0.5–2.0
L. Moderate slabbing after >1 h in massive rock	5–3	0.5–0.65	5–9	5–50
M. Slabbing and rock burst after a few minutes in massive rock	3–2	0.65–1.0	9–15	50–200
N. Heavy rock burst (strain-burst) and immediate deformations in massive rock	<2	>1	15–20	200–400
(c) Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressures				
		σ_θ/q_c		
O. Mild squeezing rock pressure		1–5		5–10
P. Heavy squeezing rock pressure		>5		10–20
(d) Swelling rock; chemical swelling activity depending on presence of water				
Q. Mild swelling rock pressure				5–10
R. Heavy swelling rock pressure				10–15

Notes: (i) Reduce these SRF values by 25–50% if the relevant shear zones only influence but do not intersect the excavation. This will also be relevant for characterization. (ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce q_c to $0.75q_c$; when $\sigma_1/\sigma_3 > 10$, reduce q_c to $0.50q_c$ (where q_c is unconfined compressive strength), σ_1 and σ_3 are major and minor principal stresses, σ_θ the maximum tangential stress (estimated from elastic theory). (iii) Few case records available where depth of crown below surface is less than span width, suggest SRF increase from 2.5 to 5 for such cases (see H). (iv) Cases L, M and N are usually most relevant for support design of deep tunnel excavation in hard massive rock masses, with RQD/J_n ratios from about 50–200. (v) For general characterization of rock masses distant from excavation influences, the use of SRF = 5, 2.5, 1.0 and 0.5 is recommended as depth increases from 0–5, 5–25, 25–250, >250 m. This will help to adjust Q for some of the effective stress effects, in combination with appropriate characterization values of J_w. Correlations with depth-dependent static modulus of deformation and seismic velocity will then follow the practice used when these were developed. (vi) Cases of squeezing rock may occur for depth $H > 350Q^{1/3}$ [Singh and Goel, 1999]. Rock mass compressive strength can be estimated from $\sigma_{cm} \approx 5\gamma(Q_c)^{1/3}$ (MPa) where γ is the rock density in t/m^3, and $Q_c = Q(\sigma_c/100)$.

the major principal stress before excavation and (iii) squeezing or swelling pressures in incompetent rock masses. SRF can also be regarded as a total stress parameter.

Note 1 – SRF should be reduced where micro-folding occurs and its axis is nearly parallel to the strike of walls of cavern or tunnels. The accumulated high stresses may be released locally during excavation (leading to failure of rock bolts in weak rocks).

Note 2 – In jointed rocks under high overburden ($H > 1000$ m), rock burst may not occur due to strength enhancement by intermediate stress (σ_2) along axis of underground opening (cases L, M, N in Table 5.6). So SRF should be selected according to the actually observed rock burst condition and not expected rock burst condition (from L, M, N cases in Table 5.6).

Ratings of all the six parameters are given in Tables 5.1 to 5.6. The ratings of these parameters obtained for a given rock mass are substituted in equation (5.1) to get rock mass quality Q.

As seen from equation (5.1), the rock mass quality (Q) may be considered as a function of only three parameters which are approximate measures of:

a. Block size (RQD/J_n)	:	It represents overall structure of rock mass
b. Inter-block shear strength (J_r/J_a)	:	It has been found that $\tan^{-1}(J_r/J_a)$ is a fair approximation to the actual peak sliding angle of friction along the clay coated joints (Table 5.7)
c. Active stress (J_w/SRF)	:	It is an empirical factor describing the active effective stress

Table 5.7 Estimation of angle of internal friction from the parameters J_r and J_a (Barton, 2002).

Description	J_r	$\tan^{-1}(J_r/J_a)$				
(a) Rock wall contact		(Thin coatings)				
		$J_a = 0.75$	1.0	2.0	3.0	4.0
A. Discontinuous joints	4.0	79°	76°	63°	53°	45°
B. Rough, undulating	3.0	76°	72°	56°	45°	37°
C. Smooth, undulating	2.0	69°	63°	45°	34°	27°
D. Slickensided, undulating	1.5	63°	56°	37°	27°	21°
E. Rough, planar	1.5	63°	56°	37°	27°	21°
F. Smooth, planar	1.0	53°	45°	27°	18°	14°
G. Slickensided, planar	0.5	34°	27°	14°	9.5°	7.1°
(b) Rock wall contact when sheared		(Thin filling)				
	J_r	$J_a = 4.0$	6	8	12	
A. Discontinuous joints	4.0	45°	34°	27°	18°	
B. Rough, undulating	3.0	37°	27°	21°	14°	
C. Smooth, undulating	2.0	27°	18°	14°	9.5°	
D. Slickensided, undulating	1.5	21°	14°	11°	7.1°	
E. Rough, planar	1.5	21°	14°	11°	7.1°	
F. Smooth, planar	1.0	14°	9.5°	7.1°	4.7°	
G. Slickensided, planar	0.5	7°	4.7°	3.6°	2.4°	
(c) No rock wall contact when sheared		(Thick fillings)				
	J_r	$J_a = 5$	6	8	12	
Nominal roughness of discontinuity rock walls	1.0	11.3°	9.5°	7.1°	4.8°	
	J_r	$J_a = 13$	16	20	–	
	1.0	4.4°	3.6°	2.9°	–	

The first quotient (RQD/J_n) represents the rock mass structure and is a measure of block size or the size of the wedge formed by the presence of different joint sets. In a given rock mass, the rating of parameter J_n could increase with the tunnel size in certain situations where additional joint sets are encountered. Hence it is not advisable to use Q-value obtained from a small drift to estimate the support pressure for a large tunnel or a cavern. It would be more appropriate to obtain J_n from drill core observations or a borehole camera.

The second quotient (J_r/J_a) represents the roughness and frictional characteristics of joint walls or filling materials. It should be noted that value of J_r/J_a is collected for the critical joint set, i.e., the joint set which is most unfavorable for stability of a key rock block in roof.

The third quotient (J_w/SRF) is an empirical factor describing "active stress condition." The stress reduction factor SRF, is a measure of: (i) loosening pressure in the case of an excavation through shear zones and clay bearing rocks, (ii) rock stress in competent rocks and (iii) squeezing pressure in plastic incompetent rocks; and can be regarded as a total stress parameter. The water reduction factor J_w is a measure of water pressure, which has an adverse effect on the shear strength of joints due to reduction in effective normal stress. Water, in addition, causes softening and possible outwash in the case of clay-filled joints.

5.2 THE JOINT ORIENTATION AND THE Q-SYSTEM

Commenting on the joint orientation, Barton et al. (1974) stated that it was not found to be an important parameter as expected. Part of the reason for this may be that the orientation of many types of excavation can be, and normally are, adjusted to avoid the maximum effect of unfavorably oriented major joints. Barton et al. (1974) also stated that the parameters J_n, J_r and J_a appear to play a more important role than the joint orientation, because the number of joint sets determines the degree of freedom for block movement (if any); and the frictional and dilatational characteristics (J_r) can counter-balance the down-dip gravitational component of weight of wedge formed by the unfavorably oriented joints. If joint orientation had been included, the classification system would be less general, and its essential simplicity lost.

However, it is suggested to collect the rating for J_r and J_a for the most critical joint set. The critical joint set or "very unfavorable joint set" with respect to tunnel axis may be obtained from the Table 4.7 given by Bieniawski (1989).

5.3 UPDATING THE Q-SYSTEM

Updating the 1974 Q-system has taken place on several occasions during the last few years, and is now based on 1260 case records where the installed rock support has been correlated to the observed Q-values. The original parameters of the Q-system have not been changed, but some of the ratings for the stress reduction factor (SRF) have been altered by Grimstad and Barton (1993). The new ratings of SRF for competent rocks are shown in Table 5.6. This was done because a hard massive rock under high stress requires far more support than those recommended by the Q-value with SRF (old) ratings as proposed by Barton et al. (1974). In the original 1974 Q-system, this problem was addressed in a supplementary note instructing how to support spalling or rock burst zones with closely spaced end-anchored rock bolts and triangular steel plates. Recent experience from tunnels under high stresses in hard rocks suggests less bolting, but extensive use of steel fiber reinforced shotcrete (SFRS), an unknown product when the Q-system was first developed in 1974. Updating the Q-system has shown that in the most extreme case of high stress and hard massive (unjointed) rocks, the maximum SRF value has to be increased from 20 to 400 in order to give a Q-value which correlates with the modern rock supports shown in Fig. 10.2. In the case of moderately jointed rocks, SRF needs to be reduced significantly according to the actually observed tunnelling conditions (Kumar, 2002).

Authors' experience suggests that overburden height H should be considered in addition to SRF in Table 5.6 for obtaining the ratings of squeezing ground conditions (Section 5.6.2). It is our feeling that old values of SRF should be used in assessment of Q-value in the jointed rocks.

5.4 COLLECTION OF FIELD DATA

The length of core or rock exposures to be used for evaluating the first four parameters (RQD, J_n, J_r and J_a) would depend on the uniformity of the rock mass. If there is little variation, a core or wall length of 5–10 m should be sufficient. However, in a few meters wide closely jointed shear zone with alternate sound rock, it will be necessary to evaluate these parameters separately if it is considered that the closely jointed shear zones are wide enough to justify special treatment (i.e., additional shotcrete); compared to only systematic bolting in the remainder of the excavation. If, on the other hand, the shear zones are less than 1/2 meters and occur frequently, then an overall reduced value of Q for the entire tunnel reach may be most appropriate since increased support is likely to be applied uniformly along the entire length of such variable zones. In such cases, a core or wall length of 10–50 m may be needed to obtain an overall picture of the reduced rock mass quality (Fig. 28.5).

Notes:

1. Values of the rock mass quality Q be obtained separately for the roof, the floor and the two walls, particularly when the geological description of the rock mass is not uniform around the periphery of an underground opening.
2. In case of power tunnels, it is suggested that the value of J_w for calculation of ultimate support pressures should be reduced assuming that seepage water pressure in Table 5.5 is equal to the internal water pressure after commissioning the hydro-electric projects.

5.4.1 Suggestions for beginners

Beginners may find difficulty in selecting a single rating for a particular parameter. They may opt for a range of rating or two ratings or values for tension-free judgment. Subsequently, a geometrical mean may be obtained from the minimum and the maximum values for obtaining a representative value of the parameter. According to authors' experience, this will not only reduce the bias but would also generate confidence among the users.

It is proposed that for the purpose of eliminating the bias of an individual, the rating for different parameters should be given a range in preference to a single value.

To overcome the problem of selecting a representative rating of various parameters, NGI has proposed a geotechnical chart (Fig. 5.1). The main body of the geotechnical chart consists of rectangular graduated areas for making numerous individual observations of joints and jointing characteristics, in the form of a histogram. They proposed that efforts should be made to estimate approximate percentages of the various qualities of each

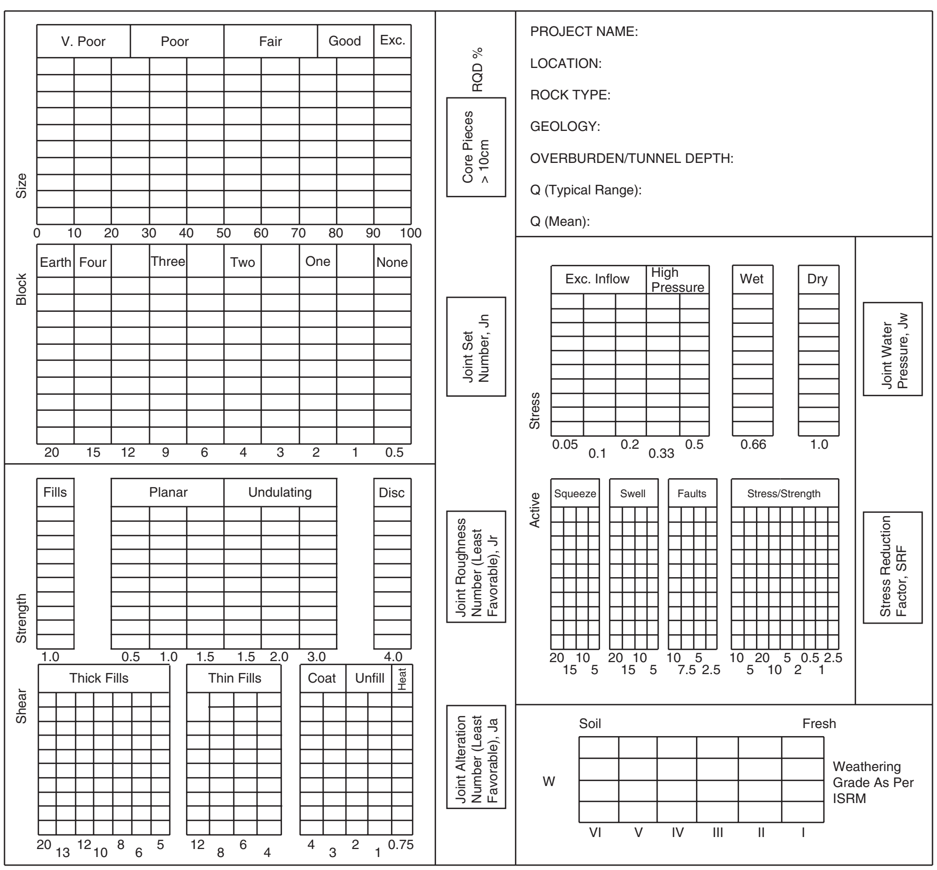

Fig. 5.1 Data sheet for recording Q parameters (Barton, 1993).

Table 5.8 Weighted average method of obtaining Q-value (Barton, 1993).

Parameter of Q	Poorest value (10%)	Most typical value (60%)	Maximum value (30%)	Weighted average
RQD	25	65	85	67
J_n	12	9	–	9.42
J_r	1.5	3	4	2.05
J_a	4	2	1	1.9
J_w	0.66	1	1	0.966
SRF	7.5	5	2.5	4.5

observed parameter, i.e., 10% poorest, 60% most typical, 30% best or maximum value, since the weighted average from all the histograms masks the extreme values. For example, the values of Q parameters collected at a location are shown in the following Table 5.8.

Using the weighted average value of each parameter, one can obtain a more realistic Q from equation (5.1). The weighted average value has been obtained using the percentage weightage mentioned above and as shown for RQD below.

A weighted average for RQD in above Table 5.8 is obtained as

$$(10 \times 25 + 60 \times 65 + 30 \times 85)/100 = 67$$

Similarly, weighted averages can be obtained for other parameters like joint wall compressive strength (JCS), joint wall roughness coefficient (JCS), etc. as proposed by NGI.

5.5 CLASSIFICATION OF THE ROCK MASS

The rock mass quality Q is a very sensitive index and its value varies from 0.001 to 1000. Use of the Q-system is specifically recommended for tunnels and caverns with arched roof. On the basis of the Q-value, the rock masses have been classified into nine categories (Table 5.9). In case the rock mass quality varies from Q_{min} to Q_{max}, the average rock mass quality of $(Q_{max} \times Q_{min})^{1/2}$ may be assumed in the design calculations.

5.6 ESTIMATION OF SUPPORT PRESSURE

5.6.1 Using approach of Barton et al. (1974)

Barton et al. (1974, 1975) plotted support capacities of 200 underground openings against the rock mass quality (Q) as shown in Fig. 5.2. They found the following empirical correlation for ultimate support pressure:

$$p_v = (0.2/J_r)\, Q^{-1/3} \qquad (5.2)$$

Table 5.9 Classification of rock mass based on Q-values.

Q	Group	Classification
0.001–0.01	3	Exceptionally poor
0.01–0.1		Extremely poor
0.1–1		Very poor
1–4	2	Poor
4–10		Fair
10–40		Good
40–100	1	Very good
100–400		Extremely good
400–1000		Exceptionally good

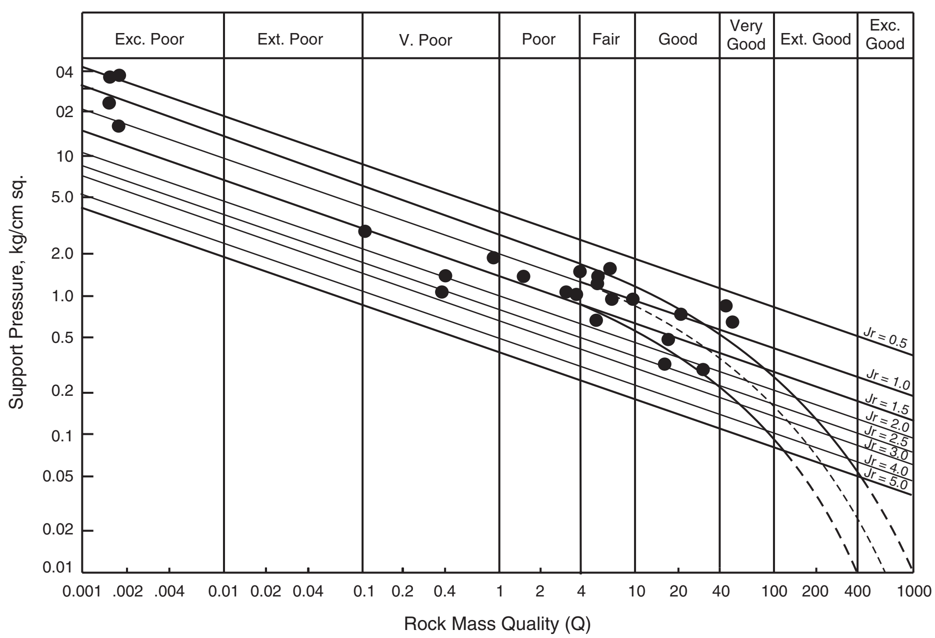

Fig. 5.2 Correlation between support pressure and rock mass quality Q (Barton et al., 1974).

$$p_h = (0.2/J_r)\, Q_w^{-1/3} \tag{5.3}$$

where

p_v = ultimate roof support pressure in MPa,
p_h = ultimate wall support pressure in MPa and
Q_w = wall factor.

It may be noted that dilatant joints or J_r values play a dominant role in the stability of underground openings. Consequently, support capacities may be independent of the opening size, unlike as believed by Terzaghi (1946).

The wall factor (Q_w) is obtained after multiplying Q by a factor which depends on the magnitude of Q as given below:

Range of Q	Wall Factor Q_w
>10	5.0Q
0.1–10	2.5Q
<0.1	1.0Q

Barton et al. (1974) further suggested that if the number of joint sets is less than three, equations (5.2) and (5.3) are expressed as equations (5.4a) and (5.4b), respectively.

$$p_v = \frac{0.2 \cdot J_n^{1/2}}{3 \cdot J_r} \cdot Q^{-1/3} \tag{5.4a}$$

$$p_h = \frac{0.2 \cdot J_n^{1/2}}{3 \cdot J_r} \cdot Q_w^{1/3} \tag{5.4b}$$

They felt that the short-term support pressure can be obtained after substituting 5Q in place of Q in equation (5.2). Thus, the ultimate support pressure is obtained as 1.7 times the short-term support pressure.

Bhasin and Grimstad (1996) suggested the following correlation for predicting support pressure in tunnels through poor rock masses (say $Q < 4$):

$$p_v = \frac{40B}{J_r} \cdot Q^{-1/3} \text{ kPa} \tag{5.5}$$

where B is diameter or span of the tunnel in meters. Equation (5.5) shows that the support pressure increases with tunnel size B in poor rock masses.

5.6.2 Correlation by Singh et al. (1992)

It may be mentioned that Q referred to in the above correlations is actually the post-excavation quality of a rock mass, because, in tunnels the geology of the rock mass is usually studied after blasting and on the spot decision is taken on support density.

5.6.2.1 Short-term support pressure

Vertical or roof support pressure The observed roof support pressure is related to the short-term rock mass quality (Q_i) for 30 instrumented tunnels by the following empirical correlation,

$$p_v = \frac{0.2}{J_r} \cdot Q_i^{-0.33} \cdot f \cdot f' \cdot f'' \text{ MPa} \tag{5.6}$$

$$f = 1 + (H - 320)/800 \geq 1 \tag{5.7}$$

where

Q_i = $5Q$ = short-term rock mass quality soon after the underground excavation,
p_v = short-term roof support pressure in MPa,
f = correction factor for overburden (Fig. 5.3),
f' = correction factor for tunnel closure (Table 5.10) obtained from Fig. 5.4 for squeezing ground condition ($H > 350\ Q^{1/3}$ and $J_r/J_a < 1/2$),
= 1 in non-squeezing ground,
f'' = correction factor for the time after excavation (equation (5.8)) and support erection and
H = overburden above crown or tunnel depth below ground level in meters.

While developing equation (5.6), the correction factors have been applied in steps. Firstly, the correction factor for tunnel depth has been applied, secondly, the correction for tunnel closure and finally the correction for time after support erection (Singh et al., 1992). Grimstad and Barton (1993) have agreed to overburden correction factor from equation (5.7).

Values of correction factors for tunnel closure (f') can be obtained from Table 5.10 on the basis of design value of tunnel closure. Table 5.10 has been derived from Figs 5.4a and 5.4b between normalized tunnel closure (u_a/a) and the correction factor for tunnel closure f' defined in equation (5.6). It may be noted that Figs 5.4a and 5.4b represent normalized observed ground response (reaction) curves for tunnel roof and walls, respectively in squeezing ground (See Section 19.7).

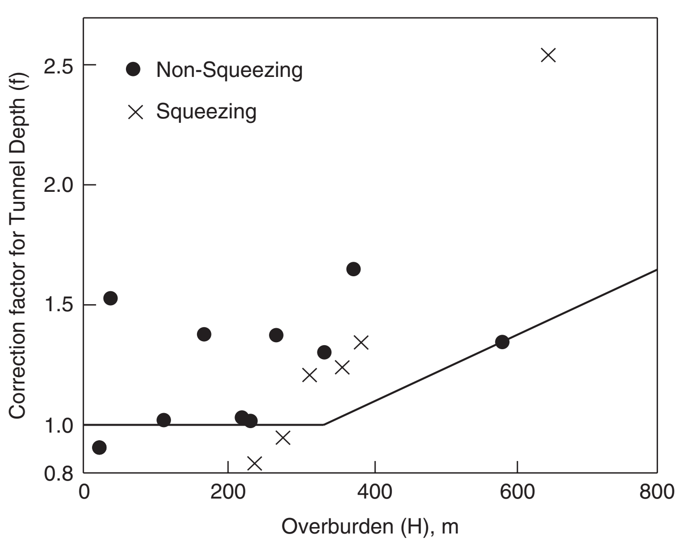

Fig. 5.3 Correction factor f for tunnel depth or overburden (Singh et al., 1992).

Table 5.10 Correction factor f' for tunnel closure (Singh et al., 1992).

S.No.	Rock condition	Support system	Tunnel closure (u_a/a), %	Correction factor, f'
1.	Non-squeezing ($H < 350\ Q^{0.33}$)	–	<1	1.1
2.	Squeezing ($H > 350\ Q^{0.33}$, $J_r/J_a < 0.5$)	Very stiff	<2%	>1.8
3.	As above	Stiff	2–4%	0.85
4.	As above	Flexible	4–6%	0.70
5.	As above	Very flexible	6–8%	1.15
6.	As above	Extremely flexible	>8%	1.8

Notes: (i) Tunnel closure depends significantly on method of excavation. In extreme squeezing ground conditions, heading and benching method may lead to tunnel closure >8%. (ii) Tunnel closures more than 4% of tunnel span should not be allowed, otherwise support pressures are likely to build-up rapidly due to failure of rock arch. In such cases, additional rock anchors should be installed immediately to arrest the tunnel closure within a limiting value of 4% of width. (iii) Steel ribs with struts may not absorb more than 2% tunnel closure. Thus, SFRS is suggested as an immediate support at the face to be supplemented with steel arches behind the face in situations where excessive closures are encountered. (iv) The minimum spacing between the parallel tunnels is 5*B* center to center in squeezing ground, where *B* is the width of a tunnel.

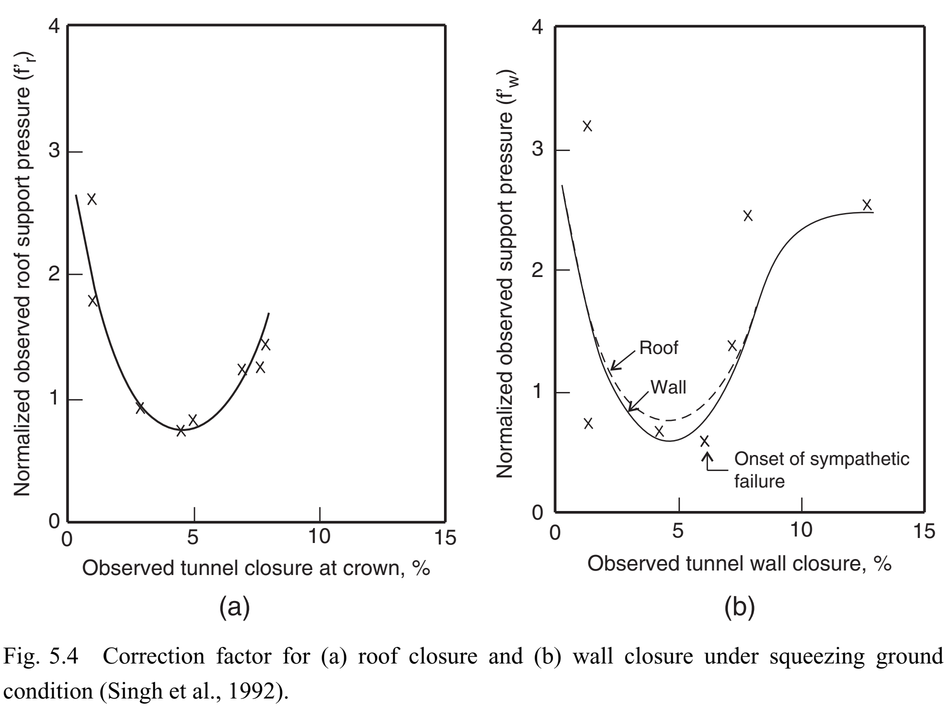

Fig. 5.4 Correction factor for (a) roof closure and (b) wall closure under squeezing ground condition (Singh et al., 1992).

The correction factor f'' for time was found as

$$f'' = \log(9.5\, t^{0.25}) \tag{5.8}$$

where t is time in months after support installation. Goel et al. (1995b) have verified correction factors f and f' for Maneri–Uttarkashi tunnel ($H = 700$ to 900 m). Kumar (2002) confirmed all the three correction factors from study of behavior of 27 km long NJPC tunnel in Himalaya, India ($H < 1400$ m). Incorporating the above three correction factors, Singh et al. (1992) proposed the following correlation for ultimate tunnel support pressure p_{ult}, after about 100 years ($f'' = 5^{1/3} = 1.7$),

$$p_{\mathrm{ult}} = \frac{0.2}{J_r} \cdot Q^{-1/3} \cdot f \cdot f' \text{ MPa} \tag{5.9}$$

Singh et al. (1992) have also studied the effect of tunnel size (2 m–22 m) on support pressures. They inferred no significant effect of size on observed support pressure. However, this aspect has been discussed in the chapter on rock mass number N.

Horizontal or wall support pressure For estimating wall support pressure, equation (5.6) may be used with short-term wall rock mass quality Q_{wi} in place of Q_i. The short-term wall rock quality Q_{wi} for short-term wall support pressure is obtained after multiplying Q_i by a factor which depends on the magnitude of Q as given below:

(i) For $Q > 10$; $Q_{wi} = 5.0.\ Q_i = 25Q$,
(ii) For $0.1 < Q < 10$; $Q_{wi} = 2.5.\ Q_i = 12.5Q$ and
(iii) For $Q < 0.1$; $Q_{wi} = 1.0.\ Q_i = 5Q$

The observed short-term wall support pressure is insignificant generally in non-squeezing rock conditions. It is, therefore, recommended that these may be neglected in the case of tunnels in rock masses of good quality of group 1 in Table 5.9 ($Q > 10$).

Note: Although the wall support pressure would be negligible in non-squeezing ground conditions, high wall support pressure is common in poor grounds or squeezing ground conditions. Therefore, invert struts with steel ribs be used when the estimated wall support pressure requires the use of wall support in exceptionally poor rock conditions and highly squeezing ground conditions. NATM or NTM are better choice otherwise.

5.6.2.2 Ultimate support pressure

Long-term monitoring at Chhibro cavern of Yamuna hydro-electric project in India has enabled the researchers to study the support pressure trend with time and with saturation. The study on the basis of 10 years monitoring has shown that the ultimate support pressure for water-charged rock masses with erodible joint fillings may rise upto 6 times the short-term support pressure (Mitra, 1990). The monitoring also suggested that for tunnels located near faults/thrusts (with plastic gouge) in seismic areas, the ultimate support pressure might be about 25 percent more due to accumulated strains in the rock mass along the fault.

On extrapolating the support pressure values for 100 years, a study of Singh et al. (1992) has shown that the ultimate support pressure would be about 1.75 times the short-term support pressure under non-squeezing ground conditions. Whereas in squeezing ground condition, Jethwa (1981) has estimated that the ultimate support pressure would be 2 to 3 times the short-term support pressure.

5.6.3 Evaluation of the approach of Barton et al. and Singh et al.

Support pressures estimated from equations (5.2) and (5.3) for various test-sections have been compared with the measured values. The estimates are reasonable (correlation coefficient $r = 0.81$) for tunnel sections through non-squeezing ground conditions. In squeezing ground conditions, the estimated support pressures never exceeded 0.7 MPa, whereas the measured values were as high as 1.2 MPa for larger tunnels. Therefore, it is thought that the Q-system may be unsafe for larger tunnels (diam. >9 m) under highly squeezing ground conditions (Goel et al., 1995a).

The estimated support pressures from equation (5.6) are also compared with the measured values for non-squeezing and squeezing ground conditions. It has been found that the correlation of Singh et al. (1992) provides reasonable estimates of support pressures.

5.6.3.1 Limitations of the Q-system

Kaiser et al. (1986) opined that SRF is probably the most contentious parameter. He concluded that it may be appropriate to neglect the SRF during rock mass classification and to assess the detrimental effects of high stresses separately. However, he has not given any alternate approach to assess high stress effect. Keeping this problem in mind, Goel et al. (1995a) have proposed rock mass number N, i.e., stress-free Q and incorporated stress-effect in the form of tunnel depth H to suggest a new set of empirical correlations for estimating support pressures. This aspect has been discussed in Chapter 7.

5.7 UNSUPPORTED SPAN

Barton et al. (1974) proposed the following equation for estimating equivalent dimension (D'_e) of a self-supporting or an unsupported tunnel

$$D'_e = 2.0\,(Q^{0.4})\,\text{m} \tag{5.10}$$

if $H < 350Q^{1/3}$ m

where

D'_e = equivalent dimension,
$= \dfrac{\text{span, diameter or height in meters}}{\text{ESR}}$,
Q = rock mass quality and
ESR = excavation support ratio.

In equivalent dimension, the span or diameter is used for analyzing the roof support, and the height of wall in case of wall support. Excavation support ratio (ESR) appropriate to a variety of underground excavations is listed in Table 5.11.

General requirements for permanently unsupported openings are,

(a) $J_n < 9$, $J_r > 1.0$, $J_a < 1.0$, $J_w = 1.0$, SRF < 2.5

Further, conditional requirements for permanently unsupported openings are given below.

(b) If RQD < 40, need $J_n < 2$
(c) If $J_n = 9$, need $J_r > 1.5$ and RQD > 90
(d) If $J_r = 1.0$, need $J_w < 4$
(e) If SRF > 1, need $J_r > 1.5$
(f) If span > 10 m, need $J_n < 9$
(g) If span > 20 m, need $J_n < 4$ and SRF < 1

The empirical design tables and charts for design of support system are presented in Chapter 10.

Table 5.11 Values of excavation support ratio ESR (Barton et al., 1974).

S. No.	Type of excavation	ESR
1	Temporary mine openings, etc.	3–5
2	Vertical shafts:	2.5
	(i) circular section	2.0
	(ii) rectangular/square section	
3	Permanent mine openings, water tunnels for hydropower (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations, etc.	1.6
4	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels, etc. (Cylindrical cavern?)	1.3
5	Oil storage caverns, power stations, major road and railway tunnels, civil defence chambers, portals, intersections, etc.	1.0
6	Underground nuclear power stations, railway stations, sports and public facilities, factories, etc.	0.8

Note: ESR should be increased by 1.5 times and Q by 5Q and Q_w by $5Q_w$ for temporary supports.

5.8 ROCK MASS CHARACTERIZATION

The Chaos theory appears to be applicable at micro-level only in nature and mostly near surface. Further, Chaos is self-organizing. For engineering use, the overall (weighted average) behavior is all that is needed. Since there is perfect harmony in nature at macro-level, the overall behavior should also be harmonious. Hence, in civil engineering, Chaos theory seems to find only limited applications. In fact in civil engineering practice, simple continuum characterization is more popular for large stable structures. Thus when one is talking about the behavior of jointed rock masses, one is really talking about the most probable behavior of rock masses.

In the case of caverns, empirical design should be checked by software packages like UDEC/3DEC, FLAC or FEM. They require the knowledge of deformation and strength characteristics of rock mass and joints.

5.8.1 Modulus of deformation of rock mass

In India a large number of hydro-electric power projects have been completed recently and several projects are under construction. These projects have generated a bulk of instrumentation data which have been analyzed by Mitra (1990), Mehrotra (1992), Verman (1993), Goel (1994) and Singh (1997). These new data and their analysis have led to a revision of the existing empirical relations and formulation of new correlations which are subsequently described in this chapter.

Modulus of deformation varies considerably. It is more in the horizontal direction than in the vertical direction. However, a mean value of modulus of deformation can be obtained by using the following relation (Barton, 2002).

$$E_{\mathrm{d}} = 10\left(\frac{\mathrm{Q}\cdot q_{\mathrm{c}}}{100}\right)^{1/3} \text{ GPa [for Q} = 0.1 \text{ to } 100 \text{ and } q_{\mathrm{c}} = 10\text{–}200\,\text{MPa]} \tag{5.11}$$

This relation gives good agreement with the correlations of Bieniawski (1978) and Serafim and Pereira (1983). The value of uniaxial compressive strength (UCS) of rock material (q_c) may be chosen from Table 5.12, where test results are not available.

Analysis of the field data has given the following correlation for modulus of deformation (E_d) of weak and nearly dry rock masses with coefficient of correlation as 0.85 (Singh, 1997),

$$E_{\mathrm{d}} = H^{0.2}\cdot \mathrm{Q}^{0.36}\,\text{GPa} \tag{5.12}$$

where Q is the rock mass quality at the time of uniaxial jacking test and H is overburden above tunnel in meters >50 m. Mehrotra (1992) found significant effect of saturation on E_d of water sensitive rocks (argillaceous). It is thus seen that the modulus of deformation of weak rock masses is pressure dependent. This correlation is suggested

Table 5.12 Average uniaxial compressive strength (q_c) of some rocks, measured on 50 mm diameter samples (Palmstrom, 2000).

Type of rock	q_c (MPa)	Type of rock	q_c (MPa)	Type of rock	q_c (MPa)	Type of rock	q_c (MPa)
Andesite (I)	150	Granite (I)	160	Marble (M)	<100>	Shale (S, M)	95
Amphibolite (M)	<160>	Granitic Gneiss (M)	100	Micagneiss (M)	90	Siltstone (S, M)	<80>
Augen Gneiss (M)	160	Granodiorite (I)	160	Micaquartzite (M)	85	Slate (M)	<190>
Basalt (I)	160	Granulite (M)	<90>	Micaschist (M)	<80>	Syenite (I)	150
Clay Schist (S, M)	55	Gneiss (M)	130	Phyllite (M)	<50>	Tuff (S)	<25>
Diorite (I)	140	Greenschist (M)	<75>	Quartzite (M)	<190>	Ultrabasic (I)	160
Dolerite (I)	200	Greenstone (M)	110	Quartzitic Phy. (M)	100	Clay (Hard)	0.7
Dolomite (S)	<100>	Greywacke (M)	80	Rhyolite (I)	85(?)	Clay (Stiff)	0.2
Gabbro (I)	240	Limestone (S)	90	Sandstone (S, M)	<100>	Clay (Soft)	0.03
				Serpentine (M)	135	Silt, sand (approx)	0.0005

Notes: (I) = Igneous; (M) = Metamorphic; (S) = Sedimentary; < > Large Variation.

for static analysis of underground openings and concrete dams. Further, the test data of 30 uniaxial jacking tests suggested the following correlation for elastic modulus E_e during unloading cycle (Singh, 1997).

$$E_e = 1.5\ Q^{0.6}\ E_r^{0.14}\ \text{GPa} \tag{5.13}$$

where

E_r = modulus of elasticity of rock material in GPa, and
Q = rock mass quality at the time of uniaxial jacking test in drift.

Equation (5.13) is valid for both dry and saturated rock masses. It is suggested for dynamic analysis of concrete dams subjected to impulsive seismic loads due to high intensity earthquake at a nearby epicenter (active fault). Some other correlations are summarized in Table 5.13. The symbols used in Table 5.13 are explained in the footnote.

Table 5.13 Empirical correlations for overall modulus of deformation of rock mass (GSI and RMR $\ll$ 100).

Authors	Expression for E_d (GPa)	Conditions
Bieniawski (1978)	$2\,\text{RMR} - 100$	$q_c > 100$ MPa and RMR > 50
Barton (2002)	$10[\text{Q}\cdot q_c/100]^{1/3}$	Q = 0.1 – 100 q_c = 10 – 200 MPa
Serafim and Pereira (1983)	$10^{(\text{RMR}-10)/40}$	$q_c \geq 100$ MPa
Nicholson and Bieniawski (1990)	$E_d/E_r = 0.0028\,\text{RMR}^2 + 0.9\,e^{(\text{RMR}/22.82)}$	–
Verman (1993)	$0.3\,H\alpha\cdot 10^{(\text{RMR}-20)/38}$	α = 0.16 to 0.30 (higher for poor rocks) $q_c \leq 100$ MPa; $H \geq 50$ m; $J_w = 1$ Coeff. of correlation = 0.91
Mitri et al. (1994)	$E_d/E_r = 0.5[1-\cos(\pi\ \text{RMR}/100)]$	–
Hoek and Brown (1997)	$\frac{\sqrt{q_c}}{10}10^{(\text{GSI}-10)/40}$	$q_c \leq 100$ MPa GSI = RMR−5
Singh (1997)	$E_d = \text{Q}^{0.36}H^{0.2}$ $E_e = 1.5\text{Q}^{0.6}E_r^{0.14}$	Q < 10; $J_w = 1$ Coeff. of correlation for E_e = 0.96; $J_w \leq 1$

Note: The above correlations are expected to provide a mean value of the modulus of deformation.

5.8.2 Anisotropy of rock mass

Jointed rock masses have very low shear modulii due to very low shear stiffness of joints. The shear modulus of a jointed rock mass has been back-analyzed by Singh (1973) as follows.

$$G \approx E_d/10\,\text{GPa} \tag{5.14}$$

The axis of anisotropy are naturally along the weakest joint or a bedding plane. Low shear modulus changes stress distribution drastically in the foundations. Kumar (1988) studied its effect on lined tunnels and found it to be significant.

5.8.3 Q vs P-wave velocity

A correlation between seismic P-wave velocity and rock mass quality Q has been proposed by Barton (2002) on the basis of around 2000 measurements for a rough estimation of Q ahead of the tunnel face using seismic P-wave velocity,

$$Q = \frac{100}{q_c} 10^{[(V_p - 3500)/1000]} \quad \text{for } 500\,\text{m} > H > 25\,\text{m} \tag{5.15}$$

$$\frac{V_s}{V_p} = 0.50 \ \text{ to } \ 0.66$$

where V_p is P-wave velocity in meters per second and q_c is UCS of rock material in MPa. The V_s is the shear wave velocity of rock masses.

For good and fair quality of granites and gneisses, an even better fit is obtained using the relation $Q = (V_p - 3600)/50$ (Barton, 1991). Fig. 5.5 gives the approximate values of rock mass quality before underground excavation for a known P-wave velocity for different values of depth of overburden (H). It should be noted that P-wave velocity increases rapidly with the depth of overburden. Fig. 5.5 also suggests the following correlation between mean static modulus of deformation in roof (in GPa) and support pressure (in MPa).

$$p_{\text{roof}} = \frac{f \cdot f'}{E_{d(\text{mean})}}\,\text{MPa} \tag{5.16}$$

The advantage of this correlation is that cross-hole seismic tomography may be used in more direct and accurate manner for specifying expected rock qualities and potential rock support needs in tender documents. There is possibility in future to assess Q values at great depths along tunnel by the seismic refraction survey on the ground level before its excavation (Chapter 2). It may be noted that the Q values after squeezing or rock burst or seepage erosion may be significantly less than Q values before tunnelling, i.e., during seismic survey.

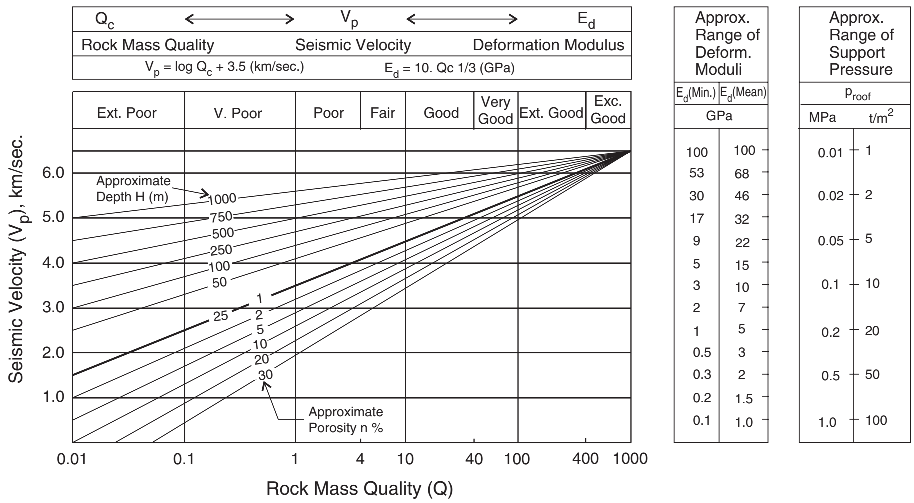

Fig. 5.5 An integration of V_p, Q, q_c, depth, porosity and static deformation modulus E_d which was developed stage by stage by trial and error fitting to field data (Barton, 2002).

5.8.4 Improvement in Q by grouting

According to Barton (2002), the in situ permeability (k) of rock mass near surface is of the order of (for Q = 0.01 to 100, $H < 25$ m and 1 lugeon = 1.3×10^{-5} cm/s),

$$k \approx Q_c = \frac{Q \cdot q_c}{100} \text{ lugeons} \tag{5.17}$$

This is surprisingly a simple correlation, yet true for Q between 0.01 and 100. Evidently the rock mass quality may be improved significantly by grouting of rock masses with cement grout, which would be proportional to the decrease in the maximum value of permeability of a grouted rock mass in any direction. Thus the required capacity of support systems for underground openings may be reduced substantially. Further, the long grout holes will drain off any water in the rock masses effectively, thereby reducing construction problems in the water-charged rock masses (flowing ground condition).

Grouting of the rock mass with permeability above 1 lugeon is feasible at sites with cement particles of maximum size of 100–150 μm (micrometer). Micro-fine and ultra-fine cements with maximum size of particles of 15–30 μm may be used in fair rock masses with physical apertures of about 0.05–0.10 μm. The thumb rule is that the maximum size of particle should be more than three or four times the physical aperture of joints (Barton, 2002). The assumption is that the grout will follow the path of least resistance which is thus the most permeable and least normally stressed joint set predominantly. Thus the least J_r/J_a value will also be improved. So, Barton (2002) has proved why the construction engineers often grouted the weak rock masses to improve its quality substantially in the past.

5.9 CONCLUDING REMARKS (BARTON, 2001, 2002)

Some doubts have been expressed whether or not in situ stress and water pressure should be considered in rock mass classification. It is because they are external and internal boundary conditions of a rock structure which are taken into account in all software packages. In fact the real response of rock masses is often highly coupled or interacting (Fig. 13.2).

The stress reduction factor (SRF) depends upon the height of overburden. Hence, it is external boundary condition. However, high overburden pressure causes damage to the rock mass structure, which needs to be considered in a rock mass classification system. It is worth seeing the time-dependent squeezing and rock burst phenomenon in deep tunnels. It helps to develop the total concept of rock mass quality.

The seepage water pressure in rock joints, on the other hand, represents the internal boundary condition. The high water pressure softens weak argillaceous rock masses due to seepage erosion and long-term weathering of rock joints particularly with coating of soft material like clay. So, joint water reduction factor (J_w) also needs to be considered for both rock mass classification and rock mass characterization.

The classification of rock mass does not mean that the correlation should be obtained with rock mass classification rating only. One should use intellect to search correlation with rock mass classification rating and other important parameters such as height of overburden, UCS, modulus of elasticity of rock material, size of opening, etc. The objective should be to improve the coefficient of correlation significantly; to the extent it is practical and simple to understand.

There is a world-wide appreciation on the utility of (post-excavation) rock mass quality Q-system for empirical design of support system for tunnels and caverns in many parts of the world in varying tunnelling conditions in over 1260 tunnels. Classification approach is really an amazing approach in civil engineering applications. Recently, Q-system has been extended to the rock mass characterization successfully (Barton, 2002).

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6
Rock mass number

"My attention is now entirely concentrated on Rock Mechanics, where my experience in applied soil mechanics can render useful services. I am more and more amazed about the blind optimism with which the younger generation invades this field, without paying any attention to the inevitable uncertainties in the data on which their theoretical reasoning is based and without making serious attempts to evaluate the resulting errors."

Annual Summary in Terzaghi's Diary

6.1 INTRODUCTION

One of the reasons why rock mass classifications has become more popular over the years is that these are easy to use and at the same time provide vital information about the stability, etc. Thus, rock mass classification is an amazingly successful approach. Classification leads to making fast decisions during tunnelling.

Despite their usefulness, one cannot deny the uncertainty in getting correct ratings of a few parameters. How to manage these uncertainties? With this objective, two rock mass indices – rock mass number N and rock condition rating RCR have been adopted. These indices are the modified versions of the two most popular classification systems, N from the Q-system of Barton et al. (1974) and RCR from the RMR-system of Bieniawski (1984).

Rock mass number, denoted by N, is stress-free rock mass quality Q. Stress effect has been considered indirectly in the form of overburden height H. Thus, N can be defined by the following equation, representing basic causative factors in governing the tunnelling conditions.

$$N = \left[\frac{\mathrm{RQD}}{J_\mathrm{n}}\right]\left[\frac{J_\mathrm{r}}{J_\mathrm{a}}\right][J_\mathrm{w}] \quad (6.1)$$

This is needed because of the problems and uncertainties in obtaining the correct rating of Barton's SRF parameter (Kaiser et al., 1986; Goel et al., 1995a).

Rock condition rating is defined as RMR without ratings for the crushing strength of the intact rock material and the adjustment of joint orientation. This is explained as below:

$$\mathrm{RCR} = \mathrm{RMR} - (\text{rating for crushing strength} + \text{adjustment of joint orientation}) \quad (6.2)$$

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B. Singh and R. K. Goel

RCR, therefore, is free from the crushing strength which is a parameter sometimes difficult to obtain at the site. Moreover, parameter wise, N and RCR have become equivalent and can be used for the purpose of interrelation.

6.2 INTERRELATION BETWEEN Q AND RMR

Interrelations between the two most widely used classification indices, the RMR of Bieniawski (1976) and the rock mass quality Q of Barton et al. (1974), have been proposed by many researchers. Bieniawski (1989) used 117 case histories involving 68 Scandinavian, 28 South African and 21 other documented case histories from the United States of America covering the entire range of Q and RMR to propose the following correlation.

$$\mathrm{RMR} = 9 \ln \mathrm{Q} + 44 \tag{6.3}$$

Based on case histories from New Zealand, Rutledge and Preston (1978) proposed a different correlation as

$$\mathrm{RMR} = 5.9 \ln \mathrm{Q} + 43 \tag{6.4}$$

Moreno (1980), Cameron-Clarke and Budavari (1981) and Abad et al. (1984) have also proposed different correlations between Q and RMR as presented in equations (6.5–6.7) respectively.

$$\mathrm{RMR} = 5.4 \ln \mathrm{Q} + 55.2 \tag{6.5}$$

$$\mathrm{RMR} = 5 \ln \mathrm{Q} + 60.8 \tag{6.6}$$

$$\mathrm{RMR} = 10.5 \ln \mathrm{Q} + 41.8 \tag{6.7}$$

Evaluation of all the correlations, given in equations (6.3) through (6.7), on the basis of 115 case histories including 77 reported by Bieniawski (1976), 4 from Kielder experimental tunnel reported by Hoek and Brown (1980) and 34 collected from India, has indicated that the correlation coefficients of these approaches are not very reliable. The correlation of Rutledge and Preston (1978) provides the highest correlation coefficient of 0.81, followed by Bieniawski (1976), Moreno (1980), Cameron-Clarke and Budavari (1981) and Abad et al. (1984) in decreasing order as shown in Table 6.1 and Fig. 6.1. These correlations, therefore, do not have high reliability for an interrelation between Q and RMR.

6.2.1 The new approach

Attempts to correlate Q and RMR in equations (6.3) through (6.7) ignore the fact that the two systems are not truly equivalent. It seems, therefore, that a good correlation can be developed if N and RCR are considered.

Table 6.1 Evaluation of various correlations between RMR and Q (Goel et al., 1995b).

Lines in Fig. 6.1	Approach	Correlation coefficient
A	Bieniawski (1976)	0.77
B	Rutledge and Preston (1978)	0.81
C	Moreno (1980)	0.55
D	Cameron-Clarke and Budavari (1981)	High scatter
E	Abad et al. (1984)	0.66

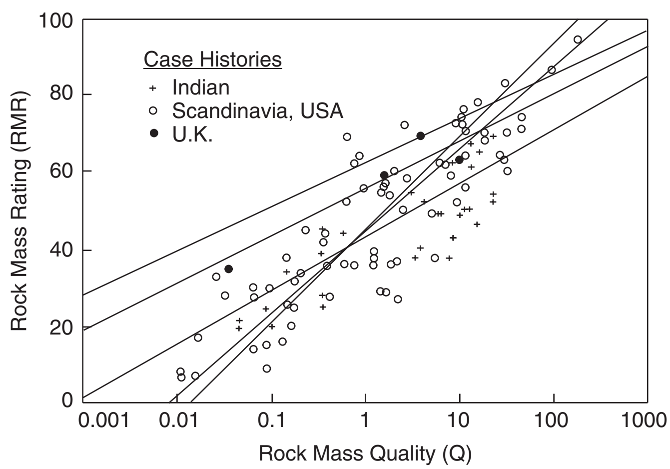

Fig. 6.1 Correlations between Q and RMR (Goel et al., 1995b).

Rock condition rating RCR and rock mass number N from 63 cases were used to obtain a new interrelation. The 63 cases comprised of 36 from India, 4 from Kielder experimental tunnel (reported by Hoek & Brown, 1980) and 23 NGI cases from Bieniawski (1984). Details about the six parameters for Q and information about joint orientation vis-à-vis tunnel axis in respect of these 23 NGI cases were picked up directly from Barton et al. (1974). Estimates of uniaxial crushing strength q_c of rock material were made from rock descriptions given by Barton et al. (1974) using strength data for comparable rock types from Lama and Vutukuri (1978). Using the ratings for joint orientation and q_c, so obtained, and RMR from Bieniawski (1984), it was possible to estimate values of RCR. Thus, the values of N and RCR for the 63 case histories were plotted in Fig. 6.2 and the following correlation is obtained:

$$\text{RCR} = 8 \ln N + 30 \quad \text{for} \quad q_c > 5 \text{ MPa} \tag{6.8}$$

Equation (6.8) has a correlation coefficient of 0.92. Equation (6.8) is not applicable on the borderline of soil and rock mass according to data of Sari and Pasamehmetoglu (2004). The following example explains how equation (6.8) could be used to obtain

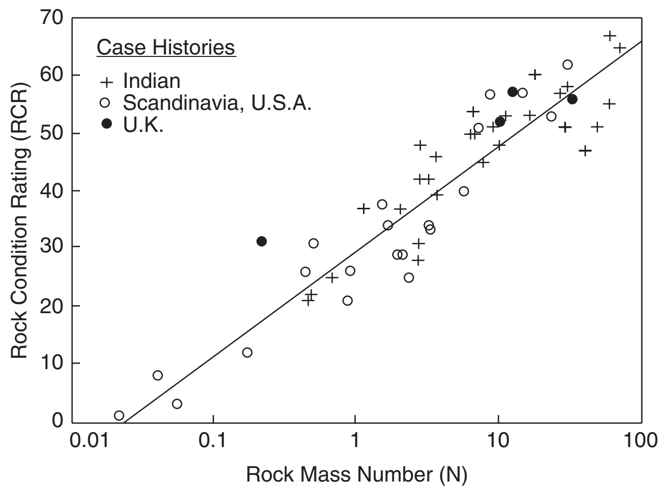

Fig. 6.2 Correlations between N and RCR (Goel et al., 1995b).

RMR from Q and vice versa. The values of the parameters of RMR and Q collected in the field are given in Table 6.2.

6.2.1.1 RMR from Q

$N = (\text{RQD}\ J_r\ J_w)/(J_n\ J_a) = 26.66$ as shown in Table 6.2
Corresponding to $N = 26.66$, RCR $= 56.26$ (equation (6.8))
RMR $=$ RCR $+$ (ratings for q_c and joint orientation as per equation (6.2)
RMR $= 56.26 + [4 + (-)12]$
RMR $= 48.26$ (It is comparable to RMR 49 obtained from direct estimation as shown in Table 6.2)

Table 6.2 Values of the parameters of RMR and Q collected in the field.

RMR-System		Q-System	
Parameters for RMR	Rating	Parameters for Q	Rating
RQD (80%)	17	RQD	80
Joint spacing	10	J_n	9
Joint condition	20	J_r	3
		J_a	1
Ground water	10	J_w	1
RCR	57	N	26.66
Crushing strength q_c	+4	SRF	2.5
Joint orientation	(−)12	–	–
RMR	49	Q	10.6

6.2.1.2 Q from RMR

RCR = RMR − (ratings for q_c and joint orientation as per equation (6.2))
RCR = 49 − (4 − 12)
RCR = 57
Corresponding to RCR = 57, N = 29.22 (equation (6.8))
Q = (N/SRF) = 29.22/2.5
Q = 11.68 (almost equal to the field estimated value, Table 6.2)

The slight difference in directly estimated values of Q and RMR and those obtained by the proposed interrelation are due to the inherent scatter in equation (6.8).

6.3 PREDICTION OF GROUND CONDITIONS

All the correlations for predicting ground conditions have been discussed in Fig. 13.1. The main advantage of rock mass number is that it does not assume ground condition but it predicts them.

In practice, the rock mass is classified into categories I, II, III, etc. Accordingly support systems are prescribed. There are unusual geological conditions at some sections. These possible conditions (flowing ground, etc.) should also be classified in the contract and support system should also be suggested. Further, there should be first and second contingency clauses in the same contract for better preparedness.

6.4 PREDICTION OF SUPPORT PRESSURE

These correlations are based on measured support pressures and other related parameters from several Indian tunnels having steel rib support. Detailed field studies have been carried out for eight tunnelling projects located in the Himalaya and the peninsular India.

Two sets of empirical correlations for estimating support pressure for tunnel sections under non-squeezing and squeezing ground conditions have been developed using N and the measured values of support pressures, the tunnel depth H, the tunnel radius a and the expected tunnel closure u_a from 25 tunnel sections (Goel et al., 1995a; Singh et al., 1997). The correlations are as follows:

Non-squeezing ground condition

$$p_v(\text{el}) = \left[\frac{0.12\,H^{0.1} \cdot a^{0.1}}{N^{0.33}}\right] - 0.038 \text{ MPa} \tag{6.9}$$

Kumar (2002) found that equation (6.9) is valid for overburden (H) up to 1400 m in case of NJPC tunnel, India.

Table 6.3 Correction factor for tunnel closure in equation (6.10) (Goel et al., 1995a).

S.No.	Degree of squeezing	Normalized tunnel closure (%)	$f(N)$
1.	Very mild squeezing ($270\,N^{0.33} \cdot B^{-0.1} < H < 360\,N^{0.33} \cdot B^{-0.1}$)	1–2	1.5
2.	Mild squeezing ($360\,N^{0.33} \cdot B^{-0.1} < H < 450\,N^{0.33} \cdot B^{-0.1}$)	2–3	1.2
3.	Mild to moderate squeezing ($450\,N^{0.33} \cdot B^{-0.1} < H < 540\,N^{0.33} \cdot B^{-0.1}$)	3–4	1.0
4.	Moderate squeezing ($540\,N^{0.33} \cdot B^{-0.1} < H < 630\,N^{0.33} \cdot B^{-0.1}$)	4–5	0.8
5.	High squeezing ($630\,N^{0.33} \cdot B^{-0.1} < H < 800\,N^{0.33} \cdot B^{-0.1}$)	5–7	1.1
6.	Very high squeezing ($800\,N^{0.33} \cdot B^{-0.1} < H$)	>7	1.7

Notations: N = rock mass number; H = tunnel depth in meters; B = tunnel width in meters.
Note: Tunnel closure depends significantly on the method of excavation. In highly squeezing ground condition, heading and benching method of excavation may lead to tunnel closure >8%.

Squeezing ground condition

$$p_v(\mathrm{sq}) = \left[\frac{f(\mathrm{N})}{30}\right] \cdot 10^{\left[\frac{H^{0.6} \cdot a^{0.1}}{50 \cdot N^{0.33}}\right]}\ \mathrm{MPa} \tag{6.10}$$

where

$p_v(\mathrm{el})$ = short-term roof support pressure in non-squeezing ground condition in MPa,
$p_v(\mathrm{sq})$ = short-term roof support pressure in squeezing ground condition in MPa,
$f(N)$ = correction factor for tunnel closure obtained from Table 6.3 and
H and a = tunnel depth and tunnel radius in meters, respectively.

The above correlations have been evaluated using measured support pressures and the correlation coefficient of 0.96 and 0.95 is obtained for equations (6.9) and (6.10), respectively (Goel et al., 1995a). It is also found that even for larger tunnels in squeezing ground conditions, the estimated support pressures (equation (6.10)) are matching with the measured values.

6.5 EFFECT OF TUNNEL SIZE ON SUPPORT PRESSURE

Prediction of support pressures in tunnels and the effect of tunnel size on support pressure are the two important problems of tunnel mechanics which attracted the attention of many researchers. The information presented here on the effect of tunnel size on support pressure has been taken from Goel et al. (1996).

Various empirical approaches of predicting support pressures have been developed in the recent past. Some researchers demonstrated that the support pressure is independent of tunnel size (Barton et al., 1974; Daemen, 1975; Jethwa, 1981; Singh et al., 1992), whereas others advocated that the support pressure is directly dependent on tunnel size (Terzaghi, 1946; Deere et al., 1969; Wickham et al., 1972; Unal, 1983; Bhasin & Grimstad, 1996). A review on the effect of tunnel size on support pressure with a concept proposed by Goel (1994) is presented for highlighting the effect of tunnel size on support pressure.

6.5.1 Review of existing approaches

Empirical approaches of estimating support pressure have been presented in Table 6.4 to study the effect of tunnel size on support pressure. A discussion is presented below.

Table 6.4 Important empirical approaches and their recommendations (Goel et al., 1996).

Approach	Results based on	Recommendations
Terzaghi (1946)	a. Experiments in sand b. Rectangular openings with flat roof c. Qualitative approach	Support pressure increases with the opening size
Deere et al. (1969)	a. Based on Terzaghi's theory and classification on the basis of RQD	Support pressure increases with the opening size
Wickham et al. (1972) RSR-system	a. Arched roof b. Hard rocks c. Quantitative approach	Support pressure increases with the opening size
Barton et al. (1974); Barton (2002) Q-system	a. Hard rocks b. Arched roof c. Quantitative approach	Support pressure is independent of the opening size
Unal (1983) using RMR of Bieniawski (1976)	a. Coal mines b. Rectangular openings with flat roof c. Quantitative approach	Support pressure increases with the opening size
Singh et al. (1992)	a. Arched roof (tunnel/cavern) b. Both hard and weak rocks c. Quantitative approach	Support pressure is observed to be independent of the opening size (2–22 m)

6.5.1.1 Influence of shape of the opening

Some empirical approaches listed in Table 6.4 have been developed for flat roof and some for arched roof. In case of an underground opening with flat roof, the support pressure is generally found to vary with the width or size of the opening, whereas in arched roof the support pressure is found to be independent of tunnel size (Table 6.4). RSR-system of Wickham et al. (1972) is an exception in this regard, probably because the system, being conservative, was not backed by actual field measurements for caverns. The mechanics suggests that the normal forces and therefore the support pressure will be more in case of a rectangular opening with flat roof by virtue of the detached rock block in the tension zone which is free to fall.

6.5.1.2 Influence of rock mass type

The support pressure is directly proportional to the size of the tunnel opening in the case of weak or poor rock masses, whereas in good rock masses the situation is reverse (Table 6.4). Hence, it can be inferred that the applicability of an approach developed for weak or poor rock masses has a doubtful application in good rock masses.

6.5.1.3 Influence of in situ stresses

Rock mass number N does not consider in situ stresses, which govern the squeezing or rock burst conditions. Instead the height of overburden is accounted for in equations (6.9) and (6.10) for estimation of support pressures. Thus, in situ stresses are taken into account indirectly.

Goel et al. (1995a) have evaluated the approaches of Barton et al. (1974) and Singh et al. (1992) using the measured tunnel support pressures from 25 tunnel sections. They found that the approach of Barton et al. is unsafe in squeezing ground conditions and the reliability of the approaches of Singh et al. (1992) and that of Barton et al. depend upon the rating of Barton's stress reduction factor (SRF). It has also been found that the approach of Singh et al. is unsafe for larger tunnels ($B > 9$ m) in squeezing ground conditions. Kumar (2002) has evaluated many classification systems and found rock mass number to be the best from the case history of NJPC tunnel, India.

6.5.2 New concept on effect of tunnel size on support pressure

Equations (6.9) and (6.10) have been used to study the effect of tunnel size on support pressure which is summarized in Table 6.5.

It is cautioned that the support pressure is likely to increase significantly with the tunnel size for tunnel sections excavated through the following situations:

(i) slickensided zone,
(ii) thick fault gouge,
(iii) weak clay and shales,

Table 6.5 Effect of tunnel size on support pressure (Goel et al., 1996).

S.No.	Type of rock mass	Increase in support pressure due to increase in tunnel span or diam. from 3 m to 12 m
A.	Tunnels with arched roof	
	1. Non-squeezing ground conditions	Up to 20 percent only
	2. Poor rock masses/squeezing ground conditions ($N = 0.5$ to 10)	20–60 percent
	3. Soft-plastic clays, running ground, flowing ground, clay-filled moist fault gouges, slickensided shear zones ($N = 0.1$ to 0.5)	100 to 400 percent
B.	Tunnels with flat roof (irrespective of ground conditions)	400 percent

(iv) soft-plastic clays,
(v) crushed brecciated and sheared rock masses,
(vi) clay-filled joints and
(vii) extremely delayed support in poor rock masses.

Further, both Q and N are not applicable to flowing grounds or piping through seams. They also do not take into account mineralogy (water-sensitive minerals, soluble minerals, etc.).

6.6 CORRELATIONS FOR ESTIMATING TUNNEL CLOSURE

Behavior of concrete, gravel and tunnel-muck backfills, commonly used with steel-arch supports, has been studied. Stiffness of these backfills has been estimated using measured support pressures and tunnel closures. These results have been used finally to obtain effective support stiffness of the combined support system of steel rib and backfill (Goel, 1994).

On the basis of measured tunnel closures from 60 tunnel sections, correlations have been developed for predicting tunnel closures in non-squeezing and squeezing ground conditions (Goel, 1994). The correlations are given below:

Non-squeezing ground condition

$$\frac{u_a}{a} = \frac{H^{0.6}}{28 \cdot N^{0.4} \cdot K^{0.35}}\% \tag{6.11}$$

Squeezing ground condition

$$\frac{u_a}{a} = \frac{H^{0.8}}{10 \cdot N^{0.3} \cdot K^{0.6}}\% \tag{6.12}$$

where

u_a/a = normalized tunnel closure in percent,
K = effective support stiffness ($= p_v \cdot a/u_a$) in MPa and
H and a = tunnel depth and tunnel radius (half of tunnel width) in meters, respectively.

These correlations can also be used to obtain desirable effective support stiffness so that the normalized tunnel closure is contained within 4 percent (in the squeezing ground).

6.7 EFFECT OF TUNNEL DEPTH ON SUPPORT PRESSURE AND CLOSURE IN TUNNELS

It is known that the in situ stresses are influenced by the depth below the ground surface. It is also learned from the theory that the support pressure and the closure for tunnels are influenced by the in situ stresses. Therefore, it is recognized that the depth of tunnel or the overburden is an important parameter while planning and designing the tunnels. The effects of tunnel depth or the overburden on support pressure and closure in tunnel have been studied using equations (6.9) to (6.12) under both squeezing and non-squeezing ground conditions which is summarized below.

(i) The tunnel depth has a significant effect on support pressure and tunnel closure in squeezing ground conditions. It has smaller effect under non-squeezing ground conditions, however (equation (6.9)).
(ii) The effect of tunnel depth is higher on the support pressure than the tunnel closure.
(iii) The depth effect on support pressure increases with deterioration in rock mass quality probably because the confinement decreases and the degree of freedom for the movement of rock blocks increases.
(iv) This study would be of help to planners and designers to take decisions on realigning a tunnel through a better tunnelling media or a lesser depth or both in order to reduce the anticipated support pressure and closure in tunnels.

6.8 APPROACH FOR OBTAINING GROUND REACTION CURVE (GRC)

According to Daemen (1975), ground reaction curve is quite useful for designing the supports specially for tunnels through squeezing ground conditions. An easy to use empirical approach for obtaining the ground reaction curve has been developed using equations (6.10) and (6.12) for tunnels in squeezing ground conditions. The approach has been explained with the help of an example.

For example, the tunnel depth H and the rock mass number N have been assumed as 500 m and 1, respectively and the tunnel radius a as 5 m. The radial displacement of the tunnel is u_a for a given support pressure $p_v(\mathrm{sq})$.

GRC using equation (6.10)

In equation (6.10), as described earlier, $f(N)$ is the correction factor for tunnel closure. For different values of permitted normalized tunnel closure (u_a/a), different values of $f(N)$ are proposed in Table 6.3. The first step is to choose any value of tunnel wall displacement u_a in column 1 of Table 6.6. Then the correction factor $f(N)$ is found from Table 6.3 as shown in column 2 of Table 6.6. Finally, equation (6.10) yields the support

Table 6.6 Calculations for constructing GRC using equation (6.10).

Assumed u_a/a (%) (1)	Correction factor (f) (2)	$p_v(\mathrm{sq})$ from equation (6.10) (MPa) (3)
0.5	2.7	0.86
1	2.2	0.7
2	1.5	0.475
3	1.2	0.38
4	1.0	0.317
5	0.8	0.25

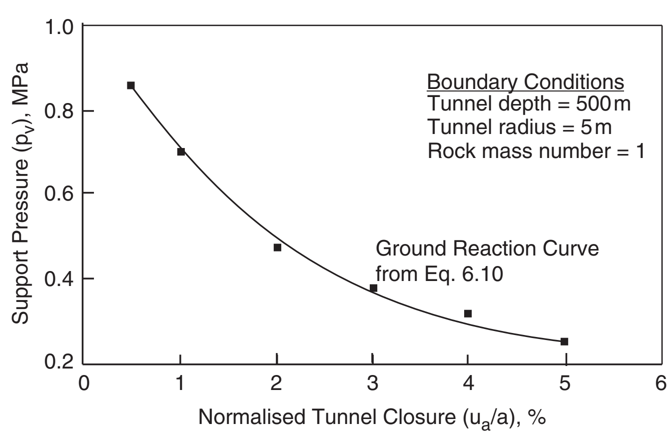

Fig. 6.3 Ground reaction curve obtained from equation (6.10).

pressure in roof (p_v) as mentioned in column 3 [Using Table 6.3 and equation (6.10), the support pressures [p_v(sq)] have been estimated for the assumed boundary conditions and for various values of u_a/a (column 1) as shown in Table 6.6]. Subsequently, using value of p_v (column 3) and u_a/a (column 1) from Table 6.6, GRC has been plotted for u_a/a up to 5 percent (Fig. 6.3).

It may be highlighted here that the approach is simple, reliable and user-friendly because the values of the input parameters can be easily obtained in the field.

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7
Strength of discontinuities

"Failure is success if we learn from it."

Malcom S. Forbes

7.1 INTRODUCTION

Rock mass is a heterogeneous, anisotropic and discontinuous mass. When civil engineering structures like dams are founded on rock, they transmit normal and shear stresses on discontinuities in rock mass. Failure may be initiated by sliding along a joint plane, near or along the foundation or along the abutments of dam. For a realistic assessment of the stability of structure with wedge, estimation of the shear resistance of a rock mass along any desired plane of potential shear or along the weakest discontinuity becomes essential. The shear strength of discontinuities depends upon the alteration of joints or the discontinuities, the roughness, the thickness of infillings or the gouge material, the moisture content, etc.

The mechanical difference between contacting and non-contacting joint walls will usually result in widely different shear strengths and deformation characteristics. In the case of unfilled joints, the roughness and compressive strength of the joint walls are important, whereas in the case of filled joints the physical and mineralogical properties of the gouge material separating the joint walls are of primary concern.

To quantify the effect of these on the strength of discontinuities, various researchers have proposed different parameters and correlations for obtaining strength parameters. Barton et al. (1974), probably for the first time, have considered joint roughness (J_r) and joint alteration (J_a) in their Q-system to take care of the strength of clay-coated discontinuities in the rock mass classification. Later, Barton and Choubey (1977) defined two parameters – joint roughness coefficient (JRC) and joint wall compressive strength(JCS) – and proposed an empirical correlation for friction of rock joints without fillings, which can be used for predicting the shear strength data accurately.

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7.2 JOINT WALL ROUGHNESS COEFFICIENT (JRC)

The wall roughness of a joint or discontinuity is potentially a very important component of its shear strength, especially in the case of undisplaced and interlocked features (e.g., unfilled joints). The importance of wall roughness declines as the thickness of aperture filling or the degree of any previous shear displacement increases.

Joint roughness coefficent, JRC_0 (JRC at laboratory scale) may be obtained by visual matching of actual roughness profiles with the set of standard profiles proposed by Barton and Choubey (1977). As such, the joint roughness coefficients are suggested for ten types of roughness profiles of joints (Fig. 7.1). The core sample will be intersected by joints at angles varying from 0 to 90° to the axis. Joint samples will therefore vary in some cases from a meter or more in length (depending upon the core length) to 100 mm (core diameter). Most samples are expected to be in the range of 100–300 mm in length.

The recommended approximate sampling frequency for the above profile-matching procedure is 100 samples per joint set per 1000 m of core. The two most adverse prominent sets should be selected, which must include the adverse joint set selected for J_r and J_a characterization.

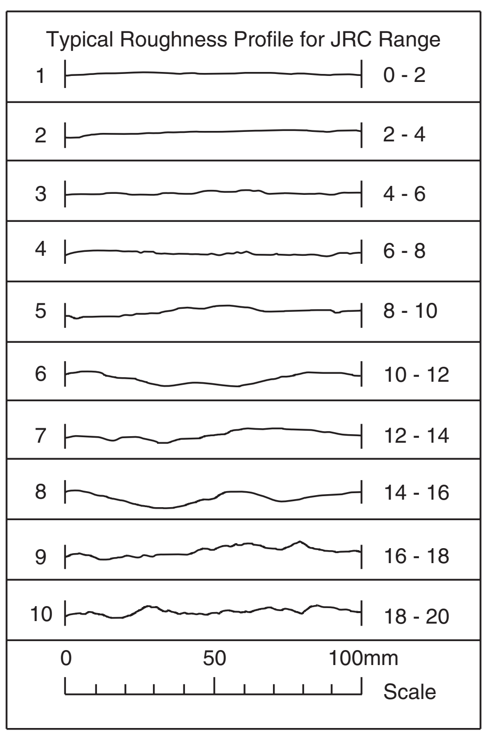


Fig. 7.1 Standard profiles for visual estimation of JRC (Barton & Choubey, 1977).

Roughness amplitude per length, i.e., *a* and *L* measurements will be made in the field for estimating JRC_n (JRC, at a natural large scale). The maximum amplitude of roughness (in millimeter) should be usually estimated or measured on profiles of at least two lengths along the joint plane, for example, 100 mm and 1 m length.

It has been observed that the JRC_n can also be obtained from JRC_0 using the following equation,

$$JRC_n = JRC_0(L_n/L_0)^{-0.02\,JRC_0} \tag{7.1}$$

where, L_0 is the laboratory scale length, i.e., 100 mm and L_n represents the natural larger scale length. A chart from Barton (1982) presented in Fig. 7.2 is easier for evaluating JRC_n according to the amplitude of asperities and the length of joint profile which is studied in the field.

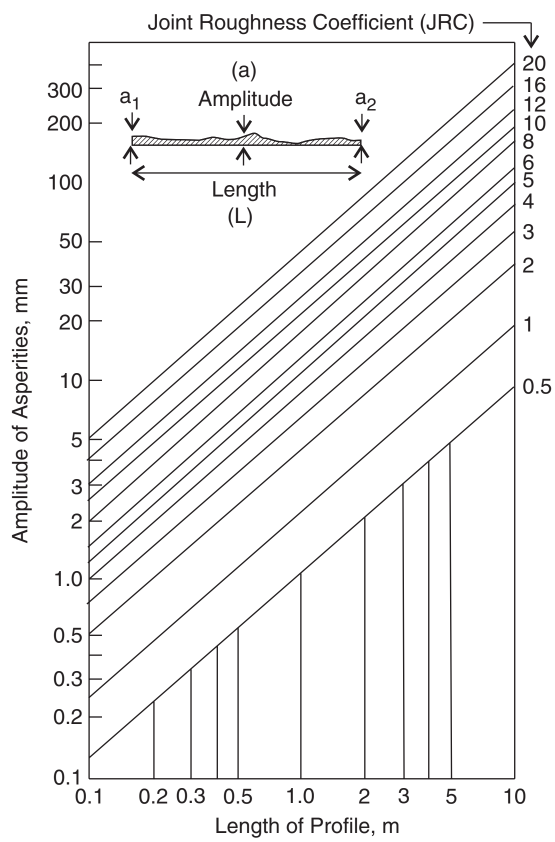
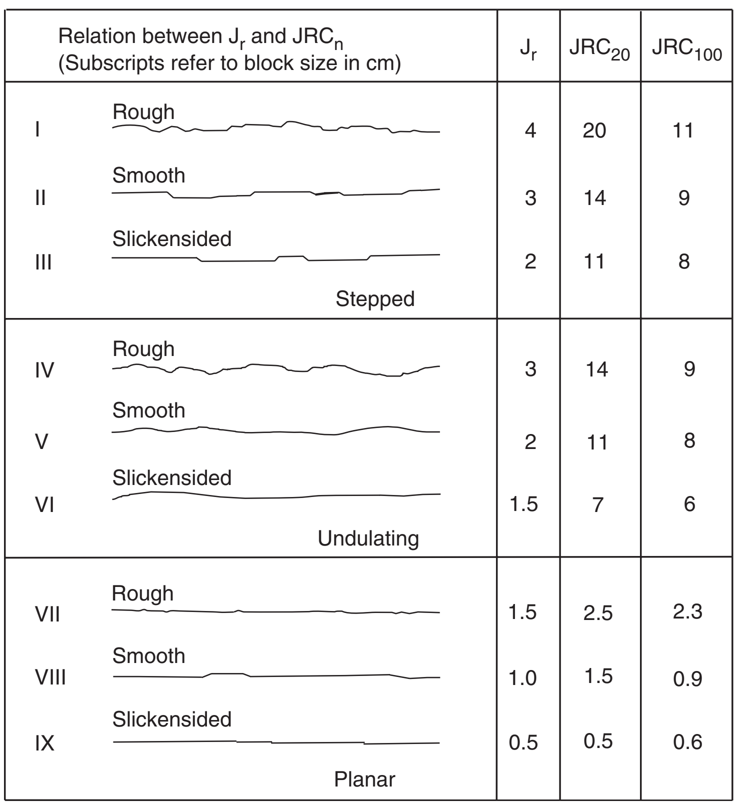


Fig. 7.2 Assessment of JRC from amplitude of asperities and length of joint profile (Barton, 1982).

7.2.1 Relationship between J_r and JRC roughness descriptions

The description of roughness given in the Q-system by the parameter J_r, and JRC are related. Fig. 7.3 has been prepared by Barton (1993) for the benefit of users of these rock mass descriptions. The ISRM (1978) suggested methods for visual description of joint roughness profiles which have been combined with profiles given by Barton et al. (1980) and with equation (7.1), to produce some examples of the quantitative description of joint roughness that these parameters provide. Increasing experience leads to better visual assessment of JRC on the basis of Fig. 7.3.

The roughness profiles shown in Fig. 7.3 are assumed to be at least 1 m in length. The column of J_r values could be used in Q-system, while the JRC values for 20 and 100 cm block size could be used to generate appropriate shear stress displacement and dilation – displacement curves.



Relation between J_r and JRC_n (Subscripts refer to block size in cm)		J_r	JRC_{20}	JRC_{100}
I	Rough	4	20	11
II	Smooth	3	14	9
III	Slickensided	2	11	8
	Stepped			
IV	Rough	3	14	9
V	Smooth	2	11	8
VI	Slickensided	1.5	7	6
	Undulating			
VII	Rough	1.5	2.5	2.3
VIII	Smooth	1.0	1.5	0.9
IX	Slickensided	0.5	0.5	0.6
	Planar			

Fig. 7.3 Suggested methods for the quantitative description of different classes of joints using J_r and JRC_n. Subscript n refers to block size in centimeter.

7.3 JOINT WALL COMPRESSIVE STRENGTH (JCS)

The joint wall compressive strength (JCS) of a joint or discontinuity is an important component of its shear strength, especially in case of undisplaced and interlocked discontinuities, e.g., unfilled joints (Barton & Choubey, 1977). As in the case of JRC, the wall strength JCS decreases as aperture or filling thickness or the degree of any previous shear displacement increases. JCS, therefore, need not be evaluated for thickly (>10 mm) filled joints.

In the field, JCS is measured by performing Schmidt hammer (L-type) tests on the two most prominent joint surfaces where it is smooth and averaging the highest 10 rebound values. JCS_0, the small scale value of wall strength relative to a nominal joint length (L_0) of 100 mm, can be obtained from the Schmidt hammer rebound value (r) as follows or by using Fig. 7.4.

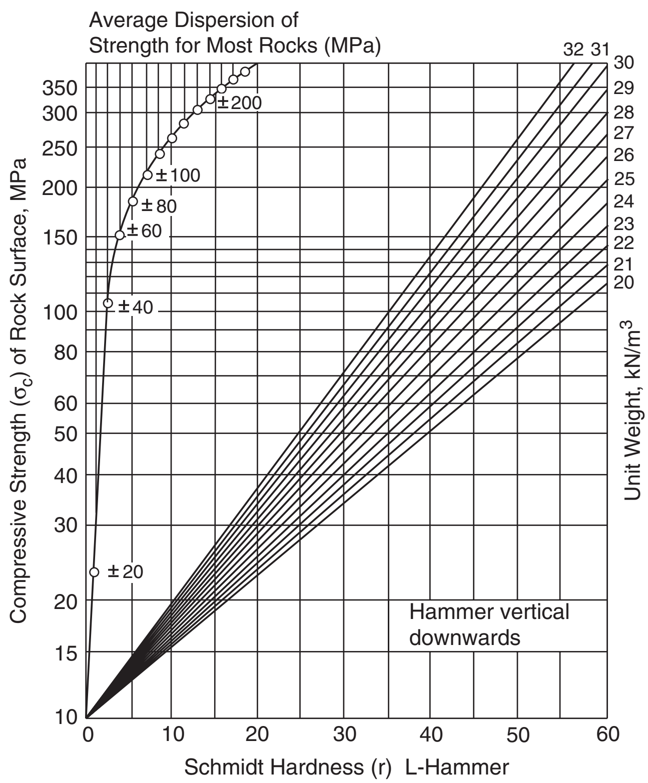


Fig. 7.4 Correlation chart for compressive strength with rock density and Schmidt hammer rebound number on smooth surfaces (Miller, 1965).

Table 7.1 Corrections for the orientation of Schmidt hammer (Barton & Choubey, 1977).

Rebound	Downward		Upward		Horizontal
r	$\alpha = -90°$	$\alpha = -45°$	$\alpha = +90°$	$\alpha = +45°$	$\alpha = 0°$
10	0	−0.8	–	–	−3.2
20	0	−0.9	−8.8	−6.9	−3.4
30	0	−0.8	−7.8	−6.2	−3.1
40	0	−0.7	−6.6	−5.3	−2.7
50	0	−0.6	−5.3	−4.3	−2.2
60	0	−0.4	−4.0	−3.3	−1.7

$$\mathrm{JCS}_0 = 10^{(0.00088\,r\,\gamma\,+\,1.01)}\ \mathrm{MPa} \tag{7.2}$$

where

r = rebound number on smooth weathered joint and
γ = dry unit weight of rocks (kN/m^3).

In case Schmidt hammer is not used vertically downward, the rebound values need correction as given in Table 7.1.

The joint wall compressive strength may be equal to the uniaxial compressive strength (UCS) of the rock material for unweathered joints; otherwise it should be estimated indirectly from the Schmidt hammer index test. It is experienced that Schmidt hammer is found to give entirely wrong results on rough joints. Therefore, it is advisable not to use Schmidt hammer rebound for JCS in case of rough joints. Lump tests on saturated small lumps of asperities will give better UCS or JCS$_0$. Quartz-coated joints in weak rocks may give high Schmidt hammer rebound number which is a surface property (Bhasin, 2004). Calcite and gypsum infillings may dissolve very slowly in hydroprojects. Coatings of chlorite, talc and graphite reduce strength on wetting. Clay minerals may be washed out by seepage.

For larger blocks or joint lengths (L_n), the value of JCS reduces to JCS$_n$, where the two are related by the following empirical equation:

$$\mathrm{JCS}_n = \mathrm{JCS}_0(L_n/L_0)^{-0.03\,JRC_0}\ \mathrm{MPa} \tag{7.3}$$

where JCS$_n$ is the joint wall compressive strength at a larger scale.

7.4 JOINT MATCHING COEFFICIENT (JMC)

Zhao (1997) suggested a new parameter, joint matching coefficient (JMC), in addition to JRC and JCS for obtaining shear strength of joints. JMC may be obtained by observing the approximate percentage area in contact between the upper and the lower walls of a joint. Thus, JMC has a value between 0 and 1.0. A JMC value of 1.0 represents a perfectly matched joint, i.e., with 100 percent surface contact. On the other hand, a JMC value close to 0 (zero) indicates a totally mismatched joint with no or minimum surface contact.

7.5 RESIDUAL ANGLE OF FRICTION

The effective basic or residual friction angle ϕ_r of a joint is an important component of its total shear strength, whether the joint is rock-to-rock interlocked or clay filled. The importance of ϕ_r increases as the clay coating or filling thickness increases, of course upto a certain critical limit.

An experienced field observer may make a preliminary estimate of ϕ_r. The quartz-rich rocks and many igneous rocks have ϕ_r between 28 and 32°, whereas, mica-rich rock masses and rocks having considerable effect of weathering have somewhat lower values of ϕ_r than mentioned above.

In the Barton–Bandis joint model, it is proposed to add an angle of primary roughness for obtaining the field value of effective peak friction angle for a natural joint (ϕ_j) without fillings,

$$\phi_j = \phi_r + i + \text{JRC} \log_{10} (\text{JCS}/\sigma) < 70°; \text{ for } \sigma/\text{JCS} < 0.3 \tag{7.4}$$

where JRC accounts for secondary roughness in laboratory tests, 'i' represents angle of primary roughness (undulations) of natural joint surface and is generally $\leq 6°$ and σ is the effective normal stress across joint.

It can be noted here that the value of ϕ_r is important, as roughness (JRC) and wall strength (JCS) reduces through weathering. Residual frictional angle ϕ_r can also be estimated by the equation:

$$\phi_r = (\phi_b - 20°) + 20\,(r/R) \tag{7.5}$$

where ϕ_b is the basic frictional angle obtained by sliding or tilt tests on dry, planar (but not polished) or cored surface of the rock (Barton & Choubey, 1977). R is the Schmidt rebound on fresh, dry–unweathered–smooth surfaces of the rock and r is the rebound number on the smooth natural, perhaps weathered and water-saturated joints ($J_w = 1.0$).

According to Jaeger and Cook (1969) enhancement in the dynamic angle of sliding friction ϕ_r of smooth rock joints can be about 2° only.

7.6 SHEAR STRENGTH OF JOINTS

Barton and Choubey (1977) have proposed the following non-linear correlation for shear strength of natural joints which is found surprisingly accurate.

$$\tau = \sigma \cdot \tan\,[\phi_r + \text{JRC}_n \log_{10} (\text{JCS}_n/\sigma)] \tag{7.6}$$

where τ is the shear strength of joints, JRC_n can be obtained easily from Fig. 7.3, JCS_n from equation (7.3) and rest of the parameters are defined above. Further, under very high normal stress levels ($\sigma >> q_c$ or JCS_n) the JCS_n value increases to the triaxial compressive strength ($\sigma_1 - \sigma_3$) of the rock material in equation (7.6) (Barton, 1976). It can be noted that at high normal pressure ($\sigma = JCS_n$), no dilation will take place as all the asperities will be sheared.

The effect of mismatching of joint surface on its shear strength has been proposed by Zhao (1997) in his JRC–JCS shear strength model as,

$$\tau = c_j + \sigma \cdot \tan\,[\phi_r + \mathrm{JMC} \cdot \mathrm{JRC_n} \log_{10} (\mathrm{JCS_n}/\sigma)] \tag{7.7}$$

and dilatation (Δ) across joint is as follows,

$$\Delta \cong \frac{1}{2} \cdot \mathrm{JMC} \cdot \mathrm{JRC_n} \cdot \log_{10} \left(\frac{\mathrm{JCS_n}}{\sigma} \right)$$

$$\therefore \Delta \cong \left(\frac{\phi_j - \phi_r}{2} \right) \tag{7.8}$$

The minimum value of JMC in the above equation should be taken as 0.3. The cohesion along discontinuity is c_j. Field experience shows that natural joints are not continuous as assumed in theory and laboratory tests. There are rock bridges in between them. The shear strength of these rock bridges add to the cohesion of overall rock joint (0−0.1 MPa). The real discontinuous joint should be simulated in the theory or computer program.

In the case of highly jointed rock masses, failure takes place along the shear band (kink band) and not along the critical discontinuity, due to rotation of rock blocks. The apparent angle of friction may be significantly lower in case of slender blocks. Laboratory tests on models with three continuous joint sets show some cohesion c_j (Singh, 1997). More attention should be given to strength of discontinuity in the jointed rock masses.

For joints filled with gouge or clay-coated joints, the following correlation of shear strength is used for low effective normal stresses (Barton & Bandis, 1990);

$$\tau = \sigma \cdot (J_r/J_a) \tag{7.9}$$

Sinha and Singh (2000) have proposed an empirical criterion for shear strength of filled joints. The angle of internal friction is correlated to the plasticity index (PI) of normally consolidated clays (Lamb & Whitman, 1979). The same may be adopted for thick and normally consolidated clayey gouge in the rock joints as follows:

$$\sin \phi_j = 0.81 - 0.23 \log_{10} \mathrm{PI} \tag{7.10}$$

Choubey (1998) suggested that the peak strength parameters should be used in the case of designing rock bolt system and retaining walls, where control measures do not permit large deformations along joints. For long-term stability of unsupported rock and soil slopes, residual strength parameters of rock joint and soil should be chosen in the analyses, respectively; as large displacement may reduce the shear strength of rock joint to its residual strength eventually.

It should be realized that there is a wide statistical variation in the shear strength parameters as found from direct shear tests. Generally, average parameters are evaluated from median values rejecting too high and too low values for the purpose of designs.

Barton et al. (1985) have related the hydraulic aperture (e) to the measured (geometric) aperture (t) of rock joints as follows when shear displacement is less than 0.75 × peak slip,

$$e = \frac{\mathrm{JRC}^{2.5}}{(t/e)^2} \tag{7.11}$$

where t and e are measured in μm (micrometer). The permeability of rock mass may then be estimated approximately, assuming laminar flow of water through two parallel plates with spacing (e) for each joint.

7.7 DYNAMIC SHEAR STRENGTH OF ROUGH ROCK JOINTS

Jain (2000) performed large number of dynamic shear tests on dry rock joints at Nanyang Technological University (NTU), Singapore. He observed that significant dynamic normal stress (σ_{dyn}) is developed across the rough rock joints. Hence there is a high rise in the dynamic shear strength. Thus, the effective normal stress (σ') in equation (7.8) can be as follows:

$$\begin{aligned} \sigma'_{\mathrm{dyn}} &= \sigma_{\mathrm{static}} - u_{\mathrm{static}} + \sigma_{\mathrm{dyn}} - u_{\mathrm{dyn}} \\ &\geq \sigma'_{\mathrm{static}} \end{aligned} \tag{7.12}$$

It is also imagined that negative dynamic pore water pressure (u_{dyn}) will develop in the water-charged joints due to dilatancy. This phenomenon is likely to be similar to undrained shearing of dilatant and dense sand/over-consolidated clay. Further research is needed to develop correlations for σ_{dyn} and u_{dyn} from dynamic shear tests on rock joints. There is likely to be a significant increase in the dynamic shear strength of rock joints due to shearing of more asperities.

7.8 THEORY OF SHEAR STRENGTH AT VERY HIGH CONFINING STRESS

Barton (1976) suggested a theory of the critical state of rock materials at very high confining stresses. It appears that the Mohr's envelopes representing the peak shear strength of rock materials (intact) eventually reach a point of saturation (zero gradient on crossing a certain critical state line).

Fig. 7.5 integrates all the three ideas on shear strength of discontinuities. The effective sliding angle of friction is about $\phi_r + i$ at a low effective normal stresses, where i = angle of asperities of rough joint. The shear strength (τ) cannot exceed shear strength

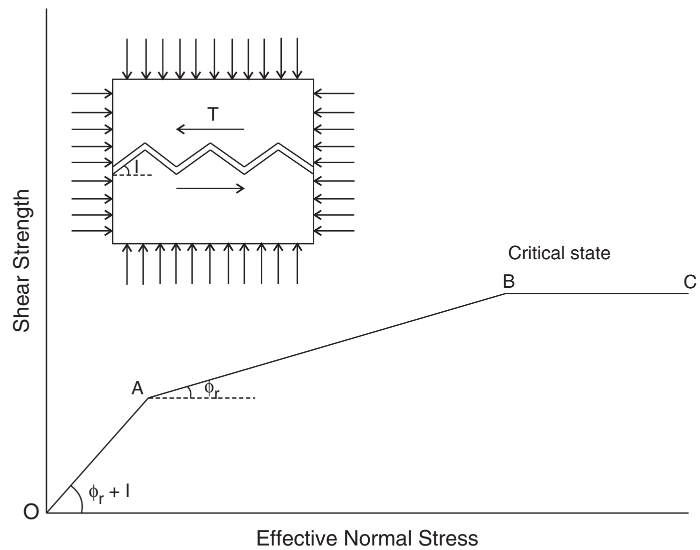


Fig. 7.5 Shear strength of discontinuities at very high confining pressure (OA – sliding above asperities, AB – shearing of rock asperities and BC – critical state of rock material at very high confining stress).

of the asperities ($= c + \sigma \tan \phi_r$), where ϕ_r = effective angle of internal friction of the ruptured asperities of rock material. In fact the non-linear equation, equation (7.7) (with JCS = triaxial strength of rock) is closer to the experimental data than the bilinear theoretical relationship.

Further there is a critical limit of shear strength of rock joint which cannot be higher than the shear strength of weaker rock material at very high confining stress. Fig. 7.5 illustrates this idea by τ = constant saturation (critical state) line. It follows that the (sliding) angle of friction is nearly zero at very high confining stresses which exist at great depth in the earth plates along interplate boundaries. It is interesting to note that the sliding angle of friction at great depth (>40 km) is back-analyzed to be as low as 5° in the Tibet Himalayan plate (Shankar et al., 2002). This analysis makes a sense. Re-crystallization of soft minerals is likely to occur creating smooth surface. The sliding angle of friction between earth plate and underlying molten rock is assumed to be zero, as the coefficient of friction between a fluid and any solid surface is governed by the minimum shear strength of the material. Thus, it is the need of the time to perform shear tests at both very high confining stresses and high temperatures to find a generalized correlation between τ and σ along mega-discontinuities. Chapter 29 summarizes further experiences on critical state.

It is interesting to note that lesser the frictional resistance along the intercontinental and colliding plate boundaries, lesser will be the locked-up elastic strain energy in the large earth plates and so lesser are the chances of great earthquakes in that area. In fact a highest earthquake of only about 7.0 M on Richter scale has occurred in Tibetan plateau.

7.9 NORMAL AND SHEAR STIFFNESS OF ROCK JOINTS

The values of static normal and shear stiffness are used in finite element method and distinct element method of analysis of rock structures. Singh and Goel (2002) list their suggested values on the basis of experiences of back analysis of uniaxial jacking tests in USA and India.

Barton and Bandis (1990) have also found correlation for shear stiffness. The shear stiffness of joint is defined as the ratio between shear strength τ in equation (7.7) above and the peak slip. The latter may be taken equal to (S/500) (JRC/S)$^{0.33}$, where S is equal to the length of a joint or simply the spacing of joints. Laboratory tests also indicate that the peak slip is nearly constant for a given joint, irrespective of the normal stress. The normal stiffness of a joint may be 10 to 30 times its shear stiffness. This is the reason why the shear modulus of jointed rock masses is considered to be very low as compared to that for an isotropic elastic medium (Singh, 1973). Of course the dynamic stiffness is likely to be significantly more than their static values. The P-wave velocity and so the dynamic normal stiffness may increase after saturation and net decrease.

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8
Strength enhancement of rock mass in tunnels

"The behaviour of macroscopic systems is generally described by non-linear laws. (The non-linear laws may explain irreversible phenomena like instabilities, dualism, unevolving socities, cycles of growth and decay of societies. The linear laws are only linear approximation of the non-linear laws at a point in time and space.)"

Ilya Prigogine, Nobel Laureate

8.1 CAUSES OF STRENGTH ENHANCEMENT

Instrumentation and monitoring of underground openings in complex geological environment is the key to success. Careful back-analysis of the data observed in the initial stages of excavation provides valuable knowledge of the constants of the selected constitutive model, which may then be used in the forward analysis to predict performance of the support system. Experience of back-analysis of data from many project sites has shown that there is a significant enhancement of rock mass strength around tunnels. Rock masses surrounding the tunnel perform much better than theoretical expectations, except near thick and plastic shear zones, faults, thrusts, intra-thrust zones and in water-charged rock masses.

Rock masses have shown constrained dilatancy in tunnels. Failure, therefore, does not occur along rough joints due to interlocking. Further, tightly packed rock blocks are not free to rotate unlike soil grains. The strength of a rock mass in tunnels thus tends to be equal to the strength of a rock material (Pande, 1997).

It has been seen that empirical criteria of rock mass failure are trusted more than the theoretical criteria. Sheorey (1997) evaluated them critically. However, designers like the linear approximtion for practical applications.

Tunnelling in Weak Rocks
B. Singh and R. K. Goel

8.2 EFFECT OF INTERMEDIATE PRINCIPAL STRESS ON TANGENTIAL STRESS AT FAILURE IN TUNNELS

The intermediate principal stress (σ_2) along the tunnel axis may be of the order of half the tangential stress (σ_1) in some deep tunnels (Fig. 8.1). According to Wang and Kemeny (1995), σ_2 has a strong effect on σ_1 at failure even if σ_3 is equal to zero. Their polyaxial laboratory tests on hollow cylinders led to the following strength criterion:

$$\frac{\sigma_1}{q_c} = 1 + A\left[e^{\sigma_3/\sigma_2}\right] \cdot \left[\frac{\sigma_2}{q_c}\right]^{1-f \cdot e^{(\sigma_3/\sigma_2)}} \tag{8.1}$$

$$\therefore \sigma_1 \approx q_c + (A + f) \cdot (\sigma_3 + \sigma_2) \quad \text{for} \quad \sigma_3 \ll \sigma_2$$

where

f = material constant (0.10–0.20),
A = material constant (0.75–2.00) and
q_c = average uniaxial compressive strength (UCS) of rock material ($\sigma_2 = \sigma_3 = 0$) for various orientations of planes of weakness.

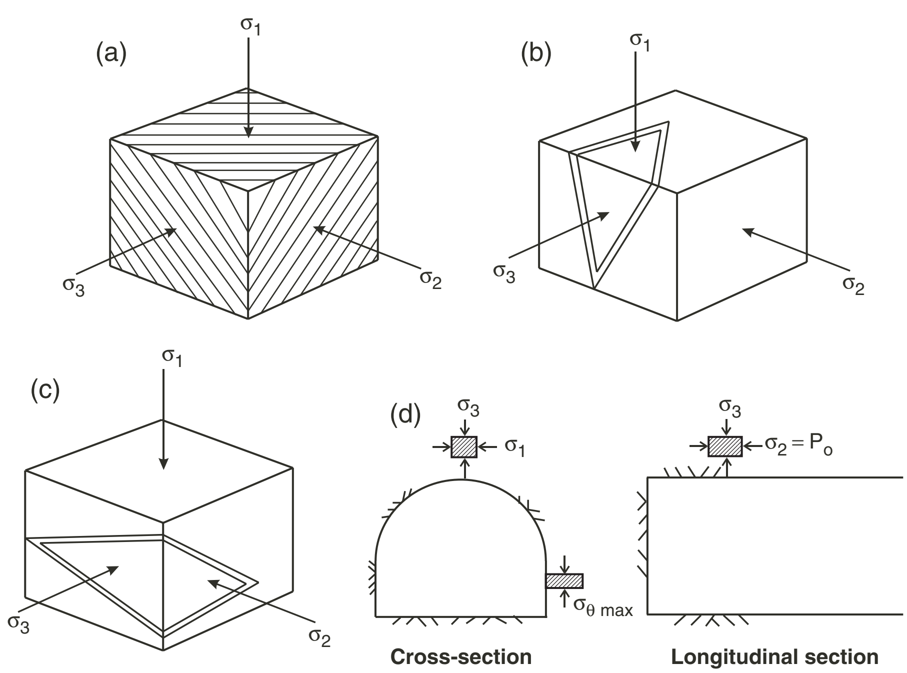

Fig. 8.1 (a) Anisotropic rock material with one joint set (slate, schist, etc.), (b) mode of failure in rock mass with 2 joint sets, (c) $p_{\text{horizontal}} \gg p_{\text{vertical}}$ and (d) direction of σ_1, σ_2 and σ_3 in the tunnel.

In the case of unsupported tunnels, $\sigma_3 = 0$ on its periphery. So, equation (8.1) simplifies to,

$$\frac{\sigma_1}{q_c} = 1 + A\left[\frac{\sigma_2}{q_c}\right]^{(1-f)} \tag{8.2}$$

It may be inferred from equation (8.2) that σ_2 will enhance σ_1 at failure by 75–200 percent when $\sigma_2 \approx q_c$. In fact, strength enhancement may be much more as propagation of fracture will be behind the excavated face (Bazant et al., 1993). Murrell (1963) suggested 100 percent increase in σ_1 at failure when $\sigma_2 = 0.5$, σ_1 and $\sigma_3 = 0$. Thus, the effective confining pressure appears to be an average of σ_2 and σ_3 and not just equal to σ_3 in the anisotropic rocks and weak rock masses.

Hoek (1994) suggested the following modified criterion for estimating the strength of jointed rock masses at high confining stresses (around $\sigma_3 > 0.10\ q_c$),

$$\sigma_1 = \sigma_3 + q_c\left[m\left(\frac{\sigma_3}{q_c}\right) + s\right]^n \tag{8.3}$$

where

σ_1 and σ_3 = maximum and minimum effective principal stresses, respectively,
m = Hoek–Brown rock mass constant,
s and n = rock mass constants,
s = 1 for rock material,
n = 0.5
= $0.65 - (\mathrm{GSI}/200) \leq 0.60$ for GSI < 25,
q_c = UCS of the intact rock core of standard NX size,
GSI = geological strength index $\approx$ RMR − 5 for RMR > 23,
(m/m_r) = $s^{1/3}$ for GSI > 25 and (8.4)
m_r = Hoek–Brown rock material constant.

Hoek and Brown (1980) criterion (equation (8.3)) is applicable to rock slopes and open cast mines with weathered and saturated rock masses. They have suggested values of m and s. Hoek and Brown criterion may be improved as a polyaxial criterion after replacing σ_3 (within bracket in equation (8.3) by effective confining pressure $(\sigma_2 + \sigma_3)/2$ as mentioned above for weak and jointed rock masses. It can be noted that parameters m_r and q_c should be calculated from the upper bound Mohr's envelope of triaxial test data on rock cores in the case of anisotropic rock materials (Hoek, 1998).

According to Hoek (2000), rock mass strength (q_{cmass}) is as follows:

$$q_{cmass} = (0.0034\, m_r^{0.8})q_c\,\{1.029 + 0.025\exp(-0.1\, m_r)\}^{\mathrm{GSI}} \tag{8.5}$$

Further, the limitations should be kept in mind that most of the strength criteria are not valid at low confining stresses and tensile stresses, as modes of failure are different.

Hoek's criteron is applicable for high confining stresses only where a single mode of failure by faulting takes place. Hence, the quest for a better model to represent jointed rock masses.

8.3 UNIAXIAL COMPRESSIVE STRENGTH OF ROCK MASS

Equation (8.3) defines that UCS of a rock mass is given by

$$q_{\text{cmass}} = q_{\text{c}} s^n \tag{8.6}$$

Past experience shows that equation (8.6) underestimates mobilized rock mass strength in tunnels. For making use of equation (8.3) in tunnels, value of constant *s* be obtained from equations (8.6) and (8.9) as follows.

$$s = \left[\left(7\gamma \text{Q}^{1/3}\right)/q_{\text{c}}\right]^{1/n} \tag{8.7}$$

Ramamurthy (1993) and his co-workers (Roy, 1993; Singh & Rao, 2005) have conducted extensive triaxial tests on dry models of jointed rock mass using plaster of Paris ($q_{\text{c}} = 9.46\,\text{MPa}$). They varied in joint frequency, inclination of joints and thickness of joint fillings, etc. and simulated a wide variety of rock mass conditions. Their extensive test data suggests the following approximate correlation for all the rock masses,

$$q_{\text{cmass}}/q_{\text{c}} = [E_{\text{mass}}/E_{\text{r}}]^{0.7} \tag{8.8}$$

where,

q_{cmass} = UCS of model of jointed rock mass in σ_1 direction,
q_{c} = UCS of model material (plaster of Paris),
= UCS of in situ block of rock material after size correction,
E_{mass} = average modulus of deformation of jointed rock mass model ($\sigma_3 = 0$) in σ_1 direction and
E_{r} = average modulus of deformation of model material ($\sigma_3 = 0$).

The power in equation (8.8) varies from 0.5 to 1.0. Griffith's theory of failure suggests that the power is 0.5, whereas Sakurai (1994) is of the opinion that the above power is about 1.0 for jointed rock masses. Further research at Indian Institute of Technology (IIT), Delhi, suggests that power in equation (8.8) is in the range of 0.56 and 0.72 (Singh & Rao, 2005). As such it appears that the power of 0.7 in equation (8.8) is realistic. Equation (8.8) may be used reliably to estimate strength of a rock mass (q_{cmass}) from the values of E_{mass} or E_{d} obtained from uniaxial jacking tests both within openings and slopes.

Considerable strength enhancement of the rock mass in tunnels has been observed by Singh et al. (1997). Therefore, on the basis of analysis of data collected from 60 tunnels, they recommended that the mobilized crushing strength of the rock mass is

$$q_{cmass} = 7\gamma Q^{1/3}\ \text{MPa} \quad (\text{for } Q < 10,\ 100 > q_c > 2\,\text{MPa},\ J_w = 1 \text{ and } J_r/J_a < 0.5) \tag{8.9}$$

$$q_{cmass} = [(5.5\gamma N^{1/3})/B^{0.1}]\,\text{MPa} \quad (\text{as per equation (13.1)}) \tag{8.10}$$

where

γ = unit weight of rock mass (gm/cc),
N = rock mass number, i.e., stress-free Barton's Q soon after the underground excavation,
Q = rock mass quality soon after the underground excavation and
B = tunnel span or diameter in meters.

Kalamaras and Bieniawski (1995) suggested the following relationship between q_{cmass} and RMR,

$$q_{cmass} = q_c \cdot \exp\left[\frac{\text{RMR} - 100}{24}\right] \tag{8.11}$$

Barton (2002) has modified equation (8.9) on conservative side as follows for calculating Q_{TBM} for tunnel boring machines (according to equation (16.1)),

$$\sigma_{cm} = 5\gamma(Q \cdot q_c/100)^{1/3}\ \text{MPa} \tag{8.12}$$

where

q_c = $I_s/25$ for anisotropic rocks (schists, slate, etc.) and
I_s = standard point load strength index of rock cores (corrected for size effect for NX size cores).

Barton (2005) has clarified that equation (8.12) should be used only for Q_{TBM}.

On the basis of block shear tests, Singh et al. (1997) have proposed the following correlation for estimating the UCS of the saturated rock mass for use in rock slopes in hilly areas.

$$q_{cmass} = 0.38\gamma \cdot Q^{1/3}\ \text{MPa} \tag{8.13}$$

Equation (8.13) suggests that the UCS would be low on slopes. This is probably because joint orientation becomes a very important factor in the case of slopes due to unconstrained dilatancy and low intermediate principal stress unlike tunnels. Further, failure takes place along joints near slopes. In slopes of deep open cast mines, joints may be tight and of smaller length. The UCS of such a rock mass may be much higher and may be found from Hoek's criterion (equation (8.5)) for analysis of the deep seated rotational slides.

The equations (8.8) and (8.9) are intended only for a 2D stress analysis of underground openings. The strength criterion for 3D analysis is presented below.

8.4 REASON FOR STRENGTH ENHANCEMENT IN TUNNELS AND A NEW FAILURE THEORY

Consider a cube of rock mass with two or more joint sets as shown in Fig. 8.1. If high intermediate principal stress is applied on the two opposite faces of the cube, then the chances of wedge failure are more than the chances of planar failure as found in the triaxial tests. The shear stress along the line of intersection of joint planes will be proportional to $\sigma_1 - \sigma_3$ because σ_3 will try to reduce shear stress. The normal stress on both the joint planes will be proportional to $(\sigma_2 + \sigma_3)/2$. Hence the criterion for peak failure at low confining stresses can be as follows ($\sigma_3 < 2q_c/3$ and $\sigma_2 < 2q_c/3$):

$$\sigma_1 - \sigma_3 = q_{\text{cmass}} + A[(\sigma_2 + \sigma_3)/2], \tag{8.14}$$

$$q_{\text{cmass}} = q_c \left[\frac{E_d}{E_r}\right]^{0.70} \cdot \left[\frac{d}{S_{\text{rock}}}\right]^{0.20}, \tag{8.15}$$

$$\Delta = \frac{\phi_p - \phi_r}{2} \tag{8.16}$$

where

q_{cmass}	= average UCS of rock mass for various orientation of principal stresses,
$\sigma_1, \sigma_2, \sigma_3$	= final compressive and effective principal stresses which are equal to in situ stress plus induced stress minus seepage pressure,
A	= average constants for various orientation of principal stress (value of A varies from 0.6 to 6.0),
	$= 2 \cdot \sin \phi_p/(1 - \sin \phi_p)$,
ϕ_p	= peak angle if internal friction of rock mass,
	$\cong \tan^{-1} [(J_r/J_a) + 0.1]$ at a low confining stress,
	< peak angle of internal friction of rock material,
	= 14–57°
S_{rock}	= average spacing of joints,
q_c	= average UCS of rock material for core of diameter d (for schistose rock also),
Δ	= peak angle of dilatation of rock mass at failure,
ϕ_r	= residual angle of internal friction of rock mass $= \phi_p - 10° \geq 14°$,
E_d	= modulus of deformation of rock mass ($\sigma_3 = 0$) and
E_r	= modulus of elasticity of the rock material ($\sigma_3 = 0$).

The peak angle of dilatation is approximately equal to $(\phi_p - \phi_r)/2$ for rock joints (Barton & Brandis, 1990) at low σ_3. This correlation (equation (7.8)) may be assumed for jointed rock masses also. The proposed strength criterion reduces to Mohr criterion for triaxial conditions.

The significant rock strength enhancement in underground openings is due to σ_2 or in situ stress along tunnels and caverns which pre-stresses rock wedges and prevents their failure both in the roof and the walls. However, σ_3 is released due to stress-free excavation boundaries (Fig. 8.1d). In the rock slopes σ_2 and σ_3 are nearly equal and negligible. Therefore, there is an insignificant or no enhancement of the strength. As such, block shear tests on a rock mass gives realistic results for rock slopes and dam abutments only; because $\sigma_2 = 0$ in this test. Thus, equation (8.14) may give a general criterion of jointed rock masses for underground openings, rock slopes and foundations.

Another cause of strength enhancement is higher UCS of rock mass (q_{cmass}) due to higher E_d because of constrained dilatancy and restrained fracture propagation near excavation face only in the underground structures. In rock slopes, E_d is found to be less due to complete stress release and low confining pressure on account of σ_2 and σ_3, and long length of weathered filled-up joints. So, q_{cmass} will also be low near rock slopes for the same Q-value (equation (8.13)).

Through careful back-analysis, both the model and its constants should be deduced. Thus, A, E_d and q_{cmass} should be estimated from the feedback of instrumentation data at the beginning of construction stage. With these values, forward analysis should be attempted carefully as mentioned earlier. At present, a non-linear back-analysis may be difficult and it does not give unique (or most probable) parameters.

The proposed strength criterion is different from Mohr's strength theory which works well for soils and isotropic materials. There is a basic difference in the structure of soil and rock masses. Soils generally have no pre-existing planes of weaknesses and so planar failure can occur on a typical plane with dip direction towards σ_3. However, rocks have pre-existing planes of weaknesses like joints and bedding planes, etc. As such, failure occurs mostly along these planes of weaknesses. In the triaxial tests on rock masses, planar failure takes place along the weakest joint plane. In polyaxial stress field, a wedge type of failure may be the dominant mode of failure, if $\sigma_2 \gg \sigma_3$. Therefore, Mohr's theory needs to be modified for anisotropic and jointed rock masses.

The new strength criterion is proved by extensive polyaxial tests on anisotropic tuff (Wang & Kemeny, 1995). It is interesting to note that the constant A is the same for biaxial, triaxial and polyaxial tests (Singh et al., 1998). Further, the effective in situ stresses (upper bound) on ground level in mountainous areas appear to follow equation (8.14) ($q_{cmass} = 3$ MPa, $A = 2.5$) which indicates a state of failure of earth crust near the water-charged ground due to the tectonic stresses.

The output of computer program SQUEEZE shows that the predicted support pressures are of the order of those observed in 10 tunnels in the squeezing ground condition in the Himalaya, India. There is a rather good cross-check between the theory of squeezing and the observations (reported by Singh et al., 1992) except in a few cases. Thus, the equations (8.14)–(8.15) assumed in the theory of squeezing are again justified partially (Singh & Goel, 2002).

In the NJPC project, tunnel excavated under high rock cover of 1400 m through massive to competent gneiss and schist gneiss, the theory predicted rock burst condition

($J_r/J_a = 3/4$, i.e., >0.5). According to site geologists, Pundhir, Acharya and Chadha (2000), initially cracking noise was heard which was followed by the spalling of 5–25 cm thick rock columns/slabs and rock falls. This is a mild rock burst condition. Another cause of rock burst is the Class II behavior of gneiss according to the tests at IIT, Delhi, India. Although according to Mohr's theory, most severe rock burst or squeezing conditions were predicted under rock cover more than 300 m ($q_c = 27$ MPa and $q_{cmass} = 15.7$ MPa). Actually mild rock burst conditions were met where overburden is more than 1000 m. However, polyaxial theory equation (8.14) suggested mild rock burst condition above overburden of 800 m. Thus, polyaxial theory of strength is validated further by SQUEEZE program (Singh & Goel, 2002). Recently, Rao, Tiwari and Singh (2003) developed the polyaxial testing system. Their results were re-plotted and parameter A was found to increase slightly from 3.8 to 4.2 for dips of joints from 0 to 60°, though q_{cmass} changed drastically.

The suggested hypothesis appears applicable approximately for the rock masses with three or more joint sets. Chapter 29 presents extension of equation (8.14) to a parabolic criterion considering the critical state rock mechanics.

8.4.1 Poor rock masses

Squeezing is found to occur in tunnels in the nearly dry weak rocks where overburden H is more than $350\text{Q}^{1/3}$ m. The tangential stress at failure may be about $2\gamma H$ assuming hydrostatic in situ stresses. Thus, mobilized compressive strength is $2\gamma 350\text{Q}^{1/3} = 700\gamma\text{Q}^{1/3}$ T/m^2. In other words (Singh & Goel, 1999),

$$q_{cmass} = 0.70\ \gamma\text{Q}^{1/3}\ \text{MPa} \quad \text{for} \quad Q < 10 \text{ and } J_w = 1.0 \tag{8.17}$$

where

γ = unit weight of rock mass in kN/m^3 (22–29) and
Q = rock mass quality soon after the underground excavation.

Singh proposed originally equation (8.17) in a lecture at Workshop of Norwegian Method of Tunnelling, New Delhi, India, in 1993 and reported it later after due confirmation (Singh et al., 1997). Since the criterion for squeezing is found to be independent of UCS ($q_c < 50$ MPa) surprisingly, so, no correction for UCS (q_c) is needed in their opinion for the weak rocks.

Many investigators have agreed with the above correlation (Barla, 1995; Barton, 1995; Grimstad & Bhasin, 1996; Choubey, 1998; Aydan et al., 2000 and others). It may be argued that q_{cmass} should be same for given RQD, J_n, J_r, J_a values irrespective of overburden depth and water pressure in joints. In fact high overburden and water pressure can cause damage to the rock mass in long-term due to induced fractures, opening of fractures, softening and seepage erosion, etc. Hence, equation (8.17) is justified logically also if Q is obtained soon after excavation in the nearly dry weak rock masses.

Ten case histories of tunnels in the squeezing ground have also been analyzed by Singh and Goel (2002). In poor rocks, the peak angle of internal friction (ϕ_p) is back-analyzed and related as follows,

$$\tan \phi_p = \frac{J_r}{J_a} + 0.1 \leq 1.5 \tag{8.18}$$

The addition of 0.1 accounts for interlocking of rock blocks. It may be visualized that interlocking is more in jointed rock mass due to low void ratio than in soils. Further, Kumar (2000) has shown theoretically that the internal angle of friction of laminated rock mass is slightly higher than the sliding angle of friction of its joints.

8.4.2 Failure of inhomogeneous geological materials

In an inhomogeneous geological material, the process of failure is initiated by its weakest link (zone of loose soil and weak rock, crack, bedding plane, soft seam, etc.). Thus, natural failure surfaces are generally three-dimensional (perhaps four-dimensional) which starts from this weakest link and propagate towards a free surface (or face of excavation). As such the intermediate principal stress (σ_2) plays an important role and governs the failure and the constitutive relations of the naturally inhomogeneous geological materials (both in rock masses and soils) in the field. Since micro-inhomogeneity is rather unknown, assumption of homogeneity is popular among the engineers. Therefore, the intuition is that the effective confining stress is about $[(\sigma_2 + \sigma_3)/2]$ in naturally inhomogeneous soils and fault-gouges also.

Further, the failure in an inhomogeneous geological material is progressive, whereas a homogeneous rock fails suddenly. Hence the advantage of inhomogeneous materials which is offered by nature is that it gives an advance warning of the failure process starting slowly from the weakest zone.

8.4.3 Failure of laminated rock mass

The laminated rock mass is generally found in the roof of underground coal mines and in the bottom of open cast coal mines. The thin rock layers may buckle under high horizontal in situ stresses first and then they may rupture progressively by violent brittle failure (Table 8.1). Therefore, the assumption of shear failure along joints is not valid here. As such, the proposed hypothesis of effective confining stress $[(\sigma_2 + \sigma_3)/2]$ may not be applicable in the unreinforced and laminated rock masses. The suggested hypothesis appears applicable approximately for the rock masses with three or more joint sets.

8.5 CRITICAL STRAIN OF ROCK MASS

The basic concept of design of structures cannot be applied in the tunnels, as stresses and strains are not known reliably. Critical strain is a better measure of failure.

Table 8.1 Overall coefficient of volumetric expansion of failed rock mass (K) within broken zone (Jethwa, 1981; Goel, 1994).

S.No.	Rock type	K
1.	Phyllites	0.003
2.	Claystones/siltstones	0.01
3.	Black clays	0.01
4.	Crushed sandstones	0.004
5.	Crushed shales	0.005
6.	Metabasics (Goel, 1994)	0.006

The critical strain (ε_{mass}) is defined as a ratio between UCS (q_{cmass}) and the modulus of deformation (E_d) of rock mass (Sakurai, 1997). He found that the critical strain is nearly independent of joints, water content and temperature. Hence equation (8.15) may be rewritten to deduce ε_{mass} as follows,

$$\varepsilon_{mass} = \varepsilon_r \left[\frac{E_r}{E_d}\right]^{0.30} \left[\frac{d}{S_{rock}}\right]^{0.20} \tag{8.19}$$

$$\geq \varepsilon_\theta = u_a/a$$

where

ε_r $= q_c/E_r =$ critical strain of rock material (obtained from tests in the laboratory),
ε_θ = tangential strain around opening,
= (observed deflection of crown in downward direction/radius of tunnel),
= u_a/a (Fig. 8.2),
S_{rock} = average spacing of joints and
q_c = UCS of rock material for core of diameter, d.

The experience in Japan is that there were not many construction problems in tunnels where $\varepsilon_\theta < \varepsilon_{mass}$ or ε_r. It can be noted that critical strain appears to be somewhat size dependent according to equation (8.19).

There is a lot of difference in predictions and actual observations in the tunnels. One has to give more attention to the joints. It is easier to observe strains than stresses in the rock mass. Sakurai (1997) classified the hazard warning level into three stages in relation to the degree of stability as shown in Fig. 8.2. He observed that where strains in the roof ($\varepsilon_\theta = u_a/a$) are less than the warning level I, there were no problems in the tunnels. Whereas tunnelling problems were encountered where strains approached warning level III. Swarup, Goel and Prasad (2000) have confirmed these observations in 19 tunnels in weak rocks in the Himalaya.

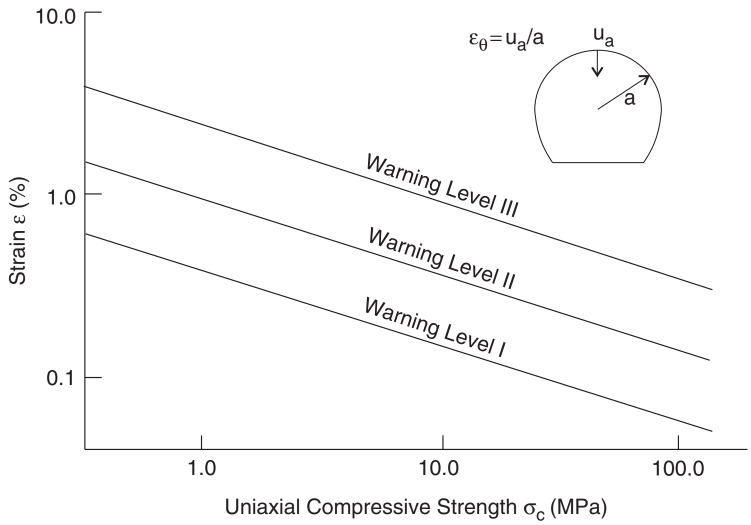

Fig. 8.2 Hazard warning levels for assessing the stability of tunnels (Sakurai, 1997).

8.6 CRITERION FOR SQUEEZING/ROCK BURST OF ROCK MASSES

Equation (8.14) suggests the following criterion for squeezing/rock burst ($\sigma_1 = \sigma_\theta$, $\sigma_3 = 0$, $\sigma_2 = P_0$ along the tunnel axis in Fig. 8.1d),

$$\sigma_\theta > q_{\mathrm{cmass}} + \frac{A \cdot P_0}{2} = q'_{\mathrm{cmass}} \tag{8.20}$$

Palmstrom (1995) has observed that $\sigma_\theta/q_{\mathrm{cmass}}$ or σ_θ/RMi may be much higher than 1, i.e., 1.5 to 3 for squeezing. Thus, his experience tends to confirm the proposed criterion (equation (8.20)) which shows that squeezing may occur when the constant A is small (<1.5). There is now need for in situ truely triaxial test data for further proof.

Experience from eleven tunnels in the Himalaya has shown that squeezing ground conditions are generally encountered where the peak angle of internal friction ϕ_p is less than 30°, J_r/J_a is less than 0.5 and overburden is higher than $350Q^{1/3}$ m in which Q is Barton's rock mass quality. The predicted support pressures using equation (8.14) are in better agreement with the observed support pressure in the roof and wall than those by Mohr's theory (Chaturvedi, 1998).

Kumar (2002) observed the behavior of 27 km long NJPC tunnel and found that the mild rock burst occurred where A is more than 2.0 and $J_r/J_a > 0.5$. In 15 sections with rock cover more than 1000 m, his studies validated equation (8.20) for predicting mild rock burst/slabbing conditions approximately, estimating q_{cmass} from equation (8.9) (Table 22.1). He also inferred from 50 tunnel sections that the ratio between tangential stress and mobilised strength (σ_θ/q'_{cmass}) is a better criterion for predicting the degree of squeezing condition than Mohr's theory (σ_θ/q_{cmass}).

8.7 TENSILE STRENGTH ACROSS DISCONTINUOUS JOINTS

The length of joints is generally less than say 5 m in tunnels in young rock masses except for bedding planes. Discontinuous joints thus have tensile strength. Mehrotra (1996) has conducted 44 shear block tests on both nearly dry and saturated rock masses. He also obtained non-linear strength envelopes for various rock conditions. These strength envelopes were extrapolated carefully in tensile stress region so that it is tangential to the Mohr's circle for uniaxial tensile strength as shown in Fig. 8.3. It was noted that the non-linear strength envelopes for both nearly dry and saturated rock masses converged to nearly the same uniaxial tensile strength across discontinuous joints (q_{tj}) within the blocks of rock masses. It is related to Barton's rock mass quality (Fig. 8.4) as follows:

$$q_{tj} = 0.029\,\gamma Q^{0.31}\ \text{MPa} \tag{8.21}$$

where γ is the unit weight of the rock mass in kN/m^3. In case of tensile stresses, the criterion of failure is as follows,

$$-\sigma_3 = q_{tj} \tag{8.22}$$

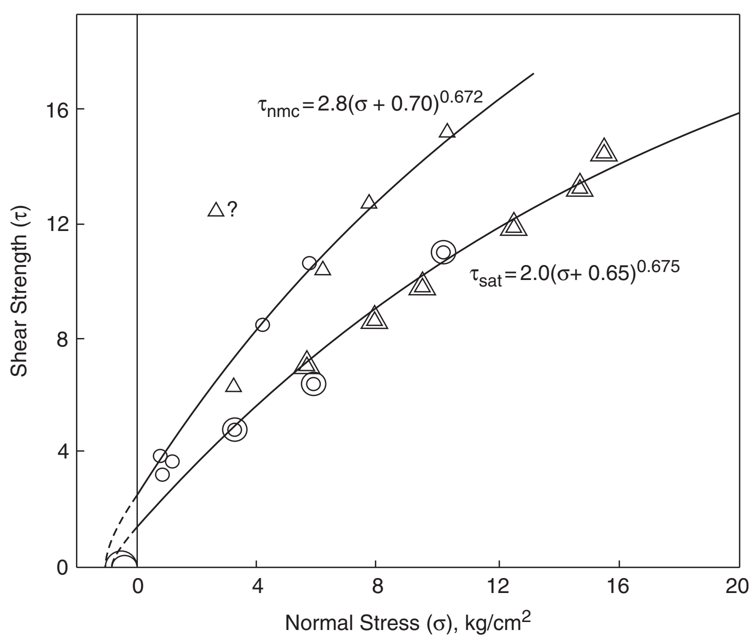

Fig. 8.3 Estimation of tensile strength of rock mass from Mohr's envelope (Mehrotra, 1992).

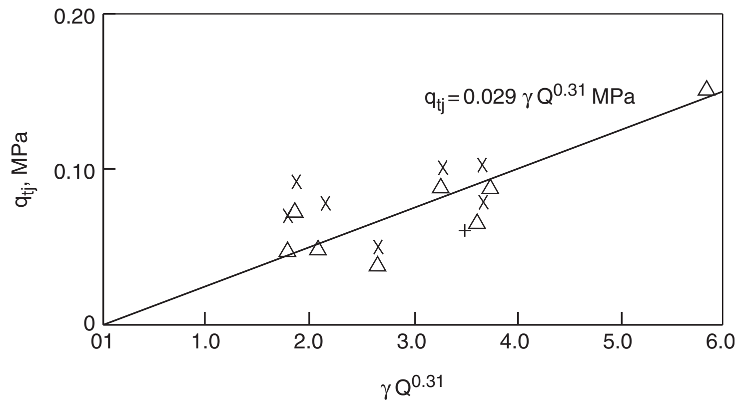

Fig. 8.4 Plot between q_{tj} and $\gamma \cdot Q^{0.31}$.

The tensile strength across discontinuous joints is not zero as generally assumed, but it is found to be significant specially in the hard rocks.

The tensile stress in tunnel roof of span B will be of the order of γB in the vertical direction. Equating this with q_{tj}, the span of self-supporting tunnels obtained from equation (8.21) would be $2.9Q^{0.31}$ m. Barton et al. (1974) found the self-supporting span to be $2Q^{0.4}$ m. This comparison is very encouraging. Thus, it is understood that the wedge analysis considering q_{tj} and in situ stress along tunnel axis may give more accurate value of the self-supporting tunnel span.

8.8 DYNAMIC STRENGTH OF ROCK MASS

It appears logical to assume that dynamic strain at failure should be of the same order as the static strain at failure for a given confining stress. Dynamic strain at failure should be proportional to modulus of elasticity of rock mass (E_e) and static strain at failure should be proportional to E_d. Therefore, a following hypothesis for dynamic strength enhancement is proposed.

$$q_{cmdyn}/q_{cmass} = (E_e/E_d)^{0.7} \tag{8.23}$$

where

q_{cmdyn} = dynamic strength of rock mass.

In seismic analysis of concrete dams, dynamic strength enhancement may be quite high, particularly for a weathered rock mass, as the instantaneous modulus of elasticity (E_e from equation (5.14)) will be much higher than the long-term modulus of deformation E_d (equation (5.13)).

Extensive research is needed to obtain more realistic correlations for dynamic strength enhancement.

8.9 RESIDUAL STRENGTH PARAMETERS

Mohr's theory will be applicable to residual failure as a rock mass would be reduced to non-dilatant soil-like condition. The mobilized residual cohesion c_r is approximately equal to 0.1 MPa and is not negligible unless tunnel closure is more than 5.5 percent of its diameter. The mobilized residual angle of internal friction ϕ_r is about 10° less than the peak angle of internal friction ϕ_p but more than 14°. The rock mechanics helps in judging the support system.

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9
The new Austrian tunnelling method

"Let us learn to live with landslide danger."

Evert Hoek (1998)

9.1 OLD TUNNELLING PRACTICE

In the conventional tunnelling practice of the past, masonry in dressed stone or brick was regarded as the most suitable lining material in unstable rock. Concrete was rejected because possible deformation during the setting and hardening process was supposed to cause irreparable damage. The space between masonry lining and rock face was dry packed. Timber lagging, which was subject to decay when left in place, generally could not be removed, particularly from the roof, because of the danger of loosening and rock falls.

The situation was further aggravated by an unfavorable time factor. Merely to bring a full section, a 9 m long section of a double-track railway tunnel by the old Austrian tunnelling method, after the bottom and top headings had been driven, it took about four weeks, and another month was needed to complete the masonry of the section. The amount of timber used in more difficult cases was so enormous that one-third and sometimes even more of the excavated space was filled by solid timber.

All these circumstances, together with the tendency of the temporary timber framework to yield, necessarily produced violent loosening pressures, which frequently caused roof settlement up to 40 cm before the masonry could be closed. Years after construction had been finished, a slow decrease in the volume of the compressible and sometimes badly executed dry packing often deformed the lining asymmetrically, causing damage and costly repairs. Damage to the surrounding rock as well as to the lining itself was further increased locally by the mechanical and chemical effects of water.

It is evident that in this period of rather inadequate methods and materials for temporary and permanent supports, loosening pressures were a source of the greatest concern to tunnel engineers. All attempts to design a lining during this period were consequently carried out with sole regard to loosening pressures. Occasional subsequent deformation of linings led

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to the erroneous conclusion that the linings designed in this way still lacked the necessary margin of safety, whereas the failures almost without exception were due to incorrect treatment of the surrounding rock and fundamental shortcomings of the methods.

Though methods and means of temporary and permanent support have improved fundamentally since the earlier period of the twentieth century, linings are still made as thick as they were about half a century ago. Loosening pressure is still considered by many to be the main active force to be reckoned with in tunnel design, although modern tunnelling methods actually make it possible to avoid loosening significantly.

9.2 DEVELOPMENT OF CONSTRUCTION AND LINING METHODS

Shortly after the turn of the twentieth century, grouting was introduced as an effective means of consolidating the rock surrounding a tunnel. By filling the voids, unsymmetrical local loads on the lining are avoided, and portions of loose or soft rock are strengthened by cementation.

The next stage was the introduction of steel for supports and which, compared with timber, constituted a remarkable improvement as a temporary lining material because of its better physical properties, its higher resistance to weathering, and its reduced tendency to yield. Decreased deformability of the temporary support made it possible to replace masonry as a lining material by concrete. Dry packing then became obsolete, since the concrete filled the spaces outside the A-line (circumference of the tunnel for payment to a contractor).

One of the most important advantages of steel supports is that they allow tunnels to be driven full-face to a very large cross section. The resulting unrestricted working area enables powerful drilling and mucking equipment to be used, increasing the rate of advance and reducing costs. Nowadays, dividing the face into headings which are subsequently widened is done only under most unfavorable geological conditions.

Remarkable progress in drilling and rock blasting especially in Sweden, has also helped to reduce damage to the surrounding rock.

9.3 MODERN TUNNELLING METHODS

Finally, during the last few decades, rock bolting and shotcrete were introduced in tunnelling practice. To judge from the results obtained up to now, the introduction of these methods of support and surface protection can be considered as a most important event, especially in the field of soft rock and earth tunnelling.

The advantages of these methods can be best shown by comparing the rock mechanics of tunnels lined by the new and by older methods [Figs 9.1 to 9.15 (Dhawan & Joshi, 1982) depicting modern and old practices of tunnelling]. Whereas all the older methods of temporary support without exception are bound to cause loosening and voids by yielding of the different parts of the supporting structure. A thin layer of shotcrete together with a suitable system of rock bolting applied to the excavated rock immediately after blasting entirely prevents loosening and reduces decompression to a certain degree, transforming the surrounding rock into a self-supporting arch.

A layer of shotcrete with a thickness of only 15 cm applied to a tunnel of 10 m diameter can safely carry a load of 45 tons/m^2 corresponding to a burden of 23 m of rock, which is more than the observed support pressure. If a steel-support structure incorporating No.20-type wide-flanged arches at 1 m centers was used under these conditions, it would fail with 65 percent of the load carried by the shotcrete lining. A timber support of the conventional Austrian type would be able to carry only a very small proportion of the same load. If the temporary support deforms or fails, the erroneous conclusion is usually drawn that the proposed permanent linings are not strong enough. In this way permanent linings that are already over-designed becomes still heavier.

9.4 TEMPORARY SUPPORTS

9.4.1 Conventional shotcrete

A temporary support designed to prevent loosening must attain a high carrying capacity as quickly as possible, and it must be strong and adhesive so that it seals off the surface closely and almost hermetically. The carrying capacity of a temporary support is determined by the material as well as by its structural design. Timber, especially when humid, is by far the worst; as it combines low physical properties with a great tendency for the structure to yield. Although steel has much better physical properties, the efficiency of steel-arch depends mainly on the quality of packing between the arches and the rock face, which is always unsatisfactory. On the contrary, concrete, particularly shotcrete, meets all the requirements for an ideal temporary support.

Shotcrete's high early strength is of the greatest importance in attaining a high support capacity rapidly, and this is particularly true of its early flexural (tensile) strength, which amounts to 30 and 50 percent of the compressive strength after one-half and two days. A recently introduced hardening accelerating admixture based on silicification gives still better results. The setting time for shotcrete is 3 min now.

The most conspicuous feature of shotcrete as a support against loosening and stress-rearrangement pressure lies in its interaction with the neighboring rock. A shotcrete layer applied immediately after opening up a new rock face acts as an adhesive surface by which a jointed rock of weak strength is transformed into a stable one. The shotcrete absorbs the tangential stresses which build up to a peak close to the surface of a cavity after it is opened up. As a result of the close interaction between shotcrete and rock blocks, the neighboring portions of rock mass remain almost in their original undisturbed state and are

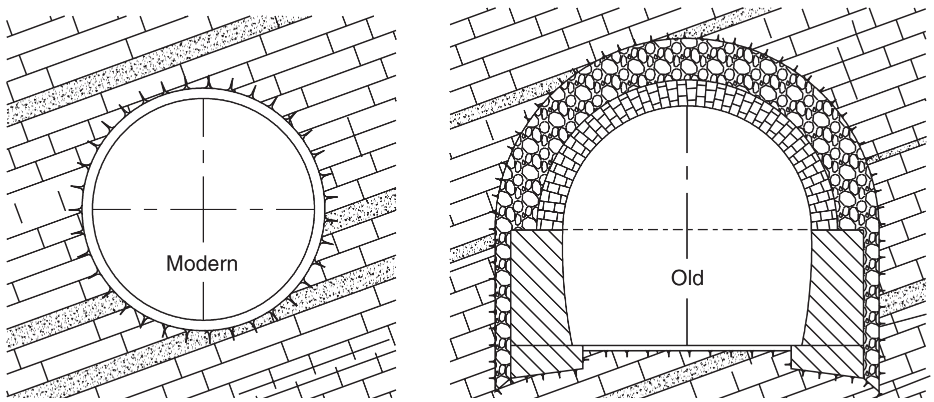

Fig. 9.1 The main load carrying member is the rock mass.

thus enabled to participate effectively in the arch action. The statically effective thickness of the zone of arch action is in this way increased to a multiple of that of the shotcrete. In this way, tensile stresses due to bending are diminished and compressive stresses are easily absorbed by the surrounding rock mass. The thickness zone of arch action can be increased at will by rock bolting.

Disintegration always starts by the opening of a thin surface fissure; if this movement is prevented at the outset by applying a shotcrete layer, the rock mass behind the shotcrete remains stable. This explains why cavities in weak rock mass lined with a skit of only a few centimeters of shotcrete remain in perfect equilibrium. Shallow tunnels in rock of medium quality built by conventional methods need a fairly strong temporary support and concrete lining. Thus only a thin layer of shotcrete, possibly locally strengthened by rock bolts, may provide both temporary support and a satisfactory permanent lining.

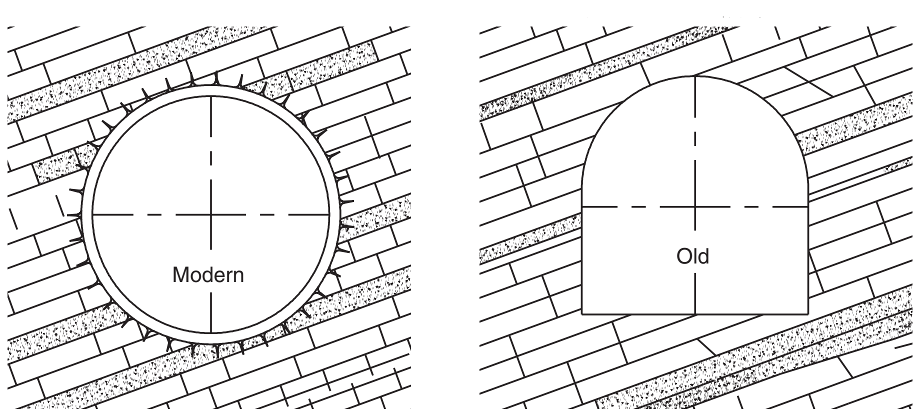

Fig. 9.2 Maintenance of original rock mass strength.

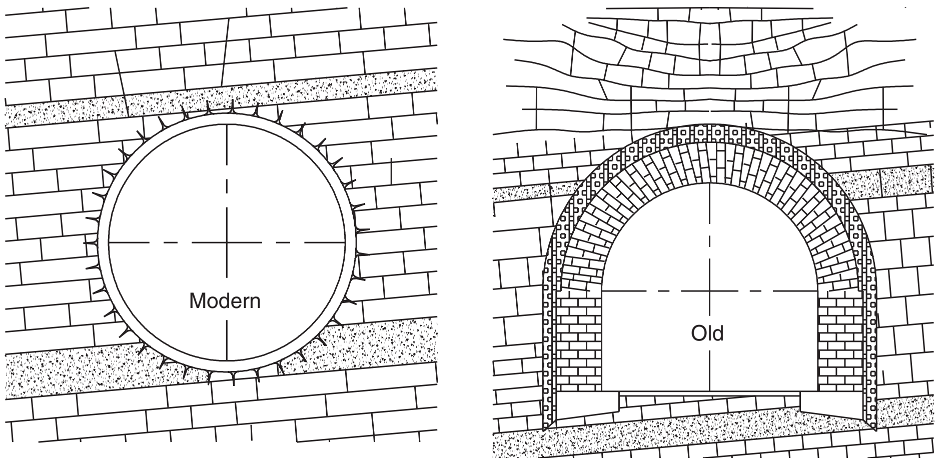

Fig. 9.3 Loosening must be prevented as it reduces strength.

Experience so far has shown that shotcrete, especially when combined with rock bolting, has proved excellent as a temporary support for all qualities of rock with standing time down to less than one hour and even for ground which normally could only be mastered by careful forepoling. Exceptionally, even almost cohesionless and plastic, ground has been successfully handled. In worst cases of plastic, water-bearing ground where steel forepoling failed, shotcrete has been successfully employed as a stabilizing reinforcement for steel support. Rock anchors can also be used to improve the behavior of rock mass.

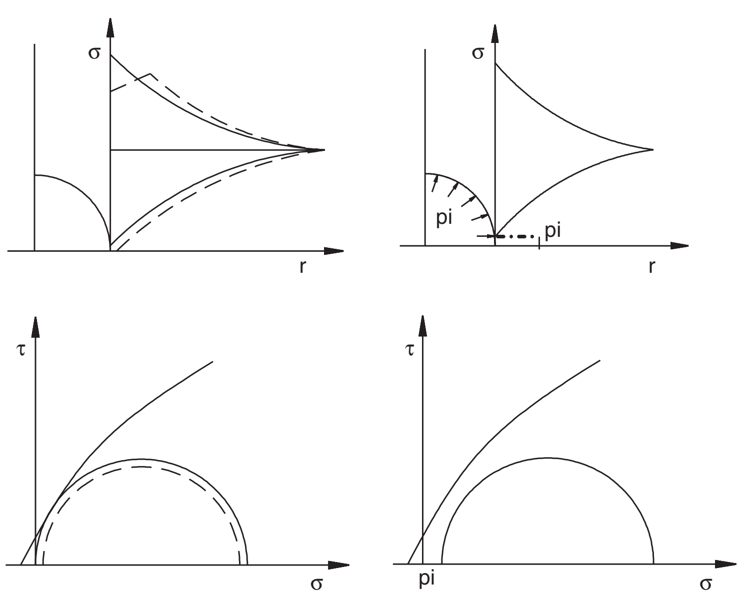

Fig. 9.4 Uniaxial stress condition should be prevented.

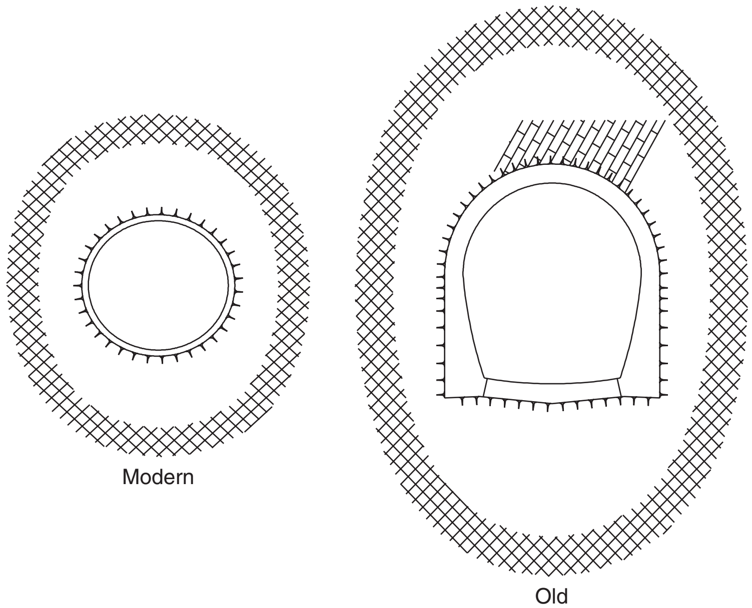

Fig. 9.5 Mobilization of the protective ring (rock carrying ring) without strength reduction.

The rock anchors stabilize the rock mass. If the anchors are placed in a radial pattern, the displacement also takes place in a radial manner. The development of shear zones can be prevented by the anchors. It also helps in improving the bearing capacity of rocks as the anchors act as reinforcement. Light steel sets and wire mesh could also be used as temporary supports. The special advantages of using these are that psychologically it looks more stable. It provides the connection between anchorage points and the weak rock and therefore increases the bearing capacity of the support system.

The name new Austrian tunnelling method (NATM) is a misnomer as it is not a method of tunnelling but a strategy for tunnelling which does have a considerable uniformity and sequence.

9.5 PHILOSOPHY OF NATM

The NATM is based on the philosophy of "Build as you go" approach with the following caution.

> "*Not too stiff*,
> *Nor too flexible*
> *Not too early*,
> *Nor too late*."

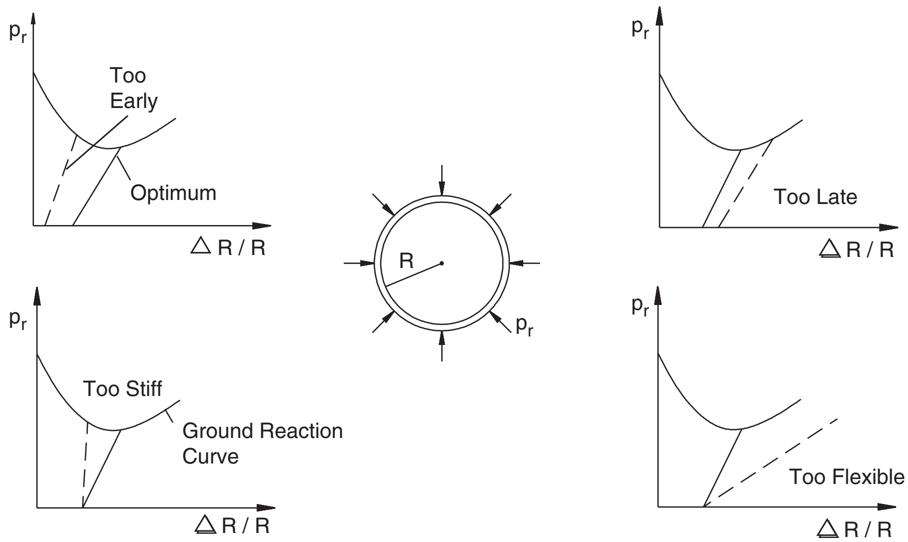

Fig. 9.6 Support (external lining) not too early, not too late, not too stiff, not too flexible.

The NATM accomplishes tunnel stabilization by controlled stress release. The surrounding rock is thereby transformed from a complex load system to a self-supporting structure together with the installed support elements, provided that the detrimental loosening, resulting in a substantial loss of strength, is avoided. The self-stabilization by controlled stress release is achieved by the introduction of the so called "Semi-Rigid Lining," i.e., systematic rock bolting with the application of a shotcrete lining. On one side, this offers a certain degree of immediate support, and on the other hand, the flexibility to allow stress release through radial deformation. The development of shear stresses in shotcrete lining in arched roof is thus reduced to a minimum. The function of NATM support system is as follows (Rabcewicz, 1964–1965; Rabcewicz et al., 1973).

(a) NATM is based on the principle that utmost advantage of the capacity of the rock mass should be taken to support itself by carefully controlling the forces in the redistribution process which takes place in the surrounding rock mass when an opening is made. This is also called "tunnelling with rock support." The main feature is that the rock mass in the immediate vicinity of the tunnel excavation is made to act as a load bearing member, together with the supporting system. The outer rock mass ring is activated by means of systematic rock bolting together with shotcrete. The main carrying member of the NATM is not only the shotcrete but also the systematically anchored rock arch.
(b) The installation of systematic rock bolting with shotcrete lining allows limited deformations but prevents loosening of the rock mass. In the initial stage it requires small forces to prevent rock mass from moving in, but once movement has started, large forces are required. Therefore, NATM advocates installation of supports within

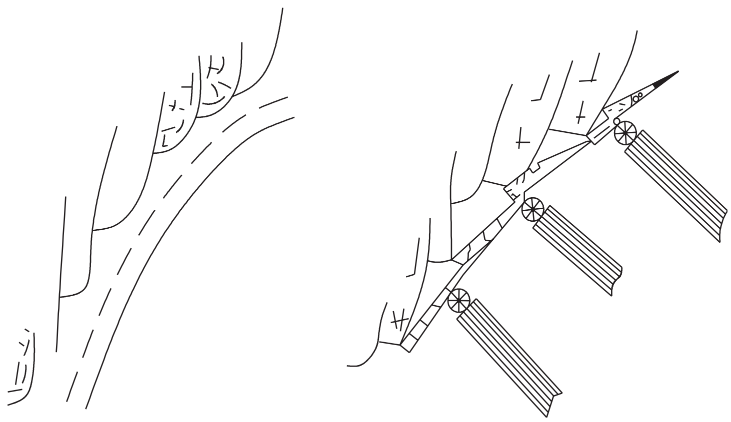

Fig. 9.7 Supports must be effective not at spots but overall.

stand-up time to prevent movements. It is also added that in non-squeezing ground conditions, the stresses in the shotcrete may be reduced significantly if the spray of the shotcrete is slightly delayed. The delay, however, should be within the stand-up time. But a safe practice is spraying first of all a sealing shotcrete layer (2.5 cm thick), immediately after excavation.

(c) In static consideration, a tunnel should be treated as a thick wall tube, consisting of a load-bearing ring of rock arch and supporting lining. Since, a tube can act as a tube only if it is closed, the closing of the ring becomes of paramount importance, specially where the foundation rock is not capable of withstanding high support pressure in squeezing ground condition.

A conduit is different than a tunnel of same diameter and depth; as trench is first excavated, then conduit is laid and soil back-filled. Thus conduit carries full cover pressure. In the case of tunnel, opening is excavated and some deformations take place before lining is sprayed. Thus the support pressures are much less than the cover pressure due to the arching action (Fig. 3.1).

(d) Due to stress-redistribution, when an opening is being excavated, a full-face heading is considered most favorable. Drivage in different stages complicates the stress-redistribution phenomenon and destroys the rock mass. In cases where full-face tunnelling is not possible, as in Chhibro–Khodri Tunnel and many other tunnels in Himalaya due to little stand-up time and the associated chances of rock falls and cavities. Consequently, engineers had to change to heading and benching method and struggled to achieve the targeted drivage rates in the absence of the beneficial effect of the shotcrete support.

(e) The question arises how to use the capacity of a jointed rock to support itself. This is accomplished by providing an initial shotcrete layer followed by systematic rock bolting, spraying additional shotcrete and using steel rib, if necessary. As in the case

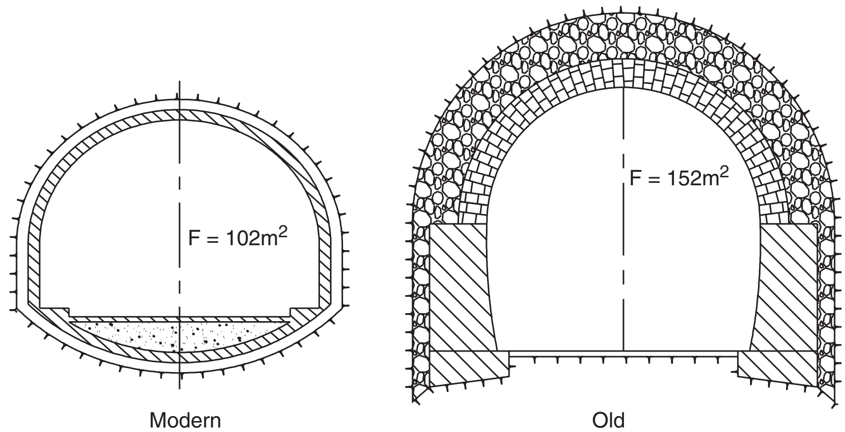

Fig. 9.8 Support should consist of thin linings which are flexible to bending. Ability to carry bending moments and bending failure is reduced.

of the Loktak Tunnel (India), NATM without steel arches in high squeezing grounds would have required several layers of shotcrete which could not be accommodated without compromising with the available finished bore. The spacing of steel arches is adjusted to suit the squeezing ground condition. The behavior of the protective support and the surrounding rock during the stress-redistribution process should be monitored and controlled, if necessary, by different measurements.

(f) Shotcrete in a water-charged rock mass should be applied in small patches leaving the radial gaps for effective drainage.

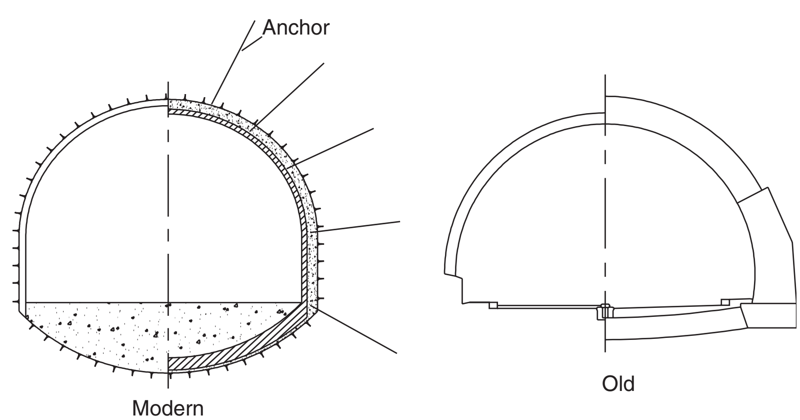

Fig. 9.9 Additional support should be provided by wire meshes, steel arches and anchorage. Not by increase of concrete thickness.

The New Austrian Tunnelling Method appears most suitable for soft ground which can be machine or manually excavated, where jointing and overbreak are not dominant, where a smooth profile can often be formed by smooth blasting and where a complete load-bearing ring can (and often should) be established. Monitoring plays a significant role in deciding the timing and the extent of secondary support.

Despite the comments by an experienced NATM pioneer that "it is not usually necessary to provide support in hard rocks," Norwegian tunnels require more than 50,000 m^3 of fiber reinforced shotcrete and more than 100,000 rock bolts each year (An article in World Tunnelling, June 1992). Two major tunnelling nations, Norway and Austria, have in fact long traditions of using shotcrete and rock bolts for tunnel supports, yet there are significant differences in philosophy and areas of application for NATM and NMT (Norwegian Method of Tunnelling).

Thus, the basic principles of NATM are summarized as

(i) Mobilization of rock mass strength,
(ii) Shotcrete protection to preserve the load-carrying capacity of the ring of rock mass,
(iii) Monitoring the deformation of the excavated rock mass,
(iv) Providing flexible but active supports and
(v) Closing of invert to form a load-bearing support ring to control deformation of the rock mass.

9.6 FINAL DIMENSIONING BY MEASUREMENT

Inseparably connected with the NATM, and a basic feature of the method, is a sophisticated measuring programme. Deformations and stresses are controlled systematically, allowing determination of whether the chosen support-capacity corresponds with the type of rock mass in question, and what kind of additional reinforcing measures are needed if any.

In case of the lining being over-dimensioned, the reinforcing measures can straight away be reduced accordingly when the same or similar mechanical conditions of the rock are encountered during further tunnel driving. An empirical dimensioning is carried out in this way, based on the scientific principles.

During the execution of a series of important tunnelling works using the NATM during the last few decades, a reasonably satisfactory measuring system has been developed.

In order to control the behavior of the outer arch and surrounding rock during the different construction stages in practice, main measuring sections are chosen at distances determined by the salient geological and rock mechanics considerations. These sections are equipped with the double extensometers and convergence measuring devices to measure deformations and the pressure pads to measure radial and tangential stresses (details on instrumentation can be seen in the Chapter 14).

In addition, roof and floor points are monitored geodetically using a modern electronic theodolite with sensors attached to the excavated faces. In between the main

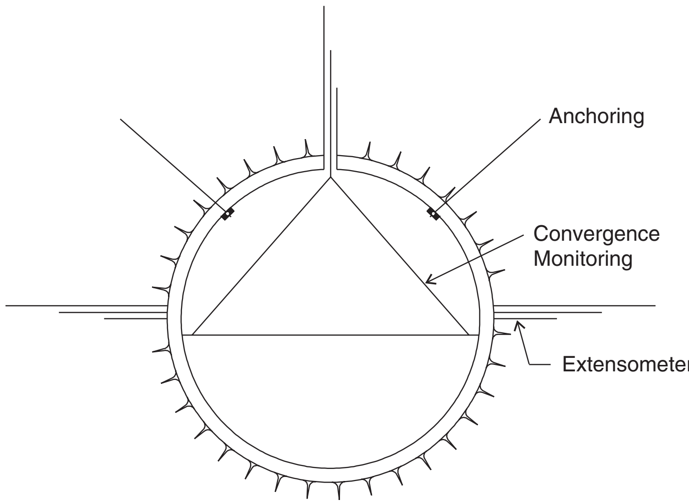

Fig. 9.10 Necessary support and its timing should be adjusted according to the measuring of the displacement.

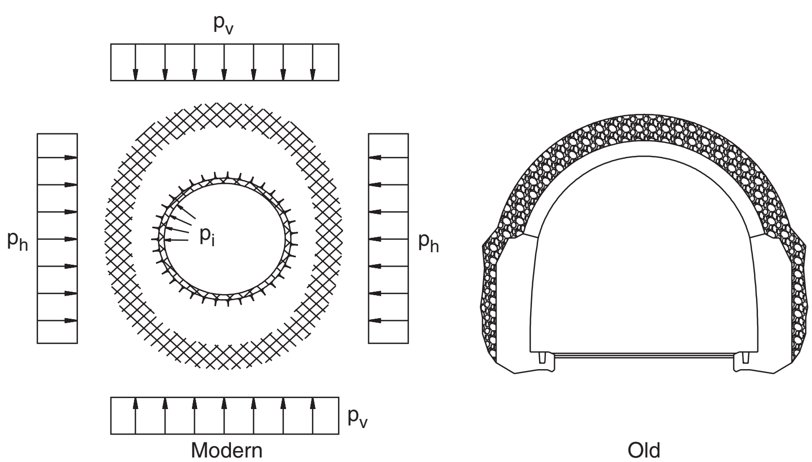

Fig. 9.11 According to rock mechanics, the tunnel is a tube which consists of the rock carrying ring and the support, not a conduit.

measuring sections, secondary ones are selected at suitable distances where only convergence readings are made between the roof and floor.

Readings are taken every other day at the beginning, decreasing to once a month according to the rate of deformation and change of stresses. The observations are plotted in graphs as a function of time. Stability of a support system is indicated if tunnel closure is stabilizing with time otherwise the reverse is true.

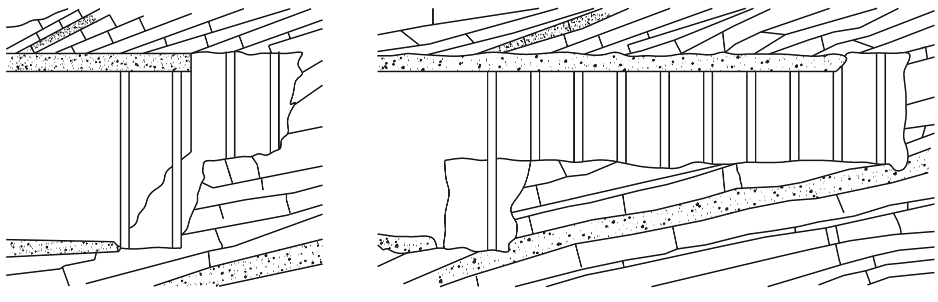

Fig. 9.12 Behavior of the rock will be influenced by the delay of ring closure, advanced crown, increase delay, bending effect on the crown and increasing loads on the rock foundation.

This method of establishing stress-time graphs gives a high degree of safety, allowing any situation to be recognized long before it becomes dangerous. They are comparable with the function of temperature charts or electrocardiograms in medical science.

Since the readjustment process takes a long time, possibly influenced locally by subsequent alterations of the geological conditions (e.g., increase in the water content of the surrounding rock), it is essential from both the practical and theoretical point of view to measure also the stresses and deformations of the inner lining. This is done by placing a series of tangential pressure pads or strain gauges, both in pairs outside and inside the lining, and also by using convergence measuring devices.

9.7 CONCLUDING REMARKS

The NATM has evolved from the long practical experience. The behavior of the linings and their surrounding rocks has been observed closely by measurements in many tunnels and galleries in all kinds of rock. The efforts have been made to find a relationship between the phenomena observed and the laws of modern rock mechanics, and also to establish possible new ones.

The greatly simplified analytical formulae have emerged from practical experience to describe complicated processes observed in nature. Greatest accuracy would certainly not suit the complexity of the problems caused by a large scattering of parameter values and frequent changes of rock types and quality even on short stretches of tunnel.

One needs both experience and theoretical knowledge to design the standard sections adequately. These qualities are even more important when applying these standard types correctly during construction. It is inevitable that alterations will be needed, following the results of in situ measurements, and this will eventually lead to the most economical solution being achieved.

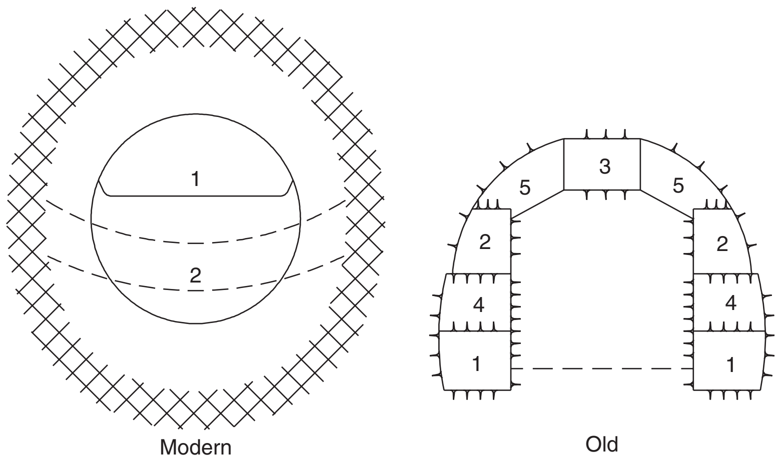

Fig. 9.13 Full-face heading helps to keep rock mass strength. Many partial headings reduce rock strength according to stress superposition.

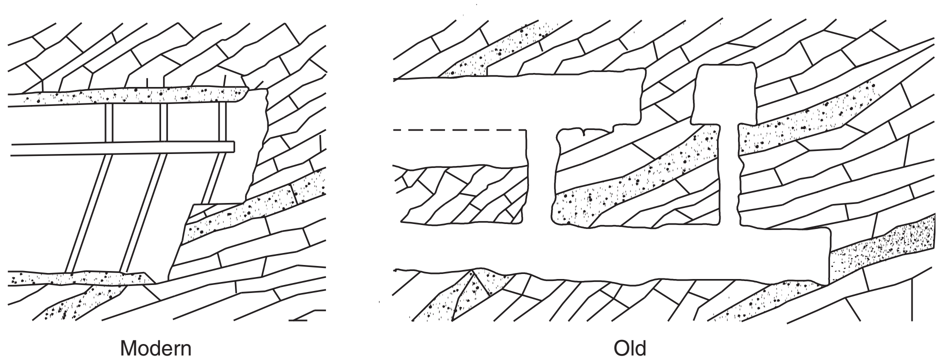

Fig. 9.14 Procedure of construction is important for safety of the structure. Variation of duration of a round, timing of support and ring closure, length of the crown and lining resistance are used to help the self-stabilization of the rock and the support.

Sometimes judgment on the support system goes wrong, the lining of shotcrete cracks, the rate of tunnel closure does not stabilize with time. In fact the best advantage of NATM over steel supports is that NATM is a flexible construction technology. One may decide to spray additional layers of shotcrete until cracking of the last layer of shotcrete does not take place. One may go for spot-bolting if instrumentation gives a clear picture of a local geological problem. Thus design of support system is by trial and error in NATM. This scientific empirical method of dimensioning seems to be downright indispensable. It can certainly be assisted, but never be replaced, by analytical considerations.

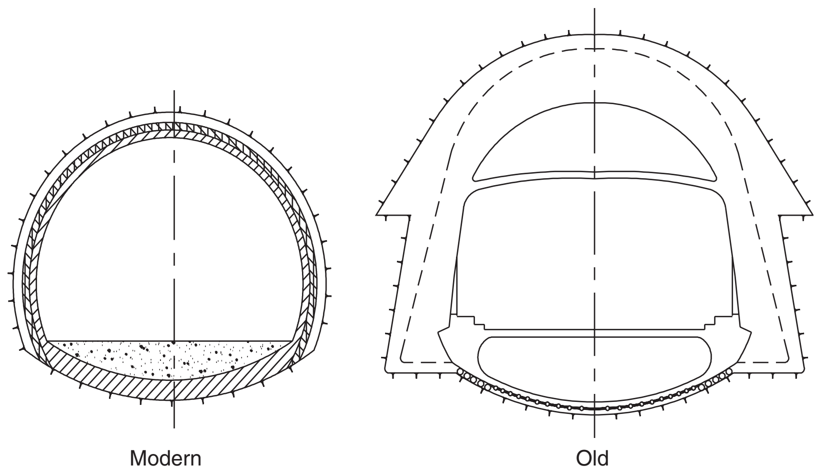

Fig. 9.15 Smoothly rounded shapes help to prevent stress concentrations.

The NATM is now not a new tunnel support method. This is based on practical experience and is designed to suit the actual field conditions. Thus it is leading to an efficient method of carrying out tunnelling operation in difficult conditions. However, it may be noted that NATM is not a method of excavation. The choice of excavation method is based on practical considerations.

The Norwegian method of tunnelling (NMT) is inspired by the NATM. NMT (Barton et al., 1974; Hoek & Brown, 1980) has evolved tables and a chart for design of NATM support system, although construction approach is quite flexible.

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10
Norwegian method of tunnelling

"The word impossible in itself says, I am possible!"

Anonymous

10.1 INTRODUCTION

According to Fairhurst (1993), designers should develop design solutions and design strategies that are robust, i.e., able to perform well and are adequate even in unknown geological conditions and fail in the desired (ductile) manner. For example, the shotcreted reinforced-rock arch is a robust design strategy. Historically, the Norwegian Method of Tunnelling (NMT) has evolved a successful strategy out of 30 years of experience which may be adopted in supporting tunnels in widely different rock conditions. There are 1260 case records to prove efficacy of this design approach.

A tunnelling revolution has occurred in the last 30 years with advent of wet-process shotcrete and stainless steel fiber reinforced shotcrete (SFRS). Since steel fibers are not continuous, they do not experience corrosion like mesh and RCC. Another revolution is the development of full-column-grouted resin (thermo-mechanically treated (TMT)) bolts. As far as life of these "light" support systems is concerned, they are stable for last 30 years. Their cost is only a fraction of the concrete lining. The key to success in polluted environment is the shotcrete of good quality which is dense, impermeable and strong (UCS > 45 MPa).

New Austrian Tunnelling Method (NATM) appears most suitable for soft ground, where a smooth profile can be formed. Thus all round load-bearing ring can be created with the help of rock anchors/bolts. It is an essential practice in NTM also. In the NTM, great emphasis is placed on extensive geological and geotechnical investigations unlike NATM. Chapter 5 describes Q-system of classification in detail. Experience has proved that a combination of RMR–Q classification is not systematic. Hence, only one system should be adopted in a tunnel.

NMT appears most suitable for good (hard) rock masses even where jointing and high overbreaks are dominant, and where drill and blast method or hard rock TBM's are the

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most usual methods of excavation. Bolting is the dominant form of rock support since it mobilizes the strength of the surrounding rock mass in the best possible way. Potentially unstable rock masses with clay-filled joints and discontinuities would increasingly need shotcrete and steel fiber reinforced shotcrete SFRS [S(fr)] to supplement systematic bolting (*B*). It is understood that NMT and NATM are the two most versatile tunnel support methods. These are devised and used extensively, because they can be applied to any profile as temporary or as a permanent support, just by changing the thickness and bolt spacing. A thick load bearing ring (reinforced ribs of shotcrete (RRS)) can be formed as needed, and it matches an uneven profile better than lattice girders or steel sets. These support requirements based on the Q-system are shown in Fig. 10.1. The essential features of the NMT are summarized in Table 10.1 (Barton et al., 1992).

10.2 UNSUPPORTED SPAN

Barton et al. (1974) proposed equation (5.11) for estimating equivalent dimension (D'_e) of a self-supporting or an unsupported tunnel. The D'_e is the ratio between tunnel width and ESR. The excavation support ratio (ESR) is given in Table 5.11. However, seepage erosion may be serious after a few decades in the initially self-supporting tunnels in water-soluble rocks near slopes.

Section 5.7 lists more conditions for no-support requirement. Needless to mention that no supports are needed in a self-supporting opening in the rock mass.

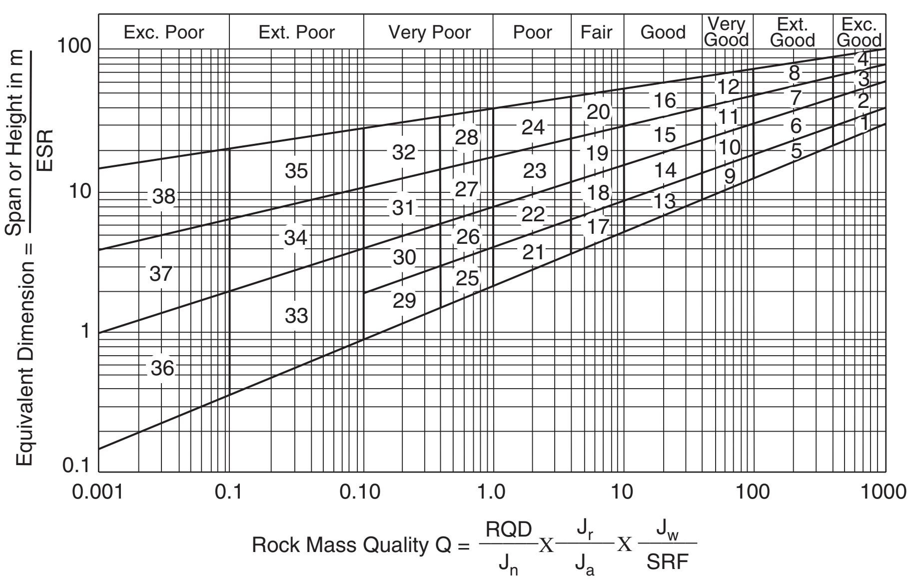


Fig. 10.1 Tunnel support chart showing 38 support categories (Barton et al., 1974).

Table 10.1 Essential features of NMT (Barton et al., 1992).

S.No.	Features
1.	**Areas of usual application:**
	Jointed rock giving overbreak, harder end of scale ($q_c = 3$ to 300 MPa) Clay bearing zones, stress slabbing (Q = 0.001 to 10 or more)
2.	**Usual methods of excavation:**
	Drill and blast, hard rock TBM, hand excavation in clay zones
3.	**Temporary rock reinforcement and permanent tunnel support may be any of the following:**
	CCA, S(fr) + RRS + *B*, *B* + S(fr), *B* + S, *B*, S(fr), S, sb, (NONE) * Temporary reinforcement forms part of permanent support * Mesh reinforced shotcrete not used * Dry process shotcrete not used * Steel sets or lattice girders not used, RRS and S(fr) are used in clay zones and in weak, squeezing rock masses * Contractor chooses temporary support * Owner/consultant chooses permanent support * Final concrete lining are less frequently used; i.e., *B* + S(fr) is usually the final support
4.	**Rock mass characterization for:**
	* Predicting rock mass quality * Predicting support needs * Updating both during tunnelling (monitoring in critical cases only)
5.	**The NMT gives low costs and**
	* Rapid advance rates in drill and blast tunnels * Improved safety * Improved environment

Notations: CCA = cast concrete arches; S(fr) = steel fiber reinforced shotcrete; RRS = reinforced ribs of shotcrete; *B* = systematic bolting; S = conventional shotcrete; sb = spot bolts; NONE = no support needed.

10.3 DESIGN OF SUPPORTS

The Q-value is related to the tunnel support requirements with the equivalent dimensions of the excavation. The relationship between Q and the equivalent dimension of an excavation determines the appropriate support measures as depicted in Fig. 10.1. Barton et al. (1974) have identified 38 support categories (Fig. 10.1) and specified permanent supports for these categories. The bolt length l, which is not specified in the support details, can be determined in terms of excavation width, B in meters using the following equations of Barton et al. (1974).

$$l = 2 + (0.15\,B/\text{ESR}), \text{m for pre-tensioned rock bolts in roof} \tag{10.1}$$

$$l = 2 + (0.15\,H/\text{ESR}), \text{m for pre-tensioned rock bolts in walls of height } (H) \tag{10.2}$$

and

$$l = 0.40\,B/\text{ESR}, \text{ m for the untensioned rock anchors in roof} \tag{10.3}$$

$$l = 0.35\,H/\text{ESR}, \text{ m for the untensioned rock anchors in walls} \tag{10.4}$$

Table 10.2 (Barton et al., 1974) suggests the type of bolt, its spacing and the thickness of conventional shotcrete for a given rock mass quality Q, equivalent span B/ESR, RQD/J_n and J_r/J_a values. For design of wall support system of a cavern, Q should be replaced by Q_w. In case of shaft, Q_w may be used for designing the support system for equivalent span (or diameter or size of shaft/ESR) and corresponding bolt length from equations (10.1) or (10.3) (Barton, 2001). Many supplementary notes are given at the end of Table 10.2. Other practical recommendations on shotcrete are compiled in Table 10.3.

It should be realized that shotcrete lining of adequate thickness and quality is a long-term support system. This is true for rail tunnels also. It must be ensured that there is a good bond between shotcrete and rock surface. Tensile bending stresses are not found to occur even in the irregular shotcrete lining in the roof due to a good bond between shotcrete and the rock mass in an arched-roof opening. Rock bolts help in better bonding. Similarly, contact grouting is essential behind the concrete lining to develop a good bond between the lining and rock mass to arrest its bending. However, bending stresses may develop in lining within the faults.

Rock has ego (Extraordinary Geological Occurrence) problems. As such, where cracks appear in the shotcrete lining, more layers of shotcrete should be sprayed. The opening should also be monitored with the help of borehole extensometers at such locations particularly in the squeezing ground. If necessary, expert tunnel engineers should be invited to identify and solve construction problems. At this point in time, NTM does not suggest the tunnel instrumentation in hard rocks, unlike NATM.

In the over-stressed brittle hard rocks, rock anchors should be installed to make the reinforced rock arch a ductile arch. Thus, a mode of failure is designed to be ductile from the brittle failure. Hence, failure would be slow giving enough time for local strengthening (or retrofitting) of the existing support system.

10.4 DESIGN OF STEEL FIBER REINFORCED SHOTCRETE

Wet process SFRS has the following advantages (Barton et al., 1992).

(i) high application-capacity rate upto 25 m^3 per hour,
(ii) efficient reinforcement,

Table 10.2 Recommended support based upon NGI rock mass quality Q (Hoek & Brown, 1980).

Rock mass quality Q or Q_w or Q_{av} or $Q_{seismic}$	Equivalent dimension (Span/ESR)	Block size (RQD/J_n)	Inter-block strength (J_r/J_n)	Approx. support pressure (p_{roof}), MPa	Spot reinforcement with untensioned grouted dowels	Untensioned grouted dowels on grid pattern	Tensioned rock bolts on grid with spacing	Chainlink mesh anchored to bolts at intermediate points	Shotcrete applied directly to rock, thickness indicated	Shotcrete reinforced with weld-mesh, thickness indicated	Unreinforced cast concrete arch, thickness indicated	Steel reinforced cast concrete arch, thickness indicated	Notes by Barton et al. (1974)	Notes by Hoek and Brown (1980)
1000–400	20–100			<0.001	≤								1	a
400–100	12–88			0.005	≤								1	a
100–40	8.5–19	>20		0.025	≤								1	a
100–40	8.5–19	>20		0.025		2.5–3 m								
100–40	14–30	>30		0.025		2–3 m								
100–40	14–30	>30		0.025		1.5–2 m		≤						b
100–40	23–72	>30		0.025			2–3 m							
100–40	23–72	>30		0.025			1.5–2 m	≤						b
40–10	5–14	>10	>1.5	0.05	≤								2	
40–10	5–14	>10	<1.5	0.05		1.5–2 m							2	
40–10	5–14	>10	>1.5	0.05		1.5–2 m							2	
40–10	5–14	>10	<1.5	0.05		1.5–2 m			20–30 mm				2	
40–10	15–23	>10		0.05			1.5–2 m	≤					2,3	b
40–10	15–23	>10		0.05			1.5–2 m			50–100 mm			2,3	c
40–10	9–15			0.05		1.5–2 m		≤					2,4	b
40–10	15–40	>10		0.05			1.5–2 m	≤					2,3,5	b
40–10	15–40	>10		0.05			1.5–2 m			50–100 mm			2,3,5	c

Continued

Table 10.2—Continued

Rock mass quality Q or Q_W or Q_{av} or $Q_{seismic}$	Equivalent dimension (Span/ESR)	Block size (RQD/J_n)	Inter-block strength (J_r/J_n)	Approx. support pressure (p_{roof}), MPa	Spot reinforcement with untensioned grouted dowels	Untensioned grouted dowels on grid pattern	Tensioned rock bolts on grid with spacing	Chainlink mesh anchored to bolts at intermediate points	Shotcrete applied directly to rock, thickness indicated	Shotcrete reinforced with weld-mesh, thickness indicated	Unreinforced cast concrete arch, thickness indicated	Steel reinforced cast concrete arch, thickness indicated	Notes by Barton et al. (1974)	Notes by Hoek and Brown (1980)
40–10	30–65	>15		0.05			1.5–2 m	≤					2,6,7,13	b
40–10	30–65	>15		0.05			1.5–2 m			50–100 mm			2,6,7,13	c
10–4	3.5–9	>30		0.10	≤								2	a
10–4	3.5–9	>10<30		0.10		1–1.5 m							2	
10–4	6–9	<10		0.10		1–1.5 m			20–30 mm				2	
10–4	<6	<10		0.10					20–30 mm				2	
10–4	10–15	>5		0.10			1–1.5 m	≤					2,4	a
10–4	7–10	>5		0.10		1–1.5 m		≤					2	a
10–4	10–15	<5		0.10			1–1.5 m		20–30 mm				2,4	
10–4	7–10	<5		0.10		1–1.5 m			20–30 mm				2	
10–4	20–29			0.10			1–2 m			100–150 mm			2,3,5	c
10–4	12–20			0.10			1–1.5 m			50–100 mm			2,3	c
10–4	35–52			0.10			1–2 m			200–250 mm			2,6,7,13	c
10–4	24–35			0.10			1–2 m			100–200 mm			2,3,5,13	c
4–1	2.1–6.5	>12.5	<0.75	0.15		1 m			20–30 mm				2	
4–1	2.1–6.5	>12.5	<0.75	0.15					20–30 mm				2	
4–1	2.1–6.5		<0.75	0.15		1 m							2	
4–1	4.5–11.5	>10	<30>1	0.15		1 m		≤					2	a
4–1	4.5–11.5	<10	>1	0.15					25–75 mm				2	

4–1	4.5–11.5	<30	<1	0.15	1 m			25–50 mm			2	c
4–1	4.5–11.5	>30		0.15	1 m						2	
4–1	15–24			0.15		1–1.5 m		100–150 mm			2,3,5,8	c
4–1	8–15			0.15		1–1.5 m		50–100 mm			2	c
4–1	30–46			0.15		1–1.5 m		150–300 mm			2,6,7,13	c
4–1	18–30			0.15		1–1.5 m		100–150 mm			2,3,5	c
1–0.4	1.5–4.2	>10	>0.5	0.225	1 m		≤				2	d
1–0.4	1.5–4.2	<10	>0.5	0.225	1 m			50 mm			2	c
1–0.4	1.5–4.2		<0.5	0.225	1 m			50 mm			2	c
1–0.4	3.2–7.5			0.225		1 m		50–75 mm			14,11,12	c
1–0.4	3.2–7.5			0.225	1 m		25–50 mm				2,10	
1–0.4	12–18			0.225		1 m		75–100 mm			2,10	c
1–0.4	6–12			0.225	1 m			50–75 mm			2,10	c
1–0.4	12–18			0.225		1 m			200–400 mm		14,11,12	c
1–0.4	6–12			0.225		1 m		100–200 mm			14,11,12	c
1–0.4	30–38			0.225		1 m		300–400 mm			2,5,6,10,13	c,f
1–0.4	20–30			0.225		1 m		200–300 mm			2,3,5,10,13	c
1–0.4	15–20			0.225		1 m		150–200 mm			1,3,10,13	c
1–0.4	15–38			0.225		1 m				300 mm–1 m	5,9,10,12,13	
0.4–0.1	1–3.1	>5	>0.25	0.3	1 m		20–30 mm					
0.4–0.1	1–3.1	<5	>0.25	0.3	1 m			50 mm				c
0.4–0.1	1–3.1		<0.25	0.3		1 m		50 mm				c
0.4–0.1	2.2–6	>5		0.3		1 m		25–50 mm			10	c
0.4–0.1	2.2–6	<5		0.3				50–75 mm			10	c
0.4–0.1	2.2–6			0.3		1 m		50–75 mm			9,11,12	c
0.4–0.1	4–14.5	>4		0.3		1 m		50–125 mm			10	c
0.4–0.1	4–14.5	<4>1.5		0.3				75–250 mm			10	c
0.4–0.1	4–14.5	<1.5		0.3		1 m			200–400 mm		10,12	c
0.4–0.1	4–14.5			0.3		1 m				300–500 mm	9,11,12	
0.4–0.1	20–34			0.3		1 m		400–600 mm			3,5,10,12,13	f
0.4–0.1	11–20			0.3		1 m		200–400 mm			4,5,10,12,13	c
0.4–0.1	11–34			0.3		1 m				400 mm–1.2 m	5,9,11,12,13	

Continued

Table 10.2—Continued

Rock mass quality Q or Q_W or Q_{av} or $Q_{seismic}$	Equivalent dimension (Span/ESR)	Block size (RQD/J_n)	Inter-block strength (J_r/J_n)	Approx. support pressure (p_{roof}), MPa	Spot reinforcement with untensioned grouted dowels	Untensioned grouted dowels on grid pattern	Tensioned rock bolts on grid with spacing	Chainlink mesh anchored to bolts at intermediate points	Shotcrete applied directly to rock, thickness indicated	Shotcrete reinforced with weld-mesh, thickness indicated	Unreinforced cast concrete arch, thickness indicated	Steel reinforced cast concrete arch, thickness indicated	Notes by Barton et al. (1974)	Notes by Hoek and Brown (1980)
0.1–0.01	1–3.9	>2		0.6			1 m			25–50 mm			10	c
0.1–0.01	1–3.9	<2		0.6						50–100 mm			10	c
0.1–0.01	1–3.9			0.6						75–150 mm			9,11	c
0.1–0.01	2–11	>2	>0.25	0.6			1 m			50–75 mm			10	c
0.1–0.01	2–11		<0.25	0.6						150–250 mm			10	c
0.1–0.01	2–11			0.6			1 m					200–600 mm	9,11,12	
0.1–0.01	15–28			0.6			1 m			300–1 m			3,10,12,13	c,f
0.1–0.01	15–28			0.6			1 m					600 mm–2 m	3,9,11,12,13	
0.1–0.01	6.5–15			0.6			1 m			200–750 mm			4,10,12,13	c,f
0.1–0.01	6.5–15			0.6			1 m					400 mm–1.5 m	3,9,11,12,13	
0.01–0.001	1–2			1.2						100–200 mm			10	c
0.01–0.001	1–2			1.2			0.5–1 m			100–200 mm			9,11,12	c
0.01–0.001	1–6.5			1.2						200–600 mm			10	c,f
0.01–0.001	1–6.5			1.2			0.5–1 m			200–600 mm			9,11,12	c,f
0.01–0.001	10–20			1.2								1–3 m	10–14	
0.01–0.001	10–20			1.2			1 m					1–3 m	3,9,11,12,14	
0.01–0.001	4–10			1.2						700 mm–2 m			10,14	c,f
0.01–0.001	4–10			1.2			1 m			700 mm–2 m			4,9,10,11,14	c,f

Table 10.3 Summary of recommended shotcrete applications in tunnelling, for different rock mass conditions.

Rock mass description	Rock mass behavior	Support requirement	Shotcrete application
Massive metamorphic or igneous rock. Low stress conditions	No spalling, slabbing or failure	None	None
Massive sedimentary rock. Low stress conditions	Surfaces of some shales, siltstones, or claystones may slake as a result of moisture content change	Sealing surface to prevent slaking	Apply 25 mm thickness of plain shotcrete to permanent surfaces as soon as possible after excavation. Repair shotcrete damage due to blasting
Massive rock with single wide fault or shear zone	Fault gouge may be weak and erodible and may cause stability problems in adjacent jointed rock	Provision of support and surface sealing in vicinity or weak fault or shear zone	Remove weak material to a depth equal to width of fault or shear zone and grout rebar into adjacent sound rock. Weld mesh can be used if required to provide temporary rockfall support. Fill void with plain shotcrete. Extend steel fiber reinforced shotcrete laterally for at least width or gouge zone
Massive metamorphic or igneous rock. High stress conditions	Surface slabbing, spalling and possible rockburst damage	Retention of broken rock and control of rock mass dilation	Apply 50 mm shotcrete over weld mesh anchored behind bolt faceplates, or apply 50 mm of steel fiber reinforced shotcrete on rock and install rock bolts with faceplates; then apply second 25 mm shotcrete layer Extend shotcrete application down sidewalls where required

Continued

Table 10.3—Continued

Rock mass description	Rock mass behavior	Support requirement	Shotcrete application
Massive sedimentary rock. High stress conditions	Surface slabbing, spalling and possible squeezing in shales and soft rocks	Retention of broken rock and control of squeezing	Apply 75 mm layer of fiber reinforced shotcrete directly on clean rock. Rock bolts or dowels are also needed for additional support
Metamorphic or igneous rock with a few widely spaced joints. Low stress conditions	Potential for wedges or blocks to fall or slide due to gravity loading.	Provision of support in addition to that available from rock bolts or cables	Apply 50 mm of steel fiber reinforced shotcrete to rock surfaces on which joint traces are exposed
Sedimentary rock with a few widely spaced bedding planes and joints, low stress conditions	Potential for wedges or blocks to fall or slide due to gravity loading. Bedding plane exposures may deteriorate in time	Provision of support in addition to that available from rock bolts or cables. Sealing or weak bedding plane exposures	Apply 50 mm of steel fibre reinforced shotcrete on rock surface on which discontinuity traces are exposed, with particular attention to bedding plane traces
Jointed metamorphic or igneous rock. High stress conditions	Combined structural and stress controlled failures around opening boundary	Retention of broken rock and control of rock mass dilation	Apply 75 mm plain shotcrete over weld mesh anchored behind bolt faceplates or apply 75 mm of steel fiber reinforced shotcrete on rock, install rock bolts with faceplates and then apply second 25 mm shotcrete layer Thicker shotcrete layers may be required at high stress concentrations
Bedded and jointed weak sedimentary rock. High stress conditions	Slabbing, spalling and possibly squeezing	Control of rock mass failure and squeezing	Apply 75 mm of steel fiber reinforced shotcrete to clean rock surfaces as soon as possible, install rock bolts, with faceplates, through shotcrete, apply second 75 mm shotcrete layer

Highly jointed metamorphic or igneous rock. Low stress conditions	Revelling or small wedges and blocks defined by intersecting joints	Prevention of progressive ravelling	Apply 50 mm of steel fiber reinforced shotcrete on clean rock surface in roof of excavation Rock bolts or dowels may be needed for additional support for large blocks
Highly jointed and bedded sedimentary rock. Low stress conditions	Bed separation in wide span excavations and revelling or bedding traces in inclined faces	Control of bed separation and ravelling	Rock bolts or dowels required to control bed separation Apply 75 mm of fiber reinforced shotcrete to bedding plane traces before bolting
Heavily jointed igneous or metamorphic rock, conglomerates or cemented rock fill. High stress conditions	Squeezing and "plastic" flow of rock mass around opening	Control of rock mass failure and dilation	Apply 100 mm of steel fiber reinforced shotcrete as soon as possible and install rock bolts, with faceplates, through shotcrete. Apply additional 50 mm of shotcrete if required. Extend support down sidewall if necessary
Heavily jointed sedimentary rock with clay coated surfaces. High stress conditions	Squeezing and "plastic" flow of rock mass around opening. Clay rich rocks may swell	Control of rock mass failure and dilation	Apply 50 mm of steel fiber reinforced shotcrete as soon as possible, install lattice girders or light steel sets, with invert struts where required, then more steel fiber reinforced shotcrete to cover sets or girders. Forepoling or spiling may be required to stabilize face ahead of excavation
Mild rockburst conditions in massive rock subjected to high stress conditions	Spalling, slabbing and mild rockbursts	Retention of broken rock and control of failure propagation	Apply 50 to 100 mm of shotcrete over mesh or cable lacing which is firmly attached to the rock surface by means of yielding rock bolts or cablebolts

(iii) lesser rebound in the range of 5–10% which is lower than that in the dry process,
(iv) uniform and high quality SFRS,
(v) less dust than in dry process,
(vi) no mesh is needed and so no air gaps behind shotcrete,
(vii) low permeability due to low water–cement ratio,
(viii) no corrosion of short-stainless steel fibers and
(ix) cost-effective in long tunnels or large caverns. However, technology calls for skilled workers, engineering geologists and rock engineers.

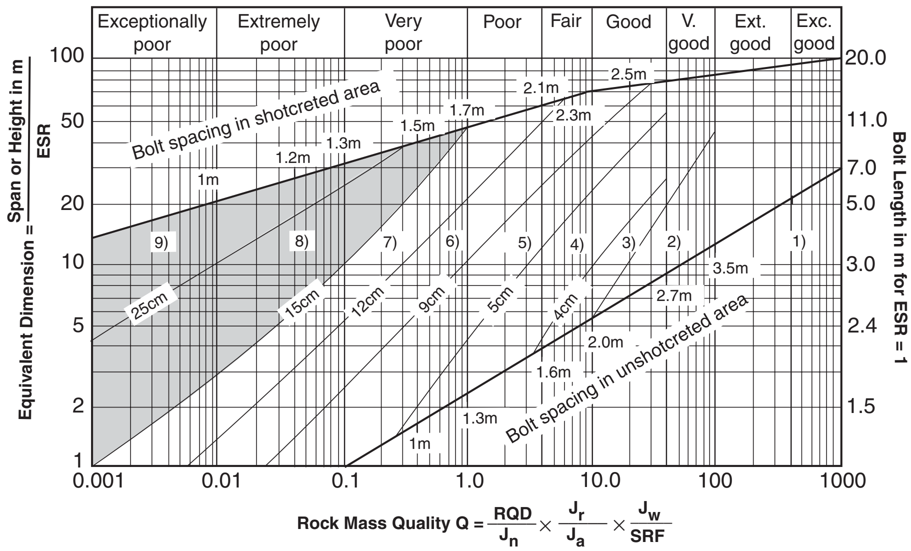
It can be noted that compression structures have longer life than the tension structures. The analysis shows that the shotcrete with good bond with the homogeneous rock mass is likely to be in compression in the tunnels with arched roof. Thus structures may have long life upto 60 years in dry rock masses.

Since the early 1980s, wet mix steel fiber reinforced shotcrete (SFRS) together with rock bolts have been the main components of a permanent rock support in underground openings in Norway. Based on the experience, Grimstad and Barton (1993) suggested a different support design chart using the SFRS on the basis of 1260 case records as shown in Fig. 10.2. This chart is recommended for tunnelling in poor rock conditions and moderate squeezing ground conditions also.

Shear zones are encountered in the underground openings specially in the tectonically disturbed geological conditions. The average value of rock mass quality Q_{av} is estimated as suggested by Bhasin et al. (1995) in Section 28.7. This value is then used in Table 10.2 and Fig. 10.2 for designing the support system in the neighborhood of shear zones. In fact, the rock masses are classified into various grades I, II, III, etc. at the tunnel projects. The drawings of temporary and or permanent support systems are prepared for all grades in advance of tunnelling. This is called flexible and robust planning strategy. Thus, all that is needed is on-the-spot decision of choice of the support system according to actual tunnelling conditions.

Supplementary notes by Barton et al. (1974):

(i) The type of support used in extremely good and exceptionally good rock will depend upon the blasting technique. Smooth wall blasting and thorough scaling-down may remove the need for support. Rough wall blasting may result in the need for a quick single application of shotcrete, especially where the excavation height exceeds 25 m.
(ii) For cases of heavy rock bursting or "popping," tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally 0.8 m). Final support is installed when "popping" activity ceases.
(iii) Several bolt lengths often used in same excavation, i.e., 3, 5 and 7 m.
(iv) Several bolt lengths often used in same excavation, i.e., 2, 3 and 4 m.



REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting, sb
3) Systematic bolting, B
4) Systematic bolting (and unreinforced 4 to10cm, B(+S)shotcrete,
5) Fiber reinforced shotcrete and bolting, 5 to 9cm, S(fr)+B
6) Fiber reinforced shotcrete and bolting, 9 to 12cm, S(fr)+B
7) Fiber reinforced shotcrete and bolting, 12 to 15cm, S(fr)+B
8) Fiber reinforced shotcrete > 15cm, reinforced ribs of shotcrete and bolting, S(fr), RRS+B
9) Cast concrete lining,CCA

Fig. 10.2 Chart for the design of SFRS support (Grimstad & Barton, 1993).

(v) Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2 to 4 m.

(vi) Several bolt lengths often used in same excavation, i.e. 6, 8 and 10 m.

(vii) Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4 to 6 m.

(viii) Several older generation power stations in this category employ systematic or spot bolting with chain link mesh, and a concrete arch roof (250–400 mm) as a permanent support.

(ix) Cases involving swelling, for instance montmorillonite clay (with access to water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.

(x) Cases not involving swelling clay or squeezing rock.

(xi) Cases involving squeezing rock. Heavy rigid support is generally used as a permanent support.

(xii) According to the experience of Barton et al. (1974), in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed, may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i.e., >1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i.e., RQD/J_n < 1.5, for example a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of layers of shotcrete to reduce the uneven loading on the concrete. But it may not be effective when RQD/J_n < 1.5 or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick setting resin anchors in these extremely poor quality rock masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches be taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.

(xiii) For reasons of safety the multiple drift method will be often needed during excavation and supporting of roof arch. For Span/ESR > 15 only.

(xiv) Multiple drift method usually needed during excavation and support of arch, walls and floor in cases of heavy squeezing. For Span/ESR > 10 in exceptionally poor rock only.

Supplementary notes by Hoek and Brown (1980):

a. In Scandinavia, the use of "Perfobolts" is common. These are perforated hollow tubes which are filled with grout and inserted into drillholes. The grout is extruded to fill the annular space around the tube when a piece of reinforcing rod is pushed into the grout filling the tube. Obviously, there is no way in which these devices can be tensioned although it is common to thread the end of the reinforcing rod and place a normal bearing plate or washer and nut on this end (see Fig. 12.4).

   In North America, the use of "Perfobolts" is rare. In mining applications a device known as a "Split set" or "Friction set" (developed by Scott) has become popular. This is a split tube which is forced into a slightly smaller diameter hole than the outer diameter of the tube. The friction between the steel tube and the rock, particularly when the steel rusts, acts in the same way as the grout around a reinforcing rod. For temporary support these devices are very effective (see Fig. 12.5).

   In Australian mines, untensioned grouted reinforcement is installed by pumping thick grout into drillholes and then simply pushing a piece threaded reinforcing rod into the grout. The grout is thick enough to remain in an up-hole during placing of the rod.

b. Chain link mesh is sometimes used to catch small pieces of rock which can become loose with time. It should be attached to the rock at intervals between 1 and 1.5 m and short grouted pins can be used between bolts. Galvanized chain link mesh should be used where it is intended to be permanent, e.g., in an underground powerhouse.

c. Weld mesh, consisting of steel wires set on a square pattern and welded at each intersection, should be used for the reinforcement of shotcrete since it allows easy access of the shotcrete to the rock. Chain link mesh should never be used for this purpose since the shotcrete cannot penetrate all the spaces between the wires, and air pockets are formed with consequent rusting of the wire. When choosing weld mesh, it is important that the mesh can be handled by one or two men working from the top of a high-lift vehicle and hence the mesh should not be too heavy. Typically, 4.2 mm diam. wires set at 100 mm intervals (designated 100×4.2 weld mesh) are used for reinforcing shotcrete.
d. In poorer quality rock, the use of untensioned grouted dowels as recommended by Barton et al. (1974) depends upon the immediate installation of these reinforcing elements behind the face. This depends upon integrating the support drilling and installation into the drill–blast–muck cycle and many non-Scandinavian contractors are not prepared to consider this system. When it is impossible to ensure that untensioned grouted dowels are going to be installed immediately behind the face, consideration should be given to use tensioned rock bolts which can be grouted at a later stage. This ensures that support is available during the critical excavation stage.
e. Many contractors would consider that a 200 mm thick cast concrete arch is too difficult to construct because there is not enough room between the shutter and the surrounding rock to permit easy access for pouring concrete and placing vibrators. The US Army Corps of Engineers suggests 10 in. (254 mm) as a normal minimum thickness while some contractors prefer 300 mm.
f. Barton et al. (1974) suggested shotcrete thicknesses of up to 2 m. This would require many separate applications and many contractors would regard shotcrete thicknesses of this magnitude as both impractical and uneconomical, preferring to cast concrete arches instead. A strong argument in favor of shotcrete is that it can be placed very close to the face and hence can be used to provide early support in poor quality rock masses. Many contractors would argue that a 50 to 100 mm layer is generally sufficient for this purpose, particularly when used in conjunction with tensioned rock bolts as indicated by Barton et al. (1974), and that the placing of a cast concrete lining at a later stage would be a more effective way to tackle the problem. Obviously, the final choice will depend upon the unit rates for concreting and shotcreting offered by the contractor and, if the shotcrete is cheaper, upon a practical demonstration by the contractor that he can actually place shotcrete to this thickness.

   In North America, the use of concrete or shotcrete linings of up to 2 m thickness would be considered unusual and a combination of heavy steel sets and concrete would normally be used to achieve the high support pressures required in a very poor ground.

Further recommendations on the application of shotcrete in different rock mass conditions are given in Table 10.3.

10.5 DRAINAGE MEASURES

The drainage system should be fully designed before the construction of a tunnel and cavern. NATM (New Austrian Tunnelling Method) and NMT specify drainage measures also. For example, radial gaps are left unshotcreted for drainage of seepage in the case of hard rock mass which is charged with water.

Very often one may observe that the seepage of water is concentrated to only one or just a few, often tubular, openings in fissures and joints. It may be worthwhile to install temporary drainage pipes in such areas before applying the shotcrete. These pipes can be plugged when the shotcrete has gained sufficient strength. Further Swellex (inflated tubular) bolts are preferred in water-charged rock masses. Cement grouted bolts are not feasible here, as grout will be washed out. Resin grout may not also be reliable. It may be mentioned that the seals used in the concrete lining for preventing seepage in the road/rail tunnels may not withstand heavy water pressure.

The pressure tunnels are grouted generally all round its periphery so that the ring of grouted rock mass is able to withstand heavy ground water pressure. Polyurethane may be used as grout in rock joints under water as it swells 26 times and cements the rock mass.

10.6 EXPERIENCES IN POOR ROCK CONDITIONS

Steel fiber reinforced shotcrete (SFRS) has proved very successful in the 6.5 km long tunnel for the Uri Hydel Project and desilting underground chambers of NJPC in Himalaya. The main advantage is that a small thickness of SFRS is needed. No weld mesh is required to reinforce the shotcrete. Provided that the shotcrete is graded and sprayed properly, there is less rebound, thanks to the steel fibers. This method is now economical, safer and faster than the conventional shotcrete. Contour blasting technique is adopted to excavate the tunnel where SFRS is to be used (Section 11.8.7). Further, selection of right ingredients and tight quality control over application are the key to success of SFRS.

Experience with the use of mesh (weld mesh, etc.) has been unsatisfactory when there were overbreaks in the tunnel after blasting. In these cases, soon after the weld mesh was spread between bolts and shotcrete, the mesh started rebounding the shotcrete and it could not penetrate inside the mesh and fill the gap between the mesh and the overbreak. Consequently, gaps were left above the shotcrete; the sound when a hammer was struck indicated the hollow areas above the mesh. Further, loosely fitted welded wire mesh vibrates as a result of blast vibrations, causing subsequent loosening of the shotcrete (Fig. 28.1a).

Because the overall experience with mesh-reinforced shotcrete has been unsatisfactory in handling overbreak situations, it is recommended that mesh with plain shotcrete should not be used where uneven surface of tunnel is available due to high overbreaks. In such cases, the thickness of shotcrete should be increased sufficiently (say by 10 mm).

10.7 CONCLUDING REMARKS

(i) In a poor rock mass, the support capacity of the rock bolts (or anchors) is small in comparison to that of shotcrete and SFRS which is generally the main element of the long-term support system for resisting heavy support pressures in tunnels in weak rock masses.
(ii) The untensioned full-column grouted bolts (called anchors) are more effective than pre-tensioned rock bolts in supporting weak rock masses.
(iii) The design experience suggests that the thickness of SFRS is about half of the thickness of plain shotcrete without reinforcement.
(iv) The SFRS has been used successfully in mild and moderate squeezing ground conditions and tectonically disturbed rock masses with thin shear zones.
(v) The NTM is based on philosophy of NATM to form a load bearing ring all round a tunnel. NTM offers site specific design tables for plain shotcrete and a design chart for SFRS. It is recommended that tunnel engineers should take the benefit of extensive experiences of the past NATM and the modern NTM.

Example 1

In a major hydroelectric project in the dry quartzitic phyllite, the rock mass quality is found to be in the range of 6 – 10. The joint roughness number J_r is 1.5 and the joint alteration number J_a is 1.0 for critically oriented joint in the underground machine hall. The width of the cavern is 25 m, height is 50 m and the roof is arched. The overburden is 450 m. Suggest a design of support system.

The average rock mass quality is $(6\times10)^{1/2} = 8$ (approx). The overburden above the crown is less than $350\ (8)^{1/3} = 700$ m. Hence the rock mass is non-squeezing. The correction factor for overburden $f = 1 + (450-320)/800 = 1.16$. The correction for tunnel closure $f' = 1.0$. Short-term support pressure in roof from equation (5.6) is ($f'' = 1$)

$$= (0.2/1.5)(5 \times 8)^{-1/3}\ 1.16 = 0.045\ \text{MPa}$$

Short-term wall support pressure is

$$= (0.2/1.5)(5 \times 2.5 \times 8)^{-1/3}\ 1.16 = 0.033\ \text{MPa (practically negligible)}$$

Ultimate support pressure in roof from equation (5.10) is given by

$$p_{\text{roof}} = (0.2/1.5)(8)^{-1/3}\ 1.16 = 0.077\ \text{MPa}$$

Ultimate wall support pressure is (see Section 5.6) given by

$$p_{\text{wall}} = (0.2/1.5)(2.5 \times 8)^{-1/3}\ 1.16 = 0.057\ \text{MPa}$$

The modulus of deformation of the rock mass is given by equation (5.13),

$$E_d = (8)^{0.36}(450)^{0.2} = 7.0\ \text{GPa}$$

The ESR is 1.0 for important structures. Fig. 10.2 gives the following support system in the roof,

Bolt length $= 6\,\text{m}$
Bolt spacing $= 2.2\,\text{m}$
Thickness of SFRS $= 90\,\text{mm}$

Fig. 10.2 is also useful in recommending the following wall support system of the cavern ($\text{Q} = 2.5 \times 8 = 20$, $\text{ESR} = 1$, $H = 50\,\text{m}$)

Bolt length $= 11\,\text{m}$
Bolt spacing $= 2.5\,\text{m}$
Thickness of SFRS $= 70\,\text{mm}$

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11

Blasting for tunnels and roadways*

"An approximate solution to the right problem is more desirable than a precise solution to the wrong problem."

U.S. Army (1971)

11.1 INTRODUCTION

Excavations of mine roadways, drifts and tunnels are common features in mining and civil engineering projects. In the absence of initial free face, solid blasting method is employed for excavation of tunnels, drifts and mine roadways, which have many similarities in configurations and in different cycles of operation followed during excavation. Henceforth, for convenience, such blasting will be termed as tunnel blasting. Extensive knowledge has been gained in mining which is relevant to the tunnelling.

In tunnelling, a greater proportion of world's annual advance is still achieved by drilling and blasting. While suffering from the inherent disadvantages of damaging the rock mass, drilling and blasting has an unmatched degree of flexibility and can overcome the limitations of machine excavations by tunnel boring machine (TBM) or road headers. In spite of no major technical breakthrough, the advantages like low investment, availability of cheap chemical energy in the form of explosives, easy acceptability to the practicing engineers, the least depreciation and wide versatility have collectively made the drilling and blasting technique prevail so far over the mechanical excavation methods. The trend seems to continue in the near future, specially in the developing nations.

Blasting for tunnelling is a difficult operation involving both skill and technique. Since tunnels of different sizes and shapes are excavated in various rock mass conditions, appropriate blast design including drilling pattern, quantity and type of explosive, initiation sequence is essential to achieve a good advance rate causing minimal damage to the surrounding rock mass. The cost and time benefit of the excavation are mostly decided by the rate of advance and undesired damage. Though faster advance at the minimum cost remains a general objective of tunnel blasting, the priorities among the various

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elementary tunnel blast results (such as the specific explosive consumption, the specific drilling, the pull and the overbreak/underbreak) may vary from site to site depending on the rock conditions. A trend has been set worldwide not to consider the blast results in isolation but in totality with due consideration to the priorities. Decisions to modify the blast pattern or tunnel configuration may be undertaken with this view to optimize the blasting practice.

Excavation of tunnels, except in geologically disturbed rock mass conditions, is preferred with full face blasting. It is common to excavate large tunnels of 80–90 m^2 cross section in sound rock masses by full face in a single round. However, tunnels larger than 50 m^2 cross-sectional area driven through incompetent ground condition are generally excavated in smaller parts and frequently applying the New Austrian Tunnelling Method (NATM) and the Norwegian Tunnelling Method (NTM), as discussed in Chapters 9 and 10, respectively. Drilling jumbos are finding increasing application to replace conventional jack hammer drilling, bringing the advantages of reduced drilling time and better precision.

Introduction of electro-hydraulic jumbo drills with multiple booms, non-electric initiation system, small diameter explosives for contour blasting and fracture control blasting are some of the recent developments in tunnel blasting. Prediction and monitoring the blast damage, application of computers in drilling, numerical modelling for advanced blast design, use of rock engineering systems for optimization and scheduling of activities have been the areas of intense research in today's competitive and high-tech tunnelling world. System approach is being applied nowadays in tunnel blasting to make it more scientific, precise, safe and economic.

In tunnel blasting, explosives are required to perform in a difficult condition, as single free face (in the form of tunnel face) is available in contrast to bench blasting where at least two free faces exist. Hence, more drilling and explosives are required per unit volume of rock to be fragmented in the case of tunnel blasting. A second free face, called "cut", is created initially during the blasting process and the efficiency of tunnel blast performance largely depends on the proper development of the cut. The factors influencing the development of the cut and the overall blast results are dependent on a host of factors involving rock mass type, blast pattern and the tunnel configurations. The results are often found to be below par when a blast pattern is designed with scant regard to the rock mass properties.

11.2 BLASTING MECHANICS

The tunnel blasting mechanics can be conceptualized in two stages. Initially, a few holes called cut holes are blasted to develop a free face or void or cut along the tunnel axis. This represents a solid blasting condition where no initial free face is available. Once the cut is created, the remaining holes are blasted towards the cut. This stage of blasting is similar to bench blasting but with larger confinement. The results of tunnel blasting depend primarily on the efficiency of the cut hole blasting. The first charge fired in cut resembles crater blasting where there is only one free face in the vicinity of the charge (Fourney, 1993). Livingston's spherical charge crater theory (Livingston, 1956) suggests that the blast induced fracturing is dominated by explosion gas pressure which is supported by Liu and Katsabanis (1998). Since then, a series of crater blasting experiments were conducted and different concepts have been reported. Duvall and Atchison (1957), Wilson (1987) and others believe that the stress wave induced radial fracturing is the dominating cause of blast fragmentation and gas pressure is responsible only for extension of the fractures developed by the stress wave. Simha et al. (1987) and Hommert et al. (1987) have similar views.

The natures of influence of the two pressures i.e., of stress and gas are different in the jointed rock mass where the stress waves are useful in fragmentation as the joints restrict the stress wave propagation. The gases, on the other hand, penetrate the joint planes and try to separate the rock blocks. The fragments' size and shape in jointed formations are dominated by the gas pressure and the joint characteristics. Forsyth (1993) and Hagan (1995) supported this concept. The experience in the footwall side blasting of Dongri Buzurg manganese opencast mine of Manganese Ore India Ltd. supplements this view. A poor fragmentation was observed due to open joints which resisted the propagation of stress waves but favored wedging through the joints by gas pressure (Chakraborty et al., 1995b).

The roles of the stress wave and the gas pressures are no different in the second stage of tunnel blasting. But with the availability of free face, the utilization of stress wave is increased. The rock breakages by rupturing and by reflected tensile stress are more active in the second stage because of cut formation in the first stage.

11.3 BLAST HOLES NOMENCLATURE

The nomenclature of blast holes in different parts of a tunnel section are shown in Fig. 11.1 (Rustan, 1998).

The back holes and the side or rib holes will be referred as contour holes hereinafter. All the holes except the contour holes are called as production holes.

The firing sequence in tunnel blasting is based on the following principles:

(i) Progressive enlargement of free face and firing of holes towards the maximum free face.
(ii) Creation of free face towards the bottom of tunnel section so that the maximum stoping can be done with favor of the gravity. However, the free face may be positioned towards the middle or the top of the tunnel section to reduce the muck tightness.
(iii) Maximum free face is made available before the contour holes are blasted so that the minimum explosive quantity is required to break rock at the contour and the blast-induced damage is restricted to a bare minimum.

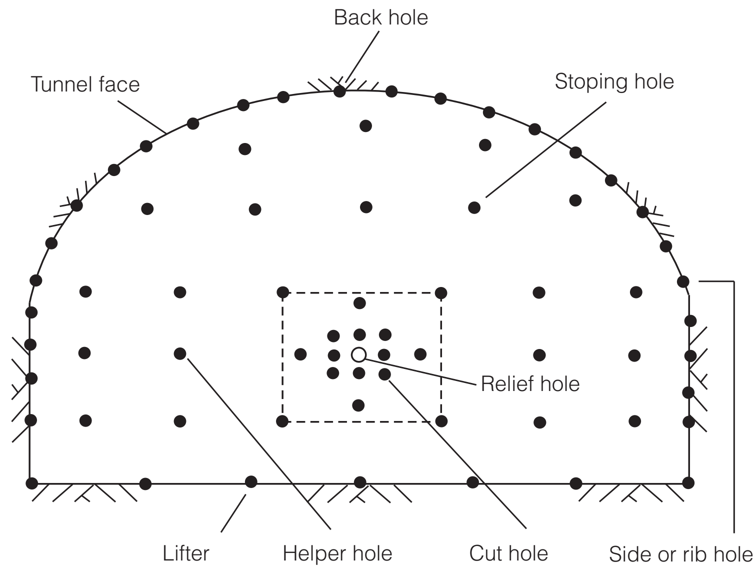

Fig. 11.1 Nomenclature of blast holes in different parts of a tunnel section (Rustan, 1998).

11.4 TYPES OF CUT

The cut can be created by blasting pairs of holes drilled at an angle to the tunnel axis. These cuts are called inclined, convergent or angled cuts and hereinafter will be referred as convergent cuts. Wedge cut or V-cut and fan cut are the most common of all the convergent cuts. However, the problem with the convergent cuts is that the depth of cut is always reduced because of angle and thus the pull is partially lost. Furthermore, the maximum drilling depth in a convergent cut is dependent on the width of the opening. Hence, not more than 50–60 percent of the tunnel width can be pulled with the convergent cut.

Sometimes, a cut is created by drilling a number of blast holes parallel to tunnel axis around one or more empty holes, called relief holes. These holes generally vary from 56 to 100 mm in diameter. Such a cut, called a parallel hole cut, will be referred hereinafter as a parallel cut. Burn cut, spiral cut and four-section parallel cut are the commonly used parallel cuts. Hagan (1980) suggested that the relief hole depth should be 0.1–0.15 m more than that of the blast holes. The void space provided by the relief holes should be at least 15 percent of the cut volume to accommodate swelling due to fragmentation (Hagan, 1980; Singh, 1995). The void requirement is less in hard, brittle and unfractured formations and more in weak and fissured ones. The formations having larger bulking factor are subjected to more of parallel cut failures because of larger void area requirements for the expulsion of the fragmented materials. The intial holes in the cut should be fired on separate delays of 100 ms or more for progressive relief of burden. The parallel cut rounds are often accompanied by sympathetic detonation, dynamic pressure desensitization and freezing. Further, the drilling accuracy plays a major role in the success of a parallel cut blasting and its importance increases in deeper rounds. To counteract the drilling error, the blast holes are generally overcharged. Templates or computer controlled jumbo drilling are used to minimize the deviations. Langefors and Kihlstrom (1973) and Hoek and Brown (1980) brought out various scopes and advantages with different types of convergent and parallel cuts.

The void space provided by the relief holes in various rock types and the pull obtained in some tunnels and drivages are listed in Table 11.1.

It is evident from Table 11.1 that larger void space is required if the formation has greater RQD or P-wave velocity (V_p).

The main differences between the parallel and convergent cuts are:

(i) The cut depth is always lower in a convergent cut than in a parallel cut as the holes are drilled at an angle in the convergent cut. Hence, a smaller pull is obtained with a convergent cut.
(ii) The advance per round in a parallel cut is designed mainly on the basis of the relief hole size, whereas, the same in a convergent cut is decided by the tunnel size.
(iii) Maintaining proper hole angle is more difficult in a convergent cut.
(iv) The cut holes in a parallel cut need to be placed very close to each other. Hence, there is a possibility of joining the holes at the bottom, if the deviation cannot be controlled.
(v) Throw in a parallel cut blasting is directed towards the relief hole side. Hence, the muck is thrown to a lesser distance. Moreover, the fragments collide due to the movements in opposite directions and generate more fines.

Both the parallel cut and the wedge or the convergent cut were practised in coal development galleries of Saoner mine, inclined drifts of Tandsi Coal Project and developmental works in the host rock and ore body in Chikla Manganese mine, India (Chakraborty, 2002). Conventionally, a decrease was observed in the specific charge and the specific drilling while the cut was extended to the full face. It was interesting to note

Table 11.1 Voids provided in parallel cut blasting in different tunnels (Chakraborty, 2002).

S. No.	Formation	q_c (MPa)	RQD	V_p (m/s)	Void by relief holes in percent of cut area	Pull (%)
1	Coal	23	Closely jointed, 10 (approx)	954	6–7	0.88
2	Rock 1	29.9	36	2800	8.7	0.75
3	Rock 2	18–46	40.75–91.4	2910–7690	10	0.75

that in all the cases, the rate of reduction in both the specific charge and the specific drilling with the parallel cuts were higher than those with the convergent cuts. Further, the reduction trend lines for the parallel and the convergent cuts in each tunnel, intersected when the tunnel area was nearly 25 m^2. Subsequently, it was concluded that a parallel cut should be economically viable when the tunnel area exceeds 25 m^2 and a convergent cut should be economical if the tunnel area was less than 25 m^2.

11.5 TUNNEL BLAST PERFORMANCE

The tunnel blast performance is generally measured in terms of one or more than one of the following blast parameters:

(i) Pull (face advance/depth of round), expressed in percent,
(ii) Specific charge (kg of explosive/m^3 or tons of yield),
(iii) Specific drilling (m of drilling/m^3 or tons of yield), or detonator or hole factor (number of holes/m^3 or tons of yield) and
(iv) Blast-induced rock mass damage and overbreak/underbreak.

The blast-induced damage is measured radially and is expressed in meter. The overbreak and the underbreak are often measured volumetrically in m^3 of in situ rock mass. These may be expressed in percent of the designed volume. However, in many projects, the permissible limit of overbreak has been defined in terms of width and height of tunnel. The Swiss Society of Engineers and Architects defines the permissible overbreak limit as $0.07\sqrt{A}$, where A is the tunnel area or 0.4 m whichever is less (Innaurato et al., 1998).

All the above mentioned blast performances jointly contribute to the safety, progress rate and economy of the tunnelling operation.

11.6 PARAMETERS INFLUENCING TUNNEL BLAST RESULTS

The parameters influencing the tunnel blast results may be classified in three groups:

(i) Non-controllable – Rock mass properties,
(ii) Semi-controllable – (a) Tunnel geometry,
(b) Operating factors and
(iii) Controllable – Blast design parameters including the explosive properties.

11.6.1 Rock mass properties

The results of rock blasting are affected more by rock properties than by any other variables (Hagan, 1995). As the mean spacing between the joints, fissures or the cracks decreases, the importance of rock material strength decreases while that of the rock mass strength increases. The blasts are required to create many new cracks in a rock mass with widely spaced joints. In a closely fissured rock mass, on the other hand, generation of new cracks is not needed and the fragmentation is achieved by the explosion gas pressure which opens the joints to transform a large rock mass into several loose blocks. The tunnel blasting efficiency is affected to a lesser degree by other rock mass properties like the internal friction, grain size and porosity. Jorgenson and Chung (1987) and Singh (1991) opined that the blast results are influenced directly by the overall rock mass strength. Chakraborty et al. (1998b) suggested strength ratings (SR) based on the uniaxial compressive strength (UCS) of rock to correlate the specific charge in a tunnel. Interestingly, it is experienced that the influence of strength on the specific charge is comparatively lower at the higher strength values.

Ibarra et al. (1996) observed in a tunnel that Barton's rock mass quality (Q) and specific charge in the contour holes have significant effect on overbreak. Chakraborty et al. (1996a) reported some typical observations in the tunnels of Koyna Hydroelectric Project, Stage IV, Maharashtra, India, through Deccan trap formations consisting of compact basalts, amygdolaidal basalts and volcanic breccias. Poor pull and small overbreak were observed in volcanic breccia having low Q value, P-wave velocity and modulus of elasticity. On the other hand, large overbreak on the sides due to vertical and sub-vertical joints and satisfactory pull were reported in the compact basalts having comparatively much higher Q value, P-wave velocity and modulus of elasticity. The fact is attributed to the presence of well-defined joints in compact basalts which is absent in volcanic breccia.

The effects of joint orientations on overbreak/underbreak and pull in heading and benching operations during tunnel excavations are explained by Johansen (1998) in Figs 11.2 to 11.6.

A) Heading

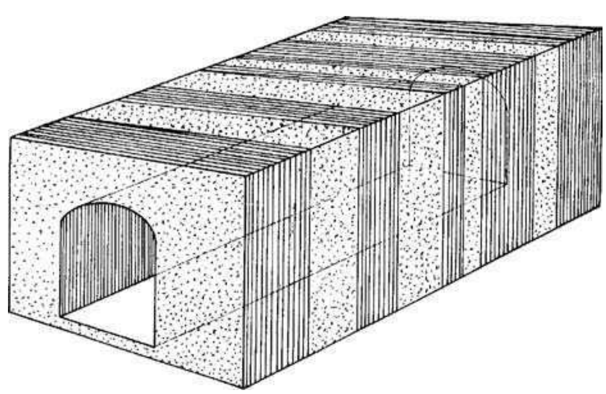

Fig. 11.2 Joints normal to tunnel direction favorable for good pull (Johansen, 1998).

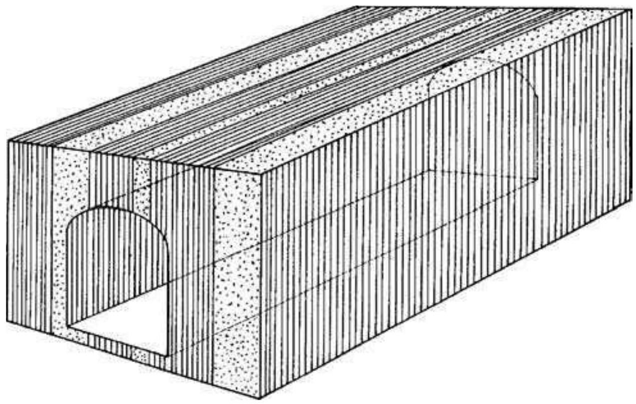

Fig. 11.3 Poor advance with joints striking parallel to tunnel advance direction (Johansen, 1998).

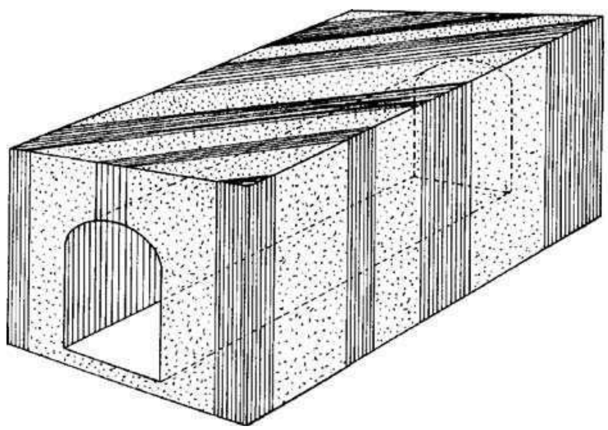

Fig. 11.4 Right side wall more prone to breakage due to obtuse angle between joints and tunnel direction (Johansen, 1998).

B) Benching

The dip direction of the blasted strata on pull could be well experienced while blasting in the development faces of Saoner coal mine where the pull was increased by 11 percent in the rise galleries compared to that in the dip galleries (Chakraborty, 2002).

Longer rounds in tunnels can be pulled when the dominant joint sets are normal to the tunnel axis as shown in Fig. 11.2. Whereas, better pull can be obtained in shaft sinking if the discontinuities are parallel to the line joining the apex of the Vs in a V-cut, Hagan (1984).

Chakraborty (2002) observed the following influences of joint directions on pull and overbreak (Table 11.2).

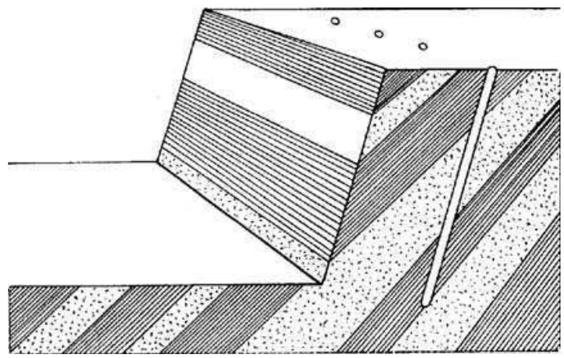

Fig. 11.5 Bench blasting with joints dipping towards the free face (Johansen, 1998). Advantages: Good forward movement of muck and reduced toe; Disadvantages: Large backbreak, poor contour and slope control problems.

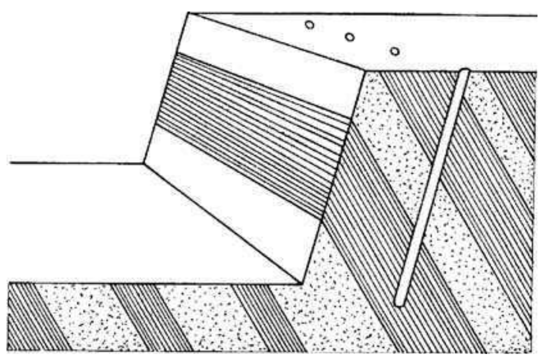

Fig. 11.6 Bench blasting with joints dipping away from the free face (Johansen, 1998). Advantages: Small backbreak; Disadvantages: Restricted forward movement, tight muck pile and increased toe.

Table 11.2 Influence of joint direction on overbreak (Chakraborty, 2002).

Joint orientation			
Dip	Strike with respect to tunnel axis	Face advance	Roof overbreak
Steep	Parallel	Very poor	Very small
Steep	Across	Very good	Very large
Gentle	Across	Fair	Large
Moderate	Across/oblique	Good	Small

The gentle, moderate and steeply dipping joint planes signify the dip angles as 0–30°, 30–60° and 60–90°, respectively. Similarly, strikes with respect to tunnel axis are mentioned as parallel, oblique and across to indicate that the joint strike intersection angle with the tunnel axis as 0–30°, 30–60° and 60–90° respectively.

If the geo-mechanical properties of the constituting formations of a tunnel are quite different, the stress energy utilization and resulting fragmentation are adversely affected. Chakraborty et al. (1996b) suggested an increase of specific charge by a percent equal to ten times the number of contact surfaces of various constituting rock formations.

11.6.2 Tunnel size

The specific charge and the specific drilling in tunnel blasting are inversely proportional to the tunnel cross-sectional area (Figs 11.7 and 11.8). It is apparent from these figures that the reduction rates of these blast results are intensely steep in small tunnels of cross-sectional area up to 10 m^2 and utterly mild in large tunnels exceeding 35 m^2 size.

Based on specific charge and specific drilling performances, tunnels can be classified into three categories in respect of their cross-sectional area as shown in Table 11.3.

As specific charge and specific drilling are less in large tunnels it may be more advantageous to construct a large tunnel, in the absence of ground control problems, instead of a number of small tunnels on blasting cost consideration.

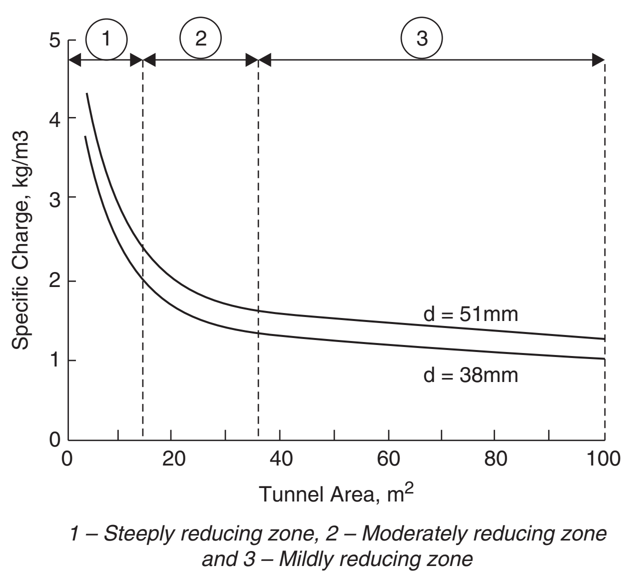

Fig. 11.7 Specific charge vs. tunnel area (Olofsson, 1988) [d = blast hole diameter].

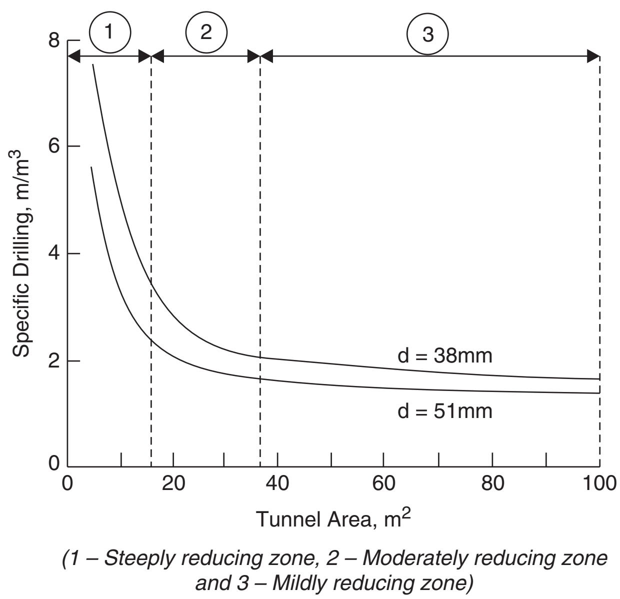

Fig. 11.8 Specific drilling vs. tunnel area (Olofsson, 1988) [d = blast hole diameter].

Table 11.3 Size-wise classification of tunnels.

S. No.	Tunnel area (m^2)	Type	Predicted specific charge and specific drilling
1	<15	Small	High
2	15–35	Medium	Moderate
3	>35	Large	Low

11.6.3 Tunnel depth

The rock masses at a great depth remain under high stress, which needs to be overcome by increased blasting energy for satisfactory fragmentation. Calder and Bauer (1983) considered the in situ stress for design of blast hole spacing in pre-split blasting. Chakraborty (2002) observed in Tandsi Project, India, that the blast performances did not show any significant variation with the increase in overburden up to a moderate height of 236 m.

11.6.4 Blast hole deviation

Blast hole deviation may occur due to poor collaring, improper alignment and wrong trajectory direction. It plays a major role in deciding the tunnel blast results. Improved drilling precision also provides better contour blasting performance. Langefors and

Kihlstrom (1973) reported that the explosive efficiency in tunnel blasting might be reduced by 50–75 percent on account of blast hole deviations leading to shorter pull.

Deviation is common in deep blast holes because of long drill rods, which are more flexible. In such cases, heavier drill rods are suggested. The following parameters influence the blast hole deviation:

(i) Drilling parameters	a) diameter, b) length and c) inclination
(ii) Drilling equipment	a) condition and type of drilling machine, b) condition of drill string and couplings and c) type, condition and shape of drill bit
(iii) Drill operation parameters	a) thrust and torque, b) rotary speed, c) rate of penetration and d) flushing medium
(iv) Rock characteristic	a) structure and strength variation
(v) Operator's skill	a) training, b) experience, c) skill and d) care

Deeper parallel cut holes in Saoner coal mine of Western Coalfields Ltd., India, got connected with the nearby relief holes because of deviations and therefore the pull could not be improved more than 1.6 m. Heavier drill rods were used to solve the problem. The need of numerically controlled jumbos, was strongly felt there. To maintain the same pull in the link tunnel of Koyna Hydroelectric Project, Stage IV, the specific charge was increased by 6 percent when the hole depth was increased from 3 to 4 m leading to greater deviation.

Hole deviation is highly detrimental to the blast performances in small tunnels. Chakraborty et al. (1998b) considered the deviation as a function of hole length for the specific charge and the specific drilling predictions in small tunnels with both wedge and parallel cuts.

11.7 MODELS FOR PREDICTION OF TUNNEL BLAST RESULTS

11.7.1 Specific charge

Pokrovsky (1980) suggested the following empirical relation to determine the specific charge (q) in tunnels (equation (11.1)):

$$q = q_1 \cdot s_t \cdot f \cdot s_{wr} \cdot d_{ef} \text{ kg/m}^3 \tag{11.1}$$

where

q_1 = specific charge for breaking the rock against a free face in kg/m^3,
= 1.5–1.2 if Protodyakonov Index is 20–15
= 1.2–1.0 if Protodyakonov Index is 15–10
= 1.0–0.8 if Protodyakonov Index is 10–8
= 0.8–0.6 if Protodyakonov Index is 8–4

$= 0.6$–0.2 if Protodyakonov Index is 4–2
$= 0.15$ if Protodyakonov Index is <2
s_t = factor for structure and texture of rock,
$= 2$ for resilient, elastic and porous rock,
$= 1.4$ for dislocated and irregular rock and
$= 1.3$ for shale and bedding planes at normal to blast holes,
f = rock confinement
$= 6.5/\sqrt{A}$, (11.1a)
A = area of tunnel (m^2),
s_{wr} = relative weight strength of explosive (ANFO = 1) and
d_{ef} = factor for diameter of explosive cartridge,
$= 1.1$ for 25 mm cartridge,
$= 1.0$ for 30 mm cartridge and
$= 0.95$ for 40 mm cartridge.

According to Langefors and Kihlstrom (1973), the specific charge (q) is related to the cross-sectional area of the tunnel (A, m^2) as given below:

$$q = \left(\frac{14}{A}\right) + 0.8\,\text{kg/m}^3 \tag{11.2}$$

The specific charge in the cut holes remain maximum and it can be upto 7 kg/m^3 in a parallel cut.

11.7.2 Rock mass damage

The aspects of blast-induced rock mass damage around a tunnel opening and its assessment have been the subjects of in-depth research for quite a long time. The type of damage can be grouped into three categories: (i) fabric damage due to fracturing, (ii) structural damage exploiting discontinuities and shears and (iii) lithological damage causing parting between two different rock units or lithological boundaries between similar rock types.

Engineers International Inc. modified basic RMR (MBR) considering blast-induced damage adjustments, as shown in Table 11.4, were suggested for planning of caving mine drift supports (Bieniawski, 1984). Chapter 4 in this book defines basic RMR.

Table 11.4 Blast damage adjustments in MBR (After Bieniawski, 1984).

Method of excavation	Damage level	Blast damage adjustment factor	Percent reduction
1. Machine boring	No damage	1.0	0
2. Controlled blasting	Slight	0.94–0.97	3–6
3. Good conventional blasting	Moderate	0.9–0.94	6–10
4. Poor conventional blasting	Severe damage	0.9–0.8	10–20

Laubscher and Taylor (1976), on the basis of their experience in 50,000 m of mining development, suggested following reductions in rock quality designation (RQD) and joint condition rating due to blasting for use in RMR (Bieniawski, 1976) determination:

(i) Machine boring: No reduction
(ii) Smooth blasting: 3 percent
(iii) Good conventional blasting: 6 percent
(iv) Poor conventional blasting: 20 percent.

Various methods suggested and used globally for blast-induced rock mass damage assessment are given by Scoble et al. (1996).

Ouchterlony et al. (1991) observed that the damage zone could be to the extent of 0.5 m with cautious tunnel blasting. McKenzie (1994) related the threshold peak particle velocity, PPV, (v_{max}) for incipient fracture with uniaxial tensile strength (q_t), Young's modulus and P-wave velocity (V_p, m/s) as shown below:

$$v_{max} = \frac{q_t \cdot V_p \cdot 10^{-3}}{E} \text{ m/s} \tag{11.3}$$

where

q_t = uniaxial tensile strength, MPa,
V_p = P-wave velocity, m/s and
E = Young's modulus, GPa.

Pusch and Stanfors (1992) and others observed that the minimum disturbance by blasting is reported when the tunnel orientation was within 15° with the strike of the joint sets.

Yu and Vongpaisal (1996) concluded that the damage is a function of blast-induced stress and rock mass resistance to damage. They proposed blast damage index (D_{ib}), as shown in equation (11.4), to estimate the type of damage due to blasting. It is the ratio of the blast-induced stress to the resistance offered against damage.

$$\begin{aligned} D_{ib} &= \text{Induced stress/damage resistance} \\ &= \frac{v_R \cdot \rho_r \cdot V_p}{k_s \cdot q_{td}} \times 10^{-6} \end{aligned} \tag{11.4}$$

where

v_R = vector sum of peak particle velocity, m/s,
ρ_r = rock mass density, kg/m^3,
k_s = site quality constant (maximum 1.0) obtained either from
(i) RMR with support adjustments, divided by 100 or
(ii) ground condition rating by sounding the ground with scaling bar with support adjustments (Tables 11.5 and 11.6),

q_{td} = dynamic tensile strength of rock mass, MPa and
V_p = P-wave velocity, m/s.

Various blast damage indices suggested by them are given in Table 11.7.

Chakraborty et al. (1998c) also suggested half cast factor (HCF) for different D_{ib} values to facilitate damage estimation from the visible half drill hole marks on tunnel walls.

Table 11.5 Ground condition rating by sounding (Yu & Vongpaisal, 1996).

Estimated by sounding	Ground condition	Rating
Sounds solid with scaling bar	Very good	0.9–1.0
Sounds solid after scaling	Good	0.7–0.9
Sounds drummy in places	Fair	0.5–0.7
Sounds drummy mostly	Poor	0.3–0.5
Loose rocks mostly	Very poor	0–0.3

Table 11.6 Rating adjustments on account of support (Yu & Vongpaisal, 1996).

Type of support	Adjustments (%)
Grouted cable bolting	4
Cable lacing, shotcreting, screening, grouted rebar	3
Bolting	2

Table 11.7 Blast damage index (D_{ib}) and type of damage (Yu & Vongpaisal, 1996).

D_{ib}	Type of damage
<0.125	No damage to underground excavations
	Maximum allowable value for key permanent workings like crusher chambers, shafts, permanent shops, ore bins, pump houses, etc.
0.5	Minor and discrete scabbing effects
	Maximum tolerable value for intermediate term workings, e.g., main drifts, main haulage ways, etc.
0.75	Moderate and discontinuous scabbing damage
	Maximum tolerable value for temporary workings, e.g., cross-cuts, drill drifts, stope access, etc.
1.0	Major and continuous scabbing failure, requiring intensified rehabilitation work
1.5	Severe damage to entire opening, causing rehabilitation work difficult or impossible
2.0	Major caving, normally resulting in abandoned access

The D_{ib} was used to assess rock mass damage around horizontal tunnel no. I in Lake Tap work of Koyna Hydroelectric Project, Stage IV, India, passing under water body.

11.7.3 Throw, muck profile and fragmentation

The loading efficiency is largely dependent on the degree of fragmentation and the muck profile. Specially, a tight muck pile affects the loading efficiency quite adversely. Factors like rock mass properties, charge parameters, initiation type and sequence along with spacing to burden ratio are known to influence the muck profile and fragmentation substantially. The control of geological factors, like pre-blast block size, on fragmentation has been clearly recognized over the years. Pre- and post-blast block size analyses have been used by many to assess the explosive energy utilization. Digital image analysis systems are currently used for such a purpose. The swell of the muck pile is also a function of fragmentation.

Throw in a tunnel blast largely depends on the specific charge, the blast hole directions, the delay intervals, the placement of the cut holes and the type of initiation. Smaller throw is reported with parallel cuts than with convergent cuts. Placement of the cut holes at the lower part of the tunnel provides lesser throw but larger fragments. The reason is that the gravitational force favors the burden movement in this case.

Short delay detonation produces a higher throw than long delay detonation. A shorter delay does not provide sufficient time for the burden to move and create space to accommodate the fragments generated in the successive delays.

11.7.4 Holistic models

Chakraborty (2002) developed models for blast results predictions based on a holistic approach. Different parameters influencing the blast results were subjected to multiple linear regression analyses to select the following seven parameters dominating the blast results.

(A) Rock mass parameters –
- (i) P-wave velocity (V_p, m/s),
- (ii) Number of contact surfaces in multiple geological conditions (n) and
- (iii) RQD.

(B) Tunnel configuration – parameters
- (i) Area A (m^2) and
- (ii) Inclination with respect to vertical upward direction (β_i, expressed in radian "r").

(C) Blast design parameters –
- (i) Cut hole angle with the face (C_α, expressed in cotangent of the value) and
- (ii) Coupling ratio between the blast hole and the explosive cartridge (R_c).

An index to characterize the tunnel blasting environment, named tunnel blasting index (TBI), was defined by the above influencing parameters as follows:

$$\mathrm{TBI} = \frac{\text{Rock mass factor (RF)}}{\text{Tunnel configuration factor (TF)} \times \text{blast design factor (BF)}} \tag{11.5}$$

where

$$\mathrm{RF} = V_{\mathrm{p}} + n + \left(\frac{\mathrm{RQD}}{10}\right), \tag{11.5a}$$

$$\mathrm{TF} = A - r, \text{ and} \tag{11.5b}$$

$$\mathrm{BF} = C_{\alpha} + R_{\mathrm{c}} \tag{11.5c}$$

The specific charge (q) and the specific drilling (b_{s}) observed during field investigations could be well related with TBI.

11.8 BLAST DESIGN

11.8.1 Depth of round

The depth of a round is an important parameter in tunnel blasting, as most of the excavation engineers desire a higher tunnelling rate. There are two options for obtaining a high drivage rate. The first one is to go for a deeper round, which may invite more strata control problems unless the ground is competent or smooth blasting practice is followed. Much unproductive time can be saved in this option because the number of rounds is reduced. The second option is based on pulling shorter rounds with smaller cycle time. This becomes a useful option, particularly when the rock mass is weak. A better pull efficiency is also expected in the second option. Therefore, it may be wise to generate a database on case to case basis for a limited period to finally decide the optimum advance per round. The advance per round in the weak rock mass is designed so that erection of support in the freshly exposed zones can be completed within the bridge action period (stand-up time), which is defined by the time interval between the blast and the first fall of roof and is dependent on the rock type, span of the excavation, support system and blasting quality (Fig. 4.1). However, the drilling resources is the dominating factor in deciding the advance per round in most of the tunnels in India.

A rough guideline on the length of blast holes in the cut and easer holes of a convergent cut is provided by Pokrovsky (1980),

$$\text{for cut holes,} \quad l_{\mathrm{c}} = 0.75\,(A)^{0.5}\ \mathrm{m} \tag{11.6}$$

$$\text{for easer holes,} \quad l_{\mathrm{e}} = 0.5\,(A)^{0.5}\ \mathrm{m} \tag{11.7}$$

where,

A = tunnel area, m^2,
l_c = length of cut hole, m and
l_e = length of easer hole, m.

According to Holmberg (1982), the depth of a round in a parallel cut depends on the size of the relief hole (equation (11.8)) as given by the following relation:

$$A_d = 0.15 + 34.1\, d_r - 39.1\, d_r^2 \text{ m} \tag{11.8}$$

where, A_d is the depth of round (m) and d_r is the diameter (m) of the relief hole.

In case of more than one relief hole of similar size, the equivalent relief hole diameter (d_{re}, m) should be considered for estimating the round depth. It is obtained by multiplying the relief hole diameter by $\sqrt{n_r}$, where n_r is the number of relief holes (Olofsson, 1988).

The depth of a round designed for drifts of various cross-sectional area in Germany is presented by Ziegler (1985) in Fig. 11.9.

11.8.2 Holes per round

The number of holes per round is decided mainly by the tunnel size and hole diameter. Ziegler (1985) reports that the number of holes per round in a drift reduces by 3 percent with every 0.001 m increase in diameter of the explosive cartridge, and the hole concentration is greater in small openings compared to large ones.

Based on US experience, Whittaker and Frith (1990) suggested the number of holes for various tunnel sizes in weak and strong formations (Table 11.8).

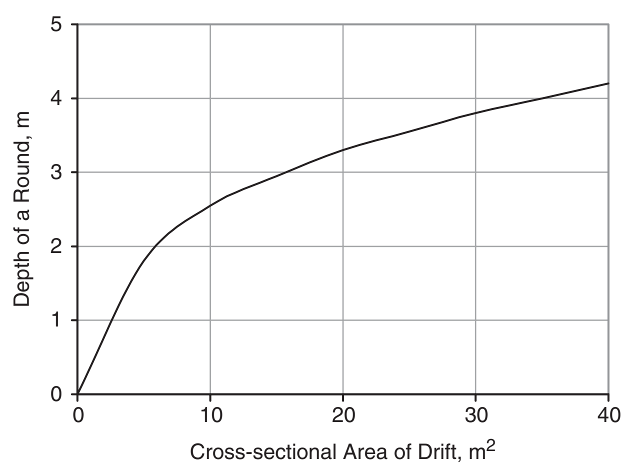

Fig. 11.9 Depth of a blast round vs. tunnel size in Germany (Ziegler, 1985).

Table 11.8 Number of holes per round (Whittaker & Frith, 1990).

	Number of holes per round	
Tunnel cross section (m^2)	Weak	Strong
10	23–27	36–50
25	45–50	60–70
50	75–85	95–110

11.8.3 Explosives and accessories

It is a known fact that more the characteristic impedance, defined as the product of density and velocity of sound wave in any medium, of rock match with that of explosive, better is the explosive energy utilization. Notwithstanding this guideline, the use of nitroglycerine (NG) explosives is common in tunnels as these are very powerful and provide good fragmentation in spite of the fact that large amount of noxious gas is generated from it leading to long defuming time. Kate (1994) from his experience in the Konkan Railway tunnels, reported that the defuming time with slurry explosive was only 25 percent with NG 90 percent explosives. Presently, manufacturing of the NG explosives is discontinued in India.

The development of ready-to-use pipe charges of various lengths has resulted in fast charging of holes. Various unconventional charges have been explored which are nowadays available in global market for contour blasting. Lopez et al. (1995) and others report that charges of 11, 17 and 22 mm diameter cartridges, called Gurit or Nabit, are commonly used for this purpose. Sometimes, such low density cartridges are surrounded by detonating cord to initiate firing. Recently, high core load detonating cord of 0.04–0.1 kg/m charge concentrations is available for contour blasting. As the total charge is distributed in the detonating cord, the stress concentration is reduced to facilitate smoother contour. Ammonium nitrate fuel oil (ANFO) diluted by common salt is also suggested as low energy explosives for this purpose. The polystyrene beads with a density of 30 kg/m^3 is also used to reduce the explosive density. A mixture of ANFO and polystyrene beads, brings down the blast hole pressure by 1/12 times with ANFO alone. The use of polystyrene beads is a common practice in Australia since 1975 (Wilson & Moxon, 1988). However, the use of ANFO is not quite common in tunnelling due to defuming problems. Adhikari and Babu (1994) and others suggested the use of bamboo spacers for longitudinal decoupling and distribution of explosive cartridges.

Blasting technology has achieved a significant development with the introduction of non-electric detonators known as NONEL system which was innovated and developed by Nitro Nobel (Olofsson, 1988). In order to meet the specific requirements of tunnel blasting, Nonel GT/T developed by Nitro Nobel has a delay range of 25–6000 ms. Such a vast range is not available with conventional detonating system.

The velocity of detonation (VOD, m/s) of the explosives generally increases with the increase in charge diameter. But no significant increase in VOD was observed if the diameter is more than 150, 125, 75 and 30 mm in case of ANFO, slurry, NG explosives and cast booster, respectively. Many a time ANFO cartridges do not yield good results in small diameter holes as the critical diameter of ANFO is 25 mm only.

Long storage has adverse effect on the VOD of explosives, which may be reduced by about 20 and 30 percent for a storage time of 6 months and 12 months, respectively.

11.8.4 Charge per hole

The explosives consumption increases if the angle of breakage is small. The easer holes in a parallel cut are blasted with small breakage angle against the free face created by the cut. Langefors and Kihlstrom (1973) suggested the following relations to estimate the linear charge concentration in a hole breaking towards a narrow opening or free face (circular or rectangular) as shown in Fig. 11.10.

$$\text{(a) Circular opening} - q_{\text{lco}} = \frac{0.55(D_\text{c} - W/2)}{(\sin \gamma_\text{a})^{3/2}} \text{ kg/m} \tag{11.9}$$

$$\text{(b) Rectangular opening} - q_{\text{lro}} = \frac{0.35(D_\text{r})}{(\sin \gamma_\text{a})^{3/2}} \text{ kg/m} \tag{11.10}$$

where

q_{lco} = linear charge concentration in case of a circular opening, kg/m,
q_{lro} = linear charge concentration in case of a rectangular opening, kg/m,
D_c = center to center distance of blast hole from circular opening, m,
D_r = distance of blast holes from rectangular opening,
W = width of the opening, m and
γ_a = half of the aperture angle (°) or angle of breakage.

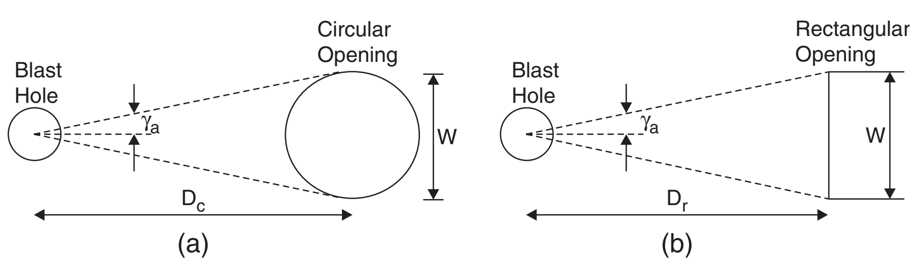

Fig. 11.10 Plan view of blasting towards narrow opening (a) circular and (b) rectangular (Langefors & Kihlstrom, 1973).

Holmberg (1982) provides a detailed design calculation of charge per hole with four-section parallel cut.

The explosion pressure at the blast hole is directly proportional to the coupling ratio raised to the power of 2.4 (Calder & Bauer, 1983). The fall in blast hole pressure due to poor coupling is more with high explosives than with low explosives. The decoupling may result in the channel effect preventing detonation of the charge column.

11.8.5 Burden

Burden is one of the most important parameters to be designed in tunnel blasting. As many as 18 empirical models are compiled by Lopez et al. (1995). Majority of the models consider the blast hole diameter, rock properties like density, compressive strength, seismic velocity and explosive properties like density, velocity of detonation and detonation pressure as the independent variables. Praillet (1980) considered separate constants for rope shovel and dragline for burden estimation.

The burden for rock fracturing without displacement of muck is called as critical burden. A burden of 0.4 to 0.9 times the critical burden gives the most satisfactory pull (Rustan et al., 1985).

Rustan (1990) suggested that the practical burden in underground blasting (B_p, m) is related to the blast hole diameter and the specific gravity of both the rock and the explosive (equation (11.11)). The practical burden is defined as the maximum burden minus the greatest blast hole deviation caused by drilling.

$$B_p = K \cdot d^{0.63} \cdot \left(\frac{\gamma_e}{\gamma_r}\right), \text{ m} \tag{11.11}$$

where

B_p = practical burden, m,
γ_e = specific gravity of explosive,
γ_r = specific gravity of rock,
d = diameter of blast hole, m and
K = site constant.

Spacing to burden ratio (m_d) of holes at different parts of a tunnel face should be different because the confinement gets released gradually as the tunnel is enlarged from the cut holes to the contour holes. The values of m_d for various types of holes in a tunnel is provided in Table 11.9.

11.8.6 Type, delay and sequence of initiation

The delay time must allow the following events to reach completion or, at least, to be well underway before initiation under subsequent delay.

Table 11.9 Recommended spacing to burden ratio (m_d) for different types of holes in a tunnel (Holmberg, 1982).

Type of hole	m_d
Helping and stoping holes	1.25
Lifters	1
Contour holes	0.8

(i) Travel of the compressive waves through the burden to face and back to the blast hole.
(ii) A subsequent readjustment of the initial stress field due to the presence of the primary radial cracks and the effect of the reflection of stress waves from the free face.
(iii) Acceleration of the broken rock mass, by the action of the explosion gases, to a high velocity to ensure the proper horizontal motion which controls the muck pile profile and the digging efficiency.

Fourney (1988) reports that the optimum delay time is much shorter for a layered formation than for a massive formation and coarser fragments are obtained if delay time is not optimum. Though the conventional concept is to go for indirect initiation in tunnel for better fragmentation, but Ester (1998) noticed at Sopac tunnel of Croatia that direct initiation produced more breakage than indirect initiation. The reason was attributed to the fact that breakage started in the least strained part of the blast hole in case of direct initiation leading to better energy utilization.

11.8.7 Contour blasting

Contour blasting in tunnelling is adopted to obtain a smooth tunnel profile and minimize damage to the surrounding rock mass. Despite a large amount of drilling required, it is preferred over conventional blasting because of the following advantages:

(i) The shape of the opening is maintained with smooth profile.
(ii) Stability of the opening and the stand-up time of the tunnel are improved.
(iii) Support requirement is reduced.
(iv) Overbreak is reduced to minimize unwanted excavations and filling to bring down the cost and cycle time.
(v) Ventilation improves due to smooth profile.

The performance of contour blasting is frequently measured in terms of "Half cast factor" (HCF) which is dominated by the design parameters of the contour holes, the joint orientation and the explosive energy distribution.

Generally, two types of contour blasting are used in tunnelling, i) pre-splitting and ii) smooth blasting. When two closely spaced charged holes are fired simultaneously the stress waves generated from the two holes collide at a plane in between the holes and create a secondary tensile stress front perpendicular to the hole axis and facilitates extension of radial cracks along the line joining the holes. The wedging action from the explosion gas acts in favor of extending the crack along the same line. It is, therefore, essential to contain the gas pressure till the cracks from both ends meet by adequate stemming. Further, the delay timing of the adjacent holes need to be very accurate so that the stress waves should collide at the mid-point and the arbitrariness of the breakage between the holes can be reduced. Yamamoto et al. (1995) observed that the depth of damage and overbreak area were reduced to 90 and 6 percent, respectively, with the use of electronic detonators, which has high accuracy compared to those with long delay detonators. Ngoma et al. (1999) reported that the electronic detonators provided 85 percent HCF compared to 34 percent with conventional delay systems. Nevertheless, the success of contour blasting is strongly dependent on its acceptance by the miners or tunnel engineers.

The contour blasting performance largely depends on the nature and the orientation of joint planes. Gupta et al. (1988) found that the joint orientation adversely influences the pre-splitting results to a maximum when these are at an angle of 1–30° to the pre-split axis.

It was observed in the machine hall of Srisailam Hydroproject in India that an arched roof in such a strata having major horizontal joint set was converted to flat roof due to roof fall and overbreak (Chakraborty, 2002). Contour blasting for arched roof was not very successful in such laminated rock masses.

In smooth blasting, the delay intervals between the contour holes and the nearest production holes are kept high to facilitate complete movement of material in production holes before the contour holes detonate so that the gas expansion in contour holes occurs towards the opening. Sometimes, holes are drilled in between two charged blast holes and are kept uncharged. These are called dummy holes (Fig. 11.11). The stress concentration at the farthest and the nearest points of the dummy holes become high to initiate cracks from the dummy holes extending towards the charged holes. The fracture is, thus, controlled along the desired contour.

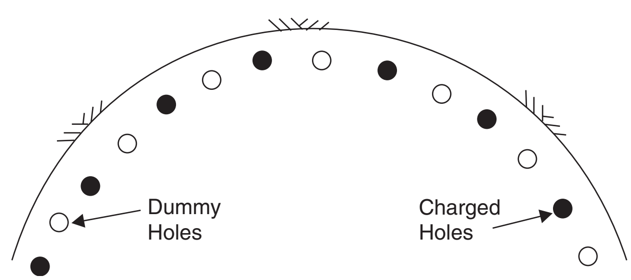

Fig. 11.11 Smooth blasting pattern with dummy holes.

In some cases, slashing or trimming techniques are used where the central core of excavation area is removed first to reduce the stress and then post-splitting is adopted to remove the remaining rock mass along the desired contour. The technique is generally referred to as "slashing" or "trimming" (Calder and Bauer (1983), Fig. 11.12).

Line drilling is adopted as an alternative technique where a number of uncharged holes are drilled along the contour with a spacing of 2–4 times the hole diameter (Du Pont, 1977). The distance of the row of empty holes from the final row of charged holes is kept as 0.5–0.75 times the normal burden. The empty holes are joined during the main blasting round and a separation is created along the contour.

According to Holmberg and Persson (1978), the spacing of pre-split holes should be 8–12 times the blast hole diameter. The following design parameters for contour hole spacing, burden to spacing ratio of contour holes and linear charge concentration in smooth blasting are suggested by Holmberg (1982):

$$S_{dc} = 16 \times d_b \text{ m} \tag{11.12}$$

$$m_{dc} = 1.25 \tag{11.13}$$

$$q_{lcc} = 90 \times (d_b)^2 \text{ kg/m} \tag{11.14}$$

where

S_{dc} = spacing of contour holes while drilling, m,
m_{dc} = burden to spacing ratio of contour holes while drilling,
q_{lcc} = linear charge concentration in the contour holes, kg/m and
d_b = diameter of blast holes, m.

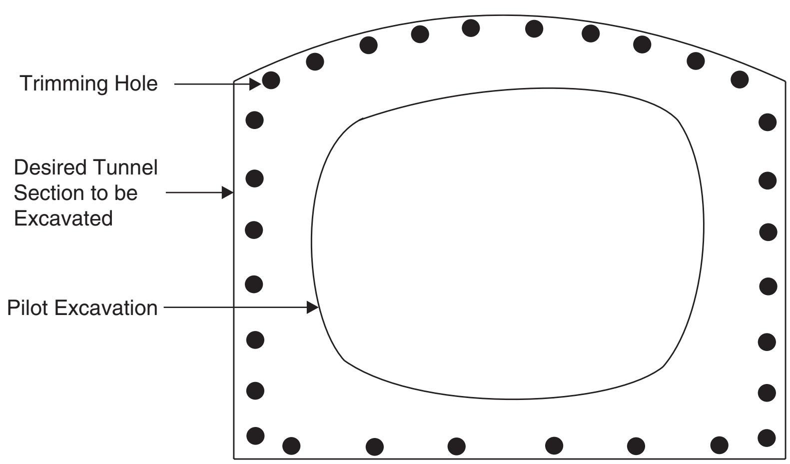

Fig. 11.12 Trimming after elimination of stress by making a pilot excavation (Calder & Bauer, 1983).

Table 11.10 Recommended blast design for contour blasting (Olofsson, 1988).

Type of blasting	Blast hole diameter (mm)	Spacing of blast holes (m)	Burden (m)	Linear charge concentration (kg/m)
Smooth blasting	25–32	0.25–0.35	0.3–0.5	0.11
	25–48	0.5–0.7	0.7–0.9	0.23
	51–64	0.8–0.9	1.0–1.2	0.42–0.45
Pre-splitting	38–44	0.3–0.45	–	0.12–0.37

Controlled blast design details recommended by Olofsson (1988) are presented in Table 11.10.

Ibarra et al. (1996) reported that the rock mass quality Q has more significant influence on overbreak while the specific charge dominates the underbreak. They suggested that the contour specific charge should be optimized considering the additional cost incurred towards the overbreak and the underbreak. For example, the curve in Fig. 11.13 indicates that the contour specific charge should be designed as 0.65–0.7 kg/m^3 when the total cost was the minimum.

Tracer blasting is used in Canadian underground mines to minimize the overbreak in development headings and stopes (Singh & Lamond, 1996). This involves placing a detonating cord along the blast hole wall before charging ANFO. The detonating cord is used to initiate the primer placed at the hole bottom. Utilizing higher VOD

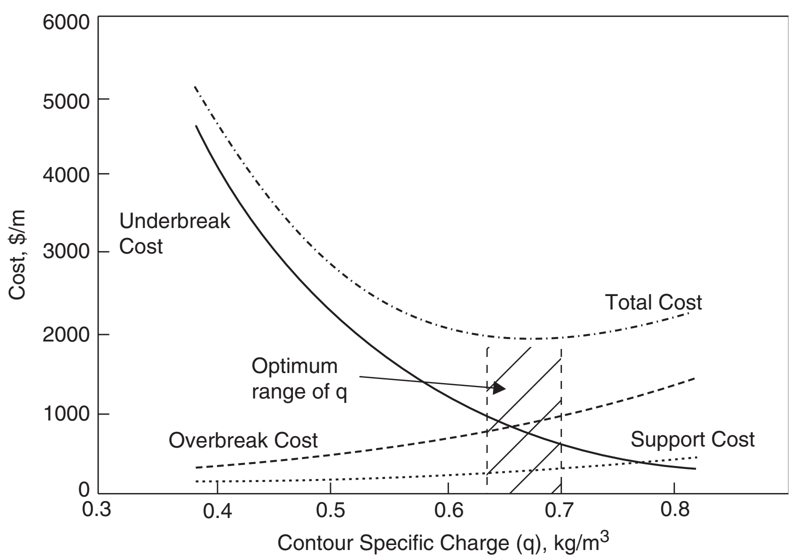

Fig. 11.13 Optimization of contour specific charge in terms of cost (Ibarra et al., 1996).

of detonating cord, it is possible to obtain partial desensitization, deflagration and decoupling which result in reduced explosive energy utilization controlling the blast damage. McDonald and Ng (1994) report the use of ANFO-spoon technique for contour blasting. The method works on the principle of air/ANFO dust explosion. Contour blasting was tried with slotted cartridges (Wang & Wei, 1987) and notch drilling also (Sperry et al., 1979; Fourney et al., 1984). In both the cases, a stress concentration is created in the pre-determined points to ensure fracture along a desired direction. Wang and Wei (1987) observed that the length and the width of a crack produced by a slotted cartridge are related to the slot width. Sperry et al. (1979) observed that 6 mm deep notch with 80° notch angle provides good results with 44 mm drill hole size. The blast-induced crack with such a notch extends to a length twenty times the diameter of the drill hole.

The contour blasting in place of a conventional one was very useful at Tandsi project of WCL. The number of holes including the dummy holes in a round was increased by more than 71 percent when the spacing to burden ratio at the contour was reduced from 1.5–1.85, as maintained in the conventional blasting, to 0.8 in the contour blasting. As a result, the specific drilling increased by nearly 30 percent. In spite of increased drilling cost and time, the excavation cost was reduced approximately by 22 percent and the tunnelling rate was improved by more than 100 percent with contour blasting and shotcrete support. *Hence, the application of contour blasting appears to be highly beneficial in reducing the overall cost and time of tunnelling.*

11.8.8 Computer-aided blast design

Some of the software developed for blast design and optimization are reported in Table 11.11. Few blasting software on tunnel blasting are commercially available and the details can be obtained through web search.

Table 11.11 Various routines for computer-aided tunnel blast design.

Name of software	Purpose	Reference
OPTES	Blast optimization in tunnels	Vierra (1984)
VOLADOR	Estimation of blast results, blast efficiency and cost analyses in tunnels	Rusilo et al. (1994)
TUNNEL BLAST	Blast design in tunnels	Chakraborty et al. (1998a)
CAD	Optimum design of ring hole blasting	Myers et al. (1990)
FLAC and UDEC	Blasting effects on the near field rock mass	Pusch et al. (1993)
ABAQUS V 5.4	Mechanics of crater blasting and the effects of air-decking and decoupling	Liu and Katsabanis (1996)
ALEGRA	Air-decking blasting	Jensen and Preece (1999)

11.8.9 Optimum tunnel blast design

Optimum drilling and blasting, which is a basic need of a rock excavation work, is traditionally achieved when the excavation cost is maintained within a band closely extended on either side of the minimum one. The blast design parameters are generally varied to achieve better results and the modified set of design parameters is used till further improvements are required. The objectives frequently change specially with the change in rock mass conditions. Hence, the optimization becomes a dynamic process in rock excavation.

Nielsen (1983), Gadberry (1984), Beattie (1982), Hagan and Gibson (1988), Hall (1986) and Michaud and Blanchet (1996) have carried out extensive work on bench blast optimization considering sub-systems utilization and fragmentation to arrive at optimum blast design.

Optimization in underground excavations cannot be complied with fragmentation assessment alone leaving other important parameters like ground stability, blast-induced damage, support cost, hole deviation and ore dilution which may have greater impact on productivity. Scoble et al. (1991) support this view.

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12
Rock bolting

"Listen to advice, but make your own decisions."

From Chinese Fortune Cookie

12.1 GENERAL

Rock bolt as the term indicates is a rod or bar shaped bolt made up of steel, tube, etc. and used to support the rock mass. With the developments in supporting technology, the rock bolt supports have become very popular. Hence, one should plan an underground engineering project with a rock bolt support. Starting from the ancient wooden or bamboo bolts there are different types of rock bolts as discussed below.

12.2 TYPES OF ROCK BOLTS

There are basically two types of rock bolt (i) pre-tensioned point-anchored bolt and (ii) untensioned full-column grouted bolt (Fig. 12.1). The evolution of rock bolting began with (mechanically) point-anchored bolt. It consists of a steel bar with a mechanical anchorage at the top and a bearing plate and a torque nut at its bottom end (Fig. 12.1a). The bolt is pre-tensioned by tightening the nut with the help of a torque wrench. However the pre-tension is lost gradually as (i) rock creeps and blast vibrations loosen the mechanical anchorage particularly in weak rocks like shale and schist, etc., (ii) humidity or ground water (with massive sulphides) corrodes the bolt, (iii) the rock is fractured at the end of borehole due to stress concentration and (iv) bolt is broken in the highly tectonically active (micro-folded) weak rocks. They are used as temporary support system.

Recently, full-column grouted bolts have been employed successfully in supporting mine roof and tunnels where point (mechanically) anchored bolts had failed. The grouted bolt is simply a steel bar grouted into a borehole throughout its length (Fig. 12.1b). The grouted bolts are much stiffer than ungrouted bolts and so resist any tendency of dislocation along the existing fractures in the rock masses. The propagation of a fracture is arrested as a reinforced crack may not open up easily. Further, interlocking between

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rock blocks is retained. A broken rock bolt is still effective as two short bolts. Most of the common bolts may be classified as illustrated in Fig. 12.2.

The various types of mechanical anchorages are shown in Fig. 12.3. Rock bolts (of about 25 mm diameter) with slot and wedge anchorages are used extensively in hard rocks. In this system, the anchorage is obtained by driving a bolt with a slot into the wedge held against the end of the borehole (Fig. 12.3a). According to Lang (1961), the bolt should be inserted by hammering with a pressure of 0.5–0.7 MPa, bolt insertion time 20 to 30 s, frequency of hammering 2000–3000 impacts per min. In this way, the slotted

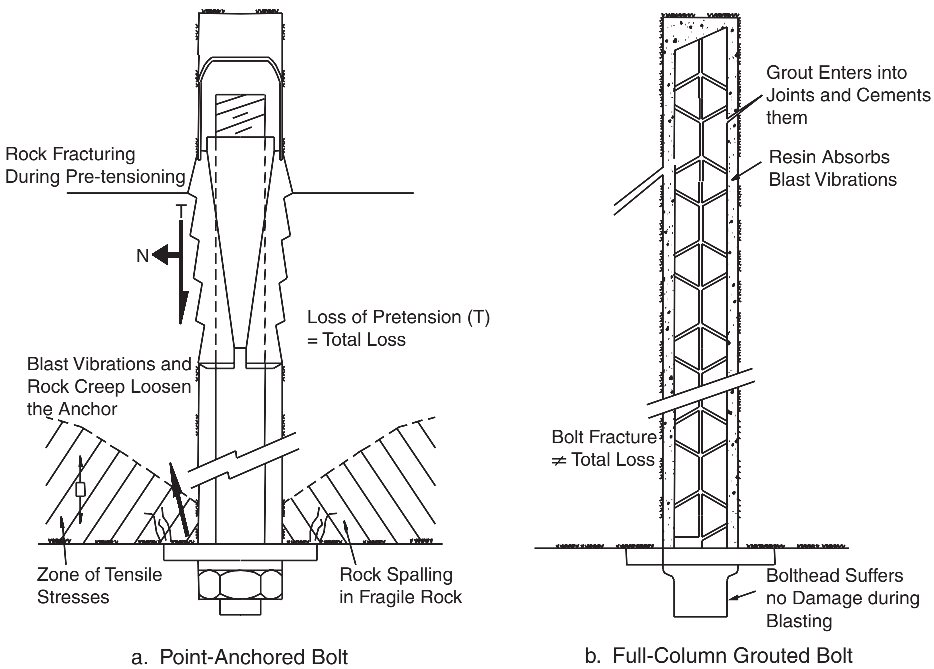

Fig. 12.1 Comparison of point-anchored and grouted bolts.

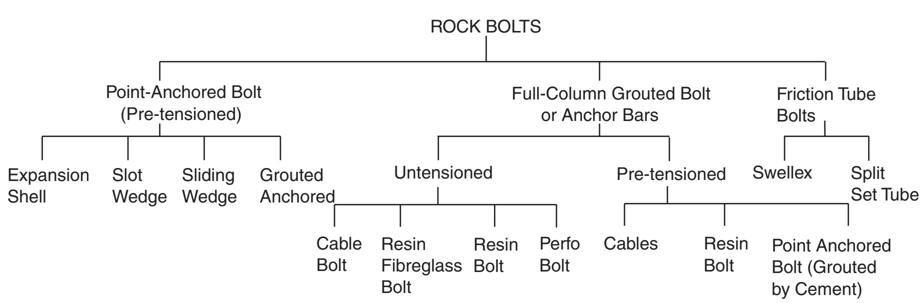

Fig. 12.2 Types of bolts.

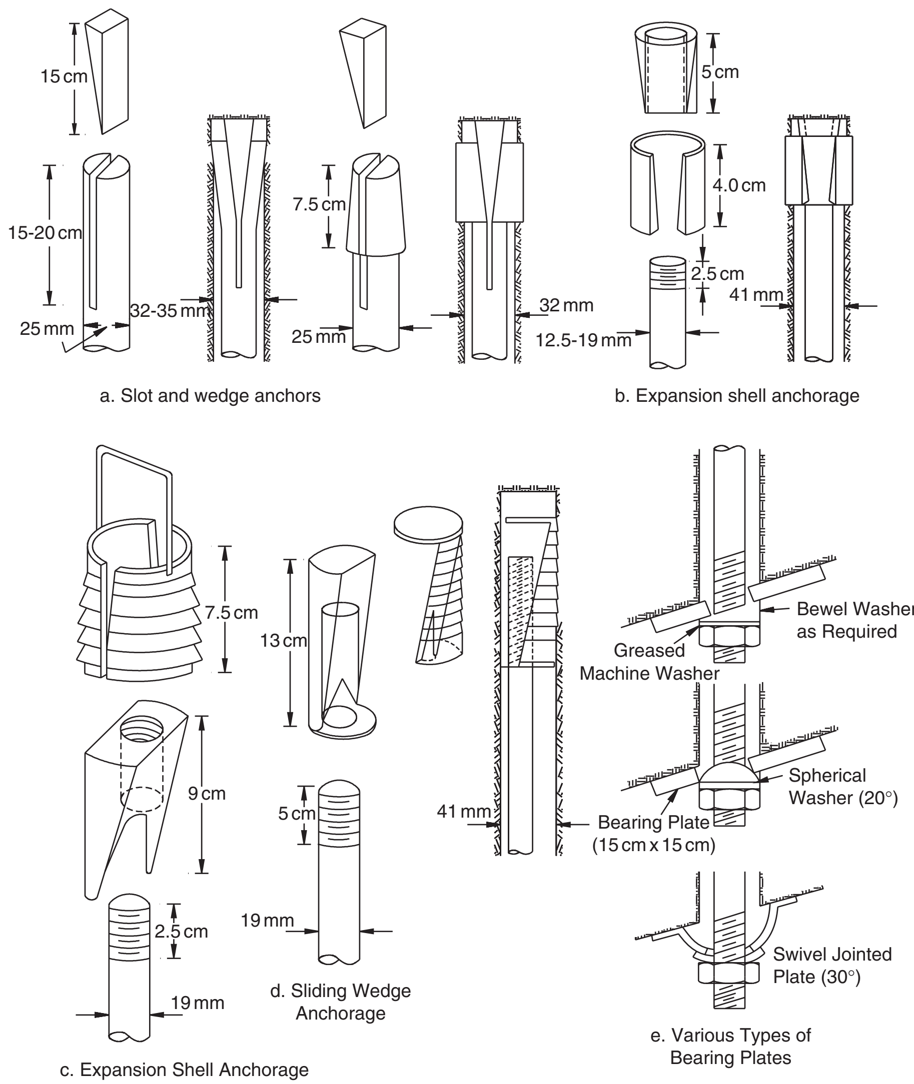

Fig. 12.3 Components of point-anchored bolts.

bolt may expand to a diameter 15 percent more than that of the borehole. If the rock is too weak, the wedge may penetrate inside the rock at borehole end without causing the expansion of slotted bolt. Thus anchorage capacity of the slot–wedge anchor may not develop to the extent of tensile strength of the bolt-shank.

The expansion shell anchorage is more versatile than slot–wedge anchorage. It consists of serrated split sleeves with a wedge cum nut screwed upon the threaded bolt (Fig. 12.3c). When the torque nut at the bottom end (Fig. 12.1a) is tightened, the wedge forces the sleeves to open against the wall of the hole. The expansion shell has two advantages over the slot–wedge (i) the length of the hole need not be precisely equal to that of the bolt and (ii) the bolt can be used in soft rocks also. For example, anchorage capacity of expansion shell for 19 mm bolt ranges from 3 to 10 tonnes for soft to medium shales. However borehole diameter has to be slightly larger than that for slot and wedge type bolt of the same diameter.

In practice, surface of the excavation is rarely flat and perpendicular to the axis of the bolt. As such steel bearing plates of size 10 × 10 cm or 15 × 15 cm are used to bridge irregularities on the rock surface and provide firm bearing surface for the washer and the nut (Fig. 12.3e).

As the bolt is tensioned, the rock asperities are crushed to provide the required bearing area. With blasting vibrations, the crushed material tends to become loose, and at times spalling of the rock above the plate occurs leaving the bolt to hang in the air. Thus the bolt should be checked periodically and retightened. This is a rule which should be strictly followed in the practice.

If rock bolts are desired to be a permanent system of support, all boltholes must be grouted completely with cement grout (Fig. 12.4a) or resin. This is for preserving the pre-tension and preventing corrosion of steel. (Steel ribs are also encased in concrete lining for the same reasons.) For this purpose either an air tube or hollow bar of high strength is used. While grouting a bolt, the rubber grout seal is used to center the bolt in the hole and to seal the collar of the hole against grout leakage. Grout injection is stopped when air has been displaced and the grout flows out from the return tube (Fig. 12.4a). A site engineer should check the flowing out of return grout to ensure the full-column grouting of rock bolts.

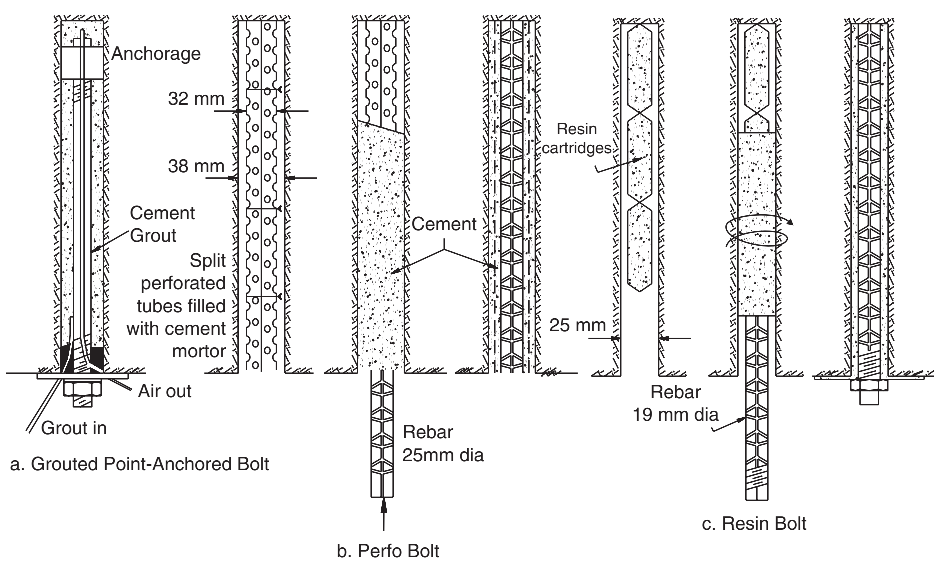

Fig. 12.4 Process of installation of grouted bolts.

In situation where very long bolts are required such as in large underground chambers (and high slopes), a steel cable may be substituted for steel bar.

The full-column grouted bolts without pre-tension are also quite effective in reinforcing the rock masses as mentioned earlier. In civil engineering construction, "Perfobolts" are used to provide a permanent system of support. It consists of a pair of semi-cylindrical perforated metallic tubes which are filled with cement mortar and tied with wire and inserted into the borehole. Then a steel bolt of a slightly smaller diameter is hammered into the tube as shown in Fig.12.4b. The mortar extrudes evenly out of perforations and fills the borehole. The modern trend is towards using resin grout because time of attaining full strength of resin is just 5 min compared to 10 h for cement. The "resin bolts" are more popular in mines and tunnels in Europe. First, resin cartridges (sausages) are inserted with the bolt and pushed to the end of the borehole. The bolt is then rotated at 100–600 rpm for about 10 s to break the cartridge and mix its contents, i.e., the polyester resin, catalyst and hardener (Fig. 12.4c). The bearing plate and the nut are fitted to suspend any loose rock mass at the rock surface because the resin may not ooze right down to the bottom of the borehole. It may be noted here that the grouted bolts are slightly costlier than point-anchored bolt, as such they are used in highly unstable (or rock burst prone) grounds or where a permanent system of support is required.

The fast rotating cartridge may dig up weak rock layers locally, preventing thorough mixing of resin in long bolts. So, bolt length should be less than 5 m in poor rocks. It is cautioned that the resin has limited shelf life in hot climates. Therefore, this must be checked before its application.

Some other types of bolts, e.g., pins driven hydraulically into soft rocks (Harrell, 1971) and roof trusses developed by Birmingham Bolt Co. (Kmetz, 1970) and explosively expanded rock bolts developed by U.S. Bureau of Mines are not commonly used.

Hoek and Brown (1980) have presented an excellent summary of new types of rock bolts. Of special interest is split tube anchor which is popular in mines where temporary stability is all that is needed. The bolt consists of 2–3 mm thick and 38 mm diameter split tube with 13 mm gap (Fig. 12.5). It is forced into a 35 mm diameter drillhole. The spring

Fig. 12.5 Split set tube bolt.

action of the tube causes the tube to jam inside the hole. The friction between drillhole and tube is increased as bolt is rusted. Grouting of this type of bolt is not possible. Rusting of split tube bolts occurs rapidly and therefore anchorage increases with time. It is difficult to install long split tube bolts.

Fig. 12.6 shows a collapsed tube called swellex bolt. It is inserted into the bore hole and expanded by air and water pressure to the shape of bore hole. The friction between tube bolt and rock reinforces the rock mass. It is ideally suited in supporting tunnels within water-charged rock masses where grouting by cement or resin is not feasible. Corrosion can be a long-term problem both in the split tube and swellex bolts.

12.3 SELECTION OF ROCK BOLTS

Following guidelines may prove useful in selection of bolts (Pender et al., 1963),

(i) Deformed bar shanks are now used for all bolts which are to be grouted with cement or resin. They are installed along unsupported free length near the tunnel face within the bridge action period of rock mass.
(ii) Plain shank bolts are used only for temporary full-column grouted bolts support or where concrete lining is to be placed for permanent support. The modern practice is

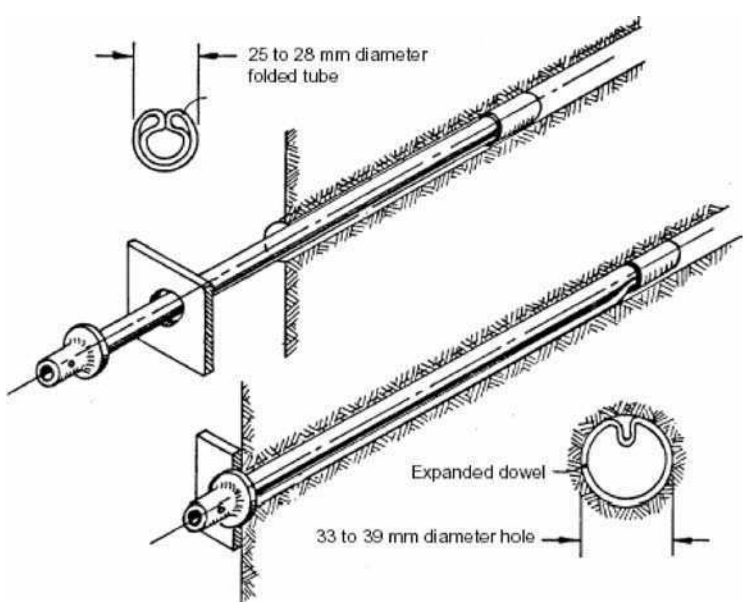

Fig. 12.6 Swellex tube bolt (Hoek, 2004).

to recommend thermo-mechanically treated (TMT) bolts as they are ductile having strength of 415 MPa (against 250 MPa of mild steel).

(iii) Bolts of high tensile strength should be used with precaution. When it breaks, it leaves a hole with high velocity. In squeezing ground or where rock bursts are likely, mild steel bolts are preferred because it meets the requirement of large plastic yielding. Special yieldable head type bolts may also be used in squeezing conditions (Barla, 1995).

(iv) The cement grout should be designed properly for flowability, slight expansion on hardening and high shear strength. These properties are obtained with grouts having water cement ratio between 0.38 and 0.44 to which commercial aluminum powder has been mixed in amounts up to 0.005 percent by weight of cement. Excessive aluminum powder may create weak, spongy and powdery grout. Other expanding agents may also be used as per specifications of manufacturers.

Mandal (2002) has suggested rock bolt and shotcrete support systems for various tunnelling ground conditions as given in Table 12.1.

Table 12.1 Suggested support for various rock conditions (Mandal, 2002).

Rock conditions	Suggested support type
Sound rock with smooth walls created by good blasting. Low in situ stresses.	No support or alternatively, where required for safety, mesh held in place by grouted dowels or mechanically anchored rock bolts, installed to prevent small pieces from falling.
Sound rock with few intersecting joints or bedding planes resulting in loose wedges or blocks. Low in situ stresses.	Scale well; install tensioned, mechanically anchored bolts to tie blocks into surrounding rock, use straps across bedding planes or joints to prevent openings. Such as in shaft stations or crusher chambers, rock bolts should be grouted with cement to prevent corrosion.
Sound rock, damaged by blasting, with few intersecting weakness planes forming blocks and wedges. Low in situ stress conditions.	Chain link or weld mesh, held by tensioned mechanically anchored rock bolts, to prevent falls of loose rock. Attention must be paid to scaling and to improving blasting to reduce amount of loose rock.
Closely jointed blocky rock with small blocks ravelling from surface causing deterioration if unsupported. Low stress conditions.	Shotcrete layer, approximately 50 mm thick. Addition of micro-silica and steel fiber reduces rebound and increases strength of shotcrete in bending. Larger wedges are bolted so that shotcrete is not overloaded. Limit scaling to control ravelling. If shotcrete not available, use chain link or weld mesh and pattern reinforcement such as split sets or swellex.

Continued

Table 12.1—Continued

Stress-induced failure in jointed rock. First indications of failure due to high stress are seen in borehole walls and in pillar corners.	Pattern support with grouted dowels. Split sets are suitable for supporting small failures. Grouted tensioned or untensioned cable can be used but mechanically anchored rock bolts are less suitable for this application. Typical length of reinforcement should be about half the span of openings less than 6 m and between half and one-third for spans of 6 to 12 m spacing should be installed before significant movement occurs. Shotcrete can add significant strength to rock and should be used in long-term openings (drill-drive etc.)
Drawpoints developed in good rock but subjected to high stress and wear during blasting and drawing of stopes.	Use grouted rebar for wear resistance and for support of drawpoints brows. Install this reinforcement during development of the trough drives and draw point, before rock movement takes place as a result of drawing of stopes. Do not use shotcrete or mesh in drawpoints. Place dowels at close spacing in blocky rock.
Fractured rock around openings in stressed rock with a potential of rock bursts.	Pattern support required but in this case some flexibility is required to absorb shock from rock bursts. Split sets are good since they will slip under shock loading but will still retain some load and keep mesh in place. Grouted resin bolts and Swellex will also slip under high load but some face plates may fail. Mechanically anchored bolts are poor in these conditions. Lacing between heads of reinforcement helps to retain rock near surface under heavy rock bursting.
Very poor quality rock associated with faults or shear zones. Rock bolts or dowels cannot be anchored in this material.	Fiber-reinforced shotcrete can be used for permanent support under low stress conditions or for temporary support to allow steel sets to be placed. Note that shotcrete layer must be drained to prevent build up of water pressure behind the shotcrete. Steel sets are required for long-term support where it is evident that stresses are high or that rock is continuing to move. Capacity of steel sets estimated from amount of loose rock to be supported. Min. 200 mm backfilling is required to develop contact between steel sets and rock surface.

12.4 INSTALLATION OF ROCK BOLTS

12.4.1 Scaling

One of the most frequent causes of accidents in underground excavations is indequate scaling soon after blasting. Scaling work consists of removal of loose pieces of rock from roof and walls before workmen move towards the face of excavation. It is generally done manually by using long steel bars. The sound of impact of a steel bar on the rock may tell the foremen whether or not the rock is loose. The same is then removed. However, there is poor visibility and walls are covered with dust and face is not easily accessible, so manual scaling may not be very much effective.

12.4.2 Installation

The rock bolts must be installed as soon as possible after scaling and within bridge action period. The delay in installation may not only jeopardize the safety of workmen due to greater chances of rock fall but it also reduces the strength of the rock mass. The good practice is:

(i) Install rock bolts concurrently with drilling of blast-holes in the (tunnel or mine) face for the next round using common jumbo. The experience is that the bolts even close to the face are seldom damaged after blasting, except that there is loss of pre-tension. The grouting may then be done if required. The grouting facilities (e.g., inlet and outlet tubes in Fig. 12.4a) should be provided at the time of rock bolting so that pre-tension in the bolt is not released while grouting.

(ii) The loosened rock particles in the roof should be pulled down rather than bolted. Scaling reduces the need for spot bolting.

(iii) Thorough inspection of the rock mass (key blocks) should be done before bolting to locate the weak zones that require special treatment or spot bolting.

12.4.3 Pre-tensioning

For efficient use of the point-anchored bolts, the pre-tension (P) must be as high as the bolt can take safely. To avoid overstressing of the bolt, adjustable automatic-cutoff (hydraulic driven or impact) torque wrenches should be used to apply the desired torque (T) on the nut. For purpose of checking the pre-tension, manually operated (lever type) torque wrenches with dial may be used. Experiences show that the greased hard nut should be used above the torque nut in order to increase the tension torque ratio (P/T) and to minimize the scatter in this ratio (Osen & Parsons, 1966; Agapito, 1970). The typical tension–torque relationship is given by

$$T = KPd \tag{12.1}$$

where d is nominal diameter of a bolt and K is a constant ($\cong 0.20$). Thus the bolt may fail due to combined stresses of tension and torque. To increase torque limit, bolts of high tensile steel are used for bolt diameter of 19 mm or less (in expansion shell). But in soft rocks, mild steel bolts are strong enough. Very often in the field, bolts of too large diameter tend to be used for psychological reasons in the poor rocks, though they cannot provide much anchorage capacity.

There is no need of tensioning full-column grouted bolts in the weak zones (Tincelin, 1970), and in fact too high pre-tension might reduce the efficiency of bolts. However, a resin bolt may be pre-tensioned by first inserting cartridge of fast setting resin, followed by cartridges of slow setting resin and thereafter rotating the bolt, and finally tightening the torque nut as for the point-anchored bolt.

12.4.4 Wiremesh

If the clear spacing between bearing plates is too large compared to the fracture spacing, rock blocks are likely to fall down leading to complete collapse of the bolted roof. The wire mesh has proved more successful than initially thought of in preventing such spalling and ravelling of highly fractured rock masses. However, the wire mesh should be stretched tightly between rock bolts and held close to the rock surface. Further it also provides an effective protection to the workmen against rock falls. Infact, even a flimsy wire netting serves the structural purpose.

Chain link mesh is used when spacing between bolts is considerable and mesh is required to hold small pieces of rock which become detached from the roof due to the poor work of scaling. This type of wire mesh consists of a woven fabric of wire such as mesh for fencing around play grounds. It is flexible. It is easy for shotcrete to penetrate behind the chain link mesh. The contact between rock surface and mesh is a difficult task in practice. Since wire mesh is easily damaged by flying pieces of rock from the nearby blast, it has been suggested (Hoek & Bray, 1980) that the mesh should not be fixed right upto face.

Another type of wire mesh is weld mesh which is generally used for reinforcing shotcrete. It consists of a square grid of steel wires, welded at junctions.

12.4.5 Rock bolt ties

In addition, continuous steel ties are also employed to support the unstable rock mass. The ties may be of steel channel sections with properly spaced holes for the bolts.

12.5 PULL-OUT TESTS

Pull-out tests on certain percentage of bolts are necessary to (i) measure the residual pre-tension in bolts after blasting, (ii) check their anchorage capacity and (iii) study creep effect, etc.

Fig. 12.7a illustrates a typical pull-out test as suggested by Franklin and Woodfield (1971). The bolt is pulled out by a 100 ton spring-return hollow ram with low friction seals for reproducible calibration. The ram is pressurized by a hand pump connected through a high pressure flexible hose. The pull is measured by a pressure gauge calibrated directly in tons. The movement of the bolt-head which is the sum of anchor slip and deformations in bolt can be monitored easily by a set of dial gauges. The bolt should be tested for a movement to the extent of 5 to 8 cm in order to study the post-failure behavior.

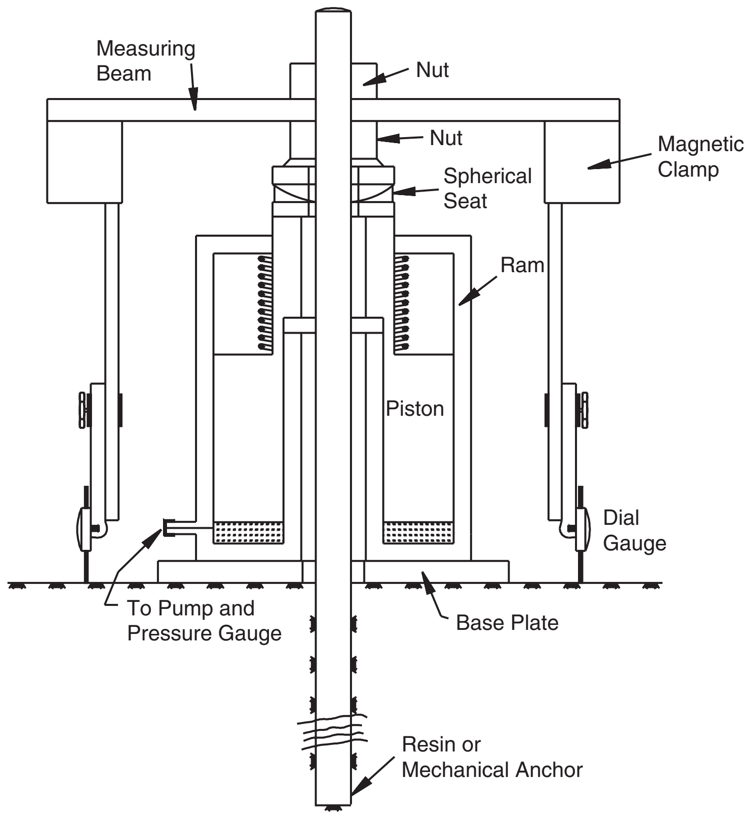

Fig. 12.7a Rock bolt testing equipment (Franklin and Woodfield, 1971).

To measure actual tension, an auxiliary shank may be coupled to the bolt-head. It is pulled out by the ram which rests on an extra packer over a bearing plate to accommodate the coupling. The actual tension is that load at which torque nut just looses contact with the bearing plate. The International Society for Rock Mechanics (ISRM) has also suggested a method for pull-out test on rock anchors and bolts. Sometimes the quality of grout is checked by overcoring a 15 cm diameter core containing the rock bolt.

Typical test results are shown in Fig. 12.7b. It is seen that mechanical anchorages may slip upto 50 mm before peak load in contrast to only 5 mm for resin bolts. In addition resin bolts are found to give much better anchorage capacity.

The quality of bolts should also be checked in laboratory by testing five bolts per 1000 according to the suggested method of ISRM (1981) as follows:

(i) Tensile test on anchorage
(ii) Tensile test on nut and bearing plate

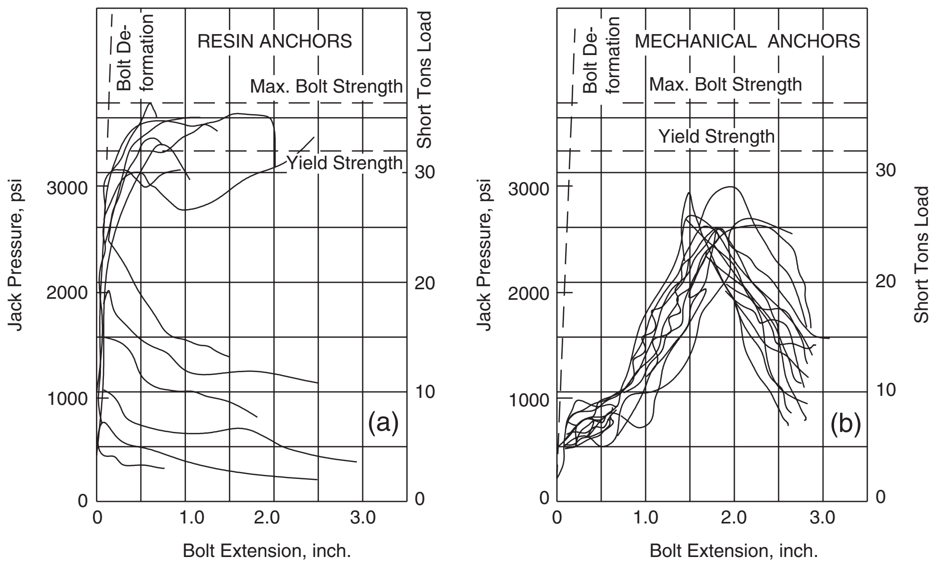

Fig. 12.7b Pull-out curves for granites (a) resin-anchored bolts, (b) mechanically anchored bolts.

(iii) Tensile test on the shank
(iv) Test for determining torque–tension ratio

Fairhurst and Singh (1974) conducted model tests on a bolted model of four layers (simply supported at the ends) to compare the reinforcement action of full-column grouted bolts and point-anchored bolts. Plexiglass beams and Masonite beams were used to represent brittle layers and ductile layers of rock masses. Both have practically same values of modulus of elasticity and modulus of rupture. The generally low stiffness of mechanically anchored bolt was modelled by interposing a spring between nut at the top end of each bolt and pre-tensioning the spring to exert on average pressure of 0.07 MPa across the layer. The grouted bolt consisted of 3 mm diameter steel rod in 5 mm hole filled with epoxy. Fig. 12.8 compares the normalized force and deflection curves for various models. It is seen that grouted bolts performed better than point-anchored bolt. This is also borne out by the field experience. Panek's (1955a, b, 1961, 1962) suspicion on efficacy of grouted bolts is not based on reality.

It is interesting to note that a fracture occurred through the grouted bolt in the Plexiglas beam presumably because of stress concentration around the bolthole. Consequently the grouted bolts lowered the ultimate load carrying capacity of the brittle beam. On the other hand the more ductile Masonite beam yielded around boltholes rather than fracturing as in the case of Plexiglas beam. Tests on thick beams of Plexiglas however exhibited the elasto-plastic shearing through bolt without any fracturing of the beam. A study of the computer model of bolted layers was taken up (Singh et al., 1973) to verify the prediction.

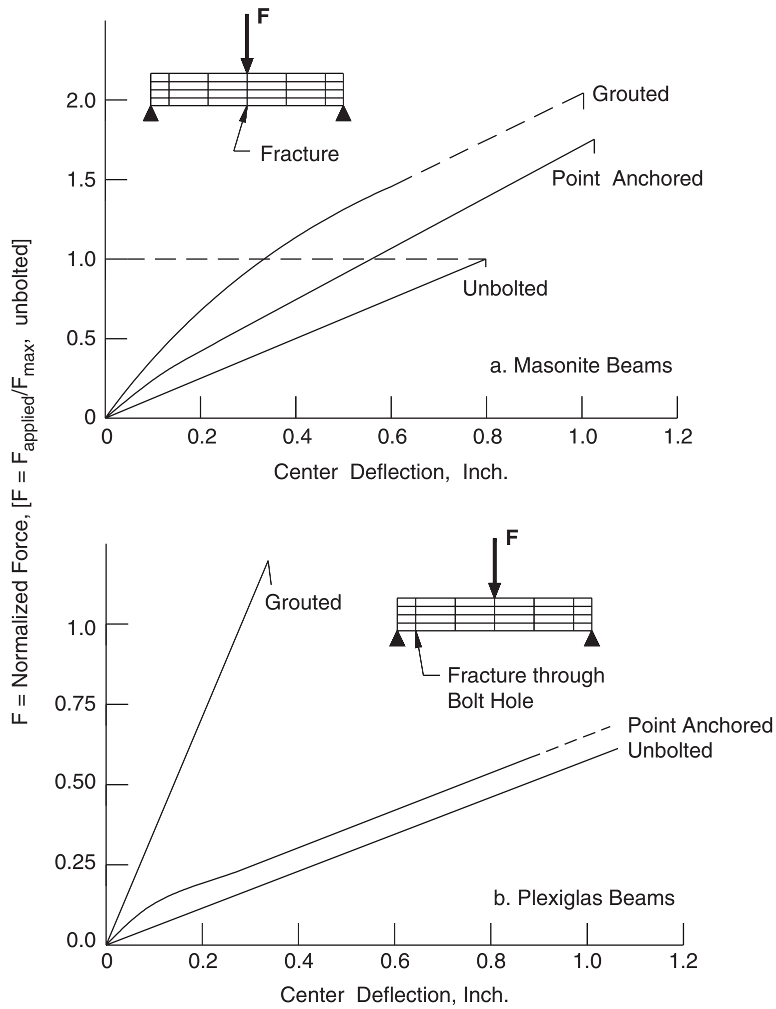

Fig. 12.8 Load deflection results from model rock bolting tests (Fairhurst and Singh, 1974).

It was shown that the untensioned grouted bolt (at usual spacing) makes a rock beam almost monolithic in behavior.

12.6 REINFORCEMENT OF JOINTED ROCK MASS AROUND OPENINGS

12.6.1 Reinforced beam

According to Lang (1961), axial pre-stress is developed due to Poisson's effect of normal stress on account of bolt's pre-tension. This pre-stress can stabilize the rock beam effectively as in the case of pre-stressed concrete beam.

A two-dimensional photoelastic study showed that the pre-tension of bolts form a zone of uniform compression between the ends of the bolts (Fig. 12.9). The only condition is that the ratio between length (l) and spacing (s) of bolts is more than 2. At this ratio, the zone is relatively narrow whereas for l/s equal to 3, it is approximately equal to two-third of the bolt length (i.e., equal to l−s). The normal stress (σ_v) within the zone may be estimated as ratio of pre-tension to the area per bolt. The horizontal stress (σ_h) equal to $k_o\sigma_v$ would be induced within this zone provided that the bolted beam is clamped laterally.

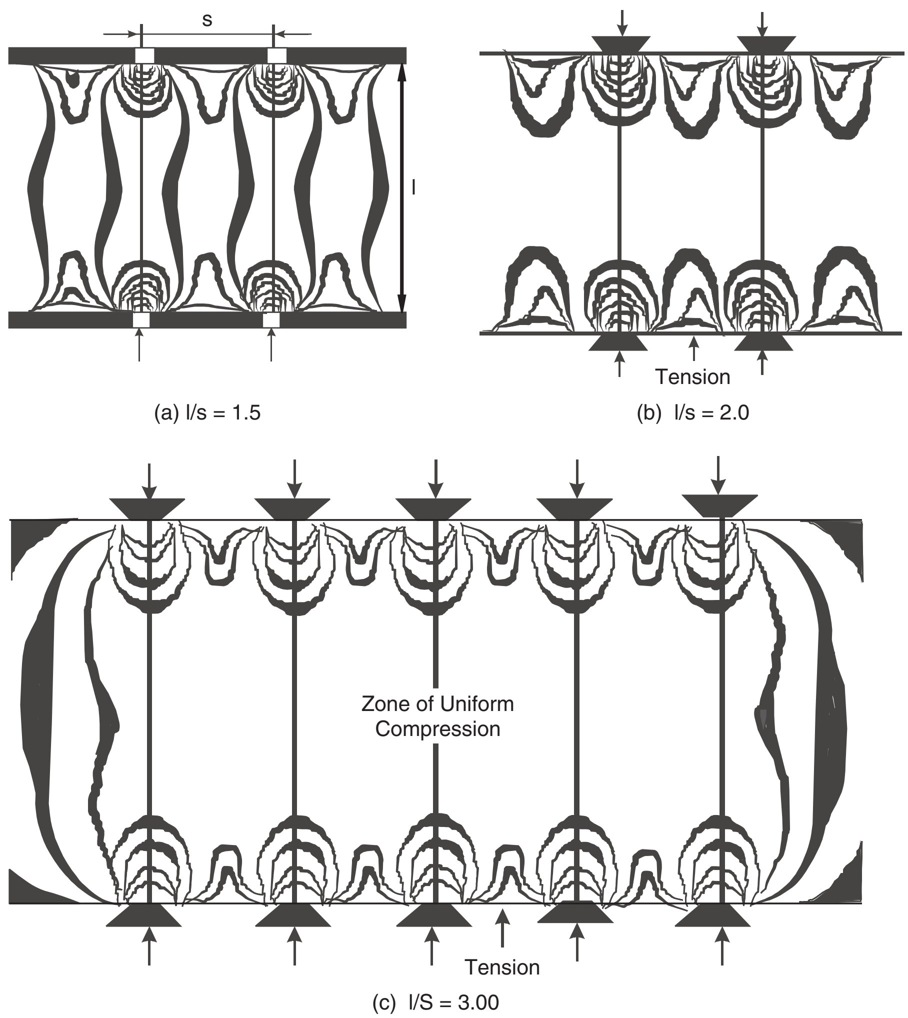

Fig. 12.9 Rock bolt – photoelastic stress pattern (Lang, 1961).

The total horizontal force is the sum of axial pre-stress (P_h) and the thrust (T) due to the arch action. Higher horizontal force means greater frictional resistance to sliding of the beam downwards.

The photoelastic model further indicated that zones of tensile stresses develop between bolts and so it may require an additional support in the form of wire-netting.

Large scale model tests to demonstrate the effectiveness of pre-tensioned bolts were also performed by Lang (1966). Crushed rock material of 38 to 57 mm in size was filled in a box of 1.2 m × 1.2 m × 1.2 m, compacted by vibration and then bolted with 58 cm long bolts. The reinforced rock mass was loaded at the center. At a load of 7000 1b (point D in Fig. 12.10), rock fragments started falling out leading to failure. The strength of the beam was almost doubled when the experiment was repeated using 24 gauge chicken wire net placed securely under the bolt-washers but not attached to the sides of the box. Note that repeated loading caused plastic deformations but without failure. This is because of some loss of pre-tension in bolts due to re-adjustment of rock fragments. Hence, the need for retightening of the bolts after vibrations or repeated loading. It was also demonstrated that only a very flimsy support is needed to hold the loose material within the tension zone between the bolt-washers.

If the clear spacing between the washer was less than 3 to 4 times the mean particle size, wire mesh was not required to prevent the ravelling as mentioned above. If this ratio was less than 7, the particles fell out between bolts but eventually a stable vault was formed. If this ratio was greater than 7, a fall out (ravelling) continued leading to total collapse. Similar conclusions have been made by Coates (1970) for block jointed models of rock mass with different orientations of joint sets.

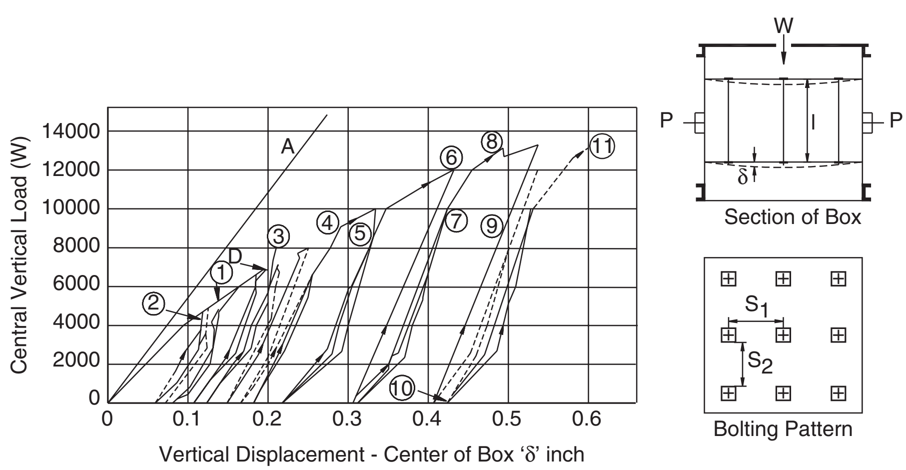

Fig. 12.10 Behavior of crushed rock model (Lang, 1961) [Rock size range was 1-1/2' to 2-1/4'; The mean (m) was 1.875 inch ($F = S_2/m = 4.3$)].

An experiment may be conducted at home by filling a bucket with crushed rock which is then bolted with single pre-tensioned bolt. The bucket is then turned upside down to see whether rock mass has been stabilized.

12.6.2 Reinforced rock arch

It may be seen from Fig. 12.11 that radial bolting pattern creates a reinforced rock arch over the tunnels. The thickness of an arch can be increased by employing supplementary bolts of shorter length. The most common practice is (Lang, 1966; Barton et al., 1974)

(i) Rock bolts should be pre-tensioned to give required ultimate support capacity (p_{roof} or p_{wall}) which is equal to $P/b{\cdot}s$ where P = pre-tension, b = bolt spacing

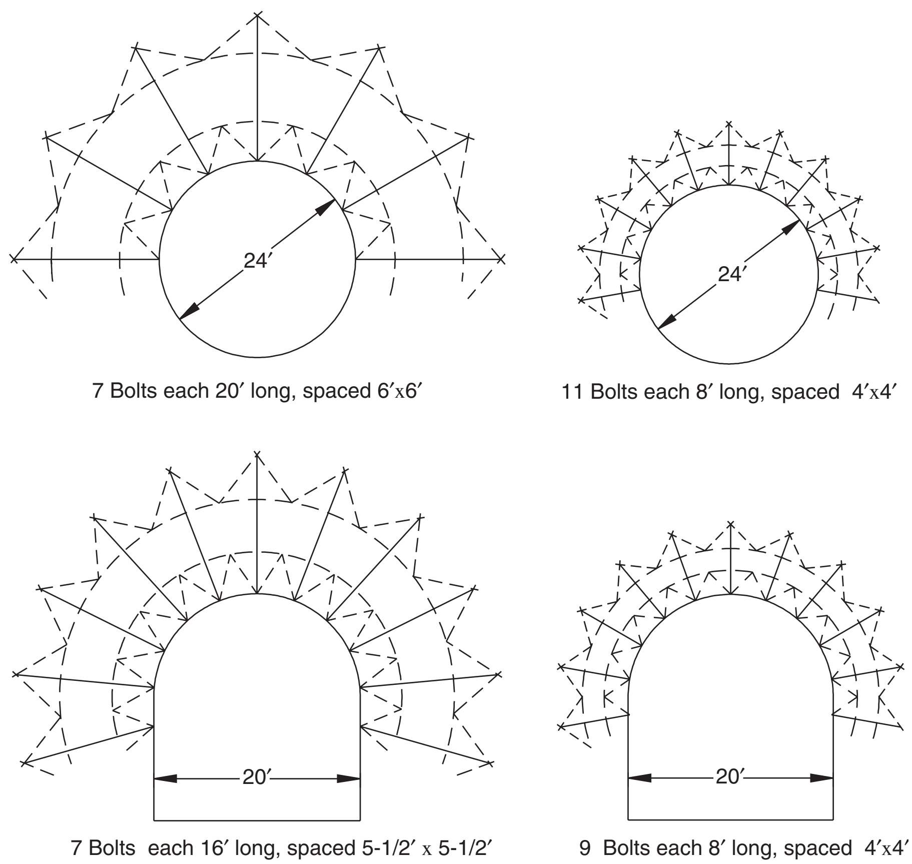

Fig. 12.11 Arch concept of rock reinforcement in circular and horse-shoe shaped tunnels (Lang, 1961).

along tunnel axis and $s =$ bolt spacing perpendicular to the tunnel axis. The pretensioned bolts are suitable for temporary support of openings in the hard rocks.

(ii) Grouted bolt anchors should be designed to provide ultimate support pressure (p_{roof} or p_{wall}) equal to P/bs where P is the tensile strength of bolts, provided bolts are adequately grouted. The bolt length should be greater than 1/4 to 1/3 of span of the tunnel.

(iii) The length of bolts (L in meters) should be calculated from the following simple relationship given by Barton et al. (1974),

$$L = 2 + (0.15\,\mathrm{B/ESR}) \quad \text{for} \quad \text{roof} \tag{12.2}$$

$$= 2 + (0.15\,\mathrm{H/ESR}) \quad \text{for} \quad \text{wall} \tag{12.3}$$

where

B = span or width of opening in meters,
H = height of opening wall in meters and
ESR = excavated support ratio (Table 5.11).

(iv) The adequate length of grouted anchors be obtained similarly as follows,

$$L = 0.40\,B/\mathrm{ESR} \quad \text{for} \quad \text{roof} \tag{12.4}$$

$$= 0.35\,H/\mathrm{ESR} \quad \text{for} \quad \text{wall} \tag{12.5}$$

(v) When single (2–3 cm thick) or double (5 cm thick) layers of shotcrete are applied usually in combination with systematic bolting, the function of shotcrete is to prevent loosening, especially in the zone between bolts. The capacity of shotcrete lining is, therefore, neglected. The application of shorcrete is essential to make grouted bolt–anchor system as permanent support.

(vi) Clear spacing between bolts should not be more than three times the average fracture spacing otherwise use wire mesh and guniting or shotcreting. *Further center to center spacing must be less than one-half of the bolt length*.

(vii) Bolts are installed on a selected pattern except near weak zones that would require special treatment. Spot bolting should be discouraged.

(viii) Bolts should be oriented to make an angle of 0 to ϕ to the normal on the critical joint sets in order to develop maximum resistance along joints (Fig. 12.12).

(ix) Bolts must be installed as early as possible within "Bridge Action Period" and close to the excavated face (Fig. 4.1).

However a tunnel is always unsupported in a certain length "t" between the last row of bolting and the newly excavated face (blasted face). Suppose rock is pulled out to a length of 3 m in each round of blasting, one may then assume the unsupported length (t) to be about 4 m. According to Rabcewicz (1955), the zone of rock mass of thickness of $t/2$ may be fractured and loosened due to blasting as shown in Fig. 12.13. Thus the bolt

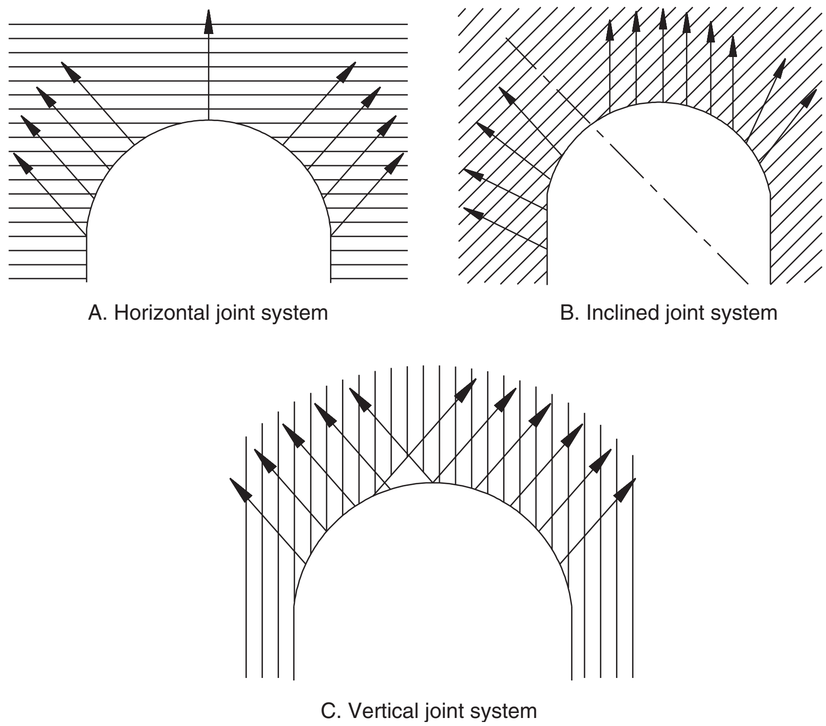

Fig. 12.12 Roof bolting in strata having various dip angles.

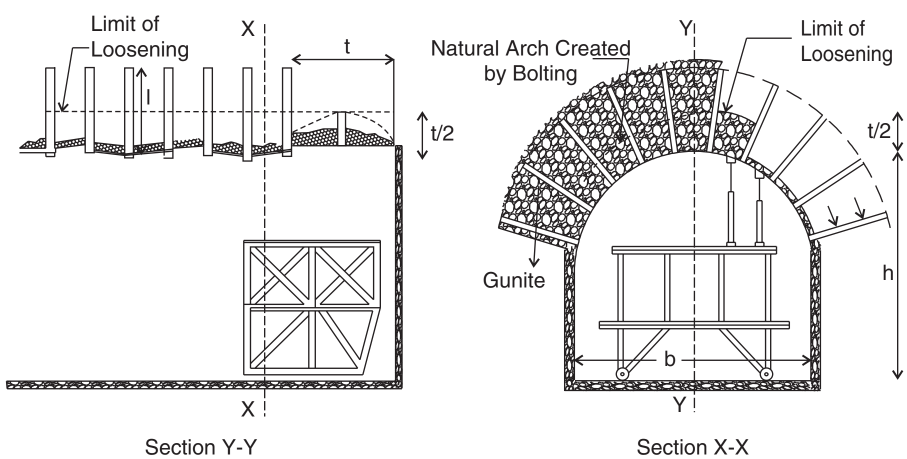

Fig. 12.13 Diagrammatic sections demonstrating principles of roof bolting.

length must be at least equal to the thickness of loosened zone (= $t/2$), so that the loose zone may be suspended by competent rock mass.

Rock bolts/anchors should be designed to absorb high longitudinal strains in the cases of weak rock masses. So the bolts of high tensile strength are failure in caverns and tunnels in weak rocks under high tectonic stresses, as in Tala Hydroproject, Bhutan (Singh, 2003).

12.7 BOLTING PATTERN

It is generally agreed that pattern bolting should be preferred over spot bolting because unknown conditions behind the surface of an excavation may be more critical than those visible at the surface. In addition, pattern bolting is advantageous from construction point of view also.

12.8 FLOOR BOLTING

Floor bolting is required to prevent floor of a tunnel from heaving in order to maintain the track properly for efficient haulage. Attempts to chop off squeezed rock mass are fruitless and may damage the wall support. The experience is that reinforcement of rock mass in the floor by rock bolts is very effective. However there is no standard practice. If swelling soft shale is found in the floor of a deep tunnel opening heaving may be serious.

In squeezing ground, rock bolting is not enough. It is important to apply steel fiber reinforced shotcrete (SFRS) layer by layer around the opening. It is necessary that invert of shotcrete lining is also laid so that it may enable the shotcrete walls to take heavy wall pressures. But one must understand the tunnelling hazards.

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13
Tunnelling hazards

"The most incomprehensible fact about nature is that it is comprehensible."

Albert Einstein

13.1 INTRODUCTION

The knowledge of potential tunnelling hazards plays an important role in the selection of excavation method and designing a support system for underground openings. The tunnelling media could be stable/competent (and or non-squeezing) or squeezing/failing depending upon the in situ stress and the rock mass strength. A weak over-stressed rock mass would experience squeezing ground condition (Dube & Singh, 1986), whereas a hard and massive over-stressed rock mass may experience rock burst condition. On the other hand, when the rock mass is not over-stressed, the ground condition is termed as stable or competent (non-squeezing).

Tunnelling in the competent ground conditions can again face two situations – (i) where no supports are required, i.e., a self-supporting condition and (ii) where supports are required for stability; which is a non-squeezing condition. The squeezing ground condition has been divided into four classes on the basis of tunnel closures by Hoek (2001) as minor, severe, very severe and extreme squeezing ground conditions (Table 13.1).

The worldwide experience is that tunnelling through the squeezing ground condition is a very slow and hazardous process because the rock mass around the opening loses its inherent strength under the influence of in situ stresses. This may result in mobilization of high support pressure and tunnel closures. Tunnelling under the non-squeezing ground condition, on the other hand, is comparatively safe and easy because the inherent strength of the rock mass is maintained. Therefore, the first important step is to assess whether a tunnel would experience a squeezing ground condition or a non-squeezing ground condition. This decision controls the selection of the realignment, excavation method and the support system. For example, a large tunnel could possibly be excavated full face with light supports under the non-squeezing ground condition. It may have to be excavated by

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Table 13.1 Classification of ground conditions for tunnelling (Singh & Goel, 1999).

S.No.	Ground condition class	Sub-class	Rock behavior
1.	Competent self-supporting	–	Massive rock mass requires no support for tunnel stability
2.	Incompetent non-squeezing	–	Jointed rock mass requires supports for tunnel stability. Tunnel walls are stable and do not close
3.	Ravelling	–	Chunks or flakes of rock mass begin to drop out of the arch or walls after the rock mass is excavated
4.	Squeezing	Minor squeezing ($u_a/a = 1$–2.5%) Severe squeezing ($u_a/a = 2.5$–5%) Very severe squeezing ($u_a/a = 5$–10%) Extreme squeezing ($u_a/a > 10\%$) (Hoek, 2001)	Rock mass squeezes plastically into the tunnel both from the roof and the walls and the phenomenon is time dependent; rate of squeezing depends upon the degree of over-stress; may occur at shallow depths in weak rock masses like shales, clay, etc.; hard rock masses under high cover may experience slabbing/popping/rock burst
5.	Swelling	–	Rock mass absorbs water, increases in volume and expands slowly into the tunnel (e.g., in montmorillonite clay)
6.	Running	–	Granular material becomes unstable within steep shear zones
7.	Flowing/sudden flooding	–	A mixture of soil like material and water flows into the tunnel. The material can flow from invert as well as from the face crown and wall and can flow for large distances completely filling the tunnel and burying machines in some cases. The discharge may be 10–100 l/s which can cause sudden flood. A chimney may be formed along thick shear zones and weak zones.
8.	Rock burst	–	A violent failure in hard (brittle) and massive rock masses of Class II* type when subjected to high stress

Notations: u_a = radial tunnel closure; a = tunnel radius; u_a/a = normalized tunnel closure in percentage; * UCS test on Class II type rock shows reversal of strain after peak failure.

heading and benching method with a flexible support system under the squeezing ground condition.

Non-squeezing ground conditions are common in most of the projects. The squeezing conditions are common in the Lower Himalaya in India, Alps and other young mountains of the world where the rock masses are weak, highly jointed, faulted, folded and tectonically disturbed and the overburden is high.

13.2 THE TUNNELLING HAZARDS

Various tunnelling conditions encountered during tunnelling have been summarized in Table 13.1. Table 13.2 suggests the method of excavation, the type of supports and precautions for various ground conditions. Table 13.3 summarizes different conditions for tunnel collapse caused by geological unforeseen conditions, inadequacy of design models or support systems (Vlasov et al., 2001).

Commission on Squeezing Rocks in Tunnels of International Society for Rock Mechanics (ISRM) has published *Definitions of Squeezing* as reproduced here (Barla, 1995).

"Squeezing of rock is the time dependent large deformation, which occurs around a tunnel and other underground openings, and is essentially associated with creep caused by (stress) exceeding shear strength (limiting shear stress). Deformation may terminate during construction or continue over a long time period."

This definition is complemented by the following additional statements:

- Squeezing can occur in both rock and soil as long as the particular combination of induced stresses and material properties pushes some zones around the tunnel beyond the limiting shear stress at which creep starts.
- The magnitude of the tunnel convergence associated with squeezing, the rate of deformation and the extent of the yielding zone around the tunnel depend on the geological conditions, the in situ stresses relative to rock mass strength, the ground water flow and pore pressure and the rock mass properties.
- Squeezing of rock masses can occur as squeezing of intact rock, as squeezing of infilled rock discontinuities and/or along bedding and foliation surfaces, joints and faults.
- Squeezing is synonymous of over-stressing and does not comprise deformations caused by loosening as might occur at the roof or at the walls of tunnels in jointed rock masses. Rock bursting phenomena do not belong to squeezing.
- Time-dependent displacements around tunnels of similar magnitudes as in squeezing ground conditions, may also occur in rocks susceptible to swelling. While swelling always implies volume increase due to penetration of the air and moisture into the rock, squeezing does not, except for rocks exhibit a dilatant behavior. However, it is recognized that in some cases squeezing may be associated with swelling.

Table 13.2 Method of excavation, type of supports and precautions to be adopted for different ground conditions.

S.No	Ground conditions	Excavation method	Type of support	Precautions
1.	Self-supporting/ competent	TBM or full face drill and controlled blast.	No support or spot bolting with a thin layer of shotcrete to prevent widening of joints.	Look out for localized wedge/shear zone. Past experience discourages use of TBM if geological conditions change frequently.
2.	Non-squeezing/ incompetent	Full face drill and controlled blast by boomers.	Flexible support; shotcrete and pre-tensioned-rock-bolt supports of required capacity. Steel fiber reinforced shotcrete (SFRS) may or may not be required.	First layer of shotcrete should be applied after some delay but within the stand-up time to release the strain energy of rock mass.
3.	Ravelling	Heading and bench; drill and blast manually.	Steel support with struts/pre-tensioned rock bolts with steel fiber reinforced shotcrete (SFRS).	Expect heavy loads including side pressure.
4.	Minor squeezing	Heading and bench; drill and blast.	Full column grouted rock anchors and SFRS. Floor to be shotcreted to complete a support ring.	Install support after each blast; circular shape is ideal; side pressure is expected; do not have a long heading which delays completion of support ring.
5.	Severe squeezing	Heading and bench; drill and blast.	Flexible support; full column grouted highly ductile rock anchors and SFRS. Floor bolting to avoid floor heaving & to develop a reinforced rock frame. In case of steel ribs, these should be installed and embedded in shotcrete to withstand high support pressure.	Install support after each blast; increase the tunnel diameter to absorb desirable closure; circular shape is ideal; side pressure is expected; instrumentation is essential.

6.	Very severe squeezing and extreme squeezing	Heading and bench in small tunnels and multiple drift method in large tunnels; use forepoling if stand-up time is low.	Very flexible support; full-column grouted highly ductile rock anchors and thick SFRS; yielding steel ribs with struts when shotcrete fails repeatedly; steel ribs may be used to supplement shotcrete to withstand high support pressure; close ring by erecting invert support; encase steel ribs in shotcrete, floor bolting to avoid floor heaving; sometimes steel ribs with loose backfill are also used to release the strain energy in a controlled manner (tunnel closure more than 4 percent shall not be permitted).	Increase the tunnel diameter to absorb desirable closure; provide invert support as early as possible to mobilize full support capacity; long-term instrumentation is essential; circular shape is ideal.
7.	Swelling	Full face or heading and bench; drill and blast.	Full-column grouted rock anchors with SFRS shall be used all-round the tunnel; increase 30 percent thickness of shotcrete due to weak bond of the shotcrete with rock mass; erect invert strut. The first layer of shotcrete is sprayed immediately to prevent ingress of moisture into rock mass.	Increase the tunnel diameter to absorb the expected closure; prevent exposure of swelling minerals to moisture, monitor tunnel closure.
8.	Running and flowing	Multiple drift with forepoles; grouting of the ground is essential; shield tunnelling may be used in soil conditions.	Full column grouted rock anchors and SFRS; concrete lining up to face, steel liner in exceptional cases with shield tunnelling. Use probe hole to discharge ground water. Face should also be grouted, bolted and shotcreted.	Progress is very slow. Trained crew should be deployed. In case of sudden flooding, the tunnel is realigned by-passing the same. Monitor rate of flow of seepage.
9.	Rock burst	Full face drill and blast	Fiber reinforced shotcrete with full column resin anchors immediately after excavation.	Micro-seismic monitoring is essential.

Table 13.3 Quality aspects related to tunnel collapses (Vlasov et al., 2001).

S.No.		Type	Phenomenon	Cause	Remedial measures
1.	Ground collapse	Ground collapse near the portal	During the excavation of the upper half section of the portal the tunnel collapsed and the surrounding ground slid to the river side.	Ground collapse was caused by the increase of pore water pressure due to rain for five consecutive days.	• Installation of anchors to prevent landslides • Construction of counter-weight embankment which can also prevent landslide. • Installation of pipe roofs to strengthen the loosened crown.
2.		Landslide near the portal	Cracks appeared in the ground surface during the excavation of the side drifts of the portal, and the slope near the portal gradually collapsed.	Excavation of the toe of the slope composed of strata disturbed the stability of soil, and excavation of the side drifts loosened the natural ground, which led to landslide.	• Caisson type pile foundations were constructed to prevent unsymmetrical ground pressure. • Vertical reinforcement bars were driven into the ground to increase its strength.
3.		Collapse of the crown of cutting face.	10 to 30 m^3 of soil collapsed and supports settled during excavation of the upper half section.	The ground loosened and collapsed due to the presence of heavily jointed fractured rock mass at the crown of the cutting face, and the vibration caused by the blasting for the lower half section (hard rock).	• Roof bolts were driven into the ground in order to stabilize the tunnel crown. • In order to strengthen the ground near the portal and talus, chemical injection and installation of vertical reinforcement bars were conducted.

4.		Collapse of fault fracture zone	After completion of blasting and mucking, flaking of sprayed concrete occurred behind the cutting face, following which, 40 to 50 m^3 of soil collapsed along a 7 m section from the cutting face. Later it extended to 13 m from the cutting face and the volume of collapsed soil reached 900 m^3.	The fault fracture zone above the collapsed cutting face loosened due to blasting, and excessive concentrated loads were imposed on supports, causing the shear failure and collapse of the sprayed concrete.	• Reinforcement of supports behind the collapsed location (additional sprayed concrete, additional rock bolts). • Addition of the number of measurement section. • Hardening of the collapsed muck by chemical injection. • Air milk injection into the voids above the collapsed portions. • Use of supports with a higher strength.
5.	Distortion of supports	Distortion of tunnel supports	During excavation by the full face tunnelling method, steel supports considerably settled and foot protection concrete cracked.	Bearing capacity of the ground at the bottom of supports decreased due to prolonged immersion by ground water.	• Permanent foot protection concrete was placed in order to decrease the concentrated load. • An invert with drainage was placed.
6.		Distortion of lining concrete due to unsymmetrical ground pressure.	During the excavation of the upper half section, horizontal cracks ranging in width from 0.1 to 0.4 mm appeared in the arch portion of the mountain side concrete lining, while subsidence reached the ground surface on the valley side.	Landslide was caused due to the steep topography with asymmetric pressure and the ground with lower strength, leading to the oblique load on the lining concrete.	• Earth anchors were driven into the mountain side ground to withstand the oblique load. • Ground around the tunnel was strengthened by chemical injection. Subsidence location was filled.

Continued

Table 13.3—Continued

S.No.	Type	Phenomenon	Cause	Remedial measures
7.	Distortion of tunnel supports due to swelling pressure	Hexagonal cracks appeared in the sprayed concrete and the bearing plates for rock bolts were distorted due to the sudden inward movement of the side walls of the tunnel.	Large swelling pressure was generated by swelling clay minerals in mudstone.	• Sprayed concrete and face support bolts on the cutting face were provided to prevent weathering. • A temporary invert was placed in the upper half section by spraying concrete.
8.	Heaving of a tunnel in service	Heaving occurred in the pavement surface six months after the commencement of service, causing cracks and faulting in the pavement. Heaving reached as large as 25 cm.	A fault fracture zone containing swelling clay minerals, which was subjected to hydrothermal alteration, existed in the distorted section. Plastic ground pressure caused by this fracture zone concentrated on the base course of the weak tunnel section without invert	• In order to restrict the plastic ground pressure, rock bolts and sprayed concrete were applied to the soft sandy soil beneath the base course. • Reinforced invert concrete was placed.

9.	Adverse effects on the surrounding environment	Adverse effects of vibration due to blasting on the adjacent existing tunnel.	During the construction of a new tunnel, which runs parallel to the side wall of the existing portal, cracks appeared in the lining (made of bricks) of the existing tunnel.	The voids behind the existing tunnel loosened and the lining was distorted due to the vibration of the blasting for construction of the new tunnel.	• Steel supports and temporary concrete lining were provided to protect the existing tunnel. • Backfill grouting was carried out. • Excavation was carried out by the non-blasting rock breaking method and the limit for chemical agent was set to mitigate the vibration.
10.		Ground settlement due to the excavation for dual-tunnel directly beneath residential area.	Considerable distortion of supports occurred in the embankment section. Although additional bolts were driven into ground and additional sprayed concrete was provided, ground surface settlement exceeded 100 mm.	Since the soil characteristics in the embankment section were worse than expected, the ground settlement was considerably increased by the construction of tunnels following the dual-tunnel.	• Pipe roofs were driven from inside the tunnel to reduce ground surface settlement.

Summary table of different conditions for tunnel collapses, caused by geological unforeseen conditions, inadequacy of design models or support systems.

Table 13.4 Comparison between squeezing and swelling phenomena (Jethwa & Dhar, 1996).

Parameter	Squeezing	Swelling
1. Cause	Small volumetric expansion of weak and soft ground upon stress-induced shear failure Compaction zone can form within broken zone	Volumetric expansion due to ingress of moisture in ground containing highly swelling minerals
2. Closure		
• Rate of closure	(i) Very high initial rate, several centimeters per day for the first 1–2 weeks of excavation (ii) Reduces with time	(i) High initial rate for first 1–2 weeks till moisture penetrates deep into the ground (ii) Decreases with time as moisture penetrates into the ground deeplywith difficulty
• Period	(iii) May continue for years in exceptional case	(iii) May continue for years if the moist ground is scooped out to expose fresh ground
3. Extent	The affected zone can be several tunnel diameters thick	The affected zone is several meters thick. Post-construction saturation may increase swelling zone significantly

- Squeezing is closely related to the excavation, support techniques and sequence adopted in tunnelling. If the support installation is delayed, the rock mass moves into the tunnel and a stress re-distribution takes place around it. Conversely, if the rock deformations are constrained, squeezing will lead to long-term load build-up of rock support.

A comparison between squeezing and swelling phenomena by Jethwa (1981) and Jethwa and Dhar (1996) is given in Table 13.4. Various approaches for estimating the ground conditions for tunnelling on the basis of Q and modified Q, i.e., rock mass number N are discussed in book by Singh and Goel (1999).

13.3 EMPIRICAL APPROACH FOR PREDICTING DEGREE OF SQUEEZING

Fig. 13.1 shows zones of tunnelling hazards depending upon the values of $HB^{0.1}$ and N (rock mass quality Q with SRF = 1). Here H is the overburden in meters, B is the width of the tunnel or cavern in meters and N is rock mass number (Chapter 6). It should

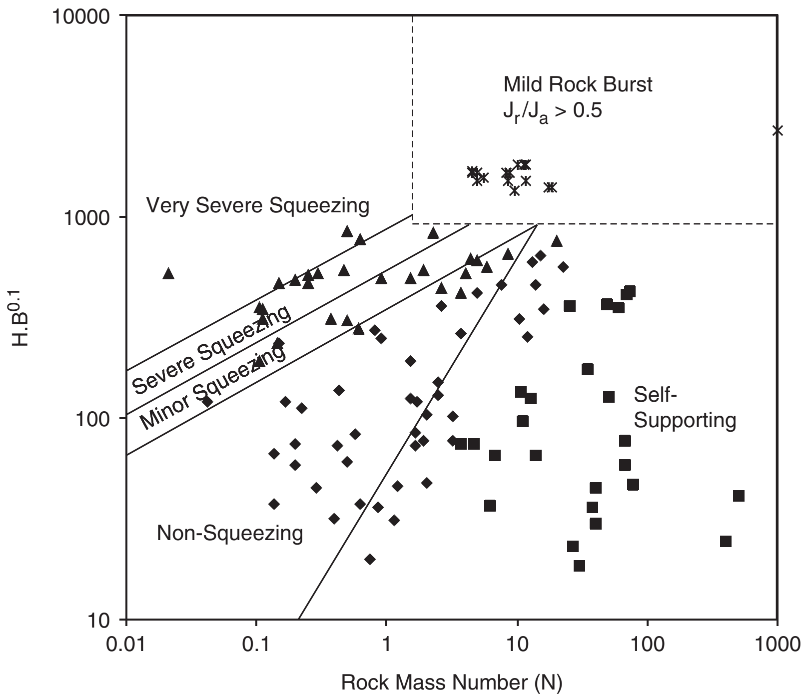

Fig. 13.1 Plot between rock mass number N and $HB^{0.1}$ for predicting ground conditions (Goel, 1994).

be noted that B should be more than the size of self-supporting tunnels (Singh and Goel, 1999).

For a squeezing ground condition

$$H >> (275\,N^{0.33}) \cdot B^{-0.1}\text{meters}$$
$$\frac{J_r}{J_a} \leq \frac{1}{2} \tag{13.1}$$

For a non-squeezing ground condition

$$H << (275\,N^{0.33}) \cdot B^{-0.1}\text{meters} \tag{13.2}$$

13.4 SUDDEN FLOODING OF TUNNELS

The inclined beds of impervious rocks (shale, phyllite, schist, etc.) and pervious rocks (crushed quartzites, sandstone, limestone, fault, etc.) may be found along a tunnel alignment. The heavy rains/snow charge the beds of pervious rocks with water like an acquifer. While tunnelling through the impervious bed into a pervious bed, seepage water may gush out suddenly. Authors have studied four similar case histories of Chhibro – Khodri, Maneri Bhali, BSL and Dulhasti hydroelectric projects in Himalaya. Experience is that sudden flood accompanied by huge out-wash of sand and boulders may occur ahead of tunnel face where several shear zones exist. This flooding problem becomes dangerous where the pervious rock mass is squeezing ground also due to the excessive overburden. The machines and tunnel boring machines are buried partly.

The seepage should be monitored near the portal regularly. The discharge of water should be plotted along chainage of the face of tunnel. If the peak discharge is found to increase with tunnelling, it is very likely that sudden flooding of the tunnel may take place on further tunnelling. It is suggested that the international experts be consulted for tackling such situations.

13.5 CHIMNEY FORMATION

There may be local thick shear zones dipping towards a tunnel face. The soil/gouge may fall down rapidly, unless it is supported carefully and immediately. Thus, a high cavity/chimney may be formed along the thick shear zone. The chimney may be very high in water-charged rock mass. This cavity should be back-filled by lean concrete completely.

13.6 ENVIRONMENTAL HAZARDS DUE TO TOXIC OR EXPLOSIVE GASES AND GEOTHERMAL GRADIENT

There are serious environmental hazards due to toxic or explosive gases while tunnelling in the argillaceous rocks. Sometimes methane gas is emitted by blasted shales. Improper ventilation also increases concentration of toxic gases like carbon monoxide, carbon dioxide, hydrogen sulfide and sulfur dioxide. Additional ventilation capacity is required. In case of methane gas emission, permissible electrical equipment may be used. Attention should be given to the physical properties of the gases, as some gases tend to collect either in high or low pockets in a tunnel complex. Table 13.5 summarizes the properties of above mentioned gases found in tunnels (Mathews, 1982). Monitoring of gases and oxygen should be carried out near the face of a tunnel specially where blast fumes and gas emissions are maximum. Oxygen must be maintained at a level of 20 percent or greater. Dust inside the tunnel should also be controlled for reducing health hazards. Therefore, wet drilling method is recommended for both blastholes and boltholes.

As rock engineers are going deeper and deeper, workers face a high temperature. The temperature may increase at a rate of about 30°C per km. This is in addition to the average ground temperature which is equal to the average temperature in the whole year. The temperature inside a 1400 m deep NJPC tunnel in Himalaya, India, was

Table 13.5 Properties of various gases which may be present in tunnel (Mathews, 1982).

Gas	Density	Color	Odor	Source	Physiological effect on workmen
Oxygen (O_2)	1.11	None	None	Air is normally 20.93% O_2	At least 20% is required to sustain normal health. Workmen become dizzy if concentration drops to 15%. Some workmen may die at 12.5%; most will faint at a concentration of 9%; and death will occur at 6% or less.
Nitrogen (N_2)	0.97	Yellow	None	Air is normally 78.10% N_2	Nitrogen has no ill effect on persons except to dilute air and decrease O_2%.
Carbon dioxide (CO_2)	1.50	None	None	Air is normally 0.03% CO_2. CO_2 is produced by decaying timber and fires, and is present in diesel exhaust.	CO_2 acts as a respiratory stimulant and may increase effects of other harmful contaminants. At 5% CO_2, breathing is laborious. A concentration of 10% can be endured for only a few minutes.
Carbon monoxide (CO)	0.97	None	None	Present in diesel exhaust and blast fumes.	CO is absorbed into the blood rather than O_2. In time, very small concentrations will produce symptoms of poisoning. A concentration slightly greater than 0.01% will cause a headache or possibly nausea. A concentration of 0.2% is fatal.
Methane (CH_4)	0.55	None	None	Present in certain rock formations containing carbonaceous materials.	Has no ill effect on persons except to dilute air and decrease O_2%. It is dangerous because of its explosive properties. Methane is explosive in the concentration range of 5.5 to 14.8%, being most explosive at a concentration of 9.5%.

Continued

Table 13.5—Continued

Gas	Density	Color	Odor	Source	Physiological effect on workmen
Hydrogen sulfide (H_2S)	1.19	None	Rotten eggs	Present in certain rock formations and sometimes in blast fumes.	Extremely poisonous – 0.06% will cause serious problems in a few minutes.
Sulfur dioxide (SO_2)	2.26	None	Burning sulfur	Present in diesel exhaust and blast fumes.	Strongly irritating to mucous membranes at low concentrations. Can be kept below objectionable levels by limiting fuel sulfur content to 0.5 percent.
Oxides of nitrogen	Approx. 1.5	Yellow-brown	Stings nose	Present in diesel exhaust and blast fumes.	NO_2 is most toxic. All oxides of nitrogen cause severe irritation of the respiratory tract at high concentrations. Acute effects may be followed by death in a few days to several weeks owing to permanent lung damage.

as high as 45°C or more. The efficiency of workers in such a high temperature was reduced drastically. They worked for two or three hours only after taking bath frequently with ice-filled water. If possible, cool fresh air should be used for ventilation to maintain a working temperature of around 30°C at the tunnel face.

13.7 INTERACTION BETWEEN ROCK PARAMETERS

The real world response of rock masses is often highly coupled or interacting. There is a non-linear complex relationship between mechanical properties and rock parameters, especially in weak argillaceous rock masses. Hudson (1992) has shown schematically such complex interaction for tunnelling (Fig. 13.2). He identified the following 12 rock parameters affecting the tunnelling conditions.

1.	Excavation dimensions	Excavation size and geometry
2.	Rock support	Rock bolts, concrete liner, etc.
3.	Depth of excavations	Deep or shallow
4.	Excavation methods	Tunnel boring machines, blasting
5.	Rock mass quality	Poor, fair, good
6.	Discontinuity geometry	Sets, orientations, distributions, etc.
7.	Rock mass structure	Intact rock and discontinuities
8.	In situ rock stress	Principal stress magnitudes and directions
9.	Intact rock quality	Hard rocks or soft rocks
10.	Rock behavior	Responses of rocks to engineering activities
11.	Discontinuity aperture	Wide or narrow
12.	Hydraulic conditions	Permeabilities, water tables, etc.

Hudson (1992) has made system's approach very simple, interesting and based on actual experiences and judgments of tunnelling experts. His approach makes decision making very easy in planning of geotechnical investigations for tunnelling projects. Figs 13.2a–d are self-explanatory. For example (7,1) means effect of 7th parameter (rock mass structure) on the first parameter (excavation dimensions). Since the problem is coupled, coordinate (1,7) means effect or excavation dimensions on the rock mass structure, e.g., opening or discontinuities and development of new fractures.

13.8 CONCLUDING REMARKS

Rock has EGO (extraordinary geological occurrence) problem. Enormous time and money is lost due to unforeseen tunnelling hazards particularly in Himalaya and such other young mountain chains in the world. It is generally said that if a shear zone or a weak zone is not seen within 200 meters in lower Himalaya, it means that it has been missed. Thus, geological uncertainties may be managed by adopting a strategy of tunnel construction

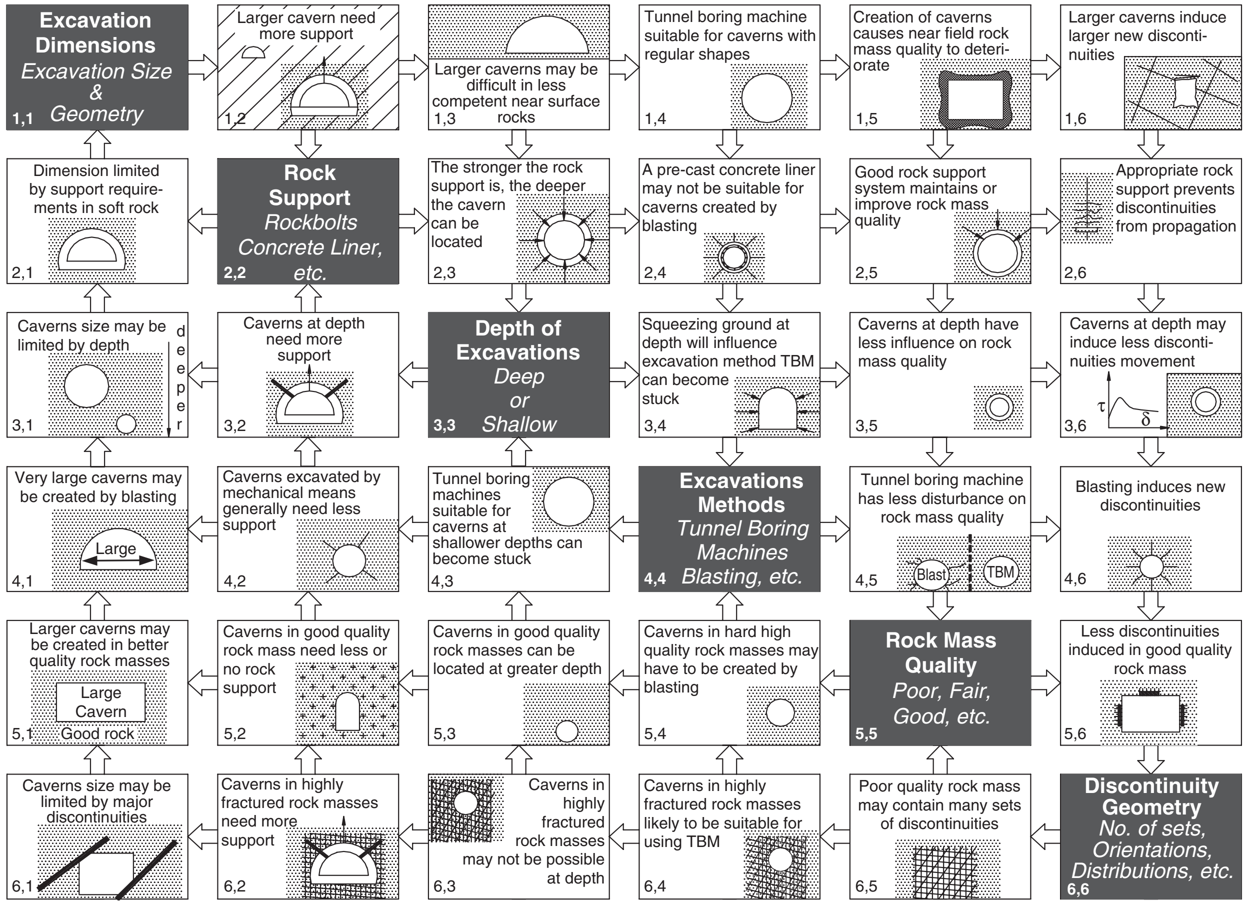

Fig. 13.2a Interaction of rock parameters in the underground excavations (Hudson, 1991).

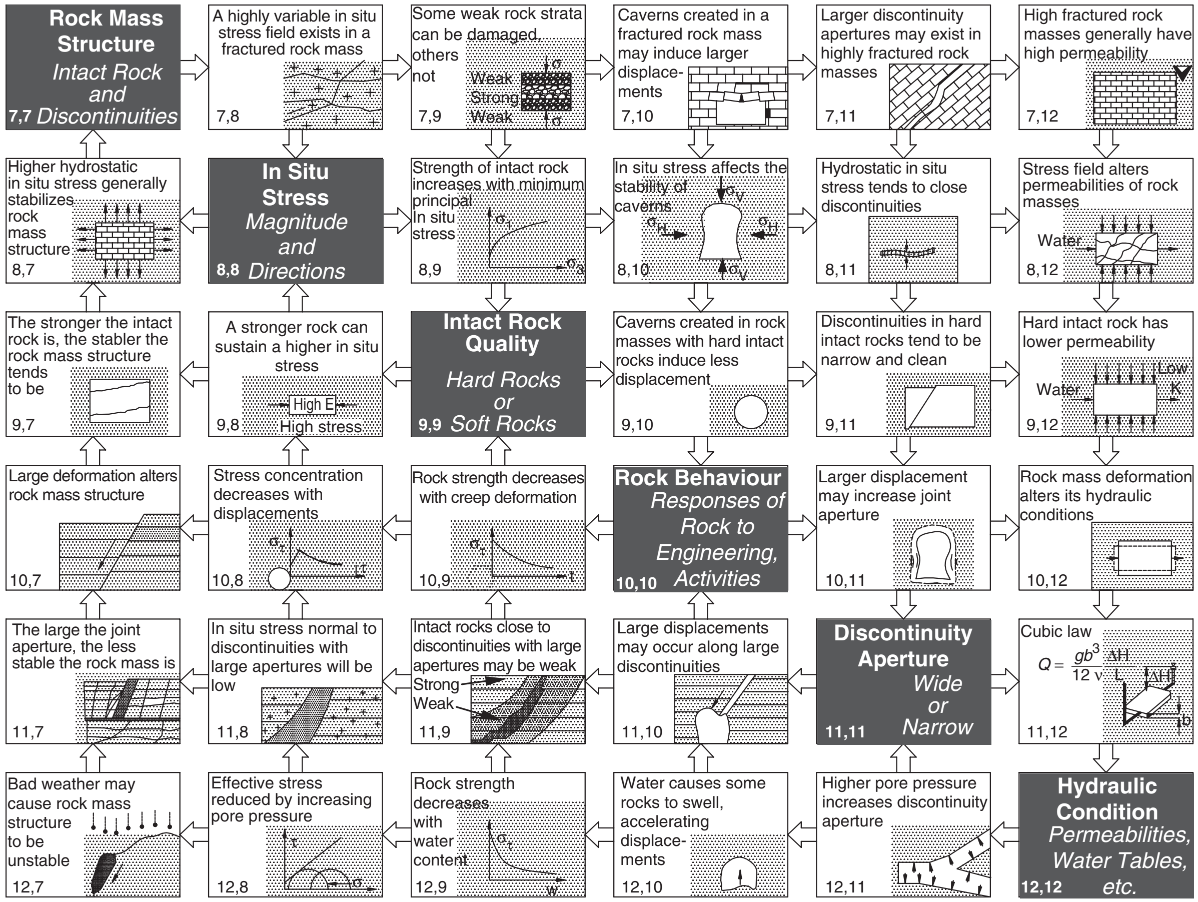

Fig. 13.2 b Interaction of rock parameters in the underground excavations (Hudson, 1991).

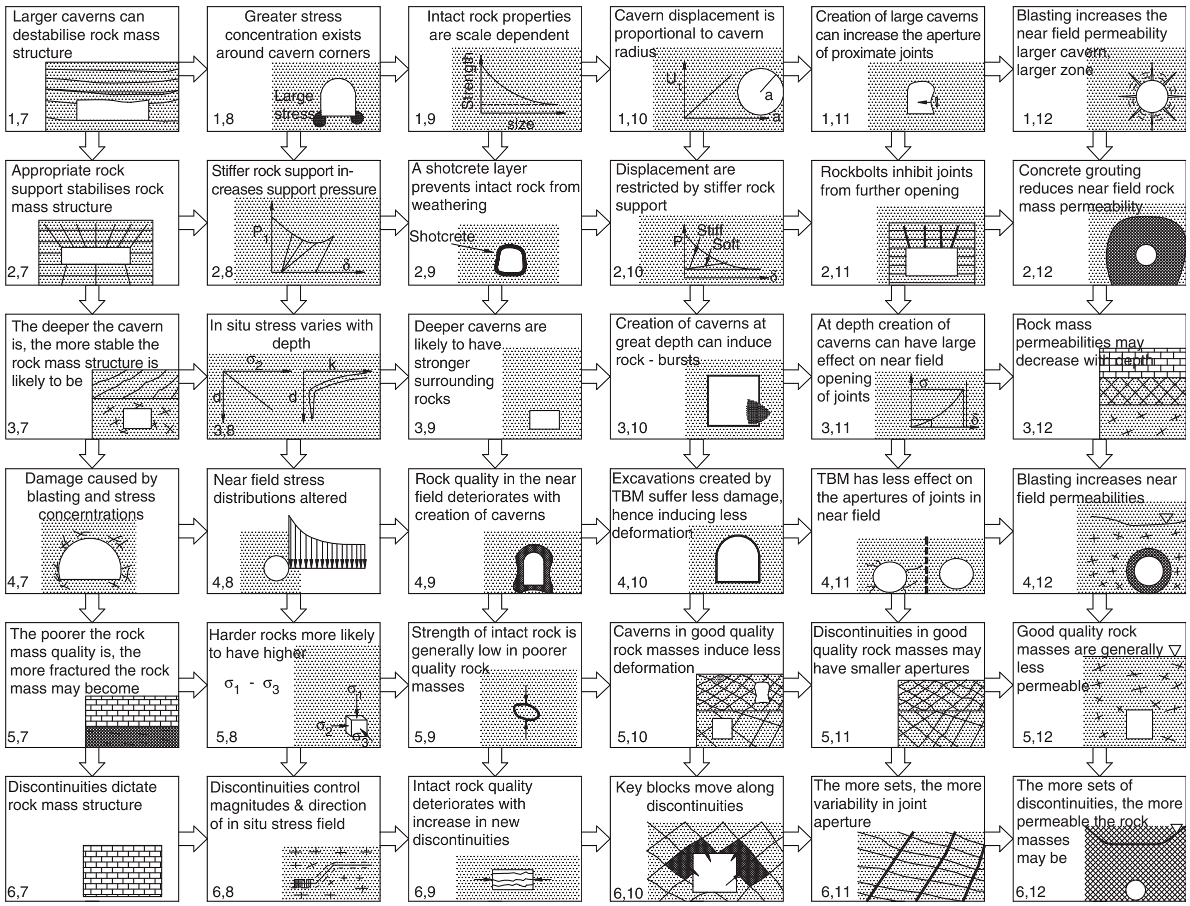

Fig. 13.2 c   Interaction of rock parameters in the underground excavations (Hudson, 1991).

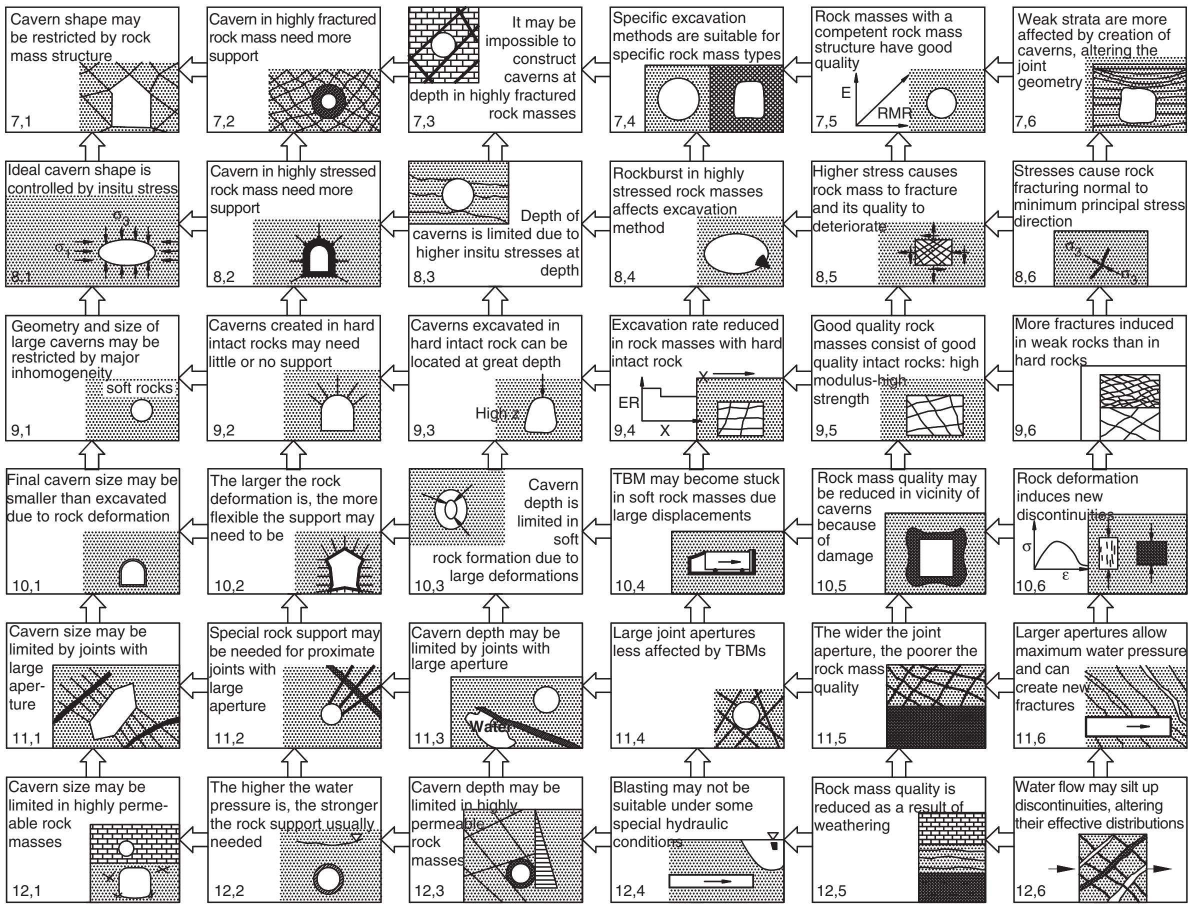

Fig. 13.2 d Interaction of rock parameters in the underground excavations (Hudson, 1991).

which can cope up with most of the tunnelling conditions. A hazard foreseen is hazard controlled. Therefore, it is desirable to use safe and effective tunnelling methodology, based on detailed explorations before and during the tunnel construction. Of course the modern trend of insuring the tunnelling machine and the losses due to delays because of unexpected geological and geohydrological conditions takes care of the contractors' interests.

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14
Tunnel instrumentation

"If you do not do the thing you fear, the fear controls your life."

Brian Tracy

14.1 INTRODUCTION

Improvements in design methods stem from the knowledge of behavior of the designed structures. Abnormal conditions indicate that the factor of safety has fallen to one. In early days of tunnel engineering, failures were a powerful motivation for developments in design techniques. In the past, there was a little emphasis on instrumentation although ad hoc observations were sometimes made if failure appeared imminent. The recent projects have created many problems, e.g., rock bursts, support failures due to intense support pressures generated by squeezing and swelling rocks, water inrush, gas explosion, etc. The construction engineers are facing these problems while constructing tunnels. For example, steel supports have failed in many cases. As such field instrumentation for support and lining design is gaining popularity among both designers and construction engineers with a little hesitation due to initial hindrance to construction progress. Of course, eventually the construction engineers did realize the net saving in the time of completion of tunnel owing to reduction in the number of tunnelling hazards and cost overruns.

14.2 BASIC CONSIDERATIONS AND REQUIREMENTS FOR THE DESIGN OF AN OPENING IN ROCK

Design of an underground structure in rock or evaluation of stability and safety of an existing structure calls for determination of (i) the deformation and/or stresses in the structure resulting from external loads applied to the structure; (ii) the physical properties of the rock in the vicinity of the opening; (iii) the ability of the structure to withstand the applied stresses or deformations; (iv) the geometry of the opening and (v) the regional geology and its influence on the stress and displacement distribution in the vicinity of the opening. Suitable analytical methods, model studies and/or numerical procedures, such

Tunnelling in Weak Rocks
B. Singh and R. K. Goel

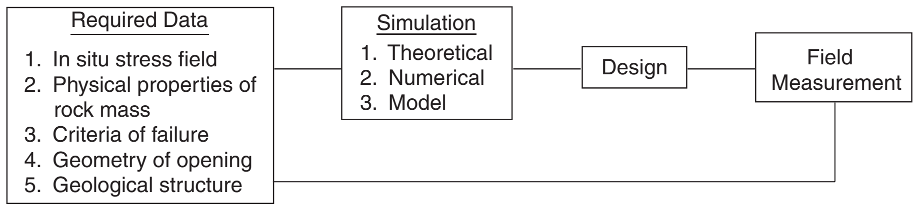


Fig. 14.1 Basic requirements for design of a structure in rock.

as the finite element or distinct element method must be used to incorporate this data into the initial stage of design. However, the initial design should be frequently evaluated by conducting full scale field tests and then by correlating the field data with predicted results. Repeated feedback of field data into the input data of the design helps in the development of the reliable design techniques. Thus design modifications can be effected if the results of instrumentation and experience warrant. Principles of this procedure are illustrated in Fig.14.1.

14.3 DATA REQUIREMENT

The data obtained from instrumentation work may be used for, (a) selection of economical support during construction period and (b) determination of basic tunnel lining design-criteria, such as in pressure tunnels.

Following information is required while dealing with tunnel supports in an effective manner:

(i) Are the tunnel supports strong enough for the purpose for which they were installed?
(ii) How the rock mass around the tunnel is behaving? Where does the rock load come from? Is the support pressure due to loosening of rock or squeezing of rock under overburden pressure or due to swelling?

The answers to these questions can come only through instrumentation, observation and geological data. Thus, zones of high support pressure may be delineated and the pressures are actually measured so that stronger lining is installed but without being overconservative and taking risk to life and structure. On the basis of measured loads, it may be possible to use lighter lining compared to that predicted from the design calculations. The actual distribution of load on the lining can also be ascertained – a factor that no theory looks into. For example, in a zone of high support pressure, there may be significant variation in the distribution of support pressure and stronger lining need to be designed for higher bending moments (if its bonding with rock mass is not good).

Another major problem faced by a designer is what direction of rock load is considered in designing a support system. Is it absolutely essential to place invert (or bottom strut) along with roof support so that mucking roof operation is least disturbed? The answer is affirmative if measurements on rock load indicate high horizontal support pressure.

In the last but not of the least importance, we must know the radius of failed rock (called coffin cover or the broken zone) in the cases of squeezing rock masses. Table 14.1 summarizes typical applications of various instruments.

Table 14.1 Typical applications for various measuring instruments.

Measurement	Instrument	Typical applications
Surface settlement	Single point borehole extensometer Multi-point borehole extensometer Vertical pipe or tube settlement gauge	Forewarning of surface settlement. Verifying adequacy of tail void filling. Monitoring basal stability of open cut excavations in clay.
Subsurface horizontal movement	Inclinometer Borehole extensometer	Forewarning of surface settlement or tunnel instability by monitoring ground movement towards excavation or heading. Verifying adequacy of rock bolting and other support.
Diameter or width change	Tape extensometer	Monitoring changing width of open cut as indication of stability. Monitoring lining distortion.
Tilt	Tilt meter	Monitoring tilting of buildings
Load or stress in structural support	Load cell Strain gauge	Verifying adequacy of structural support (rock bolts, ribs, liner plates, precast concrete liners, cross-lot bracing, tie-backs). Determining factor of safety on shoving stresses in soft ground shield tunnelling. Increasing knowledge of support behavior as input to improved design procedures.
Groundwater level	Observation well	Monitoring draw down of groundwater table due to tunnelling or shaft.
Pore water pressure	Piezometer	Forewarning of distress to buildings due to movement of soil or water towards soft ground tunnel or shaft.
Vibration	Engineering seismograph	Verifying that ground and building vibrations due to blasting do not exceed an acceptable limit.

14.4 INSTRUMENTATION FOR TUNNELLING

Instrumentation for determination of various parameters which are helpful in evaluating a suitable tunnel support system is dealt below.

14.4.1 Tunnel instrumentation: why?

Tunnel instrumentation can help in the following ways.

- Verifying design of support system
- Advancing the state-of-the-art
- Providing adequacy of new construction techniques
- Controlling quality of construction
- Reducing construction cost and extra payment
- Diagnosing the cause of a problem
- Improving construction safety by providing warning system
- Documenting as-built conditions
- Providing legal protection
- Enhancing public relations, bonding team spirit between engineers, geologists and contractors.

14.5 STRESS FIELD

Knowledge of stress field existing in a particular region prior to tunnelling is essential. Measurement of in situ stresses have shown that in many cases measured stresses are anomalous, in the sense that the horizontal stresses cannot be attributed to gravity load, even when allowances are made for variations in surface level. This anomaly is believed to be a result of tectonic stresses which are caused by movements and strains in the earth "plate" or the "continent" of the region under consideration. For example, the Indian subcontinent is drifting 25° N towards China. As such horizontal stresses in Himalayas may be high.

Generally, tectonic stresses also affect both the vertical and horizontal components of stress vector and the principal direction of stresses will be in some direction other than the horizontal or vertical. Field measurements of the in situ stress field indicate that, in general the stress fields are triaxial or polyaxial, that is, there are two or more unequal principal stresses and these are not oriented in the vertical and horizontal directions.

The determination of both the magnitude and direction of the in situ stress field plays a key role in the design of an underground structure.

14.5.1 Virgin and induced stresses

Stress distribution in rock mass prior to excavation is known as virgin or primitive stress and the stress distribution resulting from an excavation is known as induced stress. Various methods and principles for determining virgin and induced stresses are given in Table 14.2.

14.5.2 Stress measurement in the interior of rock mass

Among the various methods that can be used to measure stresses in rock mass, the most widely used is the one described here. The instrument used is a three component borehole deformation gauge developed by U.S. Bureau of Mines (Merrill, 1967).

The gauges contact the wall of a borehole at six points through movable pistons provided at diametrically opposite ends. With the deformation of borehole due to overcoring, the pistons actuate three pairs of beryllium–copper cantilevers. Four strain gauges mounted on opposite cantilevers are connected to form a single four arm bridge. The bending strain produced in the cantilevers is measured by the resistance strain gauges and is read on a resistance strain gauge indicator. The bending of cantilevers and hence the change in resistance is pre-calibrated with the movement of the pistons. The magnitude of the stress relief measurement is simply the difference between the indicator reading before overcoring and the indicator reading after overcoring, provided that the overcoring bit has penetrated sufficiently past the gauge.

After in situ stress relief measurements have been obtained, the borehole deformation gauge is used in conjunction with a biaxial pressure cell to determine the elastic modulus of the rock material (E_r). The stresses in the plane normal to the borehole can then be calculated using the following equations:

$$P + Q = \frac{E_r}{3d(1-\nu^2)}(U_1 + U_2 + U_3) \tag{14.1}$$

$$P - Q = \frac{\nu^2 \cdot E_r}{6d \cdot (1-\nu^2)}\left[(U_1 - U_2)^2 + (U_2 - U_3)^2 + (U_1 - U_3)^2\right] \tag{14.2}$$

$$\tan 2\theta = \frac{\nu^3 \left(U_2 - U_3\right)}{2U_1 - U_2 - U_3} \tag{14.3}$$

Where U_1, U_2 and U_3 are the three borehole deformation measurements, E_r and ν are elastic modulus and Poisson's ratio respectively of the rock material, d is diameter of pilot borehole and P and Q are the maximum and minimum principal stresses in the plane normal to the borehole. The angle θ is the angle from the deformation U_1 to the maximum stress P. These equations are for relatively isotropic rock material. If this is not the case, another system of equations must be used.

After borehole deformation measurements have been made in three non-parallel holes, all components of the three-dimensional stress field may be calculated in any coordinate system.

Table 14.2 Methods of measuring virgin stresses.

Nature of stress	Measuring method	Measuring principle	Instrument	Site of utilization
Virgin	Stress relieving	Electrical resistance	1. Glued strain gauges	End of borehole
			2. As above	Wall of borehole
			3. As above	At surface
			4. Borehole gauge	Along borehole
			5. Diametral deformation gauge	As above
			6. Strain gauge tensor	As above
			7. Rigid brass plug	As above
			8. Rigid inclusion gauge	As above
	Stress compensating	Induction magnetism	L.V.D.T.	Along borehole
			Borehole deformation gauge	As above
		Photo-elasticity	Drill-hole pre-stressmeter	As above
		Acoustic	Borehole deformation gauge	As above
		Mechanical	Movable strain meter	At surface
		Biaxial pressure	Biaxial compensating gauge	Along borehole
		Uniaxial pressure	1. Flat jack grouted	At surface
			2. Flat jack ungrouted	At surface
Hydraulic	Hydraulic fracturing		Equipment for grouting pressure but only water is used to create fracture along borehole	The only method for directly measuring virgin stress at great depths along a borehole
	Others	Propogation velocity of vibration		Along borehole and at surface
		Radioactive		Along borehole
Induced	Stress measurement	Electrical resistance	Stress meter	Between rock and strut
		Uniaxial pressure	Pressure cell	Between rock and strut
	Strain measurement	Electrical resistance	Strain meter – 2 m long	Along borehole
			Multiple position borehole extensometer	Along borehole
		Mechanical	Invar wires multi-point extensometers	Along borehole
				Along borehole
			Extensometers (rod and wire)	Along borehole

In situ stresses are generally measured in solid blocks of rock material surrounded by sometimes fractured rock mass. As such block is overstressed than entire rock mass locally. So measured in situ stresses are more than overall in situ stresses at such situations. Further in situ horizontal stresses are more in hard rocks than in soft rock areas. The tectonic stress is found as,

Tectonic stress = Measured horizontal stress – gravitational horizontal stress

Microcracking along drill holes may give wrong picture of in situ stresses as found from borehole deformation gauge. The hydrofracturing test is gaining popularity for in situ stress measurement, as it is simple and cost-effective. However Cornet's method of hydrofracturing along joints is better than hydrofracturing of solid blocks for reasons mentioned above (Fairhurst, 1994).

14.5.3 Stresses at the rock face

Measurement of stresses at the rock face relates to stress conditions prevailing there and should not be mistaken for the natural stress condition existing in the interior of the rock mass prior to excavation, i.e., virgin stresses. In other words, measurements at rock face of an excavation gives induced stresses. These are useful in order to check if the induced stresses have attained or going to attain ultimate uniaxial compressive strength (UCS) of the surrounding rock indicating its possibility of failure.

Strain gauges or extensometers are arranged in a delta configuration (Fig. 14.2). A stress relieving borehole is then drilled at the center. The hole should be gradually

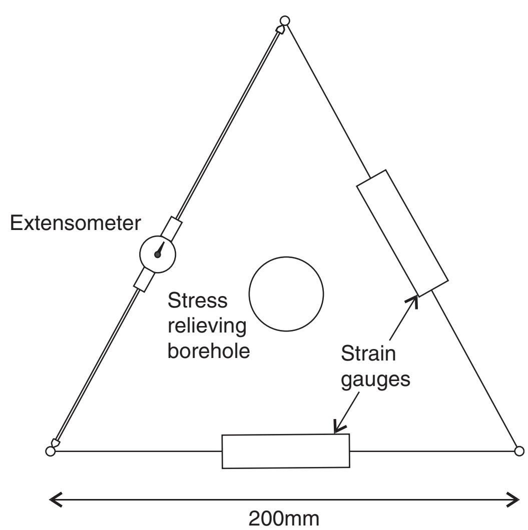


Fig. 14.2 Stress measurement at rock face by extensometers or strain gauges.

deepened and the resulting deformations should be continuously observed. Deformations are thus plotted against depth to find out local irregularities or disturbances. For a side length of 200 mm satisfactory results have been obtained with 56 mm diameter hole drilled with very fine diamond tipped bits.

14.6 SUPPORT PRESSURE IN TUNNELS

The term "support pressure" has been defined by Kastner (1962) as, referring to all effects of the induced state of stress which occur in the rock mass surrounding an unsupported excavation or which are in interaction with a support and which load the support system. Thus, support pressure relates to phenomenon caused by engineering activities but does not apply to virgin stresses.

14.6.1 Measurement by load cells and pressure cells

Many load cells are available based on various principles. Much work on their development has been done. These are in use in mines and tunnels of civil engineering projects for a considerable length of time. In the initial stages, the hydraulic load cells used in mines were adopted with some success in tunnels too. These had the disadvantages of low capacity, suitability for a smaller period only and were affected by eccentric loading, shearing action, etc. Another high capacity mechanical load cell developed for tunnels used to get damaged by blasting at face and hence could not be used at face. Now better versions of the load cells are available, which are vibrating wire type electrical load cells. The load cells are, in general, cylindrical shape device made up of steel having the height and width ratio less than one. The load measuring system is fitted inside the cylinder. The load applied on the two circular faces of cylinder is read by the readout. The load cells are installed at joints of supports and measures the hoop load on steel rib supports. Various typical installation arrangements in practice are shown in Fig. 14.3. Vertical and horizontal components of support pressure set up in Fig. 14.3b and 14.3c can be given as:

$$p_v = \frac{P_1 + P_3}{S_{rib} \cdot D} \tag{14.4}$$

$$p_h = \frac{2P_2}{S \cdot D} \tag{14.5}$$

where

p_v and p_h	=	vertical and horizontal components of support pressure, respectively exerted on the support;
P_1, P_2 and P_3	=	loads recorded by load cells at positions 1, 2 and 3, respectively;
S_{rib}	=	spacing of supports and
D	=	diameter of excavation.

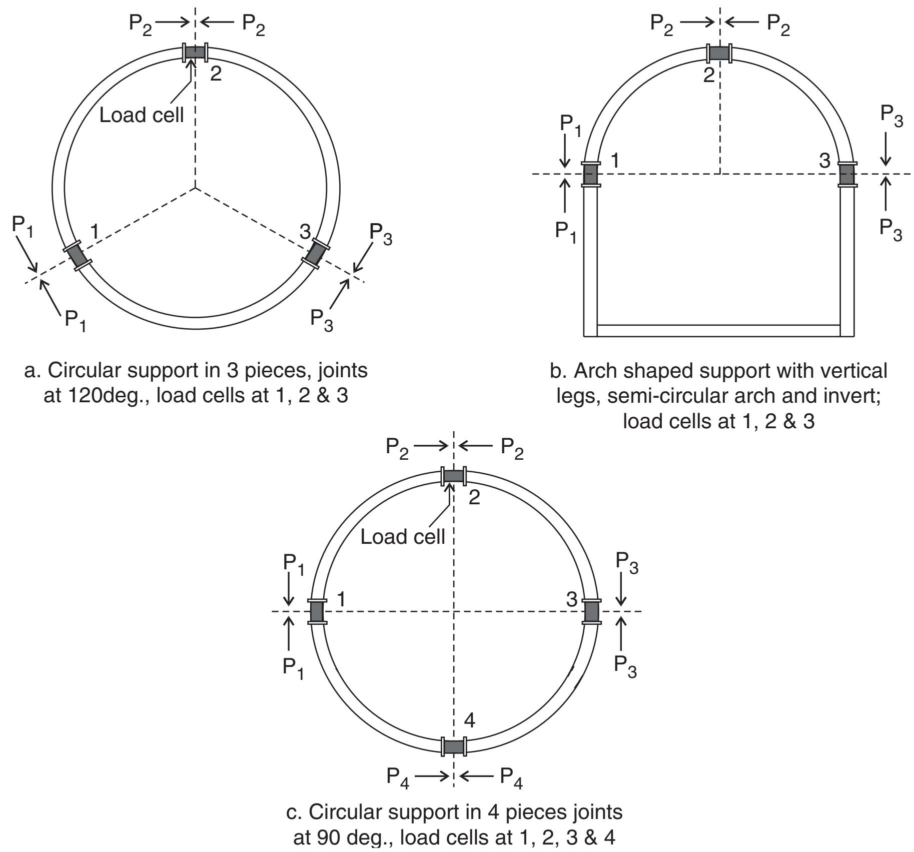


Fig. 14.3 Typical installation arrangements of load cells.

It is experienced that the observations are affected by break line, backfill material and its compactness, time interval between installation of load cells and excavation at a particular place and the face advance.

It is obvious that load cells thus used give average pressures in vertical and horizontal directions. The chances of survival of load cells may be as low as 40 percent due to blasting vibrations and failure of ribs. Hence, the need for installing statistically significant number of load cells.

A more detailed picture of radial loading of a support may be obtained by installing pressure cells at various positions (Fig. 14.4) between support and the rock. The pressure cells are pre-calibrated for changing loads in a universal testing machine.

Radial support pressure can be given by,

$$p_{\mathrm{r}} = \frac{P}{A} \tag{14.6}$$

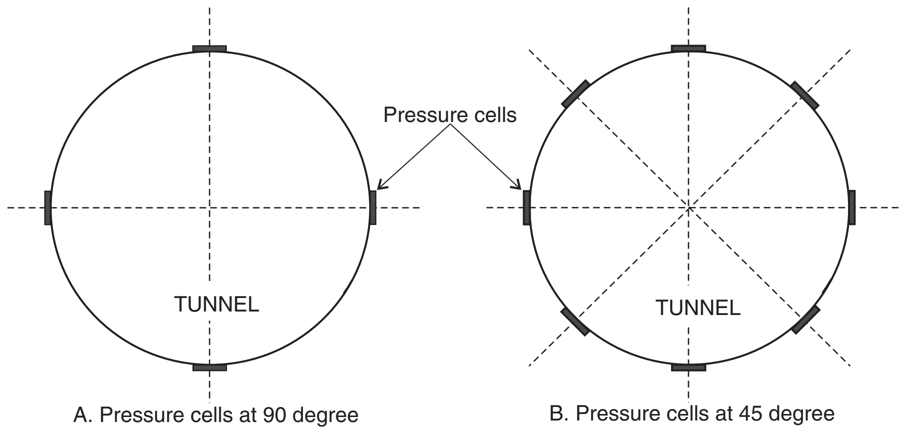


Fig. 14.4 Typical installation arrangements of pressure cells.

where

p_r = radial support pressure,
P = load indicated by a pressure cell,
(obtained from its calibration curve) and
A = area of the pressure cell.

These are also influenced by the same factors as indicated earlier in case of load cells. Pressure cells may also be used to determine radial support pressure on lining by installing them between rock and the lining.

The rock bolt load cells are also being used to monitor the pressure on (or the tension in) the rock bolt. The capacity of these rock bolt load cells depends upon the capacity of the rock bolt, but certainly quite less than the capacity of load cells installed in the steel ribs.

14.6.2 Measurement by extensometers

Extensometers can be used to calculate support pressure acting on a tunnel lining by measuring its diametric deformations. These may be arranged radially or in a star-shaped configuration. Support pressure in this case is given by,

$$p_r = \left[\frac{E_d}{1+\nu} + \frac{2h \cdot E_l}{D}\right] \cdot \frac{\delta}{D} \tag{14.7}$$

where

p_r = radial rock pressure,
E_d = modulus of deformation of rock mass,
δ = deformation,

E_l = modulus of elasticity of lining,
D = diameter of lining and
h = lining thickness.

These observations are important from the point of view of stabilization of a tunnel in squeezing and swelling rocks.

Similarly, tape extensometers may be used to determine modulus of deformation approximately by measuring diametric deformation of an unlined tunnel soon after blasting a tunnel face.

$$E_d = \frac{P \cdot D(1 + \nu)}{\delta} \tag{14.8}$$

where

P = overburden pressure,
E_d = deformation of rock mass,
δ = closure between walls,
D = diameter of the tunnel and
ν = Poisson's ratio of the rock.

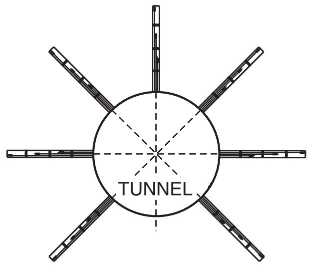
The equations described here are based on elastic theory for isotropic and elastic materials (see equation (AI.5) and Fig. AI.1).

14.6.3 Measurement by strain gauges

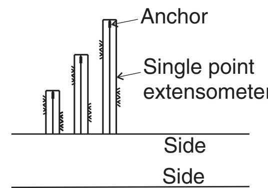
Various electrical and mechanical strain gauges may be used to measure strain on steel supports or tunnel lining and support pressures can be computed therefrom. Serious difficulties are encountered in their use, because of the elapse of time. The development of pressures is affected seriously by the proper and firm installation of gauges by all changes produced in the surroundings and by a variety of other factors, the identification and separation of which is practically impossible.

14.7 MEASUREMENT OF ROCK MASS BEHAVIOR AROUND AN UNDERGROUND OPENING BY BOREHOLE EXTENSOMETER

The rock surrounding the excavation of a tunnel may fail and produce a loose and fractured rock mass when the induced stresses exceed the uniaxial compressive strength of the surrounding rock. Thus, it is apparent that support pressure depends to a considerable degree on the depth of this zone of loose and fractured rock mass. In case of loosening type of rock, the support pressure depends on the depth of the zone of loosened rock. The utility of the depth of loosened rock mass is, therefore, obvious. A set of single point borehole extensometers or a set of multi-point borehole extensometers (Fig. 14.5) are used to determine the depth of a loosened rock mass and the extent of loosening



Vertical section of tunnel showing a typical installation arrangement for 3 point borehole extensometer



Plan of tunnel showing 3 single point extensometers installed at different depths in different holes at one location

Fig. 14.5 Typical installation arrangements of multi-point (3 points shown) and single point borehole extensometers.

at different depths. The borehole extensometers can be of wire or rod type. Relative movements are transferred from one measuring point (fixed point of rock or wire) to another measuring point (sensing head) by means of a tensioned wire or rod. Movements thus transferred are indicated mechanically in the sensing head or converted into electrical output, which may then be transferred for remote instrument reading, if required. A recording unit and an alarm may also be introduced if needed.

14.8 CASE HISTORIES

Some examples explaining how instrumentation helps in engineering decisions are given here.

14.8.1 Observations by load cells

In a particular section of a 3.0 m diameter tunnel at Yamuna Hydroelectric Project, in brecciated, crushed and sheared shales supported by 150 × 150 mm, H.H. Section R.S. joints spaced at 0.5 m; three load cells were installed at joints (Fig. 14.3a). Loading of the support as a function of time is shown in Fig. 20.10f. Support pressures of 2.7 kg/cm^2 (0.27 MPa) and 1.5 kg/cm^2 (0.15 MPa) were recorded in a period of 115 days in vertical and horizontal directions, respectively. Upon extrapolation the respective values were worked out to be about 0.5 and 0.3 MPa in about 3 years. Chapter 20 gives the detailed case history.

Based on these observations steel supports for 9.0 m diameter excavation were designed for 6.0 kg/cm^2 (0.6 MPa) of support pressure and have proved to be adequate. Earlier, weaker supports had failed in this zone.

The backfill material employed here was precast concrete blocks filled thoroughly between rock and the supports soon after excavation.

14.8.2 Observations by pressure cells

Radial support pressure was measured by installing pressure cells between steel support and the rock in yet another zone of the rock mentioned above. The supports were 300 × 140 mm R.S. Joists of 9.0 m diameter spaced at 350 mm center to center. Precast concrete blocks were used as backfill material. Method of excavation was heading and benching. The pressure cells were installed on a support in heading as shown in Fig. 20.12, which gives support pressure as a function of time both before and after excavation of the bench. First fall in support pressure values observed about 125 days after installation is attributed to removal of rock reaction due to excavation of bench at this place during this period. The support experienced a sharp rise in its rate of loading soon after completion of full ring. They were designed for 0.9 MPa support pressure. This value was exceeded upon when the pressure cells indicated pressures of about 1.2 MPa. This resulted in the deformation (buckling) of the supports which could be seen by naked eye and the pressure values decreased due to the buckling. Since then the supports had deformed very badly in this reach.

14.8.3 Observation of tunnel closure

Diametric deformations of a 3.0 m diameter tunnel were measured with a simple steel tape by installing closure pins at 45° intervals around the excavation. Tunnel closure as a function of time is shown in Fig. 14.6. An unexpected rise in the rate of tunnel closure can be seen. Upon studying the possible causes, it was found that about 60 cm deep water remained stagnant in this part of the tunnel for about two months resulting in the increased moisture content of the surrounding rock. It can be noted that the area had not stabilized although the excavation was over two years old, the rate of tunnel closures being about 2 and 1 mm per month in horizontal and vertical directions, respectively. This indicated that either the concrete lining should wait for the stabilization of the zone or it should be designed taking into consideration the time-dependent support pressures.

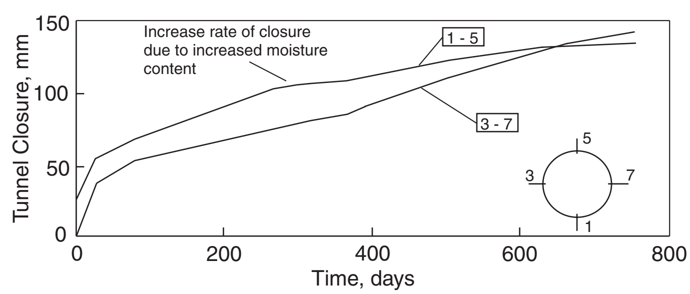


Fig. 14.6 Tunnel closure as a function of time.

14.8.4 Tunnel closure and load on supports

A very interesting feature of instrumentation in a 3.0 m diameter tunnel was relationship of tunnel closure and support pressure (Fig. 14.7). Support pressure was measured by inserting load cells at support joints and closure was measured with the help of an ordinary steel tape to an accuracy of ±1 mm. It can be seen that for both vertical and horizontal directions these have a direct straight line relationship. Its slope defines the support stiffness.

14.8.5 Observations by borehole extensometers

Non-squeezing ground condition

Single point rod type extensometers were installed at sides and crown of a 3.0 m diameter tunnel upto a depth of 3.0 m. Time and borehole extension relations (Fig. 14.8) indicated that upto this depth of 3.0 m the loosening of rock in the vertical and horizontal directions were 65 and 84 percent, respectively. In other words 65 and 84 percent of loosening was absorbed by rock mass lying at a depth greater than the diameter of excavation. In simple terms this meant that the thickness of loosened rock mass around the excavation was very high. It is obvious that multi-point borehole extensometers installed at greater depth could have given the total thickness of the zone of loose rock mass as a result of tunnelling.

Squeezing ground condition

In squeezing ground condition, displacements are very large and must be measured. Single point rod type borehole extensometers were installed on the sides of a 4.2 m diameter

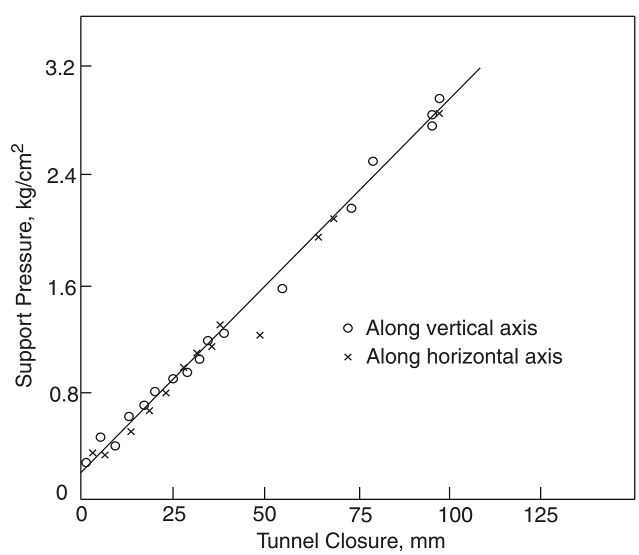


Fig. 14.7 Support pressure as a function of tunnel closure (observed by installing load cells in a support).

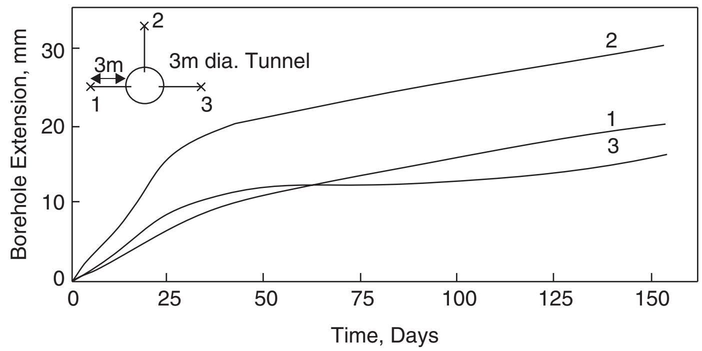


Fig. 14.8 Variation of borehole extension with time.

Giri Hydeltunnel through crushed phyllites which squeezed due to high cover pressure of about 300 m. Two extensometers of 5 and 2.5 m depths were installed on the left wall and three extensometers of 7.5, 5.0 and 2.5 m depths were installed on the right wall. No extensometer could be installed on the roof. Tunnel closures were also measured. The data were analyzed and radial displacements u_r were plotted against radial distance r for various time intervals as shown in Fig.14.9. The convergence of $u_r - \log r$ plots at point indicates stabilization of the broken zone between 200 to 300 days after excavation. The broken zone radius (b) at this period was found to be 20.7 and 20.3 m on the left and right wall, respectively. (It can be noted that the radial displacements vs. time curves tend to converge at some radial distance which is believed to be the interface between broken zone and elastic zone within a squeezing ground condition.) The steel ribs buckled after 300 days. This produced a spurt in radial displacements and the broken zone started widening again as indicated by the divergence of $u_r - \log r$ plots in Fig.14.9. The example clearly shows the usefulness of multi-point borehole extensometers to monitor the development of broken zone around a tunnel under squeezing ground conditions.

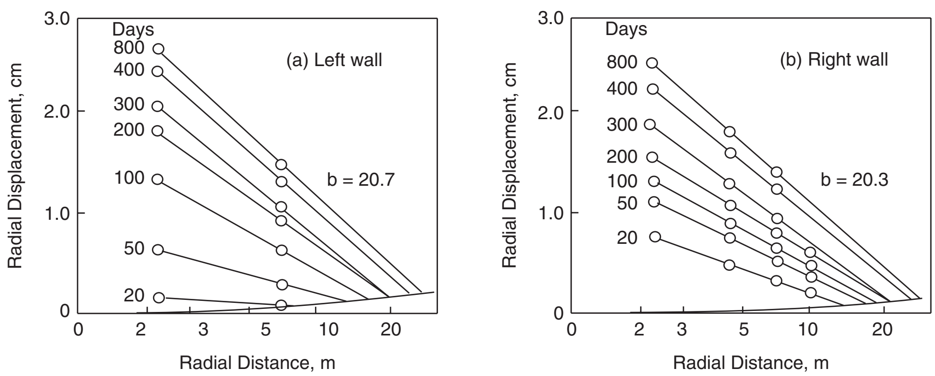


Fig. 14.9 Variation of radial displacement with radial distance within phyllites in Giri Hydeltunnel ($a = 2.12$ m and $b =$ radius of broken zone in squeezing ground).

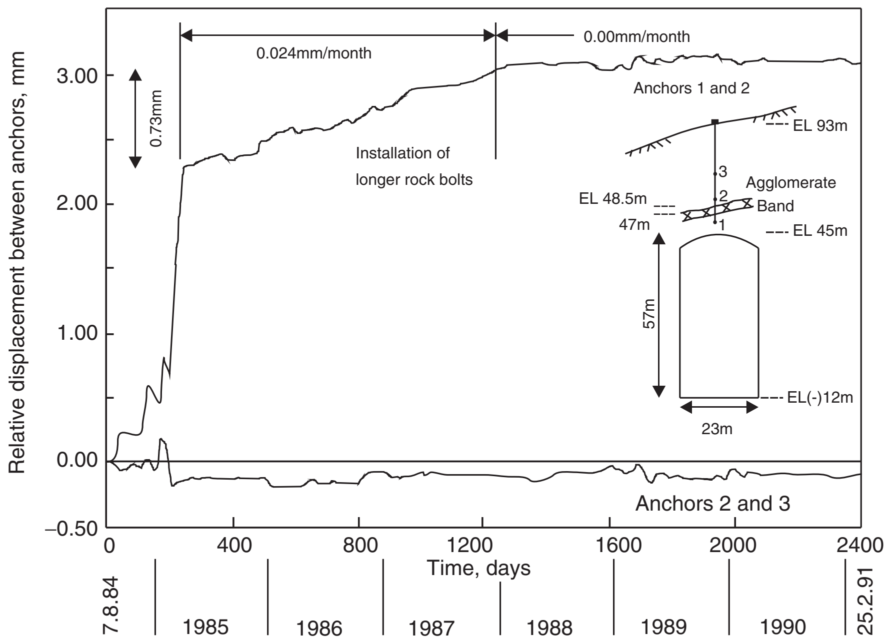


Fig. 14.10 Monitoring agglomerate band behavior with the multi-point borehole extensometer in the roof of a large underground cavity, India (Goel, 2001).

14.8.6 Observation by borehole extensometer in large underground cavity

In one of the large underground opening projects, for example, it has been possible to monitor the roof displacement of 0.024 mm/month (Fig. 14.10). The deformation remains continued for almost 30 months. At this point of time, additional supports of longer rock bolts were installed and subsequently it was observed that the roof movement/displacement had stopped.

14.9 LAYOUT OF A TYPICAL TEST SECTION

Layout of an extensively instrumented zone is shown in Fig.14.11. Measurements taken consist of following robust and valuable instruments.

(i) Radial support pressure by pressure cells
(ii) Load on support by load cells
(iii) Depth of loosened rock mass by multi-point borehole extensometers and
(iv) Rock closure and support deformation by tape extensometer.

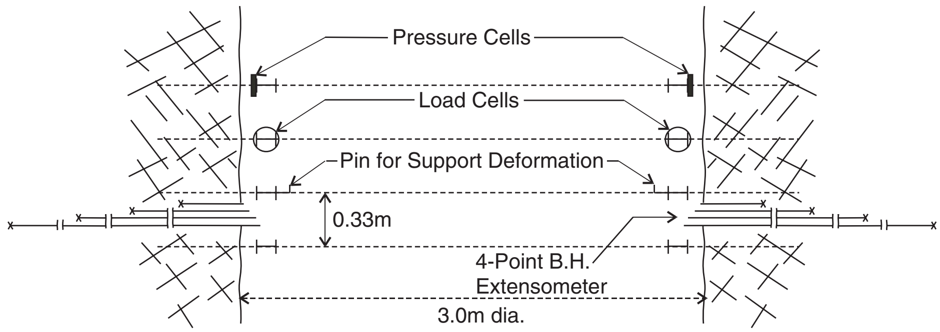


Fig. 14.11 Layout plan of a typical instrumentation zone.

Strain in support can be measured by strain gauges.

Instrumentation in the lined and concreted zone should consist of the following:

(i) Stress meters
(ii) Strain meters

Embedded in concrete

Besides the above mentioned instrumentation, following data should also be collected:

A. Geology – mapping, fracture spacing and orientation, width of fracture zone, alteration and groundwater
B. Rock mass quality (Q), rock mass rating (RMR) and geological strength index (GSI)
C. Geophysical observations – seismic activity, in situ stresses and their orientation, micro-seismic activity inside opening.

Significant researches have been done on the basis of field data from the instrumented tunnels in past. One is missing great opportunity by avoiding the tunnel instrumentation and not collecting new field data, specially in complex geological conditions.

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15
Tunnelling machines

"Any manager of a project must understand that his success depends on the success of the contractor. The contractors have to be made to succeed. They may have many problems. We cannot always talk within the rigid boundaries of a contract document. No, without hesitation. I go beyond the contract agreement document."

E. Sreedharan, Managing Director, Delhi Metro Rail Corporation

15.1 GENERAL

The age-old drill and blast technique is still being used in poor countries due to choice for labor-friendly policies. The time has come for change. We should prepare ourself mentally for change and for a fast rate of progress also. The applications of modern techniques like NATM and NTM involving automated excavation methods are the need of time.

Fig. 15.1 depicts a variety of methods of excavation as a function of strength of rock material (Jethwa, 2001). Table15.1 shows comparative study of the available techniques for tunnelling vis-à-vis some of the important parameters like cost, advance rate of tunnelling, utilization of money and geometric requirements of a tunnel. A judicious selection of tunnelling technology may be made with the help of Table 15.1 depending upon the culture of a nation. Some nations in Asia prefer to evolve slowly for sustainable growth for a very long time.

15.2 SYSTEM'S MIS-MATCH

An effort to increase the rate of tunnelling requires a system's approach. The system in totality should be improved, specially the weakest link which is the installation of support system in weak rock masses. For example, excavation by a road header will be meaningless if steel-arch supports are not replaced by SFRS (steel fiber reinforced shotcrete) support for weak rock masses. A tunnel boring machine is stuck in a thick fault or shear zone in a complex unknown geological condition, burying the machine. Excessive failure of tunnel face causes jamming of excavating head. So the choice of selection of

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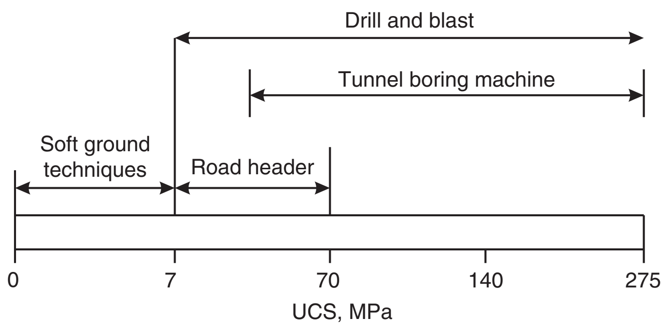

Fig. 15.1 Tunnel excavation methods as a function of rock strength (Jethwa, 2001).

tunnelling machine depends upon the complexity of geological conditions, poverty of a nation and management conditions. It is wiser to insure TBM always. Unfortunately, active participation of a rock engineer is conspicuously absent from planning to commissioning of the tunnelling projects in many nations. This results in geological surprises which have to be paid for in terms of both time and cost over-runs.

There is a great fallacy that automated tunnelling is costlier. It is not true. With advent of modern tunnelling machines, though the initial investment is high, the recurring cost is relatively low in long tunnels (>2 km), except in soft ground tunnelling (Table 15.1). Further, the tunnelling project is completed in shorter time and starts giving economic return much earlier which helps in reducing the cost of interest on the capital investment. It is painful to know that construction of hydroelectric projects is delayed greatly due to the delay in completion of very long and complex tunnel network. Hence, the justification for adopting tunnelling machines but judiciously.

15.3 TUNNEL JUMBO

The tunnel jumbo usually consist of light rock drill of high performance which are mounted on a mechanical arms. These arms are moved by hydraulic jacks. The wheeled jumbo is mobile and fast. Initial cost is only a small portion of the overall cost of tunnelling. All booms can be used to drill upwards, downwards, besides horizontally. The number of booms can go up to seven (which was used in Daniel Johnson dam in Canada). The rate of tunnelling goes up with more number of booms and the cost of jumbo also goes up.

The main advantages of modern jumbos are:

- Faster rate of penetration of drills
- Quick realignment of booms (arms)
- Versatility of boom movements
- Maneuverability of carrier
- Low power consumption

Table 15.1 Comparative study of different techniques for tunnelling projects (Jethwa, 2001).

Parameters		Conventional drill and blast	Automated drill and blast in conjunction with NATM/NTM	Roadheaders	Soft rock TBMs	Hard rock TBMs
Cost	Initial	●	● ● ●	● ● ●●	● ● ● ● ●	● ● ● ● ●
	Running	● ● ● ● ●	●●	●●	● ● ● ● ●	●●
Rate of advance	Favorable ground	50–60 m/month	200–700 m/month	350–800 m/month	150–300 m/month	500–1500 m/month
	Unfavorable ground	7–10 m/month	50–60 m/month	75–150 m/month	25–50 m/month	100–200 m/month
Utilization of money	Overall	Very inefficient	Good	Best	Best	Best
	Space at face	Least	Moderate	Moderate	● ● ● ● ●	● ● ● ● ●
Geometric requirements	Shape of tunnel	Any	Any	Horse-shoe	Circular	Circular, horse-shoe, rectangular
	Cross section of tunnel	Any	Any	2.5–10 m	1–10 m	2–13 m
	Maximum gradient	Any	Upto 30 degree	15 degree	<10 degree	<10 degree
Applicability	Geology	Universally applicable	Universally applicable	Sensitive to change	Very sensitive to change	Very sensitive to change
	Rock strength	All strength	All strength	Medium hard to hard	Soft and clayey	Hard to very hard
Operational parameters	Ground disturbance and overbreak	● ● ● ● ●●	Moderate with good controlled blasting measures	Least	Moderate to least	Least
	Operator's skill	●	Moderate	● ● ● ● ●	● ● ● ● ●●	● ● ● ● ●
	Support requirement	● ● ● ● ●●	Very high to high	●●	High to low	●●
	Speed of work	●	Good to very good	● ● ● ● ●●	Fair	Extremely good
	Public safety	●	●●	● ● ● ● ●	Moderate	● ● ● ● ●
	Quality of work	●	Poor to good	● ● ● ● ●●	● ● ● ● ●	● ● ● ● ●●

Hard Rocks – Automated D&B with NATM/NTM; **Soft Rocks** Roadheaders with NATM/NTM.
● – Very Low, ●● – Low, ● ● ● – Low to medium, ● ● ●● – Medium to high, ● ● ● ● ● – High, ● ● ● ● ●● – Very high.

- Longer bit and steel life
- Considerably less noise
- Improvement in environmental conditions

The vertical drilling mechanism is used for drilling boltholes and horizontal booms are used for drilling blast holes.

15.4 MUCK HAULING EQUIPMENT

Efficient removal of excavated rock blocks (muck) is an important operation. Use of belt conveyers is very economical and efficient. Belt conveyers load into the muck cars hauled by diesel, electricity or battery. As the area available is limited in a tunnel driving the mucking equipment should occupy minimum working space. Rail track should be well laid on rock mass and should be maintained well for efficient operation. The rail lines move upwards in squeezing rock conditions or swelling rocks. In former case, rock anchors should be installed in the floor and shotcreted using SFRS. In the latter case, swelling of rocks should be prevented by spraying shotcrete immediately all round the tunnel including the floor to prevent ingress of moisture inside the rock mass. However, the inverts delay mucking.

Fig. 15.2 shows Haggloader 10 HR which is mounted on a rubber tired chassis. It is more mobile than other Haggloaders. It uses digging and gathering arms in the front of the machine. The muck is brought into the transport equipment by a conveyer (shown by inclined line). This model is highly efficient and safe for the operator.

The classic books of Singh (1993) and Bickel and Kuesel (1982) describe various other machines used for tunnelling operations.

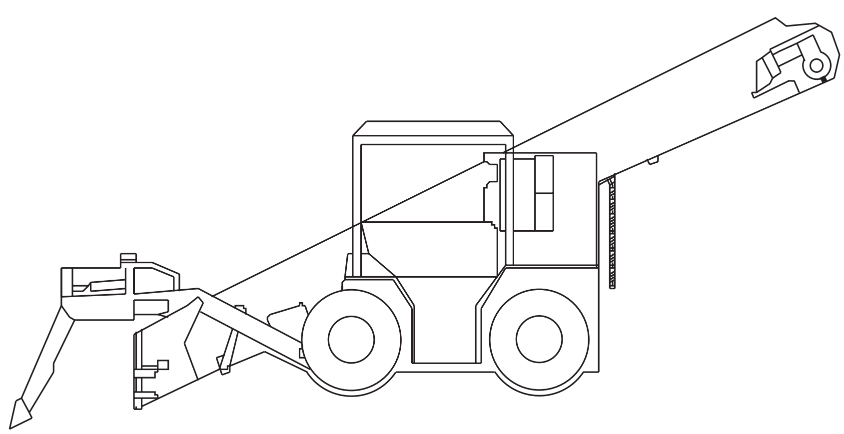

Fig. 15.2 Haggloader 10 HR, principal data.

15.5 TUNNEL BORING MACHINE (TBM)

After nearly 150 years of development, the TBM has been perfected to excavate in fair to hard rock masses. The TBM has the following technical advantages.

- Reduction in overbreaks
- Minimum surface and ground disturbance
- Reduced ground vibrations cause no damage to nearby structures, an important consideration for construction of underground metro
- The rate of tunnelling is several times of that of drill and blast method
- Better environmental conditions – low noise, low gas emissions, etc.
- Better safety of workers

Engineers should not use TBM where engineering geological investigations have not been done in detail and the rock mass conditions are very heterogeneous. Contractors can design TBM according to the given rock mass conditions which are normally homogeneous non-squeezing ground conditions. TBM is unsuitable for the squeezing or flowing grounds (Bhasin, 2004).

The principle of TBM is to push cutters against the tunnel face and then rotate the cutters for breaking the rocks in chips (Fig. 15.3).

The performance of a TBM depends upon its capacity to create largest size of chips of rocks with least thrust. Thus, rock chipping causes high rate of tunnelling rather than grinding (Kaiser & McCreath, 1994). The rate of boring through hard weathered rock mass is found to be below expectation (see Chapter 16).

Disc cutters are used for tunnelling through soft and medium hard rocks. Roller cutters are used in hard rocks, although their cost is high. A typical TBM is shown in Fig. 15.4 together with the ancillary equipment. The machine is gripped in place by legs with pads on rocks. The excavation is performed by a cutting head of welded steel and convex shape, with cutters arranged on it optimally. The long body of TBM contains the four hydraulic

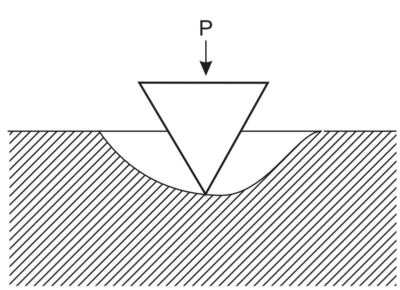

Fig. 15.3 Mechanism of failure of rock by cutter (Bickel & Kuesel, 1982).

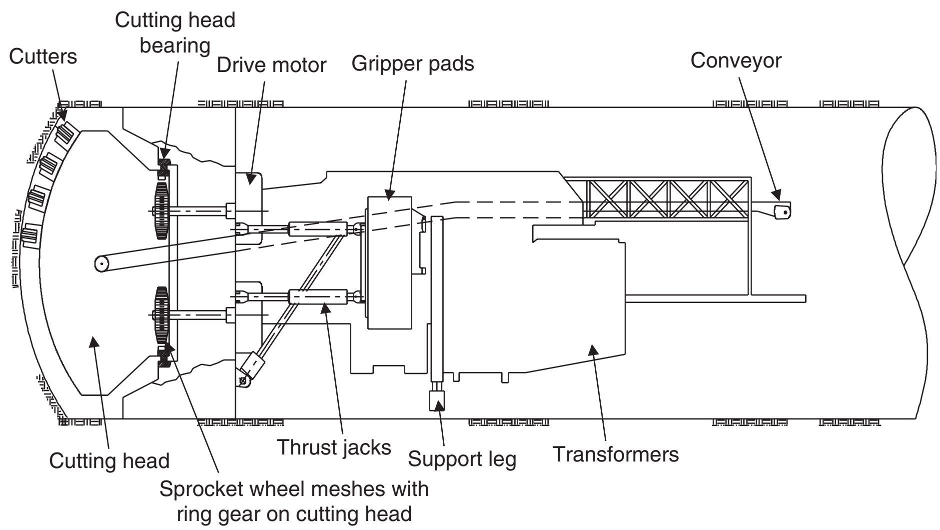

Fig. 15.4 Tunnel boring machine and ancillary equipment (Bickel & Kuesel, 1982).

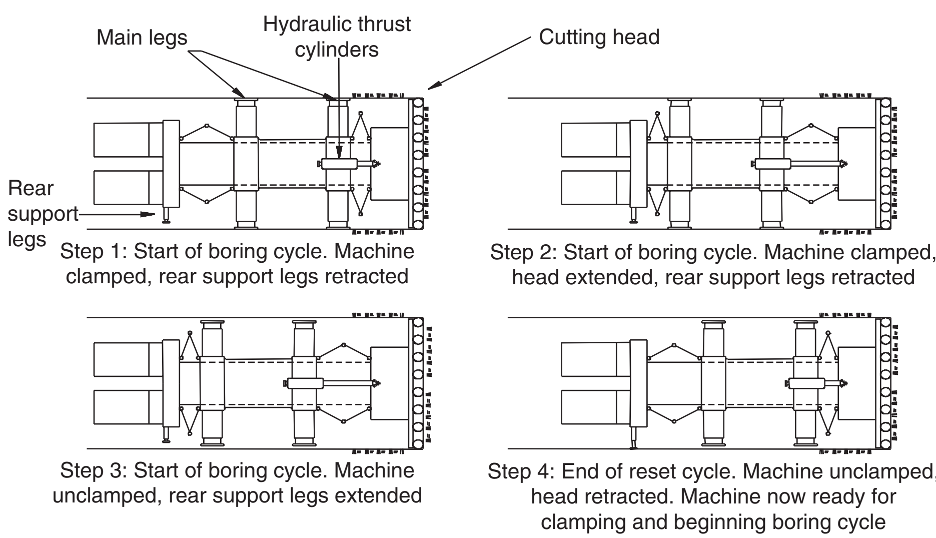

Fig. 15.5 Method of advance of a rock tunnelling machine (Bickel & Kuesel, 1982).

jacks to push forward the cutting head and also drive motors which rotate the cutting head for chipping rocks. Fig. 15.5 shows schematically a method of advance of the cutter head. This figure shows how TBM is steered and pushed ahead in self-explaining four steps. Typically even when a TBM operates well, only 30 to 50 percent of the operating time is spent on boring.

Fig. 15.4 also shows the removal system for muck (rock chips). The excavated material is collected and scooped upwards by buckets around the cutter head. These buckets then drop the rock pieces on a conveyer belt and transported it to the back end of the TBM. There, it is loaded into a train of mucking cars.

Precautions:

The following precautions should be taken:

(i) There should be adequate store-keeping of spare parts for all the tunnelling machines at the project site. Arrangement should be made to procure machine parts on a quick emergency basis by air cargo to reduce break down periods. Funds should be available for the same.
(ii) Maintenance of machines is a weakness in culture of many Asian countries, as there is no glory in the job of maintenance. Hence, maximum efforts for maintenance are needed at the project site.
(iii) Extra machines even TBM should be purchased as standby tunnelling machines. Thus standby machines can be used when there is a major breakdown of machines; as the completion of a tunnel before target date is important to start earning profit from the completed project. The completion of a project is normally delayed significantly due to the difficulties in long tunnelling.
(iv) There is high cost over-run and time over-run in long deep tunnels (>500 m). Best management conditions help.
(v) There should be good workshop of adequate capacity for repairs of machines.
(vi) There should be a preventive maintenance program, as it is of vital importance to the successful and continuous operations of all machines.
(vii) Modern fleet of tunnelling machines are more sophisticated, more versatile, more powerful and very fast, and therefore safety of workers in limited space of unsafe tunnels should be the top priority.

15.6 SAFETY DURING TUNNELLING

Safety saves. It is well-known proverb. Managing safety saves money. One dollar investment in safety recovers ten dollars of loss. Safety goes together with quality of the construction and project target. Achievement in safety creates a good public image. One may learn from the case histories. The rate of accidents should be recorded and accident reporting is very important. The risk is too high in tunnelling through water-charged rocks, wide shear zones, collapse of shallow covers in transportation tunnels and under sea tunnelling. Many times there are no contingency plans and emergency plan. There is no coordination between the design and construction engineers. There should be quick feedback of actual construction problems to the designers and managers. The real and visible commitment or involvement of senior managers is extremely important in the safety management, quality control and completion of the project in time.

Habit of safety is a way of life. Safety consciousness should be created among workers by frequent training programs at the site. There should also be interaction between all the concerned i.e., executives, planners, managers, designers, geologists, engineers and contractors, etc. Efforts should be made to reduce communication gap among them with the help of simple artistic presentation. There should be mutual respect for each other rather than distrust. There should be culture of friendship in spite of tensions and passions, as in Japan and many other nations. There are unforeseen geo-environmental conditions particularly in the long deep tunnels. So there should be contingency clause in the contract document, to be always prepared to tunnel manually through piping or flowing grounds, weak rock masses and the water-charged rock masses. Contractors should employ healthy, highly experienced and skilled workers in a tunnel. Quotations (safety saves, safety first, etc.) should be written on the boards in local languages at proper places (at inlets, etc.).

The accidents involved in tunnelling and underground construction are mainly during drilling, handling explosives and blasting, mucking and supporting the weak rock masses. The congested working space, wet and slippery floor, inadequate lighting and ventilation increase the chances of accident. Working through access shaft is an additional cause for accidents. The persons working in the tunnels should be provided with helmets and gum-boots for safety. The workers would be withdrawn from the tunnel, in case of prolonged ventilation failure or a heavy rush of ground water. Good housekeeping (maintenance) is essential for safe and successful operations of tunnelling. Proper and adequate drainage inside the tunnel leads to safe working conditions. Sump pumps, switches, crossings of rail tracks, transformers and equipment should be well lighted locally.

In the race of speedy construction, the future machines should be safe, simple, versatile and economical, sophisticated and fast and powerful (Singh, 1993). The safety of the people shall be the highest law, according to Cicero.

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16

Rock mass quality for tunnel boring machines (Q_{TBM})

"The Mother Nature is Motherly!"

Vedas, Gita and Durgasaptashati

16.1 INTRODUCTION

Tunnel boring machine (TBM) may give extreme rates of tunnelling of 15 km/year and 15 m/year, sometimes even less. The expectation of fast tunnelling places great responsibility on those evaluating geology and hydrogeology along a planned tunnel route. When the rock conditions are reasonably good, a TBM may be two to four times faster than drill and blast method. The problem lies in the extremes of rock mass quality, which can be both too bad and too good (no joints), where alternatives to TBM may be faster (Barton, 1999).

There have been continuous efforts to develop a relation between the rock mass characterization and essential machine characteristics such as cutter load and cutter wear, so that surprising rates of advance become the expected rates. Even from a 1967 TBM tunnel, Robbins (1982) has reported 7.5 km of advance in shale during four months. Earlier in the same project, 270 m of unexpected glacial debris had taken nearly seven months. An advance rate (AR) of 2.5 m/h has declined to 0.05 m/h in the same project. It needs to be explained by a quantitative rock mass classification.

Barton (2000) has incorporated a few parameters in Q-system which influence the performance of a TBM to obtain Q_{TBM}, i.e., rock mass quality for tunnel boring machine. Using Q_{TBM}, Barton (2000) opines, the performance of TBM in a particular type of rock mass may be estimated. His approach, in brief, has been presented in this chapter.

16.2 Q AND Q_{TBM}

The Q-system was developed by Barton et al. in 1974 from the drill and blast tunnel case records and now totals 1250 cases (Grimstad & Barton, 1993). The Q-values stretch

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over six orders of magnitude of rock mass quality. Continuous zones of squeezing rock and clay may have Q = 0.001, while virtually unjointed hard massive rock may have Q = 1000. Both conditions are usually extremely unfavorable for TBM advance, one stopping the machine for extended periods and requiring heavy pre-treatment and supports; the other perhaps slowing average progress to 0.2 m/h over many months due to multiple daily cutter shifts (Barton, 1999).

The general trends for a penetration rate (PR) with uninterrupted boring and the actual advance rate (AR) measured over longer periods is shown in Fig. 16.1. It is highlighted here that the penetration rate of a TBM may be high, but the real advance rate depends on the tunnel support needs and on conveyor capacity. The Q-value goes a long way to explain the different magnitudes of PR and AR but it is not sufficient without modification and the addition of some machine–rock interaction parameters.

A new method has been developed by Barton (1999) for estimating both PR and AR using both Q-value and a new term Q_{TBM}. This is strongly based on the familiar Q parameters but has additional rock-machine–rock-mass interaction parameters. Together, these give a potential 12 orders of magnitude range of Q_{TBM}. The real value depends on the cutter force.

Experience suggests that there are four basic classes of rock tunnelling conditions that need to be described in some quantitative way:

(i) Jointed, porous rock, easy to bore, frequent support;
(ii) Hard, massive rock, tough to bore, frequent cutter change, no support;
(iii) Overstressed rock, squeezing, stuck machine, needs over-boring, heavy support and
(iv) Faulted rock, overbreak, erosion of fines, long delays for drainage, grouting, temporary steel support and back-filling.

The conventional Q-value, together with the cutter life index (Johannessen & Askilsrud, 1993) and quartz content help to explain some of the delays involved.

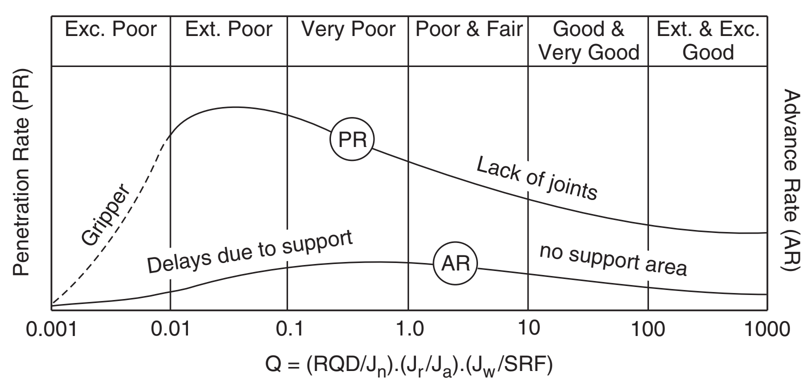

Fig. 16.1 A conceptual relation between Q, PR and AR.

The Q-value can also be used to select support once the differences between drill and blast and TBM logging are correctly quantified in the central threshold area of the Q-diagram (Fig. 10.2).

In relation to the line separating supported and unsupported excavations in the Q-system support chart, a TBM tunnel will give an apparent (and partially real) increase in the Q-value of about 2 to 5 times in this region. This is where the TBM tunnel supports are reduced. When the Q-value is lower (shaded area in Fig. 10.2) than in the *central threshold area*, the TBM tunnel will show similar levels of overbreak or instability as the drill and blast tunnel, and final support derived from Q-system can apply. However, they may be preceded by (non-reinforcing temporary) steel sets and lagging (and void formation). Each of which require due consideration while designing a support.

The Q_{TBM} is defined in Fig. 16.2, and some adjectives at the top of the figure suggest the ease or difficulty of boring. (Note the difference to the Q-value adjectives used in Fig. 16.1, which describe the rock mass stability and need of tunnel support.) The components of Q_{TBM} are as follows:

$$Q_{TBM} = \frac{RQD_0}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \times \frac{\sigma_{cm} \text{ or } \sigma_{tm}}{F^{10}/20^9} \times \frac{20}{CLI} \times \frac{q}{20} \times \frac{\sigma_\theta}{5} \tag{16.1}$$

where

RQD_0 = RQD (%) interpreted in the tunnelling direction. RQD_0 is also used when evaluating the Q-value for rock mass strength estimation,

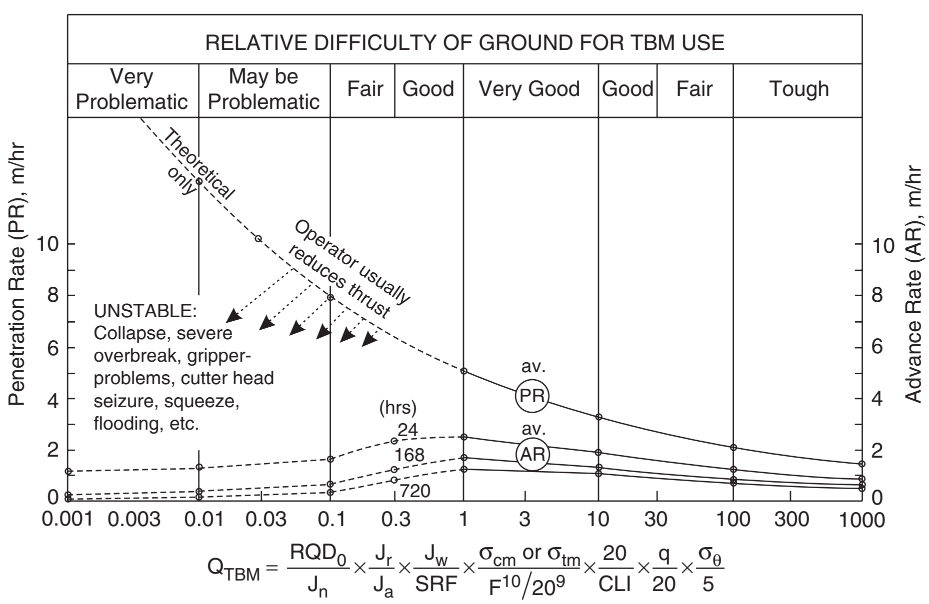

Fig. 16.2 Suggested relation between PR, AR and Q_{TBM}.

J_n, J_r, J_a, J_w and SRF = ratings of Barton et al. (1974) and are unchanged,
F = average cutter load (tnf) through the same zone, normalized by 20 tnf (the reason for the high power terms will be seen later),
σ_{cm} or σ_{tm} = compressive and tensile rock mass strength estimates (MPa) in the same zone,
CLI = cutter life index (e.g., 4 for quartzite, 90 for limestone),
q = quartz content in percentage terms and
σ_θ = Induced biaxial stress on tunnel face (approx MPa) in the same zone, normalized to an approximate depth of 100 m (= 5 MPa).

The best estimates of each parameter should be assembled on a geological/structural longitudinal section of the planned (or progressing) tunnel.

The rock mass strength estimate incorporates the Q-value (but with oriented RQD_0), together with the rock density (from an approach by Singh (1993)). The Q-value is normalized by uniaxial strength (q_c) different from 100 MPa (typical hard rock) as defined in equation (16.3a) and is normalized by point load strength (I_{50}) different from 4 MPa. A simplified (q_c/I_{50}) conversion of 25 is assumed. Relevant I_{50} anisotropy in relation to the direction of tunnelling should be quantified by point load tests in the case of strongly foliated or schistose rocks. The choice between σ_{cm} and σ_{tm} will depend on the angle between tunnel axis and the major discontinuities or foliations of the rock mass to be bored (Barton, 2000). It has been suggested to use σ_{cm} when the angle is more than 45 degree and σ_{tm} in case the angle is less than 45 degrees. It may be noted here that penetration rate is more in case the angle is zero degree.

$$\sigma_{cm} = 5 \cdot \gamma\, Q_c^{1/3} \tag{16.2}$$

$$\sigma_{tm} = 5 \cdot \gamma\, Q_t^{1/3} \tag{16.3}$$

where

$$Q_c = Q \cdot q_c/100, \tag{16.3a}$$

$$Q_c = Q \cdot q_t/100, \tag{16.3b}$$

$$= Q \cdot (I_{50}/4) \text{ and}$$

$$\gamma = \text{Density in gm/cm}^3.$$

Equations (16.2) and (16.3) for the estimation of σ_{cm} and σ_{tm} are proposed only for Q_{TBM} where it is useful as a relative measure for comparing with the cutter force (Barton, 2005).

Example

Slate Q ≈ 2 (poor stability); $q_c \approx$ 50 MPa; $I_{50} \approx 0.5$ MPa; $\gamma = 2.8$ gm/cm^3; $Q_c = 1$; and $Q_t = 0.25$. Therefore, $\sigma_{cm} \approx 14$ MPa and $\sigma_{tm} \approx 8.8$ MPa.

The slate is bored in a favorable direction, hence consider σ_{tm} and $RQD_0 = 15$ (i.e., <RQD). Assume that average cutter force = 15 tnf; CLI = 20; $q = 20\%$; and $\sigma_\theta = 15$ MPa (approx 200 m depth). The cleavage joints have $J_r/J_a = 1/1$ (smooth, planar, unaltered). The estimate of Q_{TBM} is as follows:

$$Q_{TBM} = \frac{15}{6} \times \frac{1}{1} \times \frac{0.66}{1} \times \frac{8.8}{15^{10}/20^9} \times \frac{20}{20} \times \frac{20}{20} \times \frac{15}{5} = 39$$

According to Fig. 16.2, $Q_{TBM} \approx 39$ should give fair penetration rates (about 2.4 m/h). If average cutter force were doubled to 30 tnf, Q_{TBM} would reduce to a much more favorable value of 0.04 and the PR would increase (by a factor $2^2 = 4$) to a potential 9.6 m/h. However, the real advance rate would depend on the tunnel support needs and on conveyor capacity (Barton, 1999).

16.3 PENETRATION AND ADVANCE RATES

The ratio between the advance rate (AR) and penetration rate (PR) is the utilization factor U,

$$AR = PR \cdot U \tag{16.4}$$

The decelerating trend of all the data may be expressed in an alternative format:

$$AR = PR \cdot T^m \tag{16.5}$$

where T is time in h and the negative gradient (m) values are cited in Table 16.1.

The values of m given in Table 16.1 may be refined in the future as more and more cases of TBM tunnels would be available (Barton, 2000).

16.4 CUTTER WEAR

The final gradient $(-)m$ may be modified by the abrasiveness of the rock, which is based on a normalized value of CLI, the cutter life index. Values less than 20 give rapidly reducing

Table 16.1 Deceleration gradient $(-)\ m_1$ and its approximate relation to Q-value.

Q	0.001	0.01	0.1	1	10	100	1000
m_1	−0.9	−0.7	−0.5	−0.22	−0.17	−0.19	−0.21
Unexpected events or expected bad ground. Many stability and support-related delays and gripper problems. Operator reduces PR. This increases Q_{TBM}				Most variation of $(-)m$ may be due to rock abrasiveness, i.e., cutter life index CLI, quartz content and porosity are important. PR depends on Q_{TBM}			

Note: The subscript (1) is added to m for evaluation by equation (16.6).

cutter life, and values over 20 tend to give longer life. A typical value of CLI for quartzite might be four and for shale 80. Due to the additional influence of quartz content (q%) and porosity (n%), both of which may accentuate cutter wear, these are also included in equation (16.6) to give "fine tuning" to the gradient.

It has also been felt necessary to consider the tunnel size and support needs. Although large tunnels can be driven almost as fast (or even faster) as small tunnels in similar good rock conditions (Dalton, 1993), more support-related delays occur if the rock is consistently poor in the larger tunnel. Therefore, a normalized tunnel diameter (D) of 5 m is used to slightly modify the gradient (m). (Q_{TBM} is already adjusted for tunnel size by the use of average rated cutter force.)

The fine tuned gradient $(-)m$ is estimated as follows:

$$m \approx m_1 \left(\frac{D}{5}\right)^{0.20} \left(\frac{20}{\mathrm{CLI}}\right)^{0.15} \left(\frac{q}{20}\right)^{0.10} \left(\frac{n}{2}\right)^{0.05} \tag{16.6}$$

Sometimes, PR becomes too fast due to the logistics and muck handling. There may be a local increase in gradient from 1h to 1 day as a more rapid fall occurs in AR.

16.5 PENETRATION AND ADVANCE RATE VS Q_{TBM}

Development of a workable relationship between PR and $Q_{TBM}$was based on a process of trial and error using case records (Barton, 2000). Striving for a simple relationship, and rounding decimal places, the following correlation was obtained:

$$\mathrm{PR} \approx 5\,(Q_{TBM})^{-0.2} \tag{16.7}$$

From equation (16.5), one can therefore also estimate the AR as follows:

$$\mathrm{AR} \approx 5\,(Q_{TBM})^{-0.2} \cdot T^m \tag{16.8}$$

One can also check the operative Q_{TBM} value by back-calculation from penetration rate:

$$Q_{TBM} \approx \left(\frac{5}{\mathrm{PR}}\right)^5 \tag{16.9}$$

16.6 ESTIMATING TIME FOR COMPLETION

The time (T) taken to penetrate a length of tunnel (L) with an average advance rate of AR is obviously L/AR. From equation (16.5), one can therefore derive the following:

$$T = \left(\frac{L}{\mathrm{PR}}\right)^{1/(1+m)} \tag{16.10}$$

Equation (16.10) also demonstrates instability in fault zones, until $(-)m$ is reduced by pre- or post-treatment.

Example

Slate $Q_{TBM} \approx 39$ (from previous calculations with 15 tnf cutter force). From equation (16.7), $PR \approx 2.4$ m/h. Since $Q = 2$, $m_1 = -0.21$ from Table 16.1. If the TBM diameter is 8 m and if $CLI = 45$, $q = 5\%$ and $n = 1\%$, then $m \approx -0.21 \times 1.1 \times 0.89 \times 0.87 \times 0.97 = -0.17$ from equation (16.6). If 1 km of slate with similar orientation and rock quality is encountered, it will take the following time to bore it, according to equation (16.10):

$$T = \left(\frac{1000}{2.4}\right)^{(1/0.83)} = 1433\,\text{h} \approx 2\,\text{months}$$

i.e., $AR \approx 0.7$ m/h, as also found by using equation (16.8) and $T = 1433$ h.

A working model for estimating the TBM penetration rates and advance rates for different rock conditions, lengths of tunnel and time of boring has been presented. It may be used for prediction and back-analysis. Since the model is new, Barton (2000) emphasizes that improvements and corrections may be possible as future case records are available. Shielded TBM is very useful in metro tunnels.

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17
Metro tunnels

"It is cheaper to do things right the first time."

Phil Crosby

17.1 INTRODUCTION

A new era of underground space technology has begun with extensive networks of underground metro systems all over the world, due to the grace of God. No mega city may function efficiently without a mass transit system of high performance. They offer to everyone fast, safe, comfortable and cheap access to the different areas of a city. The under-city tunnels are also being excavated for a direct by-pass traffic as in Australia.

Following are the advantages of underground metros in the mega cities (Sharma, 1998):

(i) Crossing of hills, rivers and a part of oceans (straits).
(ii) Increase in market value of adjacent land and saving in man-hours.
(iii) They also favor a more aesthetic integration into a city without blocking view of beautiful buildings, bridges, monuments and religious functions.
(iv) Very high capacity in peak hours in any direction. It forms a part of integrated total city transportation system for convenience of people.
(v) It protects the residents completely from severe round-the-clock noise pollution from surface traffic.
(vi) Efficient, safe, more reliable, faster, comfortable and environmentally sustainable and technically feasible in developing nations also. It requires just 20 percent of energy that is consumed by road traffic. It reduces road accidents and pollution due to the decrease in vehicular traffic.

With more and more use of underground transit systems, it is necessary to prepare the contingency plans accordingly to take care of emergency situation. A good example of this is of black out in USA and Canada on August 14, 2003. More than thousand persons were

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B. Singh and R. K. Goel

stranded in subways, but the police have been trained to evacuate people from subways and skyscrapers without increase in panic.

The opinion polls carried out in the USA, Japan and 14 European countries show clearly the public moral support for environmental protection, even at the expense of reducing the economic growth.

The metro rail system uses ballastless track without joints which makes it almost free of maintenance. Signals are in the driver's cabin only and the software controls automatic driving of the engine, so the train stops exactly at the same position within ± 10 cm on the dot, hence, disabled persons can enter the coaches comfortably. The driver only opens and closes the door. Underground stations (with cross passage below tracks) are air-conditioned, and there are parks above underground stations. Performance is the best publicity. The life of the metro is about 100 years. The underground stations should meet the fire safety and evacuation norms (Heijboer et al., 2004).

Tunnels are ventilated properly; one fan pumps air and the other acts as an exhaust to take out smoke, in case coaches catch fire. These fans are switched on by station masters. The train will move to pumping fan side so that passengers do not die of smoke. All coaches are connected with see-through end for further escape.

Unfortunately construction costs for underground systems are a major deterrent when city officials consider the option of underground metro. Table 17.1 compares relative costs of the various types of infrastructures on the basis of a study conducted by French Tunnelling Association.

It is the experience of road users that open cut method of construction leads to a lot of inconvenience to the society and disruption of the environment, which must be compensated financially if any justifiable comparison between cut and cover method of construction and tunnel boring is to be made.

The photograph of a rail metro tunnel is shown in Fig. 17.1, which shows the prefabricated lining which is suitable for various soil, boulder and rock conditions except squeezing grounds (due to the high overburden pressure) and flowing grounds within water-charged–wide-shear zones (due to seepage erosion or piping failure). These may not occur in shallow tunnels.

Table 17.1 Relative costs of interstations structures.

Location	Infrastructure (a)	Equipment	Total (b)	Ratio (a)/(b)
At grade (surface)	25	30	55	0.45
Elevated (super structure)	100	30	130	0.75
Long span bridge	250	30	280	0.90
Cut and cover	100 to 200	40	140 to 240	0.70 to 0.80
Tunnelled	150 to 500	50	200 to 550	0.75 to 0.90

Fig. 17.1 Precast lining in a metro tunnel (*Ref:* http://www.railwayage.com/sept01/washmetro.html).

The work culture of Delhi Metro Rail Corporation is that there is no clerk with very few peons, which is the key to success. There is no witch-hunting for a wrong decision. The decisions were not delayed. Wrong decisions were automatically noticed and get corrected. Punctuality of staff plays an important role. NATM was adopted. Ten trees were planted for every cut tree. All underground stations are built by cut and cover method. The entire site was closed by walls on all sides. Exhaustive instrumentation is done to learn lessons for construction of future metros. The rehabilitation of structures (damaged by subsidence along tunnels) is the responsibility of all contractors to save excessive time which is lost in litigation by management (see http://www.delhimetrorail.com).

17.1.1 Findings of international tunnelling association

The International Tunnelling Association (ITA, 2004) has presented the following observations after analysis of data from 30 cities in 19 countries.

(i) The typical cost for surface : elevated and : underground metro systems were found to be approximately as 1 : 2 : 4.5.
(ii) It is generally accepted that underground systems are more expensive to operate than elevated or surface system.
(iii) Due to the requirement of large investment (capital and recurring costs) and the significant urban and environmental impacts, the choice is nearly always resolved politically. The government has to subsidize the cost to reduce the cost of ticket. The metro is not commercially viable.
(iv) The over-whelming choice (of 78 percent alignment) for urban metro systems is underground with very little at grade (surface) alignment. They are typically

designed to be of high speed and capacity (20,000 passengers per hour per direction) serving the city center.

(v) In many cases – for example in the center areas of older cities (with 2–7% area of streets only) – for functional, social, historic, environmental and economic reasons; there is no alternative to the choice of an underground alignment for new transit systems.

(vi) Noiseless technology may be used in the tunnelling.

17.2 SHIELDED TUNNEL BORING MACHINES

Tunnel boring machines (TBMs) with features-purpose-built to the specific ground conditions are now the preferred mode for bored tunnelling in mega cities. The high capital cost is justified by the length of tunnel more than 2 km [Pearse, 1997 cited by Sharma (1998)]. These TBMs offer the following advantages over the drilling and blasting method in the metro tunnels.

- Explosives are not used. Hence the operations in densely built-up areas produce much lower vibrations.
- Little or no overbreak.
- Excavation is fast. Time is money.
- Lower initial support capacity saves cost.
- Less labor cost.
- Reduces surface settlement to very low levels resulting in assured safety to the existing super structures.
- Reduces risk to life of workers by (i) rock falls at face or behind the TBM, (ii) explosives, (iii) hit by vehicles and (iv) electrocution.

In case of massive rock masses, open face tunnel boring machine is used as discussed in Chapter 15. Recently, dual mode shield TBMs are developed to bore through in all soil, boulders and weak rocks (in non-squeezing ground) under high ground water table. During tunnelling, the ground water table is lowered to the bottom of the tunnel by drilling drainage holes to keep ground dry. It works on the principle of shield TBM on which both scrapper picks as well as disc cutters are mounted on the cutter head. Table 17.2 summarizes the salient features of dual mode TBM and earth pressure balance machine (EPBM). During initial excavation at New Delhi underground metro, it was found that a large number of scrappers and buckets are getting detached from the cutter head. This was probably because of the presence of too many boulders in the soil strata. As a result, the bigger boulders were entangled in the large space between the arms and thereby knocking off the scrapper and buckets. Then protective plates and deflector strips were added around the buckets to avoid direct impact of boulders on the buckets, in addition to

Table 17.2 Salient features of tunnel boring machines (Singh, 2003).

S.No.	Item	EPBM – earth pressure balance machine	Dual mode TBM shielded tunnel boring machine
1.	Manufacturers	Herrenknecht of Germany	Herrenknecht of Germany
2.	Diameter	6.490 m	6.490 m
3.	Length of shield	3.8 m (7 m including tail skin)	3.9 m (6.9 m including tail skin)
4.	Weight of shield	252 MT	325 MT
5.	Length of complete system	57 m	70 m
6.	Cutter head rotation	1 to 7 rpm	1 to 6 rpm
7.	Torque	4000 kNm	4377 kNm
8.	Tunnel lining	Precast segmental RCC	Precast segmental RCC
9.	Finished diameter	5.7 m	5.7 m
10.	No. of segments per ring	6 (5+1 key)	6 (5+1 key)
11.	Thickness of lining	280 mm	280 mm
12.	Length of ring	1.2 m	1.2 m
13.	Grade of concrete	M-45	M-45
14.	Weight of each ring	16 tons	16 tons
15.	Joint sealing	EPBM gasket and hydrophilic seal	EPBM gasket and hydrophilic seal
16.	Power required	3 MW for each machine	3 MW for each machine
17.	Planned progress	10 m per day	6 m per day
18.	Maximum progress achieved so far	28.8 m per day	7.2 m per day

other modifications. Thereafter dual TBM has succeeded (Singh, 2003). The advantage of fully shielded TBM with segment erector is that there is no unsupported ground behind the shield. That is why TBMs have failed in poor grounds yet dual TBM has succeeded (Broomfield & Denman, 2003) in soils, boulders and weak rock mass in non-squeezing ground condition ($H < 350\ Q^{1/3}$ m).

It is necessary to inject the foam along with water at the cutter head which has the following advantages:

- Reduced permeability and enhanced sealing at the tunnel face.
- Suppresses dust in rock tunnelling.
- Excavation of wet soil or weathered rock is easier.
- Soil does not stick to the cutters.

There is no experience of success in TBM tunnelling through squeezing grounds any where in the world. It is understood that TBM may stuck in the highly squeezing ground or flowing ground. Therefore, both TBM and shielded TBM are not recommended in the squeezing ground and flowing conditions.

Obviously water lines, sewer lines, etc. have to be protected during tunnelling at shallow depth below congested mega cities. Some times a sewer line is ruptured during tunnelling. Enormous stinking sewerage is spread on the roads. It is difficult to repair sewer lines quickly. In soil area the ground water table (GWT) is lowered below the tunnel base before tunnelling is done in relatively dry soil. The subsidence profile due to lowering of GWT is, however, wider compared to that due to tunnelling. Careful underpinning of the foundations or columns of the old cracked building is done to adjust to the subsidence increasing with time, such as at Delhi.

There is a tendency very often to term the "geological surprises" as the cause to justify time and cost over-runs in completion of tunnelling projects. This could be true in some cases, yet managers should be cautious.

17.3 PRECAST LINING

In some projects, fiber reinforced–precast-concrete linings have been adopted. Precast-concrete–segmental lining is now used both in soil, boulders and weak rock masses. TBM is capable of placing them in position all round the circular tunnel with the help of segment erector. Segment bolts are then tightened by impact wrenches twice. The curved alignment is achieved with the help of tapering of the lining rings. All the rings are tapered and curvature is obtained by suitably adjusting the orientation of rings. Before taking inside the tunnel, the segments are checked on ground for any cracks/damage. As water tightness is extremely important for the durability of the tunnel lining, a double gasket system comprising a durable-elastomeric gasket and a water sealing made from the hydrophobic material is used. These gaskets are located in grooves cast into the edges of the precast concrete segments. Together with the high precision casting of the segments achieved by precision steel molds, gaskets will ensure the durable and water tight tunnels. Hydrophobic seals expands upto 250 percent, once it comes into contact with water.

Thought should be given to fire-resistant design of concrete lining, as fires in trains are common these days. Extra thickness of concrete covers ($\cong$75 mm) should be provided over the steel reinforcement. Under-reinforced concrete segments may be used to ensure the failure in ductile phase, if it occurs.

Grouting is carried out simultaneously with the tunnelling. There are inbuilt ports in the tail skin of TBM. These are used in primary grouting of annulus (void between excavation profile and outer face of the precast ring). Grouting is continued upto 3 bars (0.3 MPa) pressure. Excavation is not commenced until the previous lining is completed. Secondary grouting is also done within 14 days of ring erection. Every third ring is grouted to pressure of 3 bars (0.3 MPa). Secondary grouting will fill up any void left during primary grout due to its shrinkage.

17.4 BUILDING CONDITION SURVEY AND VIBRATION LIMIT

Open trenches and shafts are excavated by drilling and blasting method for connection to the underground metro system. The controlled bench blasting method is used in open excavation, under busy and congested roads which are flanked by old or heavy buildings and monuments. Before designing the controlled blasting, the entire rock mass is explored thoroughly (Chapter 11, see article on smooth blasting). The trial blasts are detonated to determine the safe-scaled-distance ($= R/W^{1/2}$, where W is the weight of charge per delay of detonators and R is the distance from the blasting pattern), according to the nature of structures.

The next step is to assess the condition of buildings standing near the blast site to determine how much vibration can be sustained by these structures, specially old buildings and ancient monuments if any. Table 17.3 specifies permitted peak particle velocities (PPV) as per German standard. It may be reminded that ISRM has recommended almost twice PPV values.

Archaeologists suggest that no surface metro station should be built within protected 100 m periphery of a protected (heritage) monument. In such cases, an underground metro station may be a better choice.

17.5 IMPACT ON THE STRUCTURES

The blasting works may affect the surrounding structures slightly in spite of the controlled blasting. In worst case, small cracks may develop in RCC and masonry. The air over-pressure may also create cracks in glass works of doors and windows in nearby areas. Table 17.4 summarizes the various types of damages to structures. A huge compensation

Table 17.3 Permitted peak particle velocities (PPV) on structures.

S.No.	Condition of structure	Max. PPV (mm/s)
1.	Most structures in “good condition”	25
2.	Most structures in “fair condition”	12
3.	Most structures in “poor condition”	5
4.	Water supply structures	5
5.	Heritage structures/bridge structures	5

Table 17.4 Building damage classification (Burland et al., 1977; Boscardin & Cording, 1989 cited by Agarwal & Gupta, 2002).

Risk category	Description of degree of damage	Description of typical damage and likely form of repair for typical masonry buildings	Approx. crack width (mm)	Max tensile strain (%) due to subsidence
0	Negligible	Hairline cracks	–	Less than 0.05
1	Very slight	Fine cracks easily treated during normal redecorations. Perhaps isolated slight fracture in building. Cracks in exterior brickwork visible upon close inspection.	0.1 to 1	0.05 to 0.075
2	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible, some repointing may be required for weather tightness. Doors and windows may stick slightly.	1 to 5	0.075 to 0.15
3	Moderate	Cracks may require cutting out and patching. Recurrent cracks can be masked by suitable linings. Tack-pointing and possibly replacement of a small amount of exterior brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Water tightness often impaired.	5 to 15 or a number of cracks greater than three	0.15 to 0.3
4	Severe	Extensive repair involving removal and replacement of sections of walls, especially over doors and windows. Windows and door frames distorted. Floor slopes noticeable. Walls lean or bulge noticeably, some loss of bearing in beams. Utility services disrupted.	15 to 25 but also depends on number of cracks	Greater than 0.3
5	Very severe	Major repair required involving partial or complete reconstruction. Beams lose bearing, walls lean badly and require shoring. Windows broken by distortion. Danger of instability.	Usually greater than 25 but depends on number of cracks	–

may have to be given to the owners of the damaged buildings nearby according to the specified class of damage (Agarwal & Gupta, 2002).

The traffic is stopped during blasting time for a few minutes and all the roads, other exits/entries to the blasting site are closed for safety reasons. The flying of rock pieces during an urban blasting may have severe consequences.

17.6 SUBSIDENCE

The subsidence of ground and differential settlement of nearby structures takes place due to underground tunnelling. The dewatering due to excavation causes more widespread subsidence due primarily to the settlement of overlying loose deposit of soil, silt or clay specially. In totally rocky areas, the subsidence is very small and does not cause any worry. The following instruments are recommended for precision monitoring of structures.

- Precise levelling points,
- Tiltmeters,
- Crack gauges embedded in the nearby structures and
- Vibration monitoring of old/ancient structures.

In case the actual settlement is expected to go beyond the predicted subsidence, the whole construction methodology must be reviewed. Table 17.4 may be used which specifies the maximum tensile strain caused by subsidence (= increment in spacing of columns divided by the distance between columns, expressed in percentage).

17.7 PORTAL AND CUT SLOPES

It is better to locate the portals deeper into the ground or mountain where rock cover of at least equal to width of tunnel is available. The slope of the portal should be stable. Otherwise the same should be reinforced properly with the rock anchors. Alternatively a thick breast wall (1 m) of concrete should be constructed to ensure stability of portals (Singh & Goel, 2002).

It is needless to mention that the side slopes of open trenches should be stable. Deoja et al. (1991) have suggested in Table 17.5, the dip of safe cut slopes with and without protective measures for both rocks and soils. The rail lines are also being built in hilly terrains. Table 17.5 is also recommended for deciding safe cut slope angles in the hills. The utmost importance of the stable cut slopes is highlighted by the fact that landslides/rock falls have taken place just near portals suddenly after heavy rains, thereby causing very serious train accidents sometimes.

Table 17.5 Preliminary design of cut slopes for height of cut less than 10 m (Deoja et al., 1991).

S.No.	Type of soil/rock protection work	Stable cut slope without any breast wall or minor protection work (vertical : horizontal)	Stable cut slope with breast wall (vertical : horizontal)
1.	Soil or mixed with boulders		
	(a) Disturbed vegetation	1 : 1	n : 1*
	(b) Disturbed vegetation overlaid on firm rock	Vertical for rock portion and 1:1 for soil portion	Vertical for rock portion and n : 1 for soil portion
2.	Same as above but with dense vegetation forests, medium rock and shales	1 : 0.5	5 : 1
3.	Hard rock, shale, or harder rocks with inward dip	1 : 0.25 to 1 : 0.10 and vertical or overhanging	Breast wall is not needed
4.	Same as above but with outward dip or badly fractured rock/shale	At dip angle or 1 : 0.5 or dip of intersection of joint planes	5 : 1
5.	Conglomerates/very soft-shale/sand rock which erode easily	Vertical cut to reduce erosion	5 : 1

* n is 5 for $H < 3$ m; 4 for $H = 3 - 4$ m and 3 for $H = 4 - 6$ m.

The approach road/rail line to tunnel should be widened sufficiently. Catch drains of proper depth and width should be made on both sides of the track, according to the height and slope of cuts and size of boulders on the slope. A fence of 3.5 m height should be erected along both drains and tied to steel poles at about 2 m center to center with horizontal bracings at 1 m center to center. Then poles are anchored in the slopes. The price is paid back if the wire net (4 mm ϕ wires welded at 10×10 cm or alternative) withstands the impact of rock fall jumping. The wire net should then be replaced soon, if required (Hoek, 2000).

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18
Tunnelling in swelling rocks

"Be prepared for the worst and hope for the best."

Anonymous

18.1 INTRODUCTION

Swelling grounds cause major problems of supporting both during construction and during the operational life of a tunnel due to excessive tunnel wall displacement. In certain cases, invert heave of over 25 cm per year has been observed. The swell rates in the several (75–100 years) old Swiss tunnels decreased to 0.5 to 1.0 cm per year with the passage of time. However, the total invert heave was of the order of several meters requiring regular repairs of the inverts.

In the Grenchenberg tunnel of Switzerland, the use of invert arch reduced the invert heave (Golta, 1967). In Belchen tunnel of Switzerland through marl, anhydrite and opalinus clay and shale; heaving of invert and cracking of drainage pipes was observed soon after excavation (Huder & Amberg, 1970; Schillinger, 1970; Grob, 1972). A 17 m high cavity was formed at the roof of the Sallsjo Tailrace tunnel in Sweden about an year after the commissioning of the tunnel due to the presence of a 3 m wide shear zone containing montmorillonite clays (Selmer-Olsen, 1970). In the Bozberg tunnel of Switzerland, invert heave of 27 and 33 cm were observed in anhydrite (swelling mineral) and opalinus clay shale respectively in 31 years between 1923 and 1954 (Beck & Golta, 1972). Similarly, in Udhampur–Katra tunnel of Northern Railway, India, floor heaving of 40–60 cm was observed in 2004 because of swelling of claystone having montmorillonite, kaolinite and illite.

Some construction engineers harbor a wrong notion that swelling could be prevented if the ground is hermetically sealed by sprayed concrete against air moisture. When such a procedure was adopted in the Lieras tunnel of Norway, it was found that 10–20 cm thick shotcrete cracked within a week (Cecil, 1970). However a second layer, applied

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after the cracking of the first layer had ceased, proved to be adequate. According to Wahlstorm (1973),

> *"theoretically, swelling can be prevented if the swell prone ground is sealed with impermeable concrete to prevent access of atmospheric water. However, such a measure rarely helps in actual practice because of the presence of pore water within the clays."*

18.2 SUPPORT PRESSURES IN SWELLING GROUND

The swelling pressures on tunnel supports may be very high. Swelling grounds have normally low modulus of deformation and are capable of exerting high pressures even at moderate depths. Baldovin and Santovito (1973) measured contact pressures up to 3.5 MPa on provisional concrete lining (placed about 20 days after excavation) in the Foretore tunnel in Southern Italy.

Einstein and Bischef (1975) have proposed an analysis cum design method for tunnels in swelling rock. The procedure consists of the following seven steps:

(i) Determination of the primary state of stress,
(ii) Determination of the swell zones around the openings based on the primary state of stress and the stress changes caused by the opening,
(iii) Laboratory swell tests in the oedometer on samples taken from swell zones,
(iv) Determination of time-swell properties from oedometer tests by measuring the time displacement relations for several stress increments,
(v) Derivation of the swell-displacements for the stress difference between the primary state of stress and the state of stress after excavation,
(vi) Performing swell-time computations and
(vii) In situ measurements of swell-displacements and swelling pressures.

Further, they recommended the following design features based on the above procedure:

(i) Use an invert arch instead of horizontal struts,
(ii) Bolt the swelling zone with the ground below it,
(iii) Use compressible backfill between the support and the swelling ground,
(iv) Trim the floor,
(v) Employ grouting to seal off preferential paths supplying water to the swell prone zone,
(vi) Provide constraint to the swelling ground by cutting slots and injecting grout under pressure through these slots,
(vii) Avoid exposure of the swelling ground to atmospheric moisture by applying sprayed concrete (SFRS) and
(viii) Provide good drainage inside the tunnel.

In practice sometimes it is difficult to separate the squeezing and swelling components of support pressure from the total pressure acting on tunnel supports. Jethwa et al. (1977)

considered the total support pressure as the sum of the loosening pressure (p_l), squeezing pressure (p_g) and the swelling pressure (p_s). The total of first two kinds of pressures was estimated from elasto-plastic theory (Daemen, 1975), and the swelling pressure was estimated after an empirical relation of Komornik and David (1969) as given below:

$$\log p_s = 2.132 + 0.0208\, w_L + 0.000665\, \gamma_d - 0.0269\, w_n \tag{18.1}$$

where

p_s = swelling pressure (kg/cm^2) at zero swelling strain,
w_L = liquid limit (%),
γ_d = natural dry density (kg/m^3) and
w_n = natural moisture content (%).

It may be noted that the swelling pressure decreases rapidly with increasing swelling strain or tunnel closure.

Later, Daemen (1978) raised doubts on the validity of assuming the total support pressure as the arithmetic sum of the three kinds of pressures (loosening + squeezing + swelling). Subsequently, Singh (1978) emphasized on the interaction between the squeezing and swelling pressures and suggested that only the greater of the two should be considered. If radial stress is more than swelling pressure, the rock will behave as non-swelling rock in squeezing ground conditions. Thus, the short-term support pressure (p_{io}) may be given as follows:

when $p_g > p_s$

$$p_{io} = p_l + p_g \tag{18.2}$$

when $p_g < p_s$

$$p_{io} = p_l + p_s \tag{18.3}$$

$$\text{and,} \quad p_{ult} = 2 \text{ to } 3p_{io} < \text{overburden pressure} \tag{18.4}$$

where

p_g = pressure in the squeezing ground condition,
p_{io} = total short-term support pressure,
p_l = loosening pressure due to the gravity and
p_{ult} = long-term support pressure.

It may be mentioned here that the total support pressure may never exceed the overburden pressure at the crown of a tunnel, irrespective of the magnitude of swelling pressure. The zone of swelling may extend upto a radius, where the radial stress is equal to the swelling pressure corresponding to no swelling strain.

Jethwa (1981) has prepared a design chart for estimating the long-term support pressure in terms of cover pressure (p_{ult}/P) for different values of peak angle of internal friction of rock masses, according to the degree of squeezing (Fig. 18.1). He used elasto-plastic theory of Daemen (1975) and modified it for time-dependent support pressures. He assumed

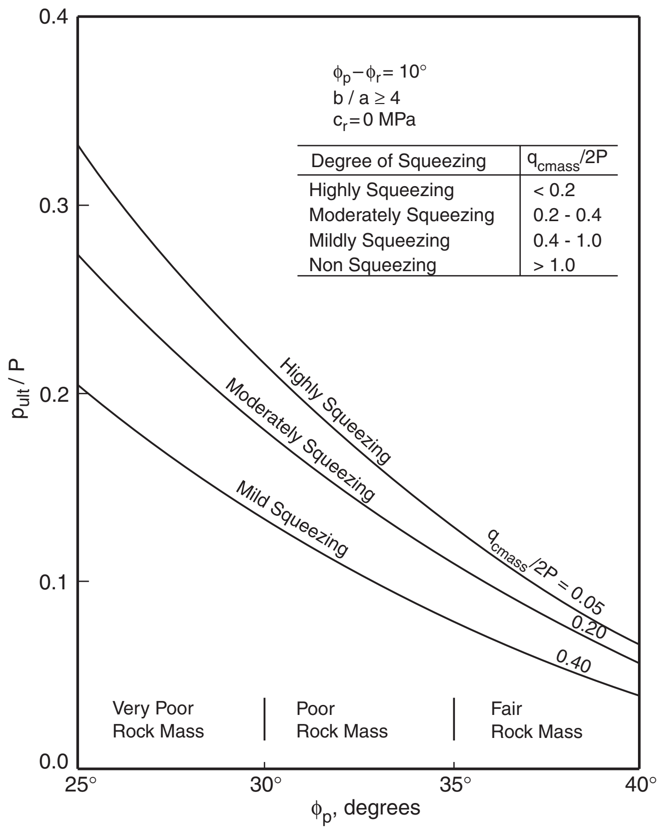

Fig. 18.1 Chart for ultimate support pressure (p_{ult}) in squeezing ground conditions (P = overburden pressure).

Mohr's theory of shear strength which is applicable to clays, crushed shales/clay stones, etc. within thick fault zones. It was found that p_{ult} is nearly constant beyond a radius of broken (plastic) zone which is more than four times the radius of tunnel. Limited experience proves the chart to be useful in estimation of support pressures approximately. The angle of internal friction of normally consolidated clays within the fault zones may be found from equation (7.10).

18.3 VARIATION OF SUPPORT PRESSURE WITH TIME

The soil parameters for the black clay within the intra-thrust zone of Chhibro–Khodri tunnel, Yamuna hydroelectric project, in lower Himalaya were found as follows (Chapter 20):

Liquid limit	= 50
Natural water content	= 6.6 percent

Dry unit weight	= 2.2 gm/cc
Plasticity index	= 27
Height of overburden above crown	= 280 m
Diameter of tunnel	= 9.0 m
Estimated swelling pressure from equation (18.1)	= 3.9 kg/cm^2 (0.39 MPa)
Observed support pressure	= 12.8 kg/cm^2 (1.28 MPa)

The support pressures on steel rib supports were observed with the help of load cells and contact pressure cells. The red shales and the black clays of the Chhibro–Khodri tunnel and the shales of the Loktak tunnel contained swelling minerals. However, the swelling pressures have been neglected as discussed earlier, because the swelling pressures (from equation (18.1)) worked out to be much less than the squeezing pressures. For example, the predicted swelling pressures (Jethwa et al., 1977) are 0.6 and 3.9 kg/cm^2 in comparison to the squeezing pressures of 3.5 and 10.0 kg/cm^2 (against observed pressures of 3.07 kg/cm^2 and 12.8 kg/cm^2) in the cases of the red shales and the black clays, respectively.

At the Yamuna project, in a test-section in clays it was observed that an early non-uniform stress distribution around steel supports changed to nearly uniform distribution with passage of time (Fig. 18.2). Likewise, placement of concrete lining in the early stages would have loaded the same, non-uniformly (Jethwa et al., 1977).

Observations up to a period of about two years on support pressures on steel supports at a test-section in clays at the Yamuna project (Jethwa & Singh, 1973), when plotted on a log–log scale, make a straight line (Fig. 18.3). These observations revealed that the

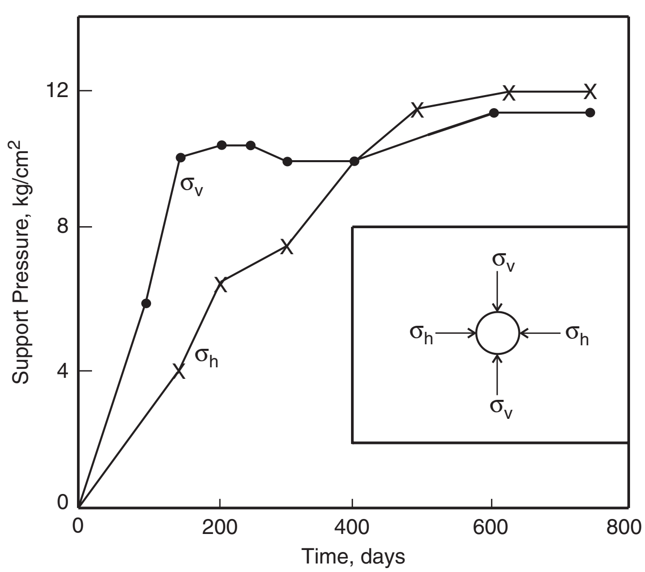

Fig. 18.2 Observed support pressure vs. time in black clays, Yamuna project (9.0 m diam. tunnel).

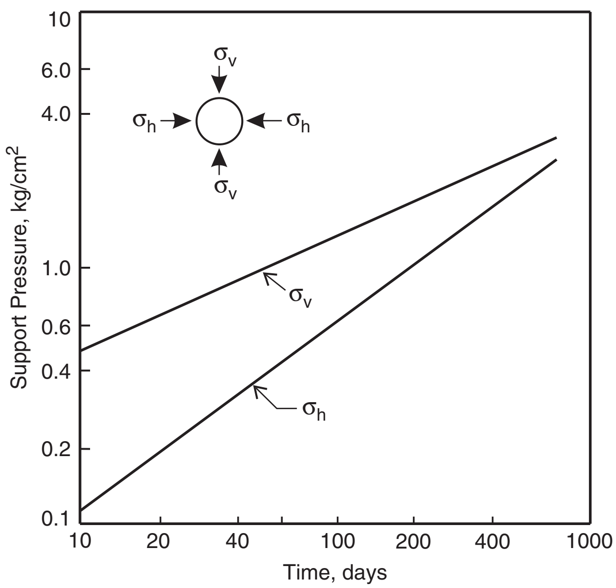

Fig. 18.3 Observed support pressure vs. time in black clays, Yamuna project (3.0 m diam. tunnel).

initial non-uniform stress distribution on tunnel supports becomes almost uniform with passage of time. Thus, swelling or time-dependent deformation may in fact prove to be beneficial for stabilizing the concrete lining, because a uniform support pressure simply tries to pre-stress the lining. It may be highlighted here that concrete lining is essential in case of water tunnels.

18.4 CASE HISTORIES

18.4.1 Malgovert tunnel

Jaeger (1972) described a case history of Malgovert tunnel in France. It is a 15 km long tunnel of Tignes hydroelectric project, subjected to a 750 m head of water. At one location, this tail race tunnel (TRT) passed through swelling ground and the strata of badly crushed shales, which was located near a fault. These shales were nearly dry and so tunnelling was easy. Soon the ground became wet causing so much of swelling that even flange-to-flange polygonal steel supports buckled under high swelling pressures. The excavation of heading was then stopped and a diversion tunnel was created. But unfortunately a failure in the crown near the same fault caused heavy in-flow of water (nearly 4000 gal/min), which flooded the tunnel. Yet again the excavation was stopped and several pumps were used to drain the water. Consequently, the tunnelling was hampered for several months.

The concrete lining was delayed until swelling and time-dependent deformations were stabilized. It was found that tunnel soffit had been lowered on average by 30–60 cm over a length of several kilometers and the size of the tunnel was thus reduced significantly. For smooth flow of water, it was decided to cut the crown to the planned height before concreting in lining of the tunnel.

It is clear that the presence of swelling clays in an otherwise competent rock mass may present serious problems of supporting. As such, it is advisable to check the presence of swelling minerals (montmorillonite, anhydrite, etc.) in clays whenever a thick shear zone or fault gouge or soft shale or mudstone is encountered in a tunnel. Once the presence of swelling clays is established, it would be better to allow the ground to swell before a concrete lining is installed. A thin layer of steel fiber reinforced shotcrete (SFRS) may be applied near the tunnel face soon after excavation for the immediate support, if the stand-up time of the rock mass is too short. It should also be realized that fragile swelling rocks may be fractured in over-stressed reaches where support pressure in squeezing ground condition may be much higher than the swelling pressure. As such, a circular shape of an opening is recommended. Its diameter should be 1.1 B, where B is the design diameter of the opening without lining.

In highly swelling rocks, one should use the closely spaced steel supports with invert struts which are embedded all around in the SFRS for its lateral stability. It is also suggested that the drainage system of the tunnel should be made better to prevent water from seeping into the rock mass. It is wise to monitor the behavior of rock mass with borehole extensometers and load cells, etc. to improve confidence. The monitoring may continue to at least whole period of stabilization of tunnel wall closure.

18.4.2 Bolu tunnel

Dalgic (2002) describes a very interesting case of tunnelling in clays of a wide intra-thrust zones of Bolu tunnel, Turkey. The twin road tunnels are 12 m wide with a rock pillar of 40 m width under rock cover of 70 m. As expected, shotcrete (with horizontal slots to allow the tunnel closure) failed, because lower most part of the lining squeezed more inside the tunnel. Further, rock bolts also suffered failure due to the little bonding with clay and adverse affect on remolding of clays. Excessive radial closure of 110 cm was observed. The ribs encased with 30 cm thick SFRS contained the deformation to 5 cm. The reinforced cement concrete lining was planned to be provided within both the intra-thrust zones after about a year when the observed rate of deformations per month are less than 2 mm. An earthquake of 7.2 M occurred causing horizontal acceleration of 0.8 g. Unfortunately, the SFRS lining failed in the intra-thrust zones only, perhaps due to dynamic earth pressures (see Appendix I.4), before RCC lining could be built. The broken or plastic zone increased upto the ground surface, causing two sink holes on the ground surface during earthquake. He concluded that NATM flexible support system failed. Yet more heavy SFRS (with ribs) support system succeeded in the clays of fault zones.

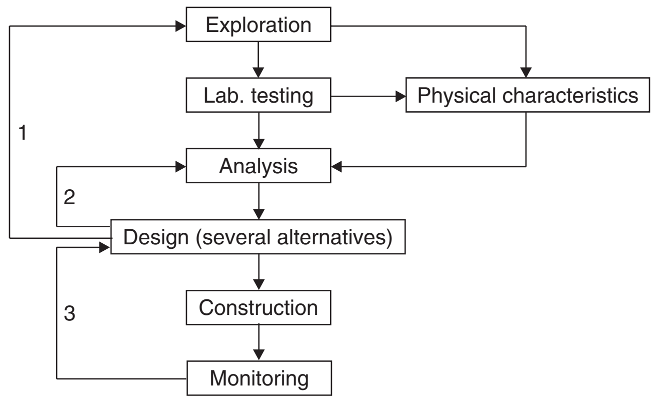

Fig. 18.4 Integrated analysis-design approach for rock structure (ISRM, 1994).

18.5 DESIGN APPROACH

ISRM (1994) recommends the integrated analysis-design approach to any engineered rock structure specially to the tunnel as shown in Fig. 18.4. It is interesting that ISRM recommends the field instrumentation.

In some road tunnels, the rock is allowed to swell freely and is regularly removed (shaved off) such that the tunnel continues to be usable.

Drainage of water flowing into the tunnel is one of the most effective measures against swelling. Sealing of all exposed surfaces is also very effective measure. However, swelling still does take place due to the migration of moisture to the zones of stress relaxation which are formed by excavation. In shales, the suction effect is very significant. Further, in the case of jointed shales, grouting may provide a barrier against further flow of moisture or seepage water through rock joints. The swelling cum heaving of tunnel bottom has been arrested by providing rock anchors below its bottom. Some engineers prefer rigid RCC tunnel lining to withstand high swelling pressures.

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19

Tunnelling through squeezing ground condition

"Strength and weaknesses go together both in matter and life. If nature has given weakness, nature will compensate. No one is perfect."

Source: IIT Roorkee

19.1 INTRODUCTION

The squeezing or elasto-plastic pressure (also called genuine mountain pressure in Europe) is mobilized due to failure of a weak rock mass around a tunnel under influence of high overburden pressure or tectonic stresses. The over stressed zone of rock mass fails where tangential stress (σ_θ) exceeds the mobilized UCS of rock mass. The failure process will then travel gradually from the tunnel boundary to deeper regions inside the unsupported rock mass. The zone of the failed rock mass is called the "broken zone." This failed rock mass dilates on account of the new fractures. A support system after installation restrains the tunnel closure and gets loaded by the support pressure. Chapter 13 presents the criteria of squeezing ground condition.

The tunnel closure may be both instantaneous and time dependent. It is the time-dependent displacement which dominates in fragile rock masses under high overburden, particularly when a broken zone is formed around an opening. Therefore the support system attempts to curb these time-dependent tunnel closure and in turn attracts higher loads (Jethwa, 1981; Dube, Singh & Singh, 1986).

Terzaghi (1946) advocated that support pressure for squeezing rocks is higher for greater overburden and is directly proportional to the tunnel width. The experience is that a weak strata in the beginning gave the impression of non-squeezing ground but the same weak strata under high overburden showed significant squeezing later on, upsetting the engineers. At another location in little stronger rock mass, squeezing did not occur at even 1400 m overburden, happily surprising every one. Thus real problems are: (i) how to anticipate squeezing condition and assess the degree of squeezing, (ii) how to estimate the short-term and long-term support pressures as well as allowable tunnel closures

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and (iii) what should be the strategy of supporting severe squeezing grounds. It is possible to tackle these challenging problems by the grace of God, if the "tunnel mechanics" of squeezing ground is better understood. Barla (2004) has given an excellent review of the state-of-the-art.

Appendix I gives the details of the equations for stress distribution within elastic and broken zones. Sections 19.2 and 19.3 below are only brief summaries of expressions for support pressures in the squeezing ground.

19.2 CRITERION FOR SQUEEZING GROUND CONDITION

According to Mohr's theory the criterion of squeezing is

$$\sigma_{\theta max} > q_{cmass} \tag{19.1}$$

where

$\sigma_{\theta max}$ = maximum tangential stress at the face of excavation and
q_{cmass} = UCS of rock mass.

Since rock mass is anisotropic due to joints, the intermediate principal stress σ_2 also enhances the deviatoric strength (differential stress at failure) of rock mass as follows:

$$\sigma_1 - \sigma_3 = q_{cmass} + \frac{\sigma_2 + \sigma_3}{2} A \tag{19.2}$$

where

A = rock mass parameter,
= $2 \sin \phi_p / (1 - \sin \phi_p)$.

Thus, the criterion of squeezing according to the modified Mohr equation is ($\sigma_3 = 0$ and $\sigma_2 = P_o$),

$$\sigma_{\theta max} > q_{cmass} + P_o A/2 = q'_{cmass} \tag{19.3}$$

where

$\sigma_{\theta max}$ = tangential stress at the periphery of a tunnel,
$\cong 2\gamma H$ in nearly circular tunnels,
q_{cmass} = compressive strength of rock mass,
= $7\ \gamma\ Q^{1/3}$ MPa,
= $5.5\ \gamma\ N^{1/3}/B^{0.1}$ MPa,
γ = unit weight of rock mass in gm/cc,
B = width of tunnel in meters,
Q = post-construction rock mass quality,
N = rock mass number (= Q assuming SRF = 1.0),
P_o = in situ stress along the tunnel axis,
$\cong \gamma H$

$A \quad = \dfrac{2 \sin \phi_p}{1 - sin\, \phi_p}$,
$\phi_p \quad$ = peak angle of internal friction of jointed rock mass,
q'_{cmass} = biaxial compressive strength of rock mass and
$H \quad$ = height of overburden above the crown of tunnel.

It may be noted that the ratio of $\sigma_{\theta max}$ and q_{cmass} may not predict the degree of squeezing realistically. In the case of hard rock masses with rough dilatant joints and higher A parameter, squeezing may not take place even if $\sigma_{\theta max} \gg q'_{cmass}$. So, equation (19.3) is a better criterion of squeezing.

19.3 ELASTO-PLASTIC THEORY OF SQUEEZING GROUND

Daemen (1975) developed the theory of support pressures in the squeezing ground condition. With modification in the Mohr strength criterion, his equations have been rederived in Appendix I. Perhaps, this 2D theory may also be applied in banded strata where thickness of weak strata is more than the width of the tunnel.

The horizontal support pressure p_h is given by the following equation in nearly hydrostatic case ($\lambda = 1$) (Fig. A1.3 in Appendix I)

$$p_h = \left[\frac{2P - q_{cmass} - (AP_o/2)}{2 + (A/2)} + c_r \cdot \cot \phi_r \right] \left(\frac{a}{b} \right)^{\alpha} - c_r \cot \phi_r \tag{19.4}$$

where P_o is the in situ stress along the tunnel axis; and P is the vertical and horizontal in situ stress across the tunnel axis. The residual cohesion is c_r and angle of internal friction is ϕ_r within the broken zone. The radius of circular tunnel is "a" and the radius of concentric broken zone is "b." It may be noted that the parameter A reduces the support pressure drastically, where A is significant such as in jointed hard rocks with rough joints.

The vertical support pressure is,

$$p_v = p_h + \gamma M_\gamma \left(b - a \right) \tag{19.5}$$

where γ is unit weight of rock mass and parameter M_γ is given by equation (AI.30) in the Appendix I.

Further, elasto-plastic theory shows that the support pressure decreases rapidly with increasing size of the broken zone. Thus development of broken zone should be permitted upto three times the radius of the opening for reducing support pressures. The vertical and horizontal support pressures may be considered of the same order.

Obviously the support pressures given by equations (19.4) and 19.5) are the short-term pressures. The long-term support pressures will be much higher, two or three times the short-term support pressures unlike non-squeezing ground conditions where generally the long-term support pressure is 1.7 times the short-term support pressure (Barton et al., 1974).

19.4 DISPLACEMENTS OF TUNNEL WALLS

It is common knowledge that the rock failure is associated with volumetric expansion due to creation and progressive widening of new fractures. Consequently, all the points within the broken zone in a circular tunnel shift almost radially towards the opening because the expanding rock mass is kinematically free to move only in the radial direction. Labasse (1949) assumed that the volume of failed rock mass increases at a constant rate (called coefficient of volumetric expansion K).

The displacement at the boundary of broken zone (u_b) is negligible compared to that at the opening periphery (u_a). So the coefficient of volumetric expansion is defined as follows:

$$K = \frac{\pi b^2 - \pi \left(a - u_a\right)^2 - \pi(b^2 - a^2)}{\pi b^2 - \pi a^2} \tag{19.6}$$

Equation (19.6) can be solved as below for obtaining u_a

$$u_a = a - \sqrt{a^2 - K(b^2 - a^2)} \tag{19.7}$$

Jethwa (1981) has found the overall values of K from instrumented tunnels in Himalaya. These values are listed in Table 8.1. Actually, K varied with time and radius vector. K was more in roof than in walls. So only peak overall values of K are reported and considered to have stabilized to a great extent with time (15–30 months). It is heartening to note that the value of K for crushed shale is the same at two different projects. It may be noted that higher degree of squeezing was associated with rock masses of higher K values.

Fig. 19.1 shows observed radial displacements within the broken zone of a tunnel in highly squeezing ground condition, at Giri Hydroelectric Project, in Himalaya, India (Dube et al., 1986). It is interesting to note that the displacement vs. radius curves tend to converge at a point X after some time. The radius at the convergence point is believed to be equal to radius (b) of the broken zone. Its ordinate is thus equal to the displacement at the boundary of the broken zone (u_b). This is a graphical method for estimating the radius of the broken zone approximately from the observations of multiple borehole extensometer(s) and tunnel closure (Dube, 1979; Dube et al., 1986).

Why squeezing rock is creeping so much, compared to the same rock mass in non-squeezing condition under shallow overburden? What is the cause of extensive creep or time-dependent deformations for more than 1000 days? Whereas the tunnel closures are stabilized within 25 days in the non-squeezing ground conditions. Further research is needed on creep models of rock mass in the failure condition. It may be the tertiary creep.

It may be pointed out that heaving of the tunnel bottom occurred in severe squeezing ground conditions. Engineers tried in vain to chop off heaving frequently to maintain the rail line for haulage of muck. The better solution is rock bolting and shotcreting of the floor to withstand the heavy support pressures in the bottom.

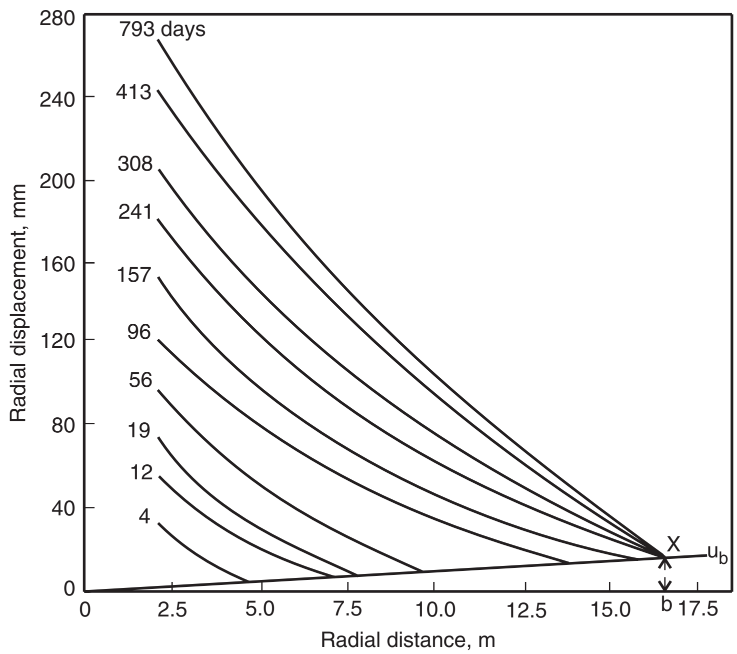

Fig. 19.1 Variation of radial displacement with radial distance within slates/phyllites of Giri Tunnel, India (Dube et al., 1986).

For tunnel under a deeper overburden, only squeezing failure occurs at both sidewalls (Fig. 19.2). The roof does not collapse, due to the higher horizontal stress (Hsu et al., 2004). The critical location of the interbedded formation is at one of the sidewalls, where the dipping direction of the bedding plane is unfavorable. For the formation located at the other sidewall, only minor squeezing failure develops. An increase of horizontal stress will help to reduce the amount of squeezing deformation for a tunnel in a steeply dipping formation. A tunnel with a steeply dipped formation at the portal section or under a shallow overburden is prone to cave-in failure. Cave-in failure starts at one sidewall, due to flexural tensile buckling, and then leads to the roof rock sliding, followed by the squeezing of the other sidewall (Hsu et al., 2004).

Shalabi (2005) analyzed the rise in contact pressure on lining of still-water tunnel (Utah, USA) in squeezing ground condition in a 200 m wide fault zone under overburden of 700 m. The rock consists of moderately to heavily jointed shale, silt and clay. All parameters of the power law creep model were determined in the laboratory. The measured support pressure on the concrete segments was about 7 percent of the overburden pressure after a time period of three months, against predicted contact pressure of 12 percent which may rise to 16 percent in 10 years. Actual delay of four days in the lining may reduce the contact pressure by 20 percent.

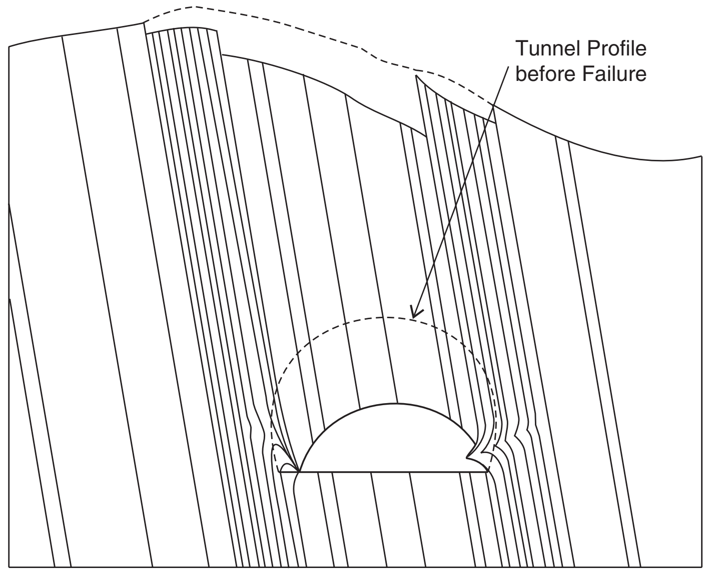

Fig. 19.2 Squeezing failures occur first at the sidewalls for a tunnel (the bedding spacing is 0.5 m and dip angle is 80°) [Hsu et al., 2004].

Yassaghi and Salari (2005) reported squeezing of 3 percent of diameter at the contact zone between andesite–basaltic bodies and the tuff rocks under an overburden of 300 m in a 6 m wide Taloun road tunnels, Iran. The GSI of contact zone is only 14. Therefore, heavy supports were applied consisting of steel arches at spacing of 1 m and concrete lining of 30 cm thickness. The initial convergence rate of 60 mm/month was reduced to less than 3 mm/month. The analysis of the lining using UDEC showed that the stress, due to bending moment is greater than that of the axial forces on the lining due to the high shear forces near the heterogeneous contact zone.

19.5 COMPACTION ZONE WITHIN BROKEN ZONE

Jethwa (1981) observed that the values of K are negative near the tunnel and increased with radius vector. Thus, he postulated the existence of compaction zone within the broken zone (Fig.19.3). The radius of the compaction zone (r_c) is estimated to be approximately equal to

$$r_c = 0.37b \tag{19.8}$$

Thus broken zone will not develop where b is equal to $a/0.37$ or $2.7a$. This is the reason why compaction zone was not observed in European tunnels in the squeezing ground conditions as b was perhaps less than $2.7a$.

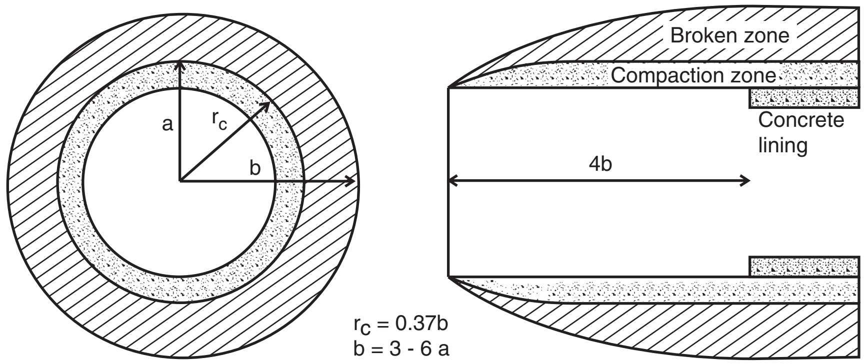

Fig. 19.3 Compaction zone within broken zone in the squeezing ground condition (Jethwa, 1981).

In an ideal elasto-plastic rock mass, the compaction zone should not be formed. The formation of the compaction zone may be explained as follows. A fragile rock mass around the tunnel opening fails and dilates under the influence of the induced stresses. The dilated rock mass then gets compacted due to the passive pressure exerted by the support system in order to satisfy the ultimate boundary condition, that is zero rate of support deformation with time. The development of the support pressure with time would reduce the deviator stresses ($\sigma_\theta - \sigma_r$) within the compaction zone which in turn will undergo creep relaxation manifested by the negative K values.

19.6 FACE ADVANCE FOR STABILIZATION OF BROKEN ZONE

Daemen (1975) and Jethwa (1981) showed that the radius of the broken zone increases with face advance (Fig. 19.3). The broken zone is stabilized after some face advance and time. Fig. 19.4 compiles data of the final radius of the broken zone (b/a) and the corresponding face advance (z/a). It is seen that

$$z = 4b \tag{19.9}$$

The concrete lining should be laid only after displacements have stabilized. This means that concrete lining should not be built in the tunnel length of $z = 4b$ from the tunnel face.

19.7 GROUND RESPONSE CURVE

Equations (19.5) and (19.7) are used to calculate the support pressure and closure in roof for different values of radius of broken zone. The curve between support pressure and tunnel closure is plotted as shown in Fig. 19.5. This curve is called ground response (reaction) curve. It is evident that the support pressure decreases rapidly with increasing tunnel closure.

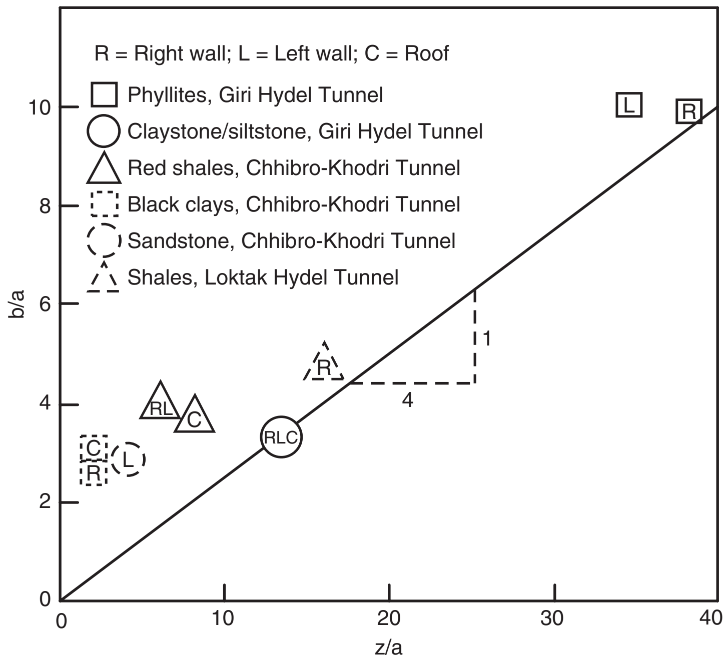

Fig. 19.4 Face advance (z) for stabilization of broken zone (b).

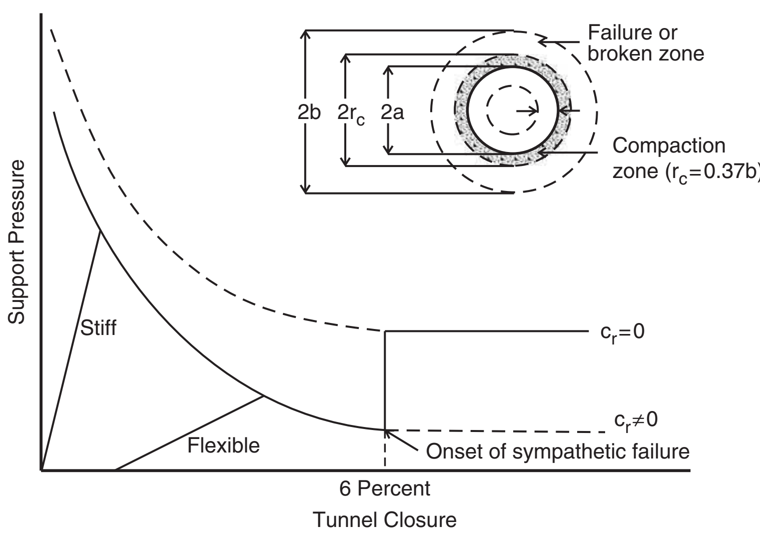

Fig. 19.5 Effect of sympathetic failure of rock mass on theoretical ground response curve of squeezing ground condition. Support reaction curve of stiff and delayed flexible supports are superposed.

Hence, the need for allowing significant displacement to reduce the cost of the support system. This is the secret of success in tunnelling through squeezing ground condition.

In case one chooses to install very stiff support system, it may be seen from Fig. 19.5 that the stiff support system will attract high support pressure as it will restrict the tunnel closure. If a flexible support system is built after some delay, it will attract much less support pressure. This is an ideal choice. However, too late and too flexible support system may attract high support pressure due to excessive loosening of rock mass in the broken zone. Yet the squeezing ground comes to equilibrium after years even in severe squeezing ground condition. Although the final deformations may be undesirable, and so corrective measures are required.

The normalized observed ground response curve is plotted in Figs 5.4a and b. It is same as the curve for correction factor f' for squeezing ground condition. Figs 5.4a and b may be replotted between $0.2(\mathrm{Q_i})^{1/3} \cdot f \cdot (f'/J_r)$ and u_a to find the actual ground response curve for the short-term support pressures in the roof and wall, respectively. Section 6.8 and Fig. 6.3 also show an observed ground response curve using the rock mass number. The data suggests that the support pressure jumps up after tunnel closure of about 5 to 6 percent. Then there is sympathetic failure of entire brittle rock mass within the broken zone, rendering its residual cohesion $c_r = 0$ in the highly squeezing ground. The theoretical ground response curve is plotted in Fig. 19.5 on the basis of this hypothesis. One gets the same trend as observed in Fig. 5.4. The sympathetic failure is in fact unstable and widespread fracture propagation in the entire failure zone, starting from the point of maximum shear strain. This brittle fracture process may be taken into account in the elasto-plastic theory by assuming $c_r = 0$ after critical tunnel closure of 6 percent. *Thus it is recommended that tunnel closure should not be permitted beyond 4 percent of tunnel radius, to be on safer side*.

19.7.1 Validation of theory in field

Singh and Goel (2002) computed tunnel closures and support pressures in 10 tunnels in Himalaya. There is a good cross-check between elasto-plastic theory based on poly-axial failure criterion (equations (19.2) and (8.15)) and observed support pressures both in the roof and walls, except in a few cases. Thus, the assumptions made in the theory are justified partially. Further the predictions are generally conservative. It is recommended that empirical equation (19.10) should be preferred for assessment of support pressures, as the same is closer to observations than that for elasto-plastic theory.

$$p_{\mathrm{ult}} = \frac{0.2}{J_r} \cdot \mathrm{Q_i}^{-1/3} \cdot f \cdot f' \cdot f'' \quad \mathrm{MPa} \qquad (19.10)$$

where

p_{ult} = ultimate roof support pressure in MPa,
$\mathrm{Q_i}$ = 5Q = short-term rock mass quality,

J_r = joint roughness number of Barton et al. (1974),
f = correction factor for overburden = $1 + (H-320)/800 \geq 1$,
f' = correction factor for tunnel closure (Table 5.10) obtained from Fig. 5.4,
= 1 in non-squeezing ground,
f'' = correction factor for the time after excavation = $\log(9.5\ t^{0.25})$,
H = overburden above crown or tunnel depth below ground level in meters and
t = time in months after excavation.

The theoretical support pressures assuming Mohr's theory for elastic zone also were too conservative when compared with the observed support pressures. So the same is not recommended.

19.8 STRAIN CRITERION OF SQUEEZING GROUND CONDITION

The experience proved that squeezing occurred when overburden exceeded $350Q^{1/3}$ m (Singh et al., 1992). One should calculate the corresponding tunnel closure which is as follows:

$$u_a = \frac{(1+\nu) \cdot a \cdot P}{E_d} \tag{19.11}$$

$$\frac{u_a}{a} = \frac{(1+\nu) \cdot \gamma \cdot H}{E_d} = \frac{(1+\nu) \cdot \gamma \cdot H}{Q^{0.36} \cdot H^{0.20}} \times 10^{-5}\ \%$$

Substituting $H = 350Q^{1/3}$, $\gamma = 2.5\ \text{t/m}^3$, $\nu = 0.20$, and Q = 0.1 to 0.01, one gets the following value of strain for squeezing to occur

$$\frac{u_a}{a} = 0.8 \text{ to } 1\% \tag{19.12}$$

On the basis of field observations and instrumentation, Sakurai (1983) concluded that tunnel closure more than 1 percent was followed by the onset of tunnel instability and difficulties in providing adequate support. Field data of Cheru et al. (1998) confirmed the observation of Sakurai (Fig. 19.6). The calculated values agree with this observation. Equation (19.12) proves that the strain criterion for squeezing is nearly independent of the rock mass quality or UCS. Therefore, degree of squeezing has been defined by Hoek (2001) as shown in Fig. 19.7. The uniaxial compressive strength of rock mass q_{cmass} may be estimated from correlations (equation (8.9) or preferably equation (8.5) of Hoek, 2001). The tunnel strain (u_a/a) may be predicted after knowing the ratio q_{cmass}/P. Then, one may have an idea of the degree of squeezing and the associated problems. The tunnel strain is reduced by the support capacity (p_i). Hoek (2001) has plotted theoretical curves and field data to get the tunnel strain (u_a/a) for a given value of q_{cmass}/P and p_i/P (Fig. 19.8).

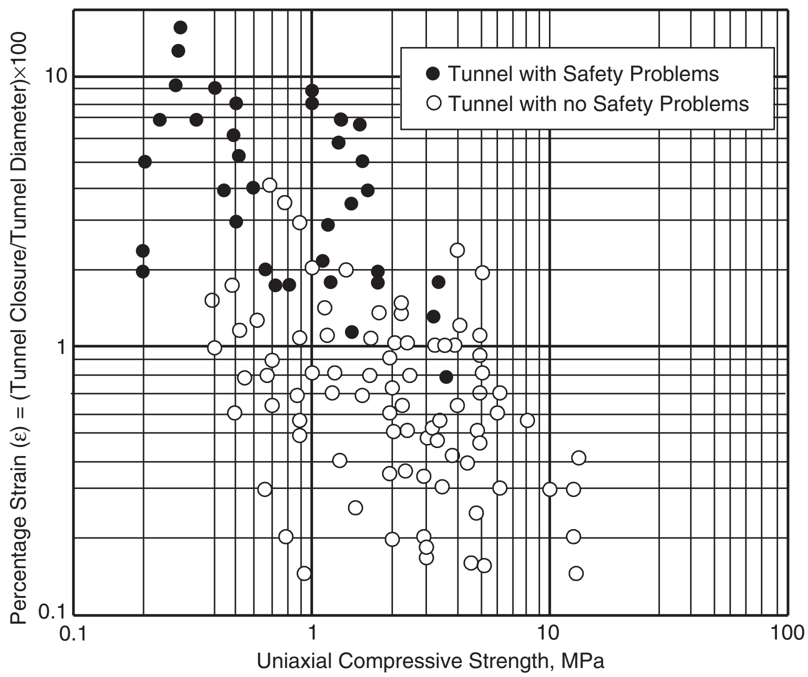

Fig. 19.6 Field observations by Cheru et al. (1998) from second Freeway, Pinglin and New Tienlun Headrace Tunnels in Taiwan.

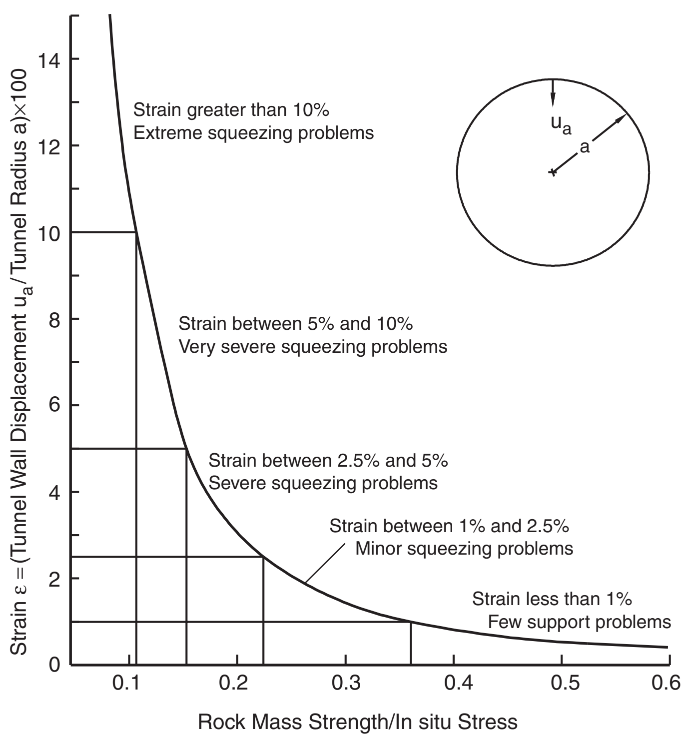

Fig. 19.7 Tunnelling problems associated with different levels of strain (Hoek, 2001).

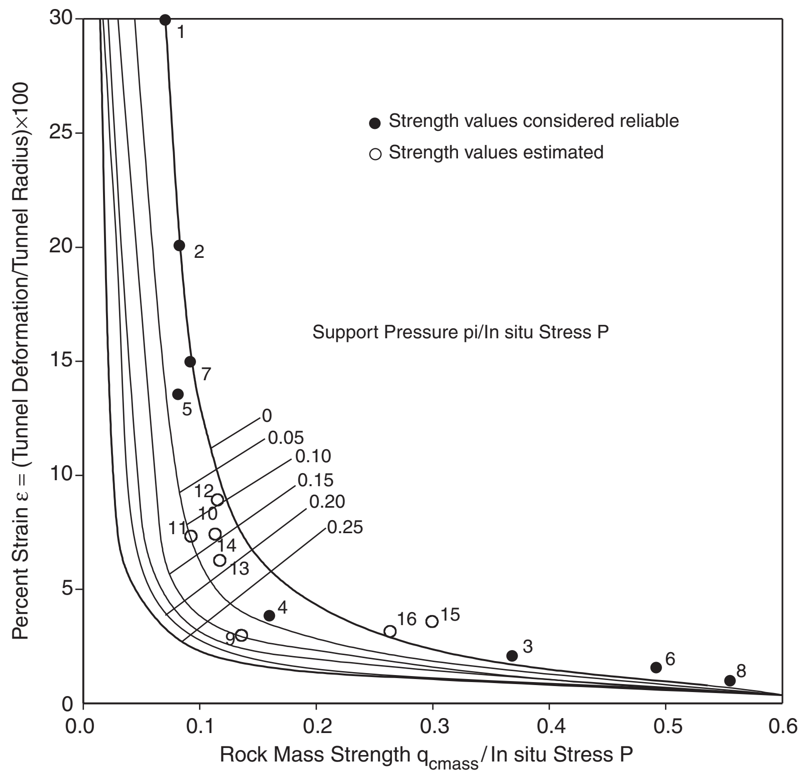

Fig. 19.8 Influence of internal support pressure p_i upon deformation of tunnels in weak ground (Numbered points are from case histories) (Hoek, 2001).

Conversely, the support pressure (p_i) may be assessed from Fig. 19.8 for a pre-planned value of tunnel strain for a given overburden pressure P.

Fig. 19.6 and experiences in Himalaya suggest that tunnels, in minor to severe squeezing ground conditions, have been completed successfully but the construction problems increased with increasing tunnel strain. Tunnelling through very severe squeezing ground condition was naturally most difficult and must be avoided by changing alignment of tunnel to reduce the overburden.

An educative case history of extreme squeezing ground conditions at Tymfristos tunnel (11 m diameter), Greece has been illustrated by Kontogianni et al. (2004). The tunnel closure was 20 percent. The redesigned supports also failed after 6 percent closure. The tunnel cost increased by 10 times. The rock mass is claystone and slickensided argillaceous schist, intensely folded and tectonized ($q_c = 5$–50 MPa). The overburden was only 153 m. It should be realized that re-excavation and installation of the new supports should be done after closure has stabilized. The latter may take several years of monitoring in very severe squeezing ground conditions.

19.9 SUPPORT DESIGN

Fortunately, the steel fiber reinforced shotcrete with embedded ribs has proved to be successful in supporting tunnels in the mild to severe squeezing ground conditions. The Fig. 10.2 may be used for the design of support system. The following detailed strategy has been adopted in squeezing grounds as shown in Fig. 19.9.

(i) Circular or horseshoe shaped tunnel should be planned in the squeezing ground condition. The tunnel width should preferably be less than 6 m in severe or very severe squeezing grounds. The excavated diameter may be 10 percent more than the design diameter.

(ii) The excavation should be by heading and benching method in minor squeezing ground and by multiple drift method in severe or very severe squeezing grounds. Drill 10 m advance probe hole ahead of the tunnel face to know the rock mass quality and drain out ground water if any.

(iii) The horizontal drill holes of 3 m length are drilled ahead of the tunnel face and the forepoles of mild steel rods are inserted and welded to the nearest steel ribs. Then smooth blasting is adopted with short length of blast holes (1 m) to cope up with the low stand-up time.

(iv) A steel fiber reinforced shotcrete (SFRS) layer of 2.5 cm thickness is sprayed immediately to prevent rock loosening. Full-column grouted bolts are installed all around the tunnel including the bottom of tunnel.

(v) Steel ribs with struts at the bottom are erected and designed to support the forepole umbrella and rock support pressure. The struts should be strong enough to resist high wall support pressures in the squeezing grounds.

(vi) The additional layers of SFRS are sprayed after some delay to embed the steel ribs. It will provide lateral stability of ribs and also create a structurally robust lining.

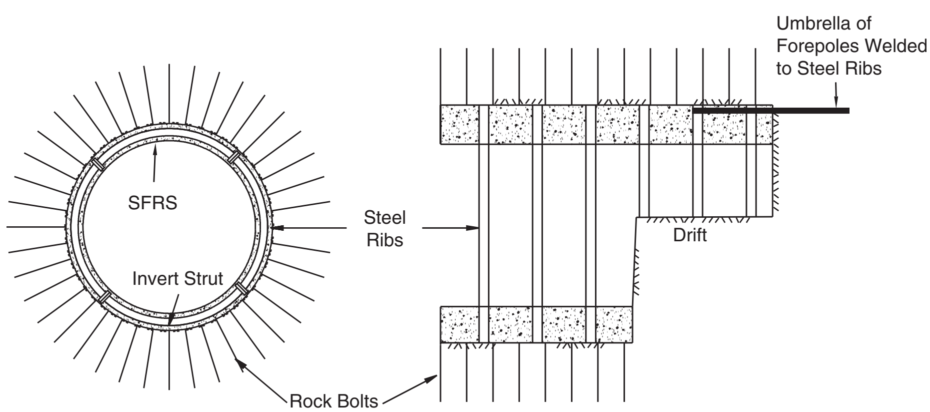

Fig. 19.9 Support system in severe squeezing ground condition.

(vii) The SFRS should also be sprayed on the floor to cover steel struts and counter heaving tendency of the squeezing ground by withstanding high bottom support pressures.

(viii) The convergence of the tunnel roof and walls should be monitored and plotted with time. In case rate of convergence/closure is not dropping with time, additional SFRS layers need to be sprayed. It is a good tunnelling practice if multiple borehole extensometers are installed to know what is happening within the broken zone particularly in severe or very severe squeezing ground conditions.

19.9.1 Precautions in tunnelling

In the cases of big tunnels (10 to 16 m span), the recommendations of Hoek (2001) need to be followed. It is a very challenging task.

It may be mentioned that TBM is obviously a failure in squeezing grounds, as it is struck inside the ground and may have to be abandoned.

In very poor ground, stand-up time is only a few hours. It is difficult to install support system within the stand-up time. So length of blast holes may have to be decreased to 1 m to increase the stand-up time for unsupported span of 1 m. In very poor ground, it is difficult to keep drill holes open for rock bolting. SFRS without rock bolt may work well in such situation. Forepoling is difficult here.

For a very severe squeezing condition, rock anchors (dowels) may be added on the tunnel face where the face is also squeezing, particularly in the big tunnels. This is in addition to the forepole umbrella. A frequent mistake is made in using the large forepoles for protecting the tunnel face. The steel ribs which support the forepoles are loaded adversely, specially in big tunnels. Full face tunnelling method may be a failure due to slow progress of tunnelling. It is good practice to install forepoles first and then make drill holes for blasting.

It may be realized that there is no time to use lengthy software packages and for academic advice at the tunnel face. Spot decisions have to be made on the basis of past experiences. *It is, therefore, justified that a tunnel engineer who understands the tunnel mechanics and has experience should be made sole in charge of supporting the ground and related works.*

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20

Case history of tunnel in squeezing ground*

"The first sound and the first sign of instability is noted initially by the foreman and the workers at the tunnel face, much before the big thud of collapse is felt in the designer's office."

Source: THDC, India

20.1 INTRODUCTION

This is a case history of tackling serious tunnelling problems in squeezing ground within the intra-thrust zone in lower Himalaya.

Stage II of the Yamuna hydroelectric scheme in the lower Himalayan region aims at complete utilization of the power potential of the river Tons between Ichari and Khodri (Fig. 20.1). A diversion dam at Ichari, and a 6.25 km long pressure tunnel of 7.0 m diameter from Ichari to Chhibro with an underground powerhouse of 240 MW capacity at Chhibro to utilize a drop of 120 m, are the major components of part I of the scheme. In part II, a 5.6 km long tunnel of 7.5 m diameter has been constructed between Chhibro and Khodri to utilize the discharge from the Chhibro powerhouse. A surface powerhouse of 120 MW capacity is built at Khodri to utilize a drop of 64 m.

Tunnel construction in part II was started from both the Chhibro and the Khodri ends. Near Kalawar, a village midway between these two places, a small incline (2×2.5 m), called the Kalawar Inspection Gallery, was driven up to the tunnel level to observe the behavior of rock masses in the fault zone (Fig. 20.1). Subsequently, this gallery was used to construct the main tunnel through this zone by opening two additional headings.

20.2 REGIONAL GEOLOGY, TUNNELLING PROBLEMS AND ALTERNATIVE LAYOUTS

The regional geology of the area was mapped by Auden (1934, 1942) followed by Mehta (1962) and Krishnaswami (1967). Additional information was presented by Shome et al.

*This chapter is reproduced from the paper by Jethwa et al. (1980).

Tunnelling in Weak Rocks
B. Singh and R. K. Goel

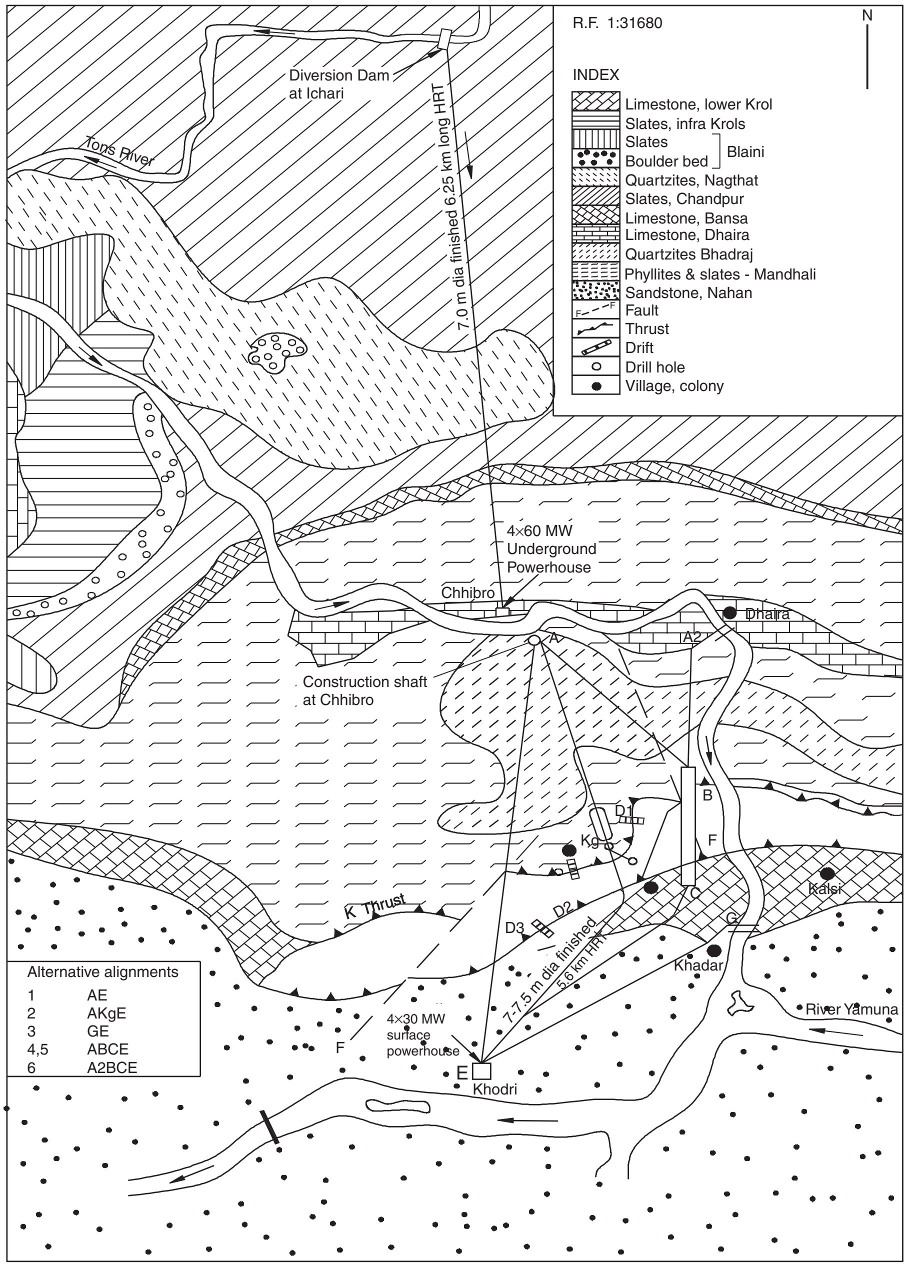


Fig. 20.1 Regional geology and alternative layout of the Yamuna hydroelectric scheme, stage II, part II.

(1973) based on their observations in a few drifts, drill holes and trenches near the villages of Kalawar and Kala-Amb and some surface features in the region (Fig. 20.1).

20.2.1 Tectonic sequence

The following tectonic sequence from north to south was postulated by Auden (1934) between Ichari and Khodri.

	Simla slates
	Nummulitics
	Tons thrust
	Nagthat quartzites
Thrust Bound	Chandpur series
Jaunsar Syncline	Mandhali series
	Krol thrust
	Nummulitics
	Nahan thrust
	Nahan series

20.2.2 Lithology

The Chhibro–Khodri tunnel passes through the following three formations from north to south (Shome et al., 1973):

Mandhali series (Palaeozoic)	Boulder slates; Graphitic and quartzitic slates; Bhadraj quartzite unit of width 5–10 m Crushed quartzites near the Krol thrust.
- - - - - - - - Krol thrust - - - - - - - -	
Subathu–Dagshai series (Lower miocene)	1–3 m thick plastic black clays along the thrust, red and purple shales and siltstones; Minor grey and green quartzites, 22 m thick black clays with thin bands of quartzites; 5–10 m thick plastic black clays along the Nahan thrust.
- - - - - - - - Nahan thrust - - - - - - - -	
Nahan series (Upper tertiary)	Greenish-grey to grey micaceous sandstone; Purple siltstone; Red, purple, grey and occasional mottled blue concretionary clays.

The regional strike of these formations is almost normal to the tunnel alignment with the dips ranging from 20° to 60° in NNW to NNE direction, i.e., towards the upstream.

20.2.3 Structural features

The major structural features in this area are the two main boundary faults running from Punjab to Assam along the foothills of the Himalaya. The faults are observed across the river Tons near Khadar and at a few gully exposures near Kala-Amb and Kalawar. These were further explored with the help of a few drill holes, drifts and trenches (Fig. 20.1). The dips of the Nahan and the Krol thrusts vary from 27° to 30° due N10°E to N10°W, and 26° due N26°W, respectively. The strike is almost normal to the tunnel alignment.

20.2.4 Anticipated tunnelling problems and alternate layouts

Krishnaswami (1967) anticipated squeezing problems in the intra-thrust zone and indicated that locked-up water was likely to be present in large quantities in the crushed Mandhali quartzites. Subsequently, Krishnaswami and Jalote (1968) attempted either to avoid the intra-thrust zone or to reduce the tunnel length through it, and they proposed several layouts, as alternatives to a straight tunnel. These are shown with costs (of 1968) in Table 20.1 and Fig. 20.1.

Table 20.1 Alternate layouts proposed for the Yamuna hydroelectric scheme, stage II, part II (Fig. 20.1).

No.	Details of layout	Increase in cost related to layout one in 1968 (million rupees)
1.	5.5 km long and 7–7.5 m diameter straight tunnel AE, width of intra-thrust zone = 800 m	Original
2.	5.6 km long and 7–7.5 m diameter tunnel AK_g, E with a kink at K_g, near Kalawar village, width of intra-thrust zone = 230 m	nominal
3.	A 51 m high earth and rockfill dam near Kalsi, a 3.0 km long and 7.0 m diameter tunnel GE, intra-thrust zone eliminated	33.8
4.	A 2.4 km long and 7.0 m diameter tunnel AB, a 30 m high and 1.6 km long reservoir BC at Kalawar and a 2.8 km long and 7.0 m diameter tunnel CE, open reservoir across intra-thrust zone	68.4
5.	Replacing of open reservoir at Kalawar in layout No.4 by a 1.45 km long open channel BC, open channel across intra-thrust zone	22.3
6.	A 50 m high concrete dam at Dhaira, a 1.2 km long and 8.0 m diameter tunnel A2B, a 1.2 km long open channel BC at Kalawar and a 2.8 km long and 7.0 m diameter tunnel CE, open channel across intra-thrust zone	21.1

Layout No.2, with a kink at Kalawar, was accepted on account of cost considerations. Although the total length of the tunnel along this layout was increased by 0.4 km as compared to the straight tunnel, the width of the intra-thrust zone was reduced from 800 to 230 m. Fig. 20.2a shows the original geological cross section along this alignment (Auden, 1942).

20.2.5 Recurrence of intra-thrust zones

In addition to their presence at Kalawar, the Subathu–Dagshai red shales were again intercepted in the tunnel between 1140 and 1300 m from the Chhibro end.

A hole drilled at 1180 m (from Chhibro) in the tunnel roof at an inclination of 60° due E established the presence of the Krol thrust over the tunnel. Finally, Jain et al. (1975) presented an ingenious interpretation of the existing geological data and predicted the existence of a series of tear faults (Fig. 20.2b) between Chhibro and Kalawar with a third intra-thrust zone between 1861 and 2166 m (from Chhibro). Thus, the total width of the intra-thrust zones was found to be 695 m against an estimated width of 230 m along the tunnel alignment. Hence, there is a need for subsurface geological and proper rock mass classification.

Considerable tunnelling difficulties were encountered within the intra-thrust zones. The multi-drift method was adopted to prevent frequent rock falls at the face. A central pilot had to be excavated by forepoling. Heavy steel arches (300 × 140 mm and 150 × 150 mm sections with 20–25 mm thick plates welded on flanges) were erected at 0.25–0.50 m spacing, (see Table 20.4 and Fig. 20.11) to cope with high squeezing pressures.

20.2.6 Branching of the main tunnel into three small tunnels

The project was delayed by over six years due to the very slow progress of tunnelling (5–6 m per month) through the intra-thrust zones. At this rate, it would have taken five and a half years to excavate the remaining 695 m (between P and Q, Fig. 20.2b) from the two ends. At this stage, the project authorities considered it wiser to replace the main tunnel by three smaller tunnels (5.0 m excavated diameter). Consequently, driving of the central tunnel was started at the end of 1976 and was completed by the middle of 1979. Assuming that the remaining two small tunnels would be excavated simultaneously during the same length of time as the central tunnel, the saving in time would be barely six months. Thus, branching of the main tunnel into three small tunnels is not proving to be a wise decision. However, simultaneous excavation of the three tunnels could have been quicker.

20.2.7 Flooding of the tunnel at Kalawar

In November 1972, the perched water of the rock mass suddenly punctured the impervious layer of argillaceous clays along the Krol thrust and rushed in from the tunnel roof at 182 m towards Chhibro from K_g (the point of inter-section of the Kalawar inspection gallery and

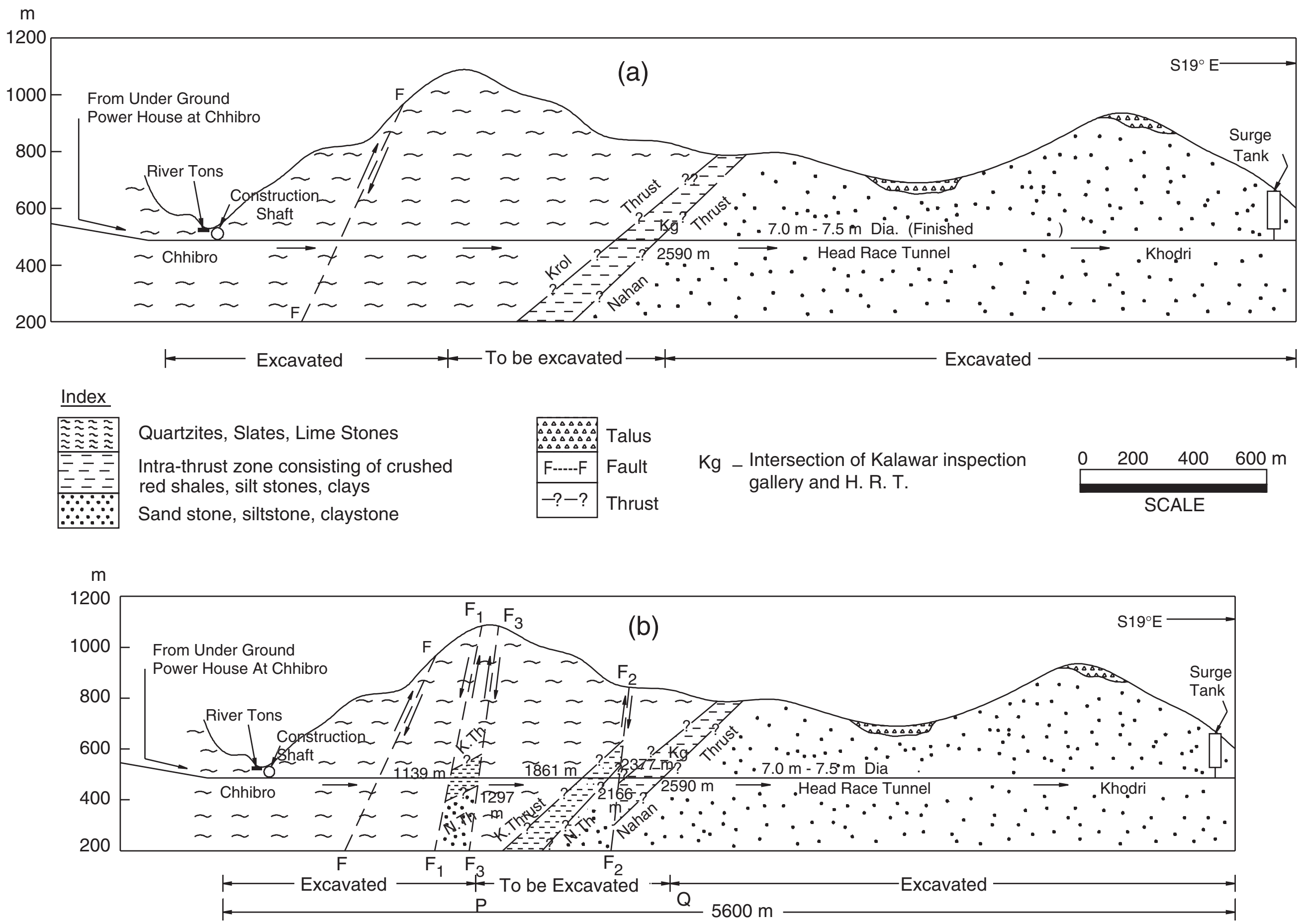


Fig. 20.2 (a) Original geological section along the Chhibro–Khodri tunnel (Auden, 1942); (b) Revised geological section along the Chhibro–Khodri tunnel (Jain et al., 1975).

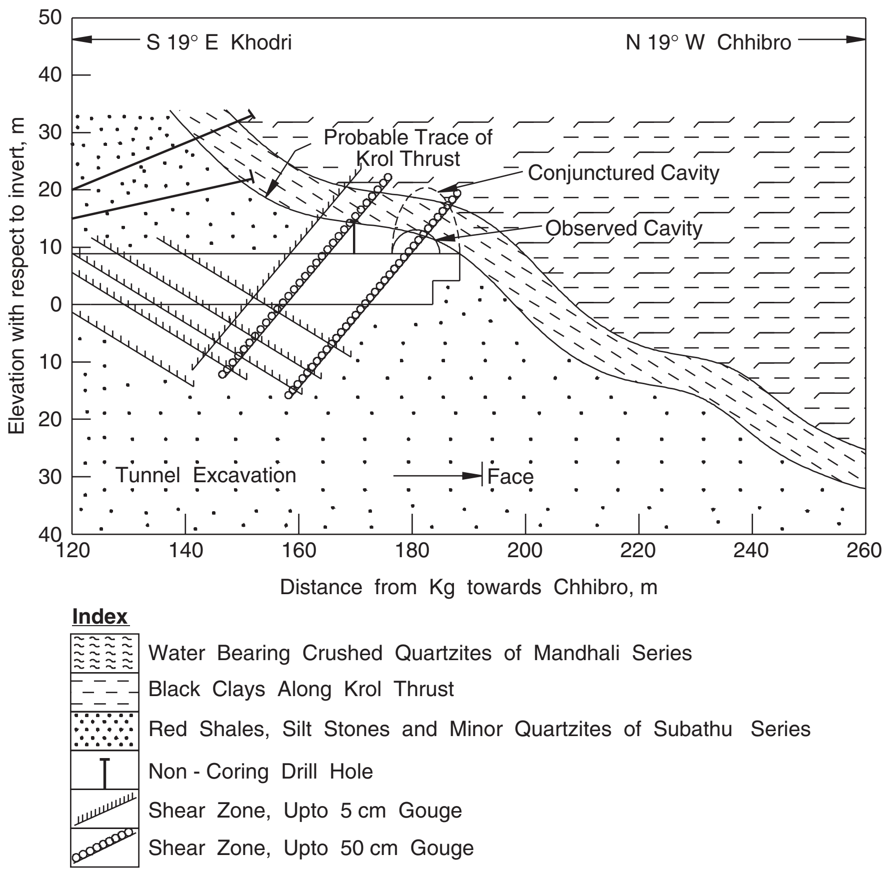


Fig. 20.3 Geological features causing flooding of tunnel at Kalawar (Shome et al., 1973).

the main tunnel) and flooded the whole tunnel at Kalawar. Fig. 20.3 illustrates the detailed geology around K_g (Shome et al., 1973). The rate of inflow was estimated to be 1.2 cusecs (34 liters/s), and 110,000 m^3 of water were pumped out in three months.

20.2.8 Properties of rock masses

The properties of the Subathu red shales and black clays are given in Tables 20.2a and b. The samples were collected from the Kalawar inspection gallery and from gully exposures near Kala-Amb.

Table 20.2a Properties of red shales and black clays.

	Red shales		Black clays	
Particulars	Kalawar	Kala-Amb	Kalawar	Kala-Amb
General properties:				
Unit weight (g/cm^3)	2.73	–	2.64	–
Natural dry density (g/cm^3)	–	1.43 − 1.68	–	2.1
Density at zero moisture content (g/cm^3)	1.86	–	1.88	–
Natural moisture content (% by wt)	8.02	9.5	11.7	18.95
Optimum moisture content (% by wt)	7.32	–	16.25	–
Grain-size distribution (%):				
Gravel (above 2.0 mm)	58.40		10.50	
Very coarse sand (1.0–2.0 mm)	2.50		3.50	
Coarse sand (0.5–1.0 mm)	2.00	78–84	2.00	38–50
Medium sand (0.25–0.5 mm)	1.50		1.50	
Fine sand (0.1–0.25 mm)	1.50		3.50	
Very fine sand (0.05–0.1 mm)	0.46		21.20	
Silt (0.005–0.05 mm)	12.00	9–14	26.05	23–29
Clay (below 0.005 mm)	21.64	7	31.75	27–33
Chemical analysis (%):				
Silicon oxide	73.54	56.42	72.55	53.00
Iron oxide	6.93	1.40	7.32	3.14
Aluminum oxide	12.93	29.00	15.52	30.98
Calcium oxide	1.40	1.00	2.1	1.10
Magnesium oxide	–	2.00	–	1.30
Sulphite	–	2.30	–	3.65
Atterburg's limits:				
Liquid limit (%)	12.0–16.0	21.0–65.0	16.0–44.3	36.0–52.5
Plastic limit (%)	13.0–22.7	15.0–16.8	12.0–22.7	19.8–21.0
Plasticity index	3.0	4.3–40.0	4.0–26.6	–
Strength parameters:				
Unconfined compressive strength (kg/cm^2)	21.0	2.54	0.33	1.35
Cohesion (kg/cm^2)				
(1) at optimum moisture content	1.0	0.29	0.20–0.42	0.18
(2) at saturation	–	–	0.35–0.61	–
Angle of internal friction (degree)				
(1) at optimum moisture content	8.54	3.0	12.0	18.0
(2) at saturation	–	28.0	11.5	–
Free swell (%)	–	60	–	55
Mineralogical composition of clay content:				
Illite	–	35	–	35
Kaolinite	–	25	–	15
Chloride	–	10	–	10

Continued

Table 20.2a—Continued

Attapulgite	–	15	–	15
Montmorillonite	–	0	–	5
Mica	–	5	–	5

Table 20.2b Modulus of deformation of red shales and Black clays.

			Modulus of deformation (10^3 kg/cm^2)	
Location	Rock type	Method	Horizontal	Vertical
Kalawar inspection gallery	Black clays	Plate-bearing test	2.7	12.18
		Flat jack test	–	6.10
3.0 m diam. pilot tunnel	Black clays	Plate-bearing test	1.405	–
3.0 m diam. pilot tunnel	Red shales	Tiwag radial	3.7	(minimum)
		Press test	35.2	(maximum)
			19.5	(average)

20.3 TECTONIC ACTIVITY AND TUNNEL LINING

20.3.1 Seismic history

The project area is located in the Garhwal Lower Himalaya, adjacent to the main boundary faults. The region is traversed by a number of secondary faults and thrusts and is known for intense tectonic activity. The age of these activities is not known, but Auden (1934) considered that the Krol thrust might have resulted from seismic activity spread over a long period of time, from pre-pliocene to post-pliocene. In recent times, the only major earthquake reported near the area was the one in 1905 with its epicenter between Kangra and Dharamshala, while a minor region of intensity greater than isoseismal seven occurred in Doon Valley. As a result of this earthquake, the town of Dehradun was lifted up by 0.13 m relative to Mussoorie. Other indications of recent tectonic activity are huge boulders of quartzites (overall size 5 m) lying in the valley near the drift at Kala-Amb and elongated spindle-shaped “boudins” of quartzite found embedded in the brecciated, pulverized and gouged material along the Nahan thrust.

20.3.2 Measurement of tectonic movement

Agrawal and Gaur (1971) fixed a pillar on the Nahan sandstone and another pillar on the Subathu clays across the Nahan thrust in the cross-cut from the Kalawar inspection gallery (Fig. 20.4). They measured the relative vertical displacement between the two pillars with the help of a water-tube tiltmeter. At the end of three years, they reported that the rate of the vertical component of the relative displacement across the Nahan thrust

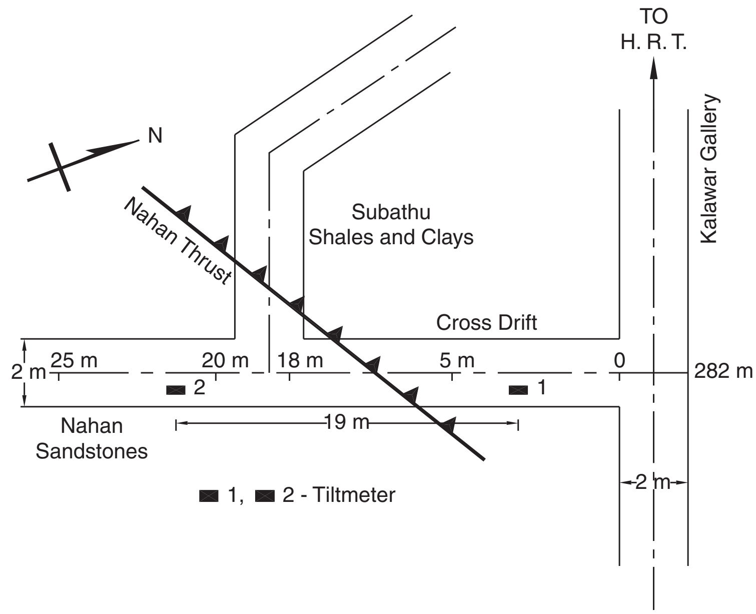


Fig. 20.4 Plan of tiltmeter bases for measuring tectonic movement along the Nahan thrust (Agrawal & Gaur, 1971).

varied from 0.4 to 1.0 mm per month. However, they conceded that a substantial portion of this movement might be attributed to the squeezing of the clays and concluded that the rate of the vertical component of the tectonic slip across the Nahan thrust was 0.5 mm per month. Subsequently, Jethwa and Singh (1973) reported that the rate of radial closure in the clays, as measured at the end of two years of excavation, was 1 mm per month in the vertical direction.

A single-point rod-type borehole extensometer was installed across the Nahan thrust in the Kalawar inspection gallery to measure the relative movement between the Nahan sandstone and the Subathu red shales (Fig. 20.5). Observations, spread over six months, did not show any movement across the Nahan thrust.

A conclusion which follows from the above measurements is that squeezing of the clays should not have been ignored while assessing the fault slip.

20.3.3 Flexible tunnel lining

Based on the work of Agrawal and Gaur (1971), Jai Krishna et al. (1974) suggested that the tunnel lining for the intra-thrust zone should be designed to withstand a total vertical dislocation of 0.5 m expected during the life of the project (100 years). Further, they considered that the total slip would be distributed uniformly along the width of the intra-thrust zone. Based on the above assumptions, they proposed a “flexible lining” to cope with the tectonic slip (Fig. 20.6). It consisted of circular segments of varying lengths

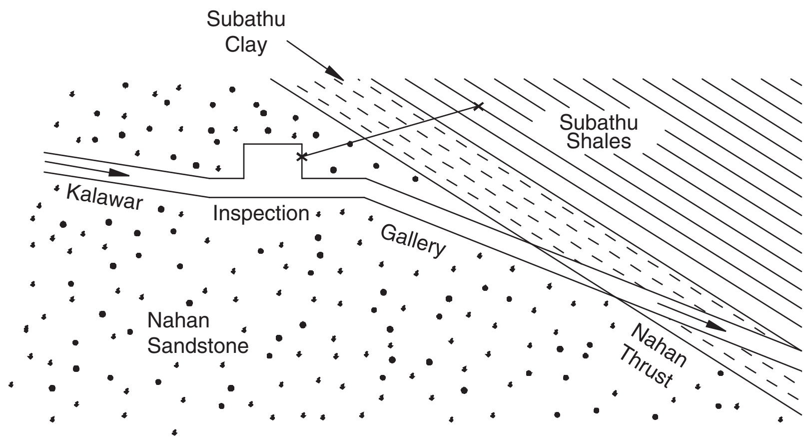


Fig. 20.5 Schematic arrangement for measuring displacement across the Nahan thrust by single-point borehole extensometer.

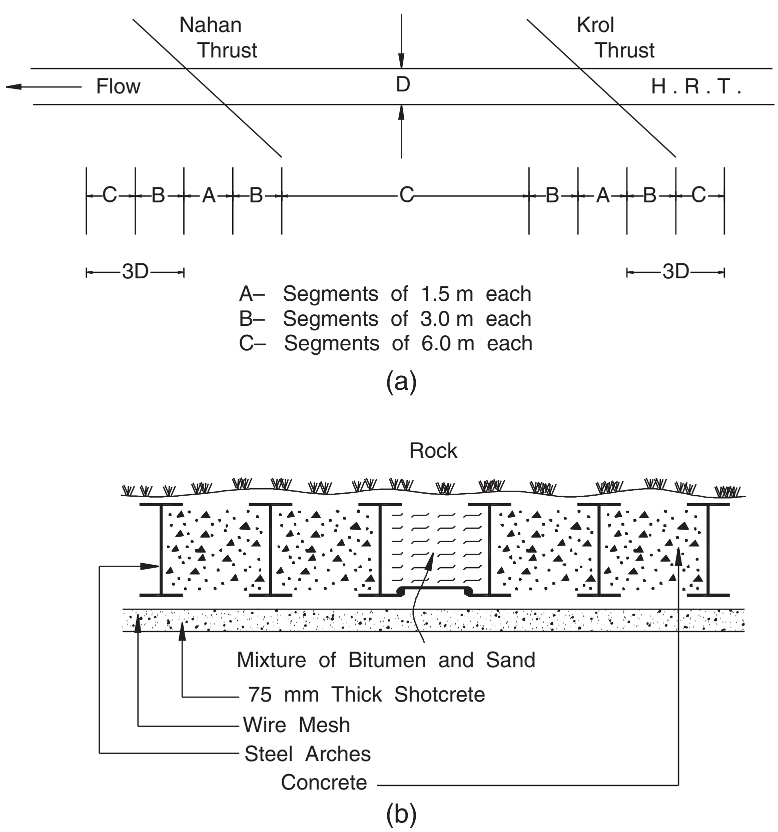


Fig. 20.6 Flexible tunnel lining in intra-thrust zone: (a) segmental lining; (b) flexible joint.

connected together by flexible joints. Contrary to the above assumption, tectonic slip in thick fault gouge may take place along any one plane as suggested by Brace and Byerlee (1967) who explained the mechanism of earthquakes by the "stick-slip" phenomenon.

It cannot be proved conclusively from the above that the faults are active. Even if this is so, it may be questionable to provide a tunnel lining on the assumption that the tectonic slip would be uniformly distributed along the entire width of the intra-thrust zone.

20.4 TUNNEL CONSTRUCTION AND INSTRUMENTATION IN THE INTRA-THRUST ZONE AT KALAWAR

20.4.1 Support behavior in Kalawar inspection gallery

Steel ribs for the Kalawar inspection gallery, under a maximum cover of 280 m were first designed for Terzaghi's (1946) rock load factor of 1.1 $(B + H_t)$ where B is width and H_t is height of the opening (Table 20.3). It corresponds to squeezing rocks at moderate depths. The water table was observed to be below the tunnel invert but was considered to be above the tunnel crown for the purpose of design. In order to arrest rib deformations, the rock load factor was gradually increased to 3.5 $(B + H_t)$, which is equivalent to squeezing rocks at "great depths."

20.4.2 Tunnel construction

A pilot tunnel of 3.0 m diameter was driven on both sides from K_g. In the Subathu red shales, this diameter was enlarged to 9.0 m towards Chhibro from a point 36 m away from K_g. The tunnel was excavated by the multi-drift method. The heading was supported by semi-circular steel arches with temporary invert struts to withstand side pressures (Fig. 20.7). The Nahan thrust was exposed in the pilot tunnel at a distance of 40 m from K_g towards Khodri, whereas the Krol thrust was exposed at a distance of 190 m from K_g towards Chhibro. The gouge in the 230 m wide intra-thrust zone consisted of soft and plastic black clays over lengths of 16 m and 2 m along the Nahan and the Krol thrusts, respectively, and of crushed, sheared and brecciated red shales and siltstones over a length of 212 m between the layers of the black clays.

20.4.3 Instrumentation

The necessity for tunnel instrumentation was felt in order to evolve a rational tunnel support system which could cope with the squeezing ground conditions encountered in the intra-thrust zone. The instrumentation program consisted of measuring: (i) hoop load in the steel arches by hydraulic load cells (ii) contact pressure at the rock-support interface by contact pressure cells (iii) "tunnel closure," defined as reduction in the size of the opening, by an ordinary steel tape to an accuracy of ± 1 mm and (iv) "borehole-extension" (defined

Table 20.3 Support details in the Kalawar inspection gallery.

Reach (m)	Rock type	Assumed rock load factor (Terzaghi, 1946)		Equivalent support pressure (kg/cm^2)		Support details		Spacing (mm)	Deformational behavior of supports (visual)
		Vertical (H_p)	Horizontal	Horizontal	Vertical	Shape and size	Cross section (mm)		
160 to 273	Black clays	1.1 $(B + H_t)$	0.3 $(H_p + H_t)$	1.30	4.48	D-shaped ribs with inverts, $H_t = 2.5$ m $B = 2.0$ m	100 × 75	500	Intolerable rib deformations, buckling of invert and bulging of vertical legs into the opening
273 to 295	Black clays	2.1 $(B + H_t)$	0.3 $(H_p + H_t)$	2.57	0.83	As above	As above	250	As above
295 to 378	Black clays	As above	As above	3.44	1.28	Circular ribs $H_t = B = 3.0$ m	As above	250	Moderate rib deformation
378 to 440	Red shades	3.5 $(B + H_t)$	3.5 $(B + H_t)$	5.73	5.73	As above	150 × 150	400 to 600	Negligible rib deformation

Notations: B = width of opening; H_t = height of opening.

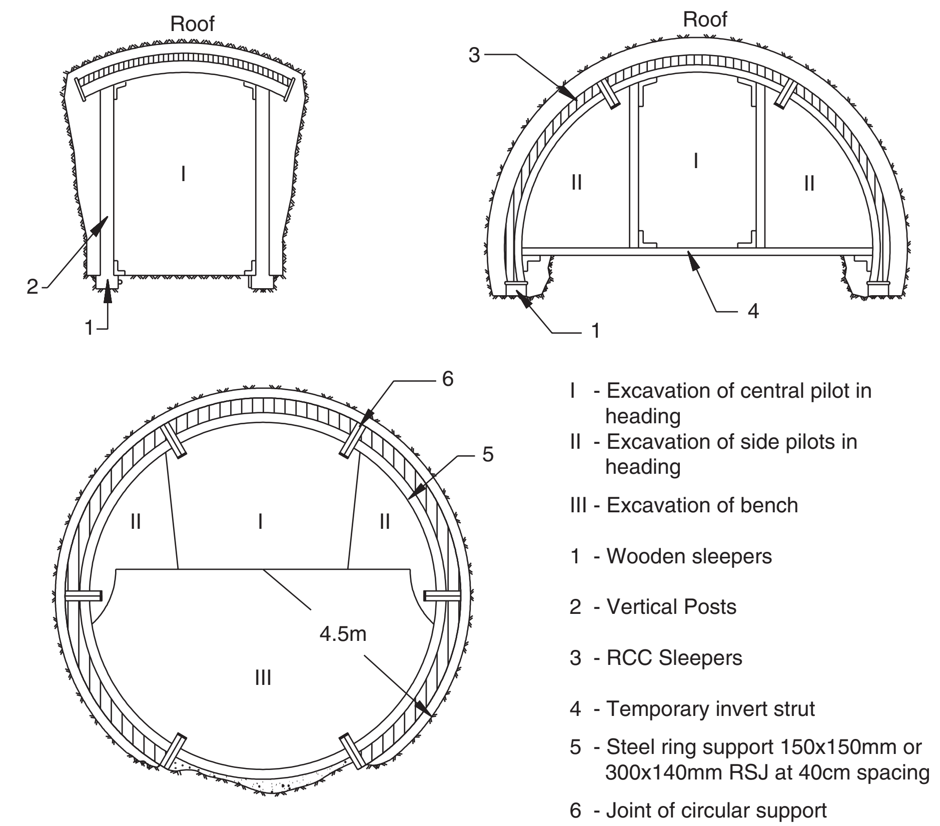


Fig. 20.7 Sequence of excavation and support for the main tunnel through the intra-thrust zone at Kalawar.

as the relative movement between the tunnel periphery and the interior of the rock mass) by single-point, rod-type borehole extensometers (depth equal to the diameter of opening) to an accuracy of ± 0.02 mm.

These instruments were designed and developed at the Central Mining Research Institute, Dhanbad (India). Test sections were established with "loose backfill" and "tight backfill" in both the red shales and the black clays. The loose backfill consisted of a 30 cm thick layer of tunnel muck thrown manually in the hollows around steel arches. The tight backfill consisted of systematically packed PCC (precast cement concrete) blocks.

20.4.4 Test sections

The instruments were installed at the tunnel face soon after excavation. Support density, type of backfill and the method of tunnelling were kept unchanged on either side of the test sections over a length equal to the tunnel diameter. Table 20.4 describes the

Table 20.4 Location of test sections, support details, type of backfill and results of instrumentation in intra-thrust zone at Kalawar.

No. of test section	Distance from Chhibro (m)	Size and shape of opening	Rock type	Method mining	Support details: Size (mm)	Support details: Spacing (mm)	Support details: Capacity (kg/cm^2)	Backfill	Period of obersvation days	Results of instrumentation (maximum observed value): Pressure (kg/cm^2) P_v	Pressure (kg/cm^2) P_H	Closure (cm) U_{rv}	Closure (cm) U_{rH}	*Borehole extension (cm) U_{by}	*Borehole extension (cm) U_{bR}	*Borehole extension (cm) U_{bL}
1.	2575	3.0 m ϕ pilot tunnel	Red shales	Full face mild blast	150×150	500	6	Tight	155	3.07	1.72	4.65	2.25	3.058	1.706	2.052
2.	2535	9.0 m ϕ main tunnel	As above	Heading and bench, mild blast	150×150 with 16 mm plate on outer flange	415	10	Loose	828	–	–	–	–	0.402	1.114	2.206
3.	2530	As above	As above	As above	As above	As above	As above	As above	824	0.8	0.4	–	–	3.772	1.250	0.332
4.	2621	3.0 m ϕ pilot tunnel	Black clays	Full face mild blast	As above	250	12	As above	758	3.20	2.70	13.40	14.30	–	–	–
5.	2631	9.0 m ϕ main tunnel	As above	Heading and bench, mild blast	300 × 140 with 20 mm plates on both flanges	275	20	Tight	719	11.50	12.20	–	–	5.512	1.620	4.408

Notations: ϕ = Diameter; P_v = Support pressure at roof; P_H = Support pressure at sides; U_{rv} = Radial tunnel closure in vertical direction; U_{rH} = Radial tunnel closure in horizontal direction; U_{by} = Borehole extension at roof; U_{bR} = Borehole extension at right wall; U_{bL} = Borehole extension at left wall; [* Borehole extension is defined as relative displacement between two points – one located on the tunnel periphery and the other located at a depth equal to tunnel diameter].

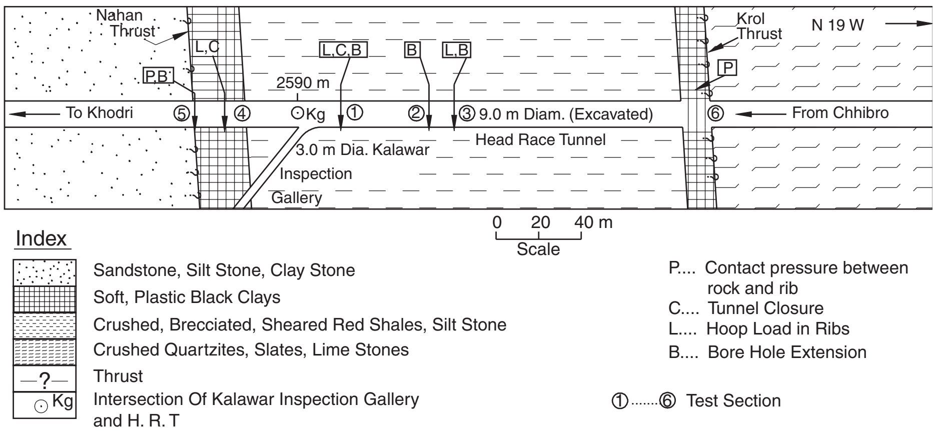


Fig. 20.8 Locations of test sections and details of instrumentation in the intra-thrust zone at Kalawar.

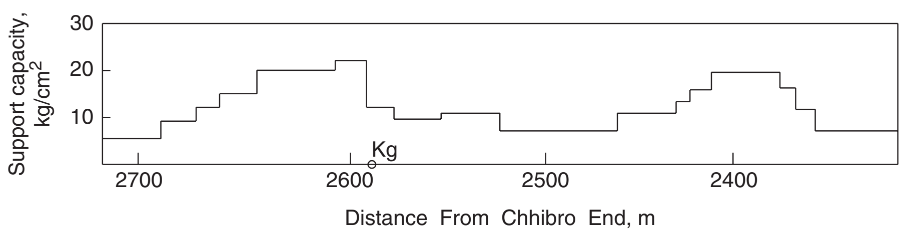


Fig. 20.9 Support density in the intra-thrust zone at Kalawar.

locations of test sections, the size and shape of the opening, details of steel arches, type of backfill and results of instrumentation. Test section 5 was set up in the black clays (near the Nahan thrust) when the 3.0 m diameter pilot tunnel was widened to 9.0 m diameter. Fig. 20.8 shows the locations of test sections and Fig. 20.9 shows the density of supports provided in this zone. Typical observations of support pressure and radial tunnel closure, and borehole extension are shown in Fig. 20.10.

20.4.5 Design of supports

Tight backfill was used to minimize the loosening of the rock mass above the tunnel crown in order to minimize the risk of flooding (although the loose backfill relieved the rock load). Hence higher support pressures were assumed; for example, 6.0 kg/cm^2 (0.6 MPa) in the red shales and 20–22 kg/cm^2 (2.0 to 2.2 MPa) in the black clays against observed support pressure of 3.07 and 12.2 kg/cm^2 (0.3 to 1.22 MPa), respectively (Fig. 20.10, a and f). The support pressure was increased gradually from 6.0 kg/cm^2 (0.6 MPa) in the middle portion of the intra-thrust zone to 22 kg/cm^2 (2.2 MPa) in the black clays along the

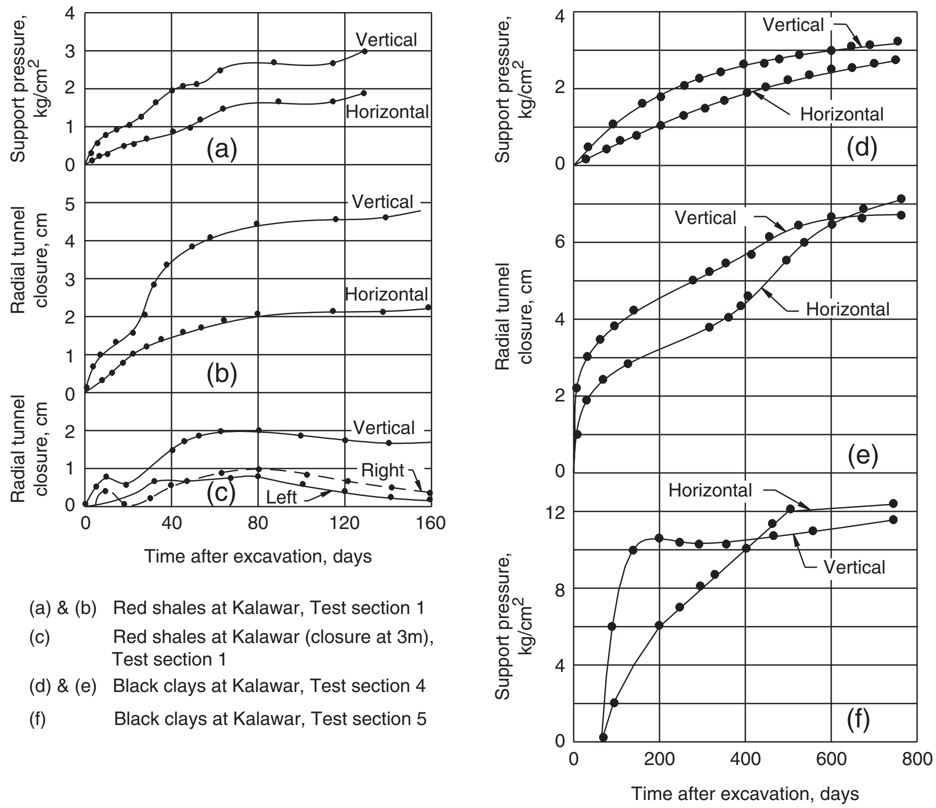


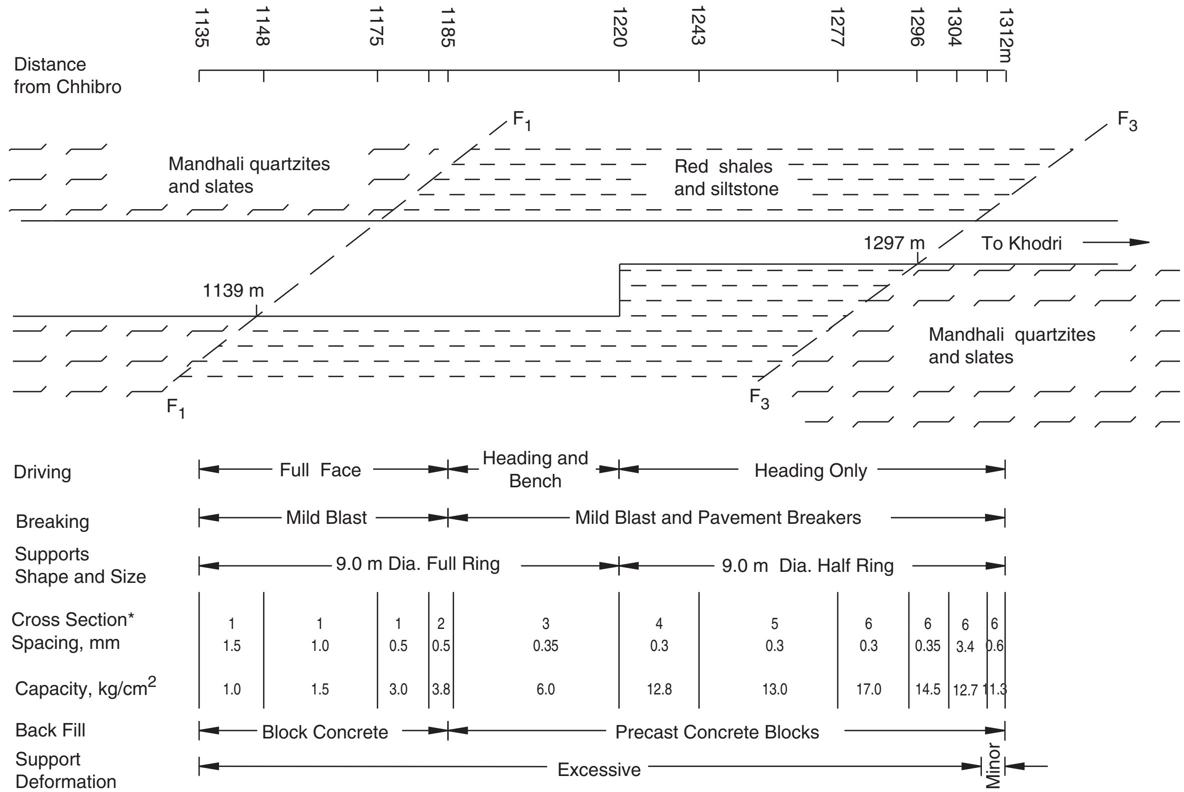
Fig. 20.10 Monitoring of support pressure and radial tunnel closure in red shales and black clays in different instrumentation test sections at Kalawar.

thrusts (Fig. 20.9). The support density was reduced gradually to 6.0 kg/cm^2 (0.6 MPa) on either side of the intra-thrust zone. Subsequent embedment of these supports in concrete has not shown any sign of distress.

20.5 TUNNEL CONSTRUCTION AND INSTRUMENTATION IN INTRA-THRUST ZONE AT CHHIBRO

20.5.1 Tunnel construction

Local geology and construction details of the tunnel through this zone are shown in Fig. 20.11. In the beginning, the unexpected exposure of the red shales at 1139 m while tunnelling from Chhibro was considered to be a local occurrence and the support density was kept unchanged. With continuation of the red shales, it was realized that a second intra-thrust zone had been intersected. The support pressure beyond 1185 m was



*1 - 150 mm x 150 mm
2 - 250 mm x 125 mm
3 - 300 mm x 140 mm
4 - 300 mm x 140 mm with 250 mm x 20 mm Plate welded on outer flange
5 - 150 mm x 150 mm with 250 mm x 25 mm plate welded on outer flange
6 - 150 mm x 150 mm with 250 mm x 25 mm plate welded on outer flange and 125 mm x 25 mm plate welded on inner flange

Fig. 20.11 Geological plan and construction details of head race tunnel through intra-thrust zone at Chhibro.

increased to 6.0 kg/cm^2 (0.6 MPa) (as at Kalawar). Full-face excavation was replaced by the heading and bench method. The heading was excavated by the multi-drift method to prevent frequent loose falls at the face. Since invert struts were not provided, the legs of the semi-circular heading supports were free to move in the opening under the influence of side pressure. Severe buckling and bending of supports followed despite raising their capacity to 17.0 kg/cm^2 (1.7 MPa). It may be noted that relatively lighter supports (6 kg/cm^2 capacity) had undergone only minor deformations in the similar rock mass at Kalawar where temporary invert struts were provided in the heading (Fig. 20.7).

20.5.2 Instrumentation

It became necessary to instrument the tunnel in this zone in view of intolerable deformations of heavy steel-arch sections found adequate in the similar rock mass at Kalawar. The instrumentation was aimed at measuring: (i) support pressure by contact pressure cells and (ii) tunnel closure by an ordinary steel tape.

Support pressure Four contact pressure cells, two at 30° and two at 60° from the vertical were installed over a 9.0 m diameter semi-circular steel rib at 1199 m in the heading (Fig. 20.12). The bench was excavated and the lower half of the rib was erected 115 days after the excavation of the heading. The backfill consisted of tightly packed PCC blocks. The observed support pressure varied from 6.5 to 13.0 kg/cm^2(0.65 to 1.3 MPa), giving an average support pressure of 10.75 kg/cm^2 (1.075 MPa) in the vertical direction. The loss of support resistance due to bench excavation reduced the observed support pressures temporarily.

Tunnel closure Tunnel closures in horizontal and vertical directions were measured at twenty-three locations in this zone. A typical plot of radial closure vs. time (Fig. 20.13) indicates that the sides of the semi-circular opening squeezed in by over 20 cm whereas the roof was pushed up by about 8 cm. A similar deformation pattern was observed in this zone from 1220 to 1295 m (where the bench excavation was delayed by a year and heading supports lacked invert struts). More details are presented in Section 14.8.

20.6 ELASTO-PLASTIC THEORY

Instrumentation of the tunnel under squeezing ground conditions encountered in the two intra-thrust zones has helped to answer the following questions:

(i) What is the effect of tunnel depth on support pressure?
(ii) Which method is suitable to assess the support pressure?
(iii) What is the performance of the flexible support system consisting of loose backfill behind steel arches?
(iv) How to ensure stability of a tunnel opening?

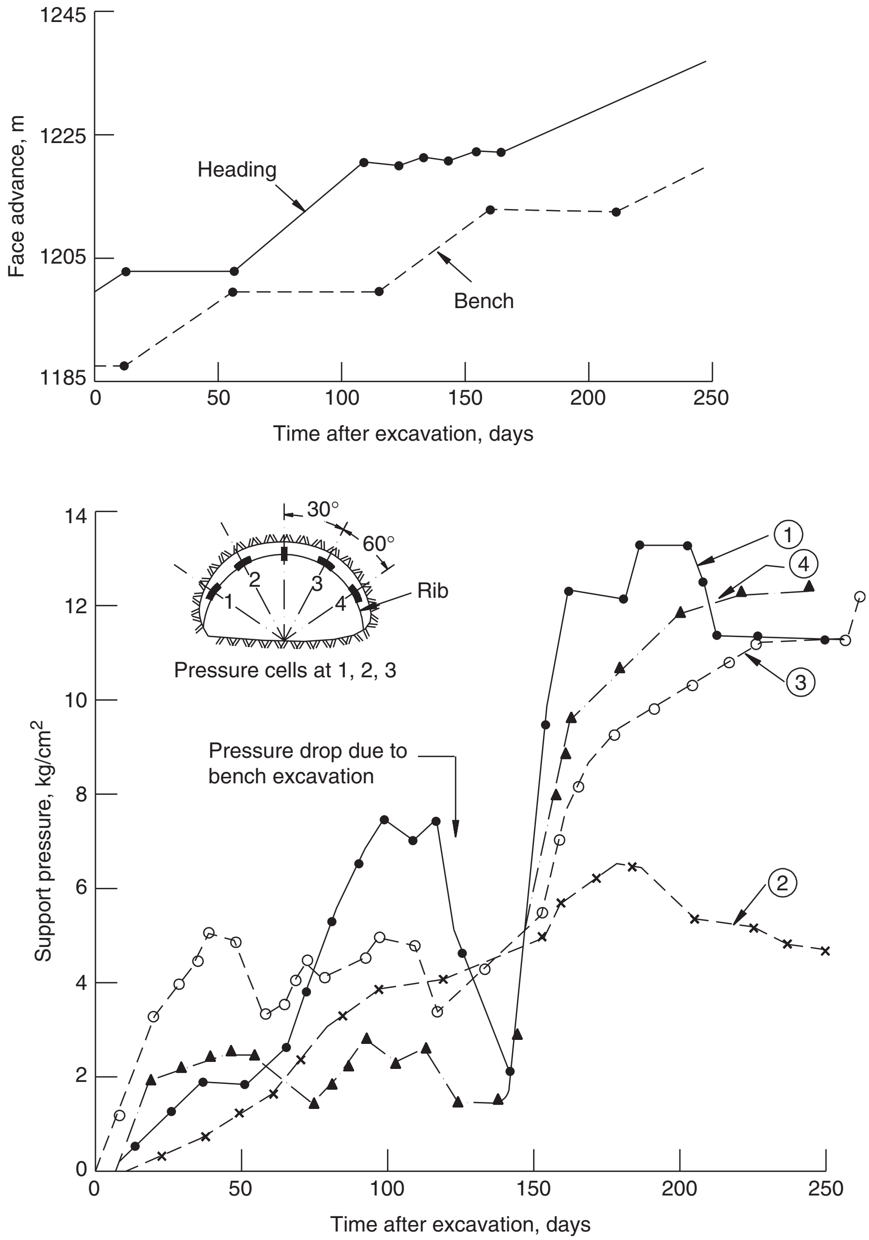


Fig. 20.12 Support pressure vs. time in red shales at Chhibro.

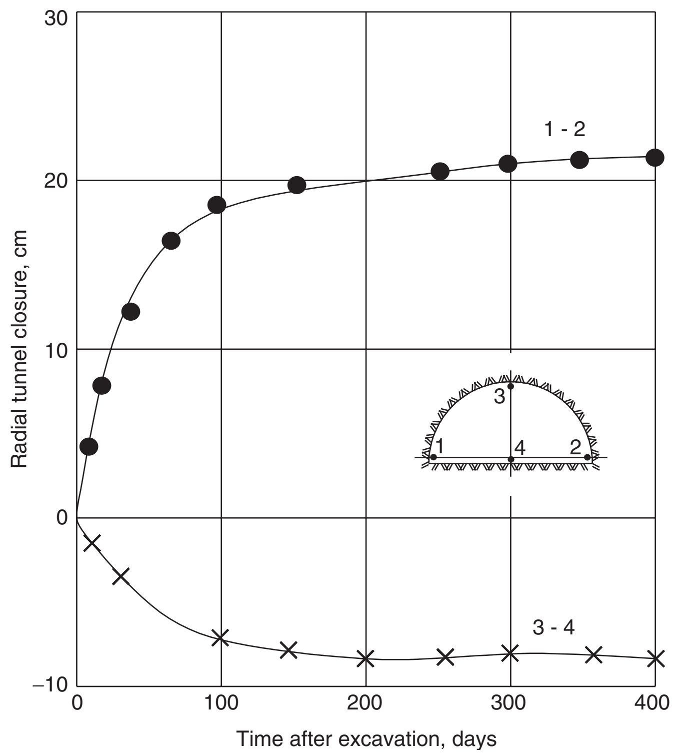


Fig. 20.13 Radial tunnel closure vs. time in red shales at Chhibro.

20.6.1 Effect of tunnel depth on support pressure

According to the elasto-plastic theory, failure of the rock mass around an opening under the influence of depth pressure forms a broken zone called "coffin cover". The failure process is associated with volumetric expansion of the broken rock mass and manifests itself in the form of squeezing into the opening (Labasse, 1949; Rabcewicz, 1964, 1965, 1969; Daemen, 1975). The "characteristic line" – or the "ground reaction curve" – concept explains that the support pressure increases with depth, provided that the tunnel closures are held constant. Further, large tunnel closures associated with the expansion of the broken zone lead to reduced support pressures (see Chapter 19).

High tunnel closures and support pressures observed in the Subathu–Dagshai red shales at a depth of 600 m at Chhibro, as compared to those observed at a depth of 280 m at Kalawar (Table 20.5), were explained by Jethwa et al. (1977) with the help of the elasto-plastic theory. They employed an empirical relation given by Komornik and David (1969) to estimate the swelling pressure and considered that the support pressure was the arithmetic sum of elasto-plastic (squeezing plus loosening) and swelling pressures. Later, J.J.K. Daemen (personal communication, 1978) raised doubts about this approach, but

Table 20.5 Comparison of estimated and observed support pressures.

Location and depth	Type of rock mass	Radius of circular opening	Estimated support pressure (kg/cm^2)														Average value of observed support pressure (kg/cm^2) and period of observation	
			After Terzaghi (1946)		After Barton et al. (1974)										After Elasto-plastic theory			
					Classification of parameters						Rock mass quality Q	Average support pressure						
			Roof	Wall *	RQD	J_n	J_r	J_a	J_w	SRF		Roof	Wall *		Roof	Wall *	Roof	Wall
Kalawar 280 m	Red shales $\gamma = 2.73$ gm/cc	1.5 m	3.4	3.4	10–20	15	1.5	4.0	1.0	5–10	+ 0.025 – 0.10	3.7	3.7		5.11	5.11	3.07 (155 – Days)	1.72
	Black clays $\gamma = 2.64$ gm/cc	4.5 m	9.98	9.98	–	–	–	–	–	–	–	–	–		11.20	11.20	11.50 (719 – Days)	12.20
Chhibro 600 m	Red shales $\gamma = 2.73$ gm/cc	4.4 m	17.20	17.20	10–20	15	1.5	4.0	1.0	10-20	± 0.0125–0.05	4.8	4.8		11.51	11.51	10.75 (257 – Days)	–

γ = Unit weight of rock mass
RQD = Rock quality designation
J_n = Joint set number
J_r = Joint roughness number
J_a = Joint alteration number
J_w = Joint water reduction factor
SRF = Stress reduction factor

* Assuming roof pressure = Wall pressure (without correction factor)
+ Corresponds to mild squeezing conditions at moderate depth
± Corresponds to highly squeezing conditions at great depth
– Not known

Singh (1978) emphasized the interaction between the swelling and squeezing pressure and suggested that only the greater of the two should be considered. The average elasto-plastic pressures, estimated according to the suggestions of Singh (1978), are close to the observed values (Table 20.5). As such, the empirical approaches, developed to estimate rock pressure for tunnel support design, must be amended to include the effect of tunnel depth in order to obtain reliable results under squeezing rock conditions. The correction factor for overburden f in Q-system (equation (5.7)) is now accepted.

20.6.2 Estimation of support pressure and support requirements

The limitations of the method of Terzaghi (1946) for estimating support requirements in the Kalawar inspection gallery have been discussed earlier. The observed pressures were significantly less than the estimated values at a greater depth (Table 20.5). Moreover, the ratio of observed support pressure at Chhibro (for the 9.0 m diameter tunnel) to that at Kalawar (for the 3.0 m diameter tunnel) in red shales is about 3.6; this ratio for estimated values rises to 5.0 (because the support pressure increases with the size of the opening in Terzaghi's approach), i.e., the pressures are overestimated for a large opening.

Table 20.5 also includes parameters of classification of rock mass according to Barton et al. (1974). A reasonable estimate of support pressure for squeezing conditions at moderate depth is obtained by this approach. However, it leads to a less reliable assessment in highly squeezing conditions associated with greater depths. Further, this approach tends to overestimate the support pressure for openings of small size and underestimates it for large openings, presumably because no consideration is given to the size of the opening and magnitude of squeezing.

It is clear from the above discussions that the elasto-plastic theory (Daemen, 1975) may be more reliable than the empirical approaches of Terzaghi (1946) and Barton et al. (1974) for estimating the support pressures in highly squeezing conditions in the clays within the intra-thrust zones.

20.6.3 Loose backfill with steel-arch supports

In a deep tunnel under squeezing rock conditions, the supports may attract huge loads unless substantial tunnel closures are allowed. It would be very expensive, possibly impracticable, to provide rigid supports under extreme conditions. Further, such a support system does not make use of the intrinsic strength of the rock mass. Various practices of tunnelling in highly squeezing conditions would fall into one of the following four categories.

(i) Excessive deformations of steel arches leading to encroachment of the required clear opening, dismantling of deformed supports, re-tunnelling and resetting new supports, e.g., the Pandoh–Baggi tunnel of Beas–Sutlej link project in India (Kochhar & Prem, 1973) and E1 Colegio tunnel in Columbia (ENR, 1966).

(ii) Cracking of concrete lining, if placed before the opening attains equilibrium, e.g., pressure tunnel of Giri hydroelectric project in India (Giri Hydel project, personal communication, 1977) and Kamuii R.R. tunnel in Japan (Takibuchhi, 1970).
(iii) Increasing the degree of support until equilibrium is reached (New Austrian Tunnelling Method (NATM), Rabcewicz, 1964, 1965, 1969).
(iv) Relieving support pressure with the help of yieldable steel arches.

Significant tunnel closures are allowed in the above examples in order to reduce the support pressures. Peck (1969) and Nussbaum (1973), while using NATM for constructing the 12 m North Tauern tunnel through Talus in Austria, have recognized the need of "tolerable yield," i.e., allowing tunnel closures in order to reduce the support pressures. The first two methods described above, are time-consuming and wasteful, whereas the last method is largely used in coal mines.

The sprayed concrete support system (shotcrete), popularly called NATM, absorbs tunnel closures to a certain extent and has shown vast potential in moderate squeezing conditions. However, its economical application under highly squeezing conditions is yet to be established. On account of the larger thickness required, Mahar et al. (1975) suggested that the shotcrete is not an economic support system to withstand high squeezing pressures. Obviously, an ideal support system for such conditions would be the one which can absorb large tunnel closures without undergoing undesirable deformations. Fig. 20.14 shows the general load-deformation characteristics of several common support systems

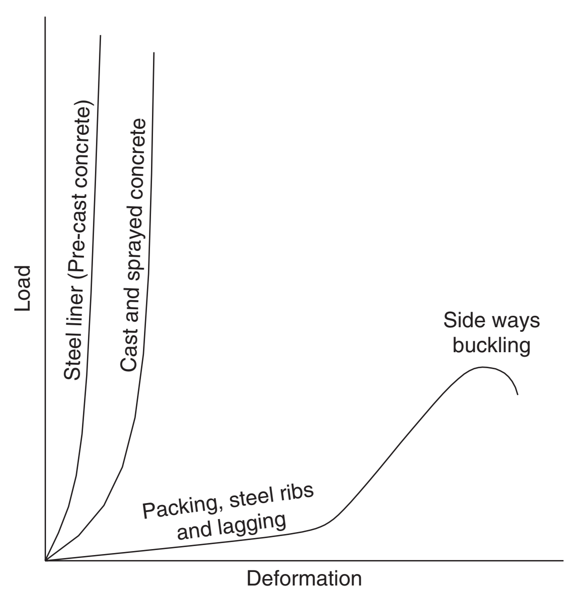


Fig. 20.14 Load-deformation characteristics of some support system (Ward, 1978).

Table 20.6 Influence of type of backfill on support pressure.

Rock type	No. of test section and tunnel radius (r)	Type of backfill	Period of observation (months)	Support pressure (p) (kg/cm^2)		p-loose/p-tight	
				Vertical	Horizontal	Vertical	Horizontal
Red shales	1($r = 1.5$ m)	tight	5	3.07	1.72	–	–
Red shales	3($r = 4.5$ m)	loose	5	0.80	0.40	0.27	0.23
Black clays	4($r = 1.5$ m)	loose	25	3.20	2.70	0.28	0.22
Black clays	5($r = 4.5$ m)	tight	25	11.50	12.20	–	–

(Ward, 1978). Steel-arch supports, which have substantial post-failure strength, may be noted for their capacity to absorb the largest deformations. Their use with loose backfill in a slightly over-excavated tunnel opening makes the system sufficiently flexible and may provide a solution to support problems in highly squeezing conditions. However, the fill material must be groutable in the case of pressure tunnel and its thickness should be decided from the considerations of compressibility and desired tunnel closures.

The p-loose and p-tight indicate support pressure with "loose" and "tight" backfill, respectively; tight backfill consisted of systematically packed PCC (precast cement concrete) blocks behind steel arches; loose backfill consisted of 30 cm thick layer of tunnel muck thrown manually around steel arches (Table 20.6).

Trials in the Chhibro–Khodri tunnel have shown that such a support system reduced the support pressures to a considerable extent (Table 20.6; Fig. 20.15). This result agrees with the observations of Spangler (1938) on conduits and Lane (1957) on Garrison dam tunnels in clay–shales. It also agrees with the concept of "tolerable yield" applicable to soft-ground tunnels (Peck, 1969).

20.6.4 Rate of tunnel closure

The monthly rate of tunnel closure ($u_r/r \times 100$) when plotted against time, indicates that the opening stabilized in about three months in the red shales at Kalawar (Fig. 20.16a). On the other hand, the stabilization time was about a year for the similar rock mass at Chhibro (Fig. 20.16b). The delayed stabilization at Chhibro is considered due to the absence of the invert supports. It is of interest to note here that the monthly rate of tunnel closure at the time of stabilization in both the cases was 0.05–0.1 percent of the opening size. It follows from the above that adequately strong temporary supports must be provided soon after excavation for an early stabilization of the opening. Further, tunnel closures must be measured to ensure that the temporary supports are adequate. Permanent supports should be provided only after the monthly rate of tunnel closure has reached safe limits of 2 mm per month.

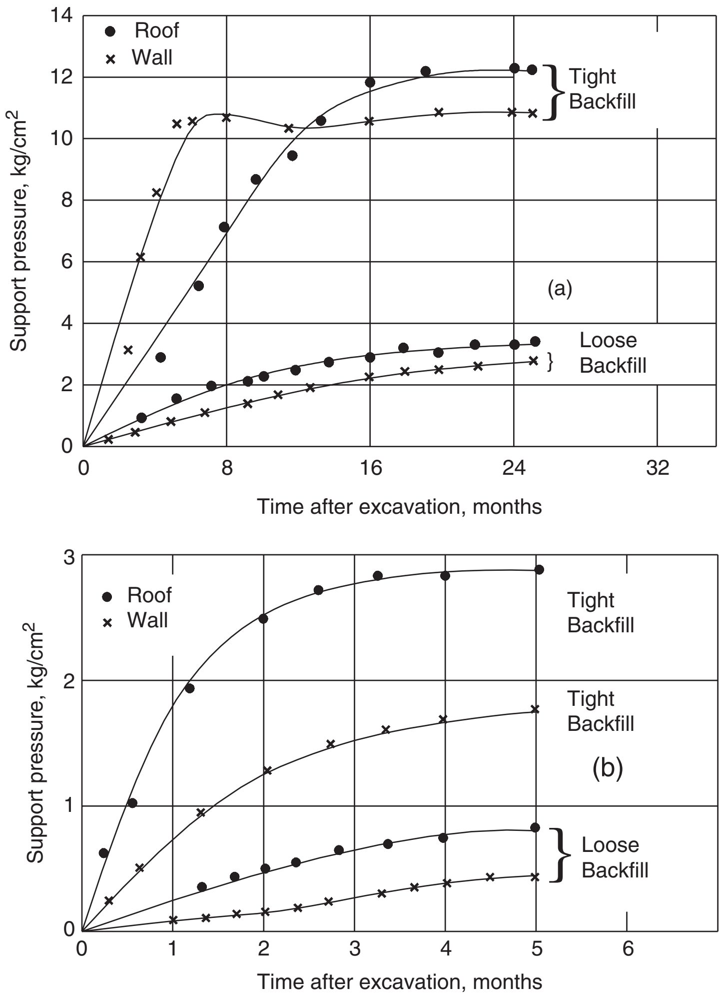


Fig. 20.15 Effect of type of backfill on support pressure in (a) black clays; (b) red shales.

20.7 CONCLUSIONS

From the case history of the Chhibro–Khodri tunnel of the Yamuna hydroelectric scheme and its instrumentation, the following conclusions are offered:

- Inadequate surface and sub-surface geological investigations have been responsible for wrong planning of the tunnel alignment. Consequently, the rate of tunnel construction has been very slow due to unforeseen changes from good rock to highly

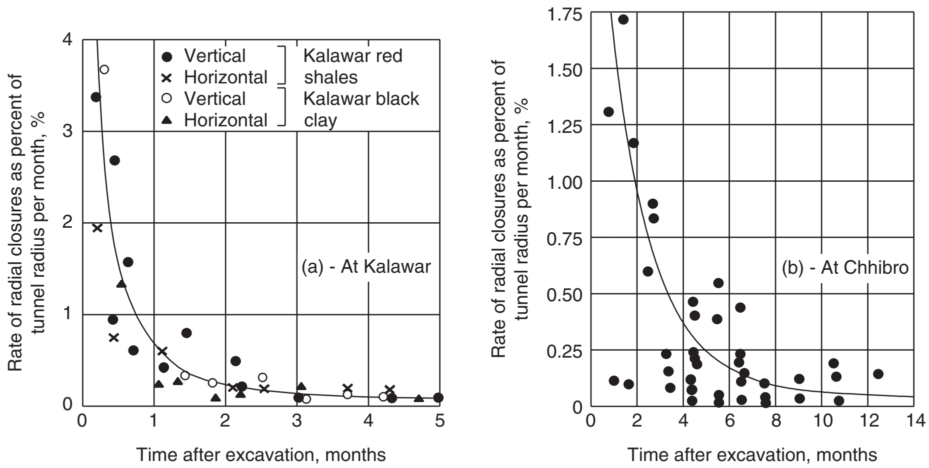


Fig. 20.16 Rate of radial tunnel closure vs. time.

squeezing rock conditions. The decision to excavate three smaller tunnels in place of a large tunnel has not proved wise.

- Tunnel closures must be considered while assessing the magnitude of tectonic slip across an active fault having thick squeezing gouge. Under these conditions, the stick-slip theory explains that the fault slip is likely to take place along any one smooth plane. Therefore, it may be questionable to design a flexible lining for an active fault zone on the assumption that the total fault slip would be distributed uniformly along the width of the fault gouge.
- Tunnel depth and the effect of opening size are not considered adequately in the empirical approaches of Terzaghi (1946) and Barton et al. (1974). As such, these methods (without correction factors) failed to provide reliable guidelines for support design under highly squeezing rock conditions. A deeper tunnel under these conditions is likely to attract higher rock loads unless greater closures are allowed. The elasto-plastic theory, which is based on the concept of tolerable yield, provided a better assessment of rock pressure under these conditions.
- Stiffer supports were found to attract higher support pressures under squeezing rock conditions. A flexible support system, consisting of steel arches with loose backfill, caused significant reduction in support pressure.
- The heading supports underwent severe buckling and bending under the influence of side pressure in squeezing rock conditions when the invert struts were not provided and the bench excavation was unusually delayed. Under such conditions, the bench must follow the heading closely to facilitate early erection of the invert supports. This practice mobilizes the full capacity of the support to sustain side pressure. If excavation of the bench is likely to be delayed, the heading supports must be provided with temporary invert struts.

- The monthly rate of radial closure at the time of stabilization was observed to be in the range of 0.05–0.10 percent of the radius of the opening (2 mm/month). Tunnel closures must be measured not only to ensure that the temporary supports are adequate, but also to find out when it is safe to provide permanent supports, i.e., concrete lining in the case of a pressure tunnel within squeezing ground.

Section 18.4.2 describes another interesting case history of two road tunnels within the intra-thrust zone.

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21

Tunnels in seismic areas

"Winners don't do different things. They do things differently."

Shiv Khera

21.1 INTRODUCTION

A study of the published literature indicates that the tunnels and caverns in rock medium do not suffer as much damage as the surface structures during major earthquakes ($M \leq 8.5$), particularly if they are located at a depth of more than 20 m and there is no fault zone in the neighborhood.

The explanation of drastic damage to surface structures during shallow major earthquakes is that surface waves (called Rayleigh waves) have more energy than primary and shear waves. The amplitude of Rayleigh waves decays exponentially with depth and it becomes negligible at a depth of about 15–20 m below the ground level in rock masses (just like surface waves in ocean).

A dynamic analysis of an underground structure is essential when it is meant to accommodate human activities. Other situations requiring a dynamic analysis are (Kumar & Singh, 1998),

- The underground structure may be located in the area of high seismic activities and the active fault may be crossing it or may be very near to it,
- The underground structure is to be used for testing of weapons,
- The ammunition stored in the structure may explode,
- The blasting technique is used in the excavation,
- A power tunnel is shut down under emergency resulting in oscillations and transient conditions due to effect of water hammer.

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In the dynamic analysis of such structures, two situations may arise. In the first case, the source of the dynamic loading is located within the structure itself so that an analysis for impact and over pressure is to be performed. In the second case, the source of dynamic loading may be far away and the structure is subjected to loading due to the traveling waves. This chapter is devoted to the second case.

21.2 RESPONSE OF AN UNDERGROUND STRUCTURE TO DYNAMIC LOADING

When a dynamic disturbance strikes an underground structure, some deformations result. These deformations may be decomposed in three components, namely, radial, axial and tangential. The axial component may be further decomposed into the longitudinal and transverse (wave) components. The radial deformation of the underground structure is important when the source of the dynamic disturbance is located within the structure, which is not covered in this chapter.

The longitudinal (axial) deformations are represented by alternating regions of compressive and tensile strains that travel as a wave train along the tunnel axis. The transverse (axial) component creates alternate regions of negative and positive curvatures propagating along the tunnel. A tunnel lining that is stiff compared with the surrounding soil responds as an elastic beam. For a positive bending associated with the transverse (axial) deformations, the top of the lining is in compression while its bottom is in tension. The same is not true, however, for rock tunnels with flexible or no lining at all. In such cases, the tunnel in positive curvature experiences tensile strain on top and compressive strain at bottom. This dynamic effect consisting of alternating cycles of compressive and tensile strain superimpose on the existing static state of strain in the rock and lining.

The tangential deformations result when waves propagate normal or nearly normal to the tunnel axis. These may result into distortion of the tunnel cross section and may lead to additional stress concentration. This effect is not severe as the tunnel diameter is much less than half the wavelength. Another aspect associated with the tangential deformational characteristic of the dynamic disturbance is that of ringing, i.e., entrapment and circulation of dynamic wave energy around the tunnel (Owen et al., 1979). This is not possible as the wavelength of the dynamic disturbance is much more than the tunnel radius. In general, the seismic wavelengths are very large (25–500 m) compared to the normal tunnel sizes.

Bickel et al. (1997) have analyzed maximum longitudinal strains in the concrete lining from snaking and racking motions during earthquakes in the case of tunnels in the soil. Software packages may also be used to check whether or not maximum strain is within the elastic range. Experience suggests that there is no cause for worry for tunnel stability because of earthquakes in rock masses below 20 m from ground surface, except in the active fault zones.

21.3 OBSERVED RESPONSE

In the case of a nuclear waste repository, it may be possible to select a site, which is relatively free of seismic disturbance threat. However, in the case of a metro or tunnels in a hydroelectric power project no such choice is usually available. In such cases, a quantitative assessment becomes essential. One of the major difficulties is that the earthquakes are recorded on the ground surface which is used in the designing of surface structures. Relatively much less is known about the variation of seismic disturbance intensity with depth.

Dowding and Rozen (1978) have compiled the seismic response of 71 tunnels. Fig. 21.1, extracted from this study, shows that the tunnels are less susceptible to damage

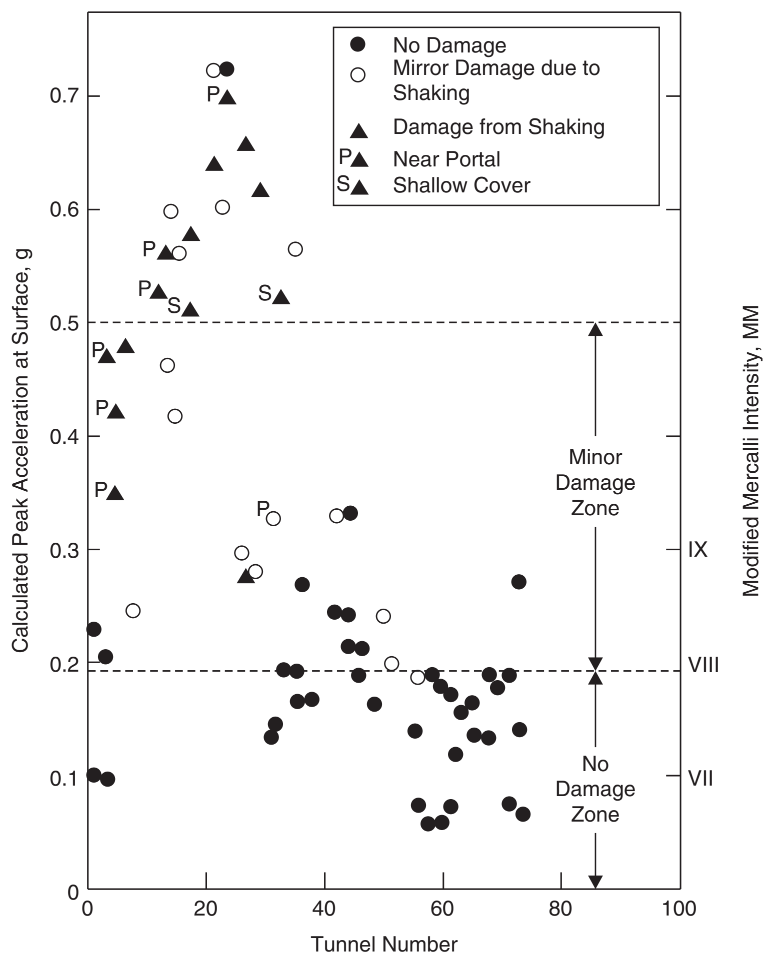

Fig. 21.1 Calculated peak acceleration at the surface and associated tunnel damage (Dowding & Rozen, 1978).

than the surface structures. The peak acceleration at the surface of less than 0.2 g magnitude did no damage to the tunnels. The accelerations between 0.2 and 0.5 g did only minor damage. The damage was found to be significant only when the peak ground acceleration exceeded 0.5 g. In such cases, most of the damage that occurred was located near portals. One may say that the portals are essentially surface structures.

Several Japanese investigators measured earthquake motion simultaneously at the ground surface and at depth. The findings of these studies may be summarized as follows. Nasu (1931) determined the ratio of displacements due to earthquakes at the surface and tunnels up to depths of 160 m. The geology consisted of lake deposit on the surface and volcanic andesite underneath. The surface/depth displacement ratios were 4.2, 1.5 and 1.2 for periods of 0.3, 1.2 and 4 s, respectively. Kanai and Tanaka (1951) measured acceleration at depth up to 600 m in copper mines in paleozoic rock. The ratio of maximum surface displacement to that at the depth of 300 m was about 6:1. Iwasaki et al. (1977) obtained acceleration records up to a depth of 150 m during a period of 5 years. The borehole accelerometers were installed at four locations around the Tokyo bay. Three of these sites were in sand and clay while the fourth was in siltstone. During the period of measurement, 16 earthquakes with magnitudes ranging from 4.8 to 7.2 were recorded. The analysis showed that the maximum acceleration heavily depended upon the soil conditions. The ratio of surface/depth accelerations are about 1.5 on a rocky ground, 1.5 to 3.0 in sandy ground and 2.5 to 3.5 in clayey ground. Although, the acceleration magnitude at depth was smaller, the frequency contents were similar.

The study of the Alaskan earthquake which was one of the largest earthquake of the twentieth century ($M = 8.5$) showed that while the surface damage was extreme, the underground structures escaped without any significant damage (Eckel, 1970). Similar results were reported by Cooke (1970) on the Peru earthquake of May 31, 1970. The earthquake of 7.7 magnitude on Richter scale did no damage to 16 rail road tunnels of combined length of 1740 m under small ground cover located in MM-VII and MM-VIII intensity zones. Similarly, no damage was reported to the underground hydroelectric power plant, three coal mines and two lead zinc mines located in MM-VII intensity zone.

The Himalayan experience may be added to the above. A large number of shrines are located in the caves deep inside the Himalayas. Although, this is a seismically active region and several big earthquakes have rocked this area, over the centuries nothing has happened to these shrines. It is understood that the size of the natural caves, tunnels and caverns is smaller than the quarter wavelength of seismic waves. Hence, openings are not noticed by the seismic waves and so there is no resonance and damage of the openings.

21.4 CASE HISTORY OF 1991 UTTARKASHI EARTHQUAKE

21.4.1 Project description

The Yamuna Hydroelectric Scheme Stage II harnesses the hydropower potential of river Tons which is a tributary of river Yamuna. The available head of 188 m is being utilized in two stages. Stage I utilizes the head of about 124 m along the first river loop between

Ichari and Chhibro to generate 240 MW of power. To avoid large scale excavation of steep slopes, the powerhouse chamber is located underground. Its size is 18.2 m wide, 32.5 m high and 113.2 m long. This cavern is excavated in a band of limestone of 193 × 217 m horizontal extent. A major shear zone passes within 10 m of the lowest draft tube level in the powerhouse area.

21.4.2 Seismic response

An earthquake of 6.3 magnitude occurred on 21 October 1991 which was centered near Uttarkashi and about 100 km away from the project site. The earthquake devastated the entire Uttarkashi area. The recorded damage in the Chhibro powerhouse cavern on account of this earthquake is limited to minor cracks in the region closest to the shear fault zones. The damage is described by Mitra and Singh (1995) as follows:

a) Out of the eight extensometers installed on the side walls of the powerhouse, only two on the downstream wall adjacent to the control room (nearest to the underlying shear fault zone) recorded any significant rock deformation. These deformations were of the order of 1 to 4 mm. Besides, a deep crack of 2 to 4 mm width formed diagonally up to a length of 3.5 m between these two extensometers.
b) Horizontal hairline cracks were observed on each column of the control room and the downstream side wall at heights of 0.5 to 2.5 m.
c) Two horizontal 1 mm wide cracks of lengths, about 5 m, were found in the portal at the main entrance of the powerhouse adit.
d) Two vertical cracks of 0.5 and 4 mm width with a spacing of about 80 m were observed inside the adit at a height of about one meter. But these cracks appeared to have formed in the shotcrete lining.
e) The anchor plates supporting the pre-stressed rock anchors in the expansion chamber adit appeared to have stretched slightly and may have caused the lining cracks.

There was no damage in the sections of powerhouse complex away (upto a distance of width of opening B) from the shear/fault zone. An analysis by Mitra and Singh (1997) shows that the dynamic support pressures are negligible compared to the long-term support pressures in the roof of the chamber near the shear fault zone due to residual strains in the nearby rock mass.

The above study shows that the seismographs should be installed inside a tunnel across active faults to record seismic peak acceleration in the roof, walls and base.

21.4.3 Segmental concrete lining across active fault (Jethwa & Singh, 1980)

Krishna et al. (1974) suggested an innovative segmental lining for the tectonically active intra-thrust zone along Chhibro–Khodri tunnel of Yamuna Hydroelectric Scheme to withstand a total vertical dislocation of 0.5 m expected during the 100 years life of the project (Agrawal & Gaur, 1997). Further, they considered that the slip would be distributed uniformly along the width of the intra-thrust zone. Based on the above assumptions, they proposed a "segmental lining" to cope with the tectonic slip (Fig. 20.6). It consisted of circular segments of varying lengths connected together by flexible joints. Contrary to the above assumptions, tectonic slip in thick fault gouge may take place along any one of the plane as suggested by Brace and Byerlee (1967) who explained the mechanism of earthquakes by the "strike–slip" phenomenon. However, the power tunnel with segmental lining is working satisfactorily since 1980.

The sewer tunnel of Los Angeles passed through an active fault zone. It was decided to design a 30 m long articulated concrete lining, surrounded by the back-packing of cast-in-place cellular concrete, which may withstand 20 cm of lateral displacement along the fault zone. The dynamic compressive stress in the lining was estimated as 0.13 MPa. This proved practical and cost-effective (Bickel et al., 1997). Bolu tunnel is another educative case history (Section 18.4.2).

21.5 PSEUDO-STATIC THEORY OF SEISMIC SUPPORT PRESSURE

A pseudo-static approach is proposed to estimate the support pressure under dynamic conditions in the underground openings. Suppose the vertical peak acceleration is $\alpha_v \cdot g$ in roof and horizontal peak acceleration is $\alpha_h \cdot g$ in the wall of the tunnel, where g is the acceleration due to gravity (Fig. 21.2). It is reasonable to assume in the case of jointed rock masses that the vibrating mass is the mass of rock wedge which is naturally formed by three

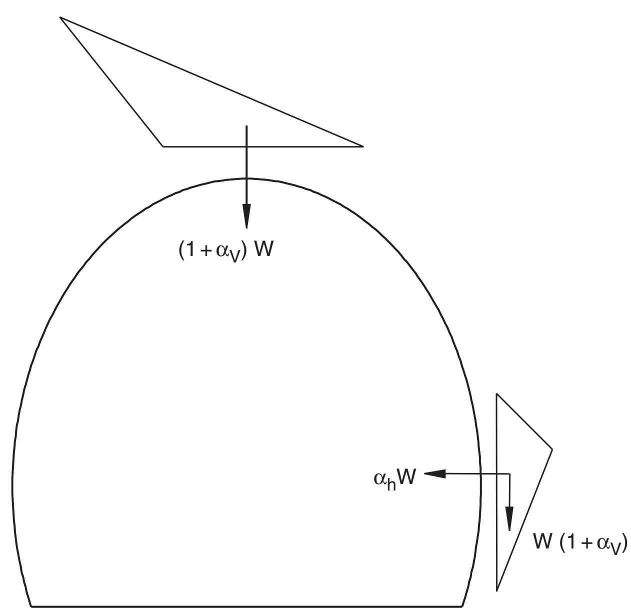

Fig. 21.2 Peak acceleration experienced by rock wedges during earthquakes.

critical rock joints. Pseudo-static analysis is quite popular in geotechnical engineering and it assumes that the unit weight of rock mass (γ) is modified to $(1 + \alpha_v) \cdot \gamma$. It follows that the increase in the support pressure because of earthquake ($p_{seismic}$) may be taken approximately as follows.

In roof:

$$p_{seismic} = (\alpha_v) \cdot p_{roof} \tag{21.1}$$

$$= 0.25\, p_{roof} \quad \text{(Barton, 1984)} \tag{21.2}$$

In walls:

$$p_{seismic} = (\alpha_h) \cdot p_{wall} \tag{21.3}$$

$$= 0.25\, p_{wall} \quad \text{(Barton, 1984)} \tag{21.4}$$

where
α_v = coefficient of the vertical peak acceleration at roof = 0.25,
α_h = coefficient of the horizontal peak acceleration at walls = 0.25 and
γ = unit weight of the rock mass.

Another cause of seismic support pressure is continuous building up of the residual strains around an opening with successive earthquakes, particularly near the faults, etc. Nevertheless the hypothesis of Barton (1984) appears to be realistic in view of the fact that tunnels have seldom failed during even major earthquakes. The design of support system may be selected from the chart (Fig. 10.2) and Table 10.2 of Barton et al. (1974) for the following seismic rock mass quality (see Chapter 10),

$$Q_{seismic} = Q/(1 + \alpha_v)^3 \tag{21.5}$$

$$= Q/(1 + 0.25)^3 = \frac{Q}{2} \quad \text{(Barton, 1984)} \tag{21.6}$$

Alternatively the software TM may be used considering the total support pressure of $p_{roof} + p_{seismic}$ (see Appendix II). Seismic support pressure in the squeezing ground may be assessed approximately as discussed in Section AI.4.

The dynamic increment in support pressure in rail tunnels may perhaps also be assumed to be negligible and of the same order as that of earthquakes. However, where overburden is less than $2B$ (where B is the width of the opening), the roof support pressure is taken equal to the overburden pressure. This conservative practice is due to errors inherent in the survey of hilly terrain. In case the shallow rail or road tunnels are excavated in the seismic rocky areas, concrete lining is provided with contact grouting between concrete lining and rock mass. Consolidation grouting of loosened rock mass should also be done to prevent further loosening of the rock mass during earthquakes. Back grouting ensures intimate contact between concrete lining and rock surface which may not allow bending of the lining, and no bending stresses are likely to develop during earthquakes.

21.6 SUPPORT SYSTEM FOR BLAST LOADING

It is being realized now that underground openings may provide safety against nuclear or missile attacks. The depth of overburden is the most important factor. Rock engineers are now approached to design support systems which are safe against blast loading. The concept is same as for seismic loading, except that the peak acceleration may be of high intensity ($\alpha_v > 1$, sometimes 5).

The experience of tunnelling or mining through rock burst prone areas may be relevant here. Long resin bolts/anchors (without pre-tension) have been successfully used as they are able to withstand vibrations of high intensity and arrest propagation of fractures in the rock mass. The steel fiber reinforced shotcrete (SFRS) is also a ductile material and has high fracture toughness and high shearing resistance. The principle for transforming a catastrophic brittle failure into the plastic failure is that the brittle rock mass is converted into the ductile reinforced rock arch. The SFRS is also ductile obviously due to steel fibers.

It may be mentioned here that the peak acceleration of blast waves do not attenuate rapidly in hard rocks. The damping coefficient of hard rocks is also low. As such the coefficient of peak acceleration (α_v) is likely to be quite high in shallow openings. Engineering judgment is the best guide here. Conservative approach is the need of design of underground structures of strategic importance, as future weapons and atomic bombs are going to be unimaginably disastrous in its lifetime.

The dynamic model tests show that rock wedge in the roof tends to slide down slightly on shaking. Hence wedge theory of support pressure would perhaps be applicable under heavy dynamic loading such as blast loading. Field research is needed in this area.

The dynamic support pressures are likely to be high according to equations (21.1) and (21.3). In case $\alpha_v > 1$, the rock wedge at the bottom of the opening may also be dislodged in upward direction. Thus the required dynamic support pressure at the bottom of an opening is estimated by assuming the unit weight of rock mass equal to $(\alpha_v - 1) \cdot \gamma$ (Fig. 21.3),

$$p_{\mathrm{bottom}} = (\alpha_v - 1) p_{\mathrm{roof}} \tag{21.7}$$

Hence rock anchors and SFRS may also be needed at the bottom of the opening. Perhaps it is not necessary to make bottom of the opening curved surface to reduce dynamic tensile stresses. The software package UWEDGE and TM can be used confidently to design support system in the roof, walls and the bottom (Singh & Goel, 2002). The chart (Fig. 10.2) of Grimstad and Barton (1993) may be used considering a down-graded rock mass quality approximately by equation (21.5).

One may also keep in mind that the overburden of rock mass at portal of the tunnel should be $5 \cdot B$ in the blast prone area, where B is the span of opening. Further the maximum overburden over an opening should be much less than $350Q^{1/3}$ m where $J_r/J_a < 0.5$, this will ensure non-squeezing condition in the openings. Yet a minimum of cover of 300 m above underground opening should be ensured for safety against mega nuclear attacks right above them. Needless to mention that the rock mass quality near portals is down graded to Q/3 and it is Q/2 near intersection of openings (Barton et al., 1974). So additional

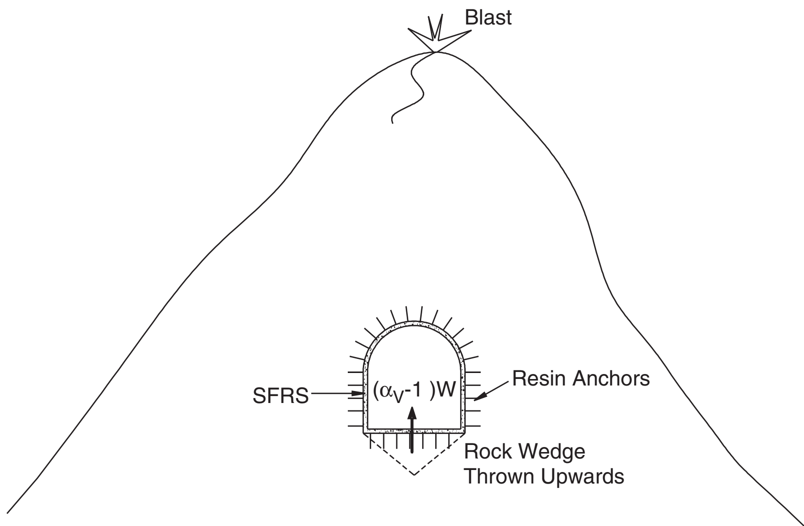

Fig. 21.3 Support system for blast loading.

down grading of rock mass quality may be done using equation (21.5) near portals and intersections of underground openings.

The designer should also check the peak particle velocity at the roof level to save the support system from damage by shock waves. The peak particle velocity (v) is,

$$v = \alpha_v \cdot g/2\pi f < 7.5\,\text{cm/s} \quad \text{(Dowding, 1993)} \tag{21.8}$$

where f approximately is the frequency of the blast waves or inverse of time period of the shock waves. Damage to the support system is unlikely to occur if the particle velocity is less than 7.5 cm/s, which is the permissible peak particle velocity for structures on or within rock masses. Damages if occurred are minor and localized due to blast shock waves and are easily repairable. Further the design of concrete lining may be checked by software FLAC^{3D} considering realistic dynamic forces at the top of openings and elasto-plastic behavior of nearby fault zones.

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22
Rock burst in tunnels

22.1 INTRODUCTION

Experience shows that deeper an opening is made in hard rocks, more vulnerable it becomes to rock burst. The rock burst is defined as any sudden and violent expulsion of rock pieces from an apparently (temporarily) stable opening. The manifestation of slabbing and release of microseismic energy may be the first sign but suddenly several thousands of tons of rocks may break out like an explosion releasing seismic energy of a mild earthquake (approx. 4M). The loss of life is not difficult to imagine. For example, in a very deep mine, rock bursts may account for 50 percent of total fatalities.

As such a sequence of excavation or mining must be so designed that rock fails in a controlled manner. At least no rock burst should occur near working face during working hours for protection of workers. It may be noted that there is a significant departure in the philosophy of design from that of some surface structures (hill roads, bridges, etc) which are allowed to fail catastrophically.

What is the best strategy of sequence of excavation which minimizes both frequency and severity of rock bursts? The conventional philosophy of minimizing stress concentration is too conservative and irrational in comparison to recent theories.

Is it necessary to make an opening without sharp corners for avoiding stress concentration? The answer is negative. The fear of stress concentration among rock engineers perhaps has found the way from structural and mechanical engineers who do not wish any part of the structure to crack. In fact many roadways of rectangular shape have been successfully used. In practice even circular openings are seldom excavated as truly circular, yet the actual openings do remain stable in spite of unwanted notches.

The secret follows from the Griffith theory that a notch and associated stress concentration is harmless if it does not cause a significant release in the magnitude of strain energy to cause a sustained fracture propagation in the rock masses.

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22.2 CONDITIONS FOR ROCK BURST IN DEEP TUNNELS

Let us consider a simple case of stability of a rock pillar between two closely spaced tunnels. The roof above the pillar may be characterized by a spring of stiffness *K*. If pillar is a unit cube, its post-failure stiffness will be equal to *K* (Fig. 22.1).

Thus rock burst is likely to occur if roof stiffness (*K*) is less than the post-peak stiffness of the rock mass E_f. This happens usually in a laboratory while testing rock samples in

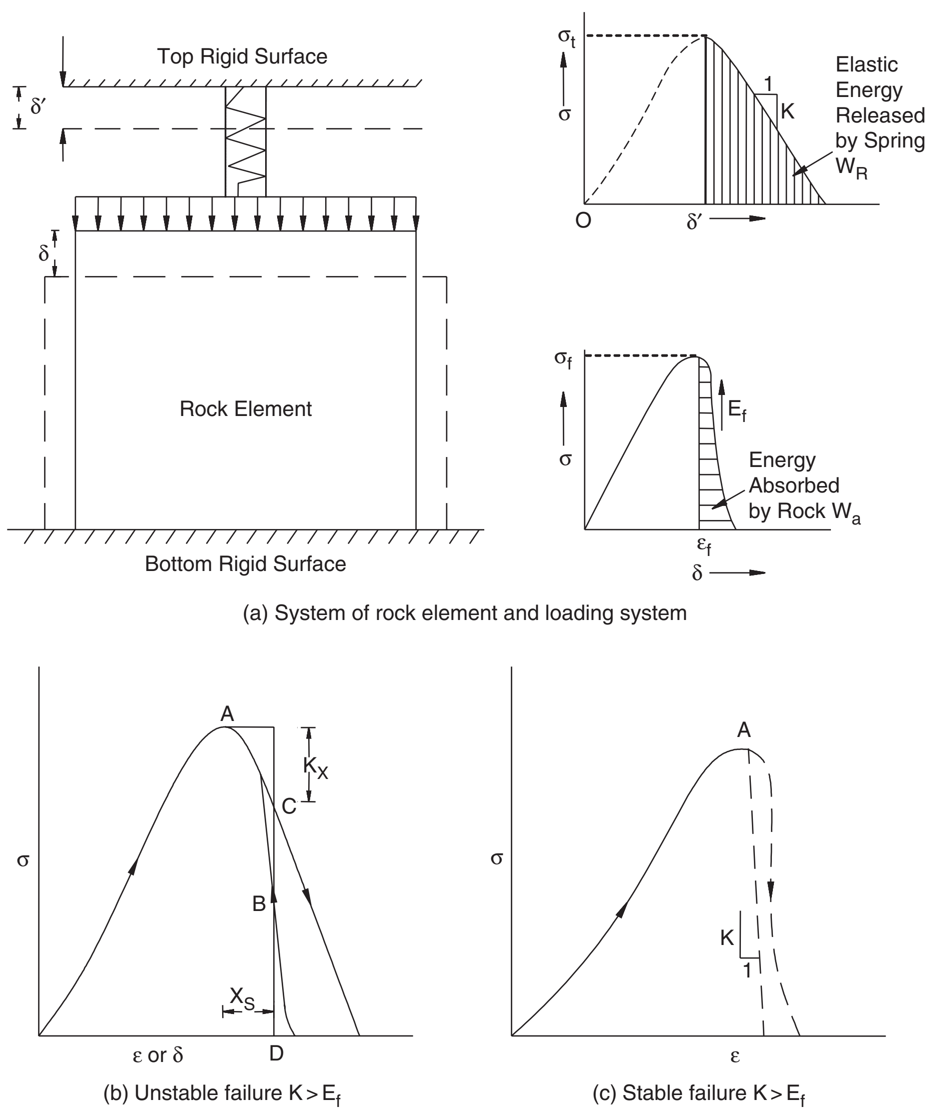

Fig. 22.1 Conditions for slow or sudden failure of a rock element (1 × 1 × 1) compressed by a loading system of stiffness *K*.

not-too-stiff conventional testing machines. The stress applied by spring (or machine or roof) is more than the load carrying capacity of rock block after the peak failure as shown in Fig. 22.1b (i.e., CD > BD). The kinetic energy of flying rock mass is $W_R - W_a$ as shown in Fig. 22.1a. Obviously this pillar is likely to be stable where $K > E_f$ (Fig. 22.1c).

Another interesting example is of a deep tunnel around which a broken zone is developed (Fig. 22.2a). The stiffness of elastic zone (K_e) will be equal to $2G/b$, where G is the shear modulus of isotropic elastic rock mass and b is the radius of broken zone.

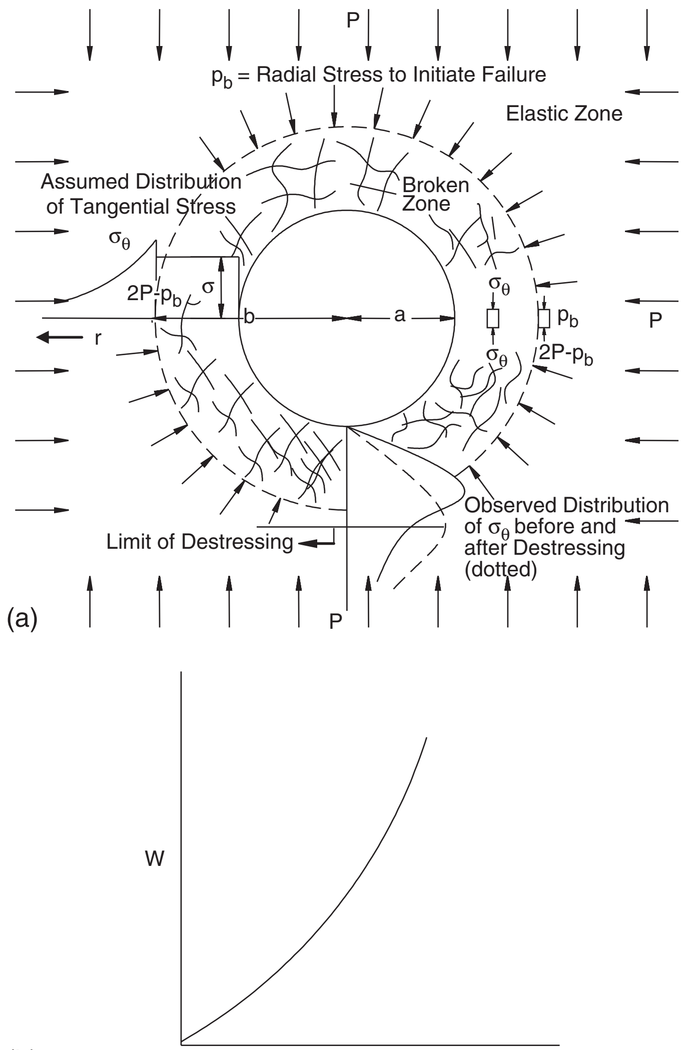

Fig. 22.2 Energy released in a rock burst; W is the difference in energy released by elastic zone and that absorbed by broken zone.

In other words, the stiffness of elastic zone will decrease drastically with increasing size of the broken zone. However, the post-failure stiffness (K_f) of the broken ring will increase with its increasing thickness. The post-failure stiffness of the broken ring is defined as the loss in support capacity (p_b) per unit radial deflection (u_b). So the severity of rock burst will tend to be more for thicker broken zones as $K_e < K_f$ (Jaeger & Cook, 1969).

It would be interesting to derive an expression for a limiting broken zone assuming uniform stress distribution released across the ring (Fig. 22.2a). According to Jaeger and Cook (1969), the criterion of rock burst is as follows:

$$\left|\frac{(b-a)E_f}{b}\right| > 2G \tag{22.1}$$

Further, the strain energy released by the elastic zone is equal to W_e. The energy absorbed by the broken zone is W_b. Thus, energy released as rock burst is equal to W or $W_e - W_b$ (Fig. 22.2b). This energy will be converted into kinetic energy of rock pieces and energy of seismic waves. Simple calculations will show that it is likely to increase rapidly with increasing size of the opening (Fig. 22.2b).

A design of tunnel support system based on the plastic theory (Fig. 22.2) may thus turn out to be unsafe if the mode of failure of rock mass is not checked out. One should know whether it is going to fail as rock burst or plastically. Rock burst is likely to take place in most situations where rock mass shows class II behavior (E_f is negative) and is overstressed.

22.3 CONCEPT OF STRAIN ENERGY RELEASE RATE

This concept is useful in understanding why a rock burst can occur. According to Griffith theory, if the strain energy released per unit area of new crack surface is more than the surface energy of the new crack surface, a crack will propagate, otherwise not. This principle may be applied in tunnel openings also. For example, if strain energy released per unit surface area of excavation is more than a limiting value, rock burst will occur. In other words, strain energy release rate as defined above should be controlled by a planned sequence of excavation that it is minimum and does not exceed a limiting value.

The strain energy release rate is equal to half of the product of primitive stresses and displacements at the boundary of new opening which is just excavated full face.

In reality, rock mass is a non-linear material with time-dependent characteristics. So the concept of strain energy release rate requires generalization. Nevertheless, the simple concept of strain energy release rate does give some idea about the problems of rock bursts in openings within massive hard rock masses.

22.4 SEISMIC ENERGY RELEASED IN A ROCK BURST

Evidently the center of seismic event leading to rock burst is the region of highest stress concentration in the elastic zone. Seismic studies of Cook (1962) indicated that such events occur generally not more than 30 meters from the face of an excavation (Jaeger & Cook, 1969). Seismic events that end up in rock burst were only 5 percent of all events recorded and the seismic energy of the order of 10^5 to 10^8 ft 1b. was released in bursts. Otherwise in the remaining 95 percent of the cases, the energy released at the epicenter of the violent failure and propagating towards the excavation is most probably absorbed in the deformation of the previously fractured zone of rock mass. This zone in this manner provides adequate cushion between the epicenter and the face of excavation.

Experience shows that rock masses which are fractured either naturally or artificially are not prone to rock burst. This is explained by the relatively ductile behavior of jointed rock masses. It is only the massive hard and brittle rocks (Q perhaps greater than 2) that pose problem because of low value of E/E_f. Further, since a fault will render the masses more flexible as if it has reduced the elastic modulus, the chances of rock burst at the intersection between the fault and the tunnel or roadway are increased.

Another important factor is the rate of excavation which cannot however be accounted in the theory. Laboratory tests show that the ratio E/E_f increases with decreasing rate of deformation. Thus a slower rate of excavation may cut down the frequency and severity of rock bursts.

22.5 SEMI-EMPIRICAL CRITERION OF PREDICTING ROCK BURST

It is obvious that failure of rock mass will occur where tangential stress exceeds its biaxial (plain strain) compressive strength. Singh et al. (1998) have suggested that the effective confining stress is nearly the average of minimum and intermediate principal stresses. Thus the biaxial strength is given by equation (19.3) in Chapter 19.

In situ stresses should be measured in drifts in areas of high tectonic stresses to know P_o and σ_θ realistically. It will help in predicting rock burst conditions in massive rock masses.

Kumar (2002) has studied the rock burst and squeezing rock conditions at NJPC head race tunnel in Himalaya, India. The field data is compiled in Table 22.1 for 15 tunnel sections of 10 m diameter where overburden is more than 1000 m. No rock burst occurred at lesser overburden. According to Barton et al. (1974), heavy rock burst was predicted as σ_θ/q_c was more than 1.0, where q_c is the uniaxial compressive strength of rock material (gneiss). Fortunately, values of σ_θ/q'_{cmass} are between 0.55 and 1.14, which predict very mild rock burst conditions. Actually there were no heavy or moderate rock burst conditions along the entire tunnel. Slabbing with cracking noise was observed after more than one hour of blasting. According to site geologists, Pundhir et al. (2000), initially cracking noise was heard which was followed by the spalling of 5–25 cm thick rock

Table 22.1 Comparison of Mohr's and Singh's criteria of strength of rock mass (Kumar, 2002).

S.No.	Chainage, *m*	Rock cover (m)	UCS (MPa)	Q Parameters RQD	J_n	J_r	J_a	J_w	SRF	Q	ϕ_p (deg)	P_o (MPa)	σ_θ (MPa)	q_{cmass} (MPa)	q'_{cmass} (MPa)	$\sigma_\theta / q'_{cmass}$	$\sigma_\theta / q'_{cmass}$	Predicted rock behavior	Rock behavior (observed)
1.	11435–11446	1430	50	70	6	2	2	1	2.5	4.7	45	38.6	77.2	31.6	124.8	2.4	0.62	Heavy burst	Mod. slabbing with noise
2.	11446–11459	1420	32	60	6	2	2	1	2.5	4.0	37	38.3	76.7	30.0	87.9	2.6	0.87	Heavy burst	Mod. slabbing with noise
3.	11459–11525	1420	50	67	6	2	2	1	2.5	4.5	45	38.3	76.7	31.1	123.7	2.5	0.62	Heavy burst	Mod. slabbing with noise
4.	11621–11631	1320	32	55	9	1.5	2	1	2.5	1.8	37	35.6	71.3	23.1	77.0	3.1	0.93	Heavy burst	Mod. slabbing with noise
5.	11634–11643	1300	50	70	6	1.5	2	1	2.5	3.5	45	35.1	70.2	28.7	113.4	2.4	0.62	Heavy burst	Mod. slabbing with noise
6.	11643–11650	1300	60	60	6	1.5	3	1	2.5	2.0	45	35.1	70.2	23.8	108.6	2.9	0.65	Heavy burst	Mod. slabbing with noise
7.	11656–11662	1300	55	55	6	1.5	3	1	2.5	1.8	45	35.1	70.2	23.1	107.9	3.0	0.65	Heavy burst	Mod. slabbing with noise
8.	11662–11796	1300	50	65	6	1.5	2	1	2.5	3.3	45	35.1	70.2	28.0	112.7	2.5	0.62	Heavy burst	Mod. slabbing with noise

9.	11860–11917	1230	50	67	6	1.5	3	1	2.5	2.2	45	33.2	66.4	24.7	104.9	2.7	0.63	Heavy burst	Mod. slabbing with noise
10.	12044–12070	1180	42	70	6	2	2	1	2.5	4.7	55	31.9	63.7	31.6	175.9	2.0	0.36	Heavy burst	Mod. slabbing with noise
11.	12070–12077	1180	34	60	6	1.5	3	1	2.5	2.0	30	31.9	63.7	23.8	55.7	2.7	1.14	Heavy burst	Mod. slabbing with noise
12.	12087–12223	1180	42	67	6	1.5	2	1	2.5	3.4	45	31.9	63.7	28.3	105.2	2.3	0.61	Heavy burst	Mod. slabbing with noise
13.	12223–12267	1100	42	75	4	2	2	1	2.5	7.5	45	29.7	59.4	37.0	108.7	1.6	0.55	Heavy burst	Mod. slabbing with noise
14.	12273–12322	1090	50	70	4	3	3	1	2.5	7.0	45	29.4	58.9	36.2	107.2	1.6	0.55	Heavy burst	Mod. slabbing with noise
15.	12359–12428	1060	50	75	6	1.5	2	1	2.5	3.8	45	28.6	57.2	29.4	98.5	1.9	0.58	Heavy burst	Mod. slabbing with noise

Notations: $P_o = \gamma H$; $\sigma_\theta = 2\gamma H$; $q_{cmass} = 7\gamma Q^{1/3}$ MPa; q'_{cmass} = biaxial compressive strength from equation (19.3); Q = post-construction rock mass quality; ϕ_p = peak angle of internal friction in degrees and H = height of overburden in meters.

columns or slabs and rock falls. This is very mild rock burst condition. Another cause of rock burst is the class II behavior of gneiss according to tests at IIT, Delhi, India (i.e. axial strain tends to reduce in comparison to peak strain after failure, although lateral strain keeps on increasing due to slabbing). Further, only the light supports have been installed in the rock burst prone tunnel even under very high overburden of 1400 m. These light supports are stable. It may also be noted from Table 22.1 that according to Mohr's criterion, σ_θ/q_{cmass} is estimated to be in the range of 1.6 to 3.1 which implies that moderate rock burst conditions should have occurred. Kumar (2002), therefore, made an observation that Singh et al.'s (1998) criterion (equation 19.3) considering σ_θ/q'_{cmass} is a better criterion than Mohr's criterion for predicting the rock burst conditions in tunnels. It is interesting to note that q'_{cmass} is much greater than uniaxial compressive strength (UCS) of rock materials. However, q'_{cmass} would be less than biaxial strength of rock material. Hence equation (19.3) appears to be valid. It is important to note that q'_{cmass} (biaxial strength) is as high as four times or more of uniaxial rock mass strength (q_{cmass}). The peak angle of internal friction (ϕ_p) in Table 22.1 is found from the triaxial tests on the rock cores. It is assumed to be nearly same for moderately jointed and unweathered rock mass. This appears to be a valid hypothesis approximately for $q_c > 10$ MPa as micro reflects the macro. There is difference in the scale only. The ϕ_p is not affected by the size effect. Table 29.1 offers more explanation considering non-linear effect in Chapter 29.

It is important to know in advance, if possible, the location of rock burst or squeezing conditions, as the strategy of support system are different in the two types of conditions. Kumar (2002) could fortunately classify mode of failures according to values of joint roughness number (J_r) and joint alteration number (J_a) as shown in Fig. 22.3. It is observed

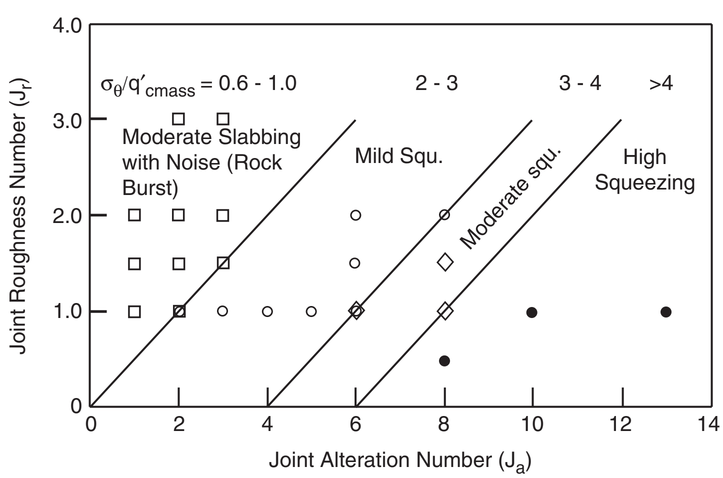

Fig. 22.3 Prediction of ground condition (Kumar, 2002).

that mild rock burst occurred only where J_r/J_a exceeds 0.5. This observation confirmed the study of Singh and Goel (2002). If J_r/J_a is significantly less than 0.50, squeezing phenomenon was encountered in many tunnels in the Himalaya. Thus, a semi-empirical criterion for mild rock burst in the tunnels is suggested as follows:

$$\frac{\sigma_\theta}{q'_{cmass}} = 0.60 - 1.0 \tag{22.2}$$

and

$$\frac{J_r}{J_a} > 0.50 \tag{22.3}$$

The support pressure may be assessed from modified Barton's criterion which is found to be valid upto an overburden of 1430 m by Kumar (2002),

$$p_{roof} \cong \frac{0.2(Q)^{-1/3}}{J_r} f \text{ MPa} \tag{22.4}$$

where

f = correction factor for overburden,
= $1 + (H-320)/800 \geq 1$,
H = overburden above crown of tunnel in meters and
Q = post-construction rock mass quality.

The dynamic support pressure may be $\alpha_v p_{roof}$ like equation (21.1) where $\alpha_v \cdot g$ is the observed maximum acceleration of rock pieces. The α_v may be as high as 0.35.

22.6 SUGGESTION FOR REDUCING SEVERITY OF ROCK BURSTS

Suppose a tunnel opening is supported by very stiff supports so that support pressure develops to the extent of cover pressure, no rock burst will occur. But, this is a very costly way of solving the problem.

Another way of reducing chances of rock burst is to make opening of small size. This is because amount of strain energy released per unit area of excavation will be reduced considerably.

Since stress concentration is responsible for initiation of cracking, it may help to select a shape of excavation which gives minimum stress concentration. For example, an elliptical opening is best suited in non-hydrostatic stress field. Its ratio of span to height should be equal to ratio of horizontal stress to vertical stress. In hydrostatic stress field, circular openings are better than square openings. As mentioned earlier, it may also help to slow down the rate of excavation in the zone of stress concentration, as rocks will be able to absorb more strain energy due to creep.

It may be recalled that the de-stressing technique has been used with some success in mines. In tunnel opening, if rock is broken intentionally by blasting or drilling, etc. to radius, in excess of b, the stress concentration is pushed inside the rock mass (Fig. 22.2a). Further the maximum tangential stress in elastic zone will be reduced below the in situ strength. Consequently chances of rock burst are reduced. The data of Reax and Den Khaus (Obert and Duvall, 1967) from South African mine supports the above hypothesis only partially. The de-stressing of the overstressed rock behind the face of excavation postponed the bursts from on-shift to off-shift period. Even then, in this way number of fatalities had been cut down drastically. Further destressing holes in areas of stress concentration are not effective.

Not only should the support system be designed to be safe, its safe mode of failure should also be designed to be slow and ductile (Fairhurst, 1973).

The modern trend is to convert the brittle rock mass into a ductile rock mass by using full-column grouted resin bolts. The plastic behavior of mild steel bars will increase the overall fracture toughness of a rock mass. So the overstressed rock mass will tend to fail slowly, as the propagation of fractures will be arrested by the reinforcing bars. The length of the rock bars may be equal to the thickness of the broken zone ($b - a$). The capacity of the reinforced rock arch should be equal to p_{roof} (equation (22.4)).

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23
Pressure tunnels

23.1 INTRODUCTION

The modern trend is construction of small dams with very long tunnels and shafts to generate a high head of water for generation of electricity. Head race tunnels or pressure tunnels are therefore used extensively in the hydroelectric projects. The water flows through pressure tunnel under internal pressure which depends upon the height of the dam. The pressure tunnels are also employed as diversion tunnels to discharge floods during construction of a dam. The water tunnels are also useful to carry drinking water from lakes to cities. The tunnels are being excavated to discharge storm water from mega cities to rivers after some treatment in modern times. Concrete lined canal tunnels are also being made passing through hills. It may be mentioned that pressure tunnels of medium size ($B = 5$ to $6\,\text{m}$) are most economical for generation of electricity.

Unlined pressure tunnels are provided within massive hard rock masses as it is self-supporting (Section 5.7). Discharge will be less due to rough surfaces of excavations. The permissible velocity of water in unlined tunnels is also less (<1 m/s).

Most pressure (power) tunnels are lined with concrete to reduce head loss due to friction at the tunnel boundary. This reduces water loss due to seepage and also stabilizes the unstable rock wedges. Plain cement concrete (PCC) lining has been used in many long power tunnels in hydroelectric projects in U.P., India. No hoop reinforcement has been provided though internal water pressure is quite high. These PCC linings have been working satisfactorily since 1980 without any closure for repairs. It is heartening to know that PCC lining has worked in squeezing rock conditions also. Millions of dollars and construction time can be saved if unnecessary hoop reinforcement is eliminated in the conventional design of power (pressure) tunnels. Reinforcement though increases the tensile strength of the concrete, it hampers the construction of a good dense cement concrete lining. Good and compact concrete capable of withstanding high velocities and abrasion is desirable (see Section 24.8).

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23.2 MINIMUM OVERBURDEN ABOVE A PRESSURE TUNNEL

It must be ensured in a pressure tunnel that the minimum in situ principal stress is more than the internal water pressure along the entire water tunnel. In other words, the overburden of rock mass should be more than the internal water head. According to field experience, the errors of surveying are higher in mountainous terrain because of many difficulties. As such, the depth of rock cover (*H*) cannot be estimated reliably. Re-surveying may be recommended in critical areas where overburden is not adequate.

Fig. 23.1 shows the overburden (*H*) which is perpendicular distance between a safe slope profile and the pressure tunnel. The following criterion should be considered for safety of the pressure tunnel,

$$p_i < \gamma \cdot H \cos \psi_f \tag{23.1}$$

where

p_i = internal water pressure,
= $\gamma_w H_w$
ψ_f = stable slope angle of the hill,
H_w = maximum head of water considering the effect of water hammer,
H = perpendicular distance between safe slope profile and pressure tunnel (Fig. 23.1),
> three times the diameter of the tunnel (to absorb vibration energy due to the water hammer during sudden closure of a pressure tunnel).

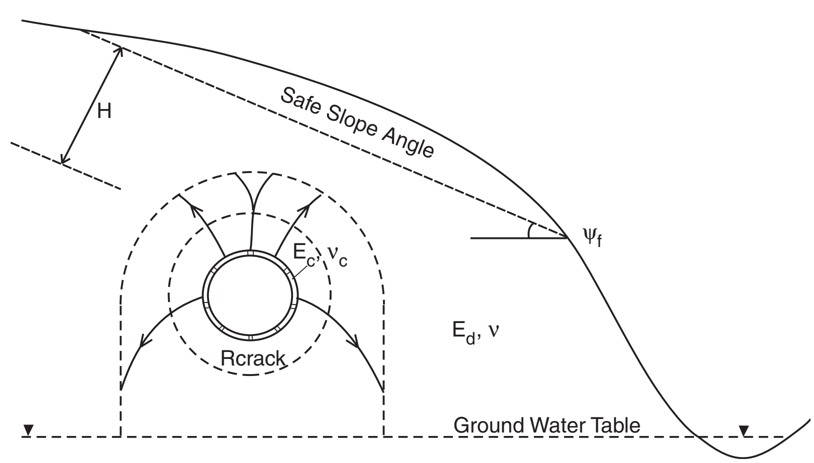

Fig. 23.1 Safe overburden above a pressure tunnel.

23.3 SOLID CONCRETE LINING

Jaeger (1972) derived an expression for stresses in the solid plain concrete lining within an isotropic, homogenous and elastic rock mass in plane stress condition. The solution for plain strain situation will be more realistic. The modified expression for rock sharing (reaction) pressure is given below (Kumar & Singh, 1990).

$$\lambda = \frac{p_c}{p_i} = \frac{2a^2(1-\nu_c)}{\left[(1+\nu)/(1+\nu_c)\right](E_c/E_d)\left(C^2-a^2\right)+(1-2\nu_c)C^2+a^2} \tag{23.2}$$

where

p_i = maximum internal water pressure,
p_c = support reaction pressure at the interface of lining and rock mass,
E_d = modulus of deformation of rock mass,
ν = Poisson's ratio of rock mass,
E_c = modulus of elasticity of concrete lining,
ν_c = Poisson's ratio of concrete lining,
a = internal radius of lining and
C = outer radius of lining.

The tensile stress within the lining is calculated by the elastic solution for thick cylinder. It should be less than the permissible tensile stress of the concrete. Hence rich concrete mix is used. A nominal reinforcement of 1.0 percent of volume of lining is provided to stop shrinkage cracks.

23.4 CRACKED PLAIN CEMENT CONCRETE LINING

A PCC lining for a water power tunnel is likely to crack radially at number of places where the hoop tensile stress exceeds its tensile strength (Fig. 23.1). In practice six construction joints are provided while concreting. These joints are also likely to open up due to internal water pressure. Further, cracks may also develop where the surrounding rock mass is poor. These radial cracks will be distributed nearly uniformly along the circumference due to good bond between concrete and rock mass.

Fig. 23.1 shows a crack pattern in a plain concrete lining. The actual number of cracks and the width of cracks may be smaller than that predicted due to percolation of water inside the rock mass through cracks. The number of cracks should be limited so that the length of the segment is approximately more than three times the thickness of the lining or about 1.75 m so that the segment is not eroded by the fast flowing water.

The spacing of cracks is likely to be uniform along the entire lining due to a built-in good bond between concrete and the rock mass. The spacing of cracks (S) is derived by

Singh et al. (1988a,b) as follows:

$$S = \frac{(f_t + p_i)(C - a)}{p_i} \tag{23.3}$$

where f_t is ultimate tensile strength of the concrete.

The average opening (u) of cracks is given approximately by the following equation (23.4).

$$u = \frac{(1 + \nu)(C - a)(f_t + p_i)}{E_d} \tag{23.4}$$

The lining is designed properly to ensure that the crack opening or width is within safe limit (<3 mm) and length of segments is more than three times the thickness of the lining or 1.75 m. This would ensure self-healing of the crack by precipitation of $CaCo_3$ etc. within cracks and the cracked segments will not be washed away by the water flowing with high velocity. In order to minimize the cracking of the lining, it is recommended that water pressure be applied to the tunnel lining slowly and not abruptly.

In case of PCC lining also, reinforcement must be provided in the lining (i) at the tunnel intersections, (ii) at the enlargements, (iii) at inlet and outlet ends, (iv) in plug areas, (v) in the areas where the power tunnel passes through a relatively poor rock mass and (vi) where the overburden pressure due to rock cover is inadequate to counter-balance the internal water pressure.

It may also be noted that the rock mass is saturated all around the lining as shown in Fig. 23.1 after charging of the water conductor system. In argillaceous rocks, this saturation reduces the modulus of deformation of the rock mass significantly. Consequently, high support pressures are developed on the lining after saturation of the rock masses (equation (24.8)). The worst condition of design occurs when the power tunnel is empty. Thus, the PCC lining must be able to support these unusually high support pressures as well as the ground water pressure, which is nearly equal to the internal water pressure in the tunnel. The elastic solution for thick cylinder should be used to calculate the maximum hoop (tangential) stress in compression within a lining, which should be less than the permissible compressive strength of the concrete. This criterion gives the minimum thickness of the PCC lining. The recommended factor of safety in hoop compression is 3.0 for PCC/RCC lining (Jethwa, 1981). *To make PCC lining ductile, nominal reinforcement of about 1 percent of volume of concrete is suggested so that mode of failure of lining is ductile and slow due to unexpected rock loads. Nominal reinforcement will also prevent shrinkage cracks in the concrete lining.*

It may be recalled that temporary support system for a power tunnel is designed by considering the existing ground water condition for rock mass quality Q. However, it is the post-construction ground water condition around a power tunnel which will govern the long-term support pressure even in non-swelling rock masses. Hence J_w in rock mass quality Q should be taken corresponding to the internal water pressure of power tunnel. There is no cause for anxiety as the extra long-term support pressure on the lining is negligible compared to the ground water pressure. Both pressures act simultaneously on the concrete lining.

The design methodology for PCC lining was developed by Singh et al. (1988a & b). Later Kumar and Singh (1990) proposed a design procedure for reinforced concrete lining for water/power tunnels. The program LINING has been developed on the basis of this research work (Singh & Goel, 2002). The program also calculates seepage loss through lining using the analytical solution (Schleiss, 1988). Concrete lining tunnels are also being used as canals. The seepage loss may be estimated by the expression developed by Swamee and Kashyap (2001).

Indeed a water conductor is never charged instantly as assumed in the design. The power tunnel is pressurized slowly. Thus, seepage takes place through construction joints into the rock mass. The seepage pressure tries to counteract the internal water pressure on the concrete lining. Consequently, the actual number of cracks are limited to construction joints mostly. The actual crack opening may be much less than that predicted by theory. As such, use of PCC lining may be encouraged in good and fair rock masses where overburden is adequate.

Table 23.1 summarizes case histories of various pressure tunnels in the hydroelectric projects in Himalaya, India, where PCC (M25) lining has been functioning successfully since 1980. Further contact grouting and consolidation grouting has been done around all these tunnels (see Section 28.11). It may be mentioned that Kopli tunnel failed because of inadequate overburden of 31 m to sustain very high internal pressure of 1.6 MPa.

The modern practice is to build PVC waterstops across the construction joints between segments of concrete lining (both PCC and RCC). Then joints are filled with bitumen. It should be checked that PVC waterstops are able to withstand the high internal water pressure. However, the construction of PVC waterstops requires skilled workers.

Gysel (2002) cited case histories where the water tunnels developed cavities around the lining in anhydrite karst rocks due to dissolution and erosion by seepage within 6 months. Anhydrite also created swelling pressure on the lining. The repair was done by adding steel liner and grouting the cavities. The same problem may arise in water-soluble rocks like salt and gypsum. It is, therefore, recommended that RCC lining should be provided across the soluble rocks and consolidation grouting is done thoroughly.

23.5 STEEL LINER IN PENSTOCK

The steel liner is provided in the underground penstocks which connect the power tunnel (head race tunnel, HRT) and the underground powerhouse cavity. The steel liner can sustain very high velocities of flow of water (1.6 to 9 m/s) and reduce the hoop tensile stresses in the surrounding PCC lining. The computer program LINING helps in calculating thickness of the steel liner and the spacing of stiffeners. The worst condition for steel liner is also the empty tunnel, as the seepage pressure of the order of the internal water pressure may act upon the steel liner. As such the stiffeners are provided to prevent the

Table 23.1 Details of PCC pressure tunnels for various projects in India (Singh et al., 1988).

S. No.	Project	Shape	Diameter in meters (2a)	Lining thickness at crown t (mm)	Water pressure (MPa)	E_d = Modulus of (rock) deformation (MPa)	Crack opening (mm)	Crack spacing $t = C - a$	No. of cracks
1.	Ram Ganga River Project	Circular	9.0	750	0.45	850–3500	3.1–0.8	$6.6t$	9
2.	Maneri Bhali Hydel Scheme Stage I	Circular but excavation of horseshoe shape	4.75	300–500	0.18–0.62	7500	0.1–0.25	$(14.6–5)t$	2–6
3.	Yamuna Hydroelectric Scheme Stage II (U.P.)	Circular	7.0	300–600	0.44–0.62	500–7000	2.1–0.3	$(6.7–5)t$	7–9
4.	Maneri Bhali Hydel Scheme Stage II	Horseshoe	6.0	300–500	0.15–0.35	3000–10,000	3.2–0.17	$(17.7–8.1)t$	3–5
5.	Tehri Dam Project - Head Race Tunnel	Horseshoe	11.0	375–900	0.4–0.6	500–3000	2.6–1.1	$(7.3–5.2)t$	10–14
6.	Tehri Dam Project - Head Race Tunnel	Circular	8.0	600	0.2–1.2	800–7000	2.4–0.4	$3.1t$	17
7.	Kopli Hydel Project in jointed granite and gneiss	Circular	4.5	150–200	1.6	570–1500	1.3–0.70	$2.6t$	11

steel liner from buckling (Timoshenko & Goodier, 1987). The steel liner is painted with anti-corrosive paint like epoxy paint. However, the minimum thickness of the liner (t_s) is $D/400$; where D is the diameter of the penstock. This will provide adequate stiffness to the liner which is required during its fabrication and handling.

It is assumed that concrete lining is cracked radially due to high internal pressure. There is a gap of Δ_c between steel liner and the concrete lining due to shrinkage of concrete, thermal effect and rock creep. But rock is not cracked radially due to lack of hoop tensile stresses. In this situation the reaction contact pressure (p_c) at the concrete-rock periphery is given by the following equation:

$$p_c = \frac{p_i \cdot a_s^2(1-\nu_s^2) - \Delta_c \cdot t_s \cdot E_s}{t_s \cdot E_s \cdot (1+\nu) \cdot \left(C/E_d\right) + (1-\nu_c^2) \cdot t_s \cdot \left(E_s/E_c\right) \cdot C \cdot L_n \left(C/a\right) + C \cdot (1-\nu_s^2) \cdot a_s} \tag{23.5}$$

where

t_s = thickness of steel liner,
E_s = modulus of elasticity of steel,
ν_s = Poisson's ratio of steel,
a_s = internal radius of the steel liner = $D/2$,
Δ_c = gap between steel liner and concrete lining and
p_i = maximum internal pressure inside liner, considering the effect of water hammer due to sudden closure of turbines.

Then hoop tensile stress in the steel liner is calculated. Adequate thickness of the liner is provided so that tensile stress is less than safe tensile strength of welded steel. Factor of safety of 1.7 is recommended. The liner should be anchored into the concrete lining. The diameter of anchors is generally 25–40 mm and its length is 30–50 cm. Suitable spacing should be adopted.

The thickness of the steel liner should be reduced within competent rocks naturally. But thickness of the liner in poor rock mass should be continued upto a distance of diameter ($D = 2a_s$) of penstock inside the adjoining competent rock. Thereafter, liner thickness is reduced in steps of 5 mm till smaller thickness required for competent rock mass is obtained. Where the liner emerges from the tunnel, it should be designed for maximum internal pressure and due care should be taken of stresses in the tunnel.

Finally contact grouting between concrete lining and rock mass (and also between steel liner and concrete lining) is executed at low pressure. This is followed by the consolidation grouting of the surrounding ring of the rock mass under high pressure (Section 28.11). Some experts recommend high grouting pressure to pre-stress the concrete lining. In view of the authors, pre-stressing is not needed where PCC lining is feasible. Vaidya and Gupta (1998) have reported failure of grout plugs in the steel liner due to the seepage pressure. The repair was done successfully.

23.6 HYDRAULIC FRACTURING NEAR JUNCTION OF PRESSURE TUNNEL AND PENSTOCK

The following caution should be kept in mind.

It is observed by Barton (1986) that a rock joint opens near the junction of unlined HRT and the steel lined penstock. The ground water table is very high above the HRT at the time of full head, but it drops suddenly above the penstock due to the impervious steel lining. Hence, consolidation of rock mass above the penstock may take place due to the drastic reduction in seepage water pressure. Consequently, this leads to the development of the horizontal tensile strain in the upstream adjoining rock mass around the HRT. Thus, a rock joint opens at this junction within the ungrouted rock mass and this fracture may propagate upto the top of the hill in some cases. This phenomenon of fracturing underlines the need for extensive consolidation and contact grouting of rock mass near this junction, specially in the case of unlined HRT.

The problem at junction is further complicated due to square shape of steel lining which cannot bear high outside water pressure. The building of steel liners of the two penstocks of the Pong Dam Project after reservoir filling may be due to the above reasons. This damage was repaired successfully (Oberoi & Gupta, 2000).

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24
Shafts

24.1 INTRODUCTION

Shafts refer to a vertical or near vertical excavation in rocks. But, some shafts are inclined depending upon its function. Shafts are used in tunnelling works. In general, shafts have the following functions.

(i) Tunnel shafts provide vertical access to the level of a tunnel or cavern for its construction. Tunnel shafts provide additional working faces for rapid excavation of tunnels.
(ii) Mining shafts for access of workers up to the mine face.
(iii) Surge shafts for absorbing excess energy of water hammer near penstocks in the hydroelectric projects.
(iv) Transformer shaft carries electrical cables from powerhouse to the transmission lines on the ground.
(v) Bunker shafts to connect underground tunnels for protection against atomic wars.
(vi) Ventilation shafts along the long tunnels and mines.

Tunnel shafts can be a temporary shaft which is used only for construction. Other shafts are permanent shafts with permanent support system.

The shaft should be located on ground having sufficient vacant area. The vacant space on the ground surface is required (i) for providing space for temporary buildings, (ii) to dispose off muck from the shaft and (iii) to discharge seepage water from inside the shaft, etc.

The depth of shaft varies with its purpose. Some shafts are very deep ($H > 1000$ m). With depth, the excavation and the supporting problem also increases. While excavating the shaft, management of ground water may also sometimes pose construction problems. Unexpectedly large inflows can occur while passing through water bearing strata. The pump should be of sufficient capacity to pump out maximum anticipated inflow of seepage.

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24.2 SHAPES OF SHAFT

The shape depends upon the function of shafts. Commonly three shapes are used.

1. *Circular shaft* – It is popular in civil engineering projects. It is used in mines also. It is a structurally stable section in weak rocks.
2. *Rectangular shaft* – It is used in mines generally.
3. *Elliptical shaft* – It is not used frequently.

Fig. 24.1 shows various shapes of shafts along with temporary and permanent support systems. Rock bolts or anchors are treated as temporary support system. In modern times, shotcrete may be considered as permanent support system. The minimum safe clear distance between vertical shafts may be taken as 1.5B in non-squeezing ground and 3B in the squeezing ground, where B is the diameter of the shaft.

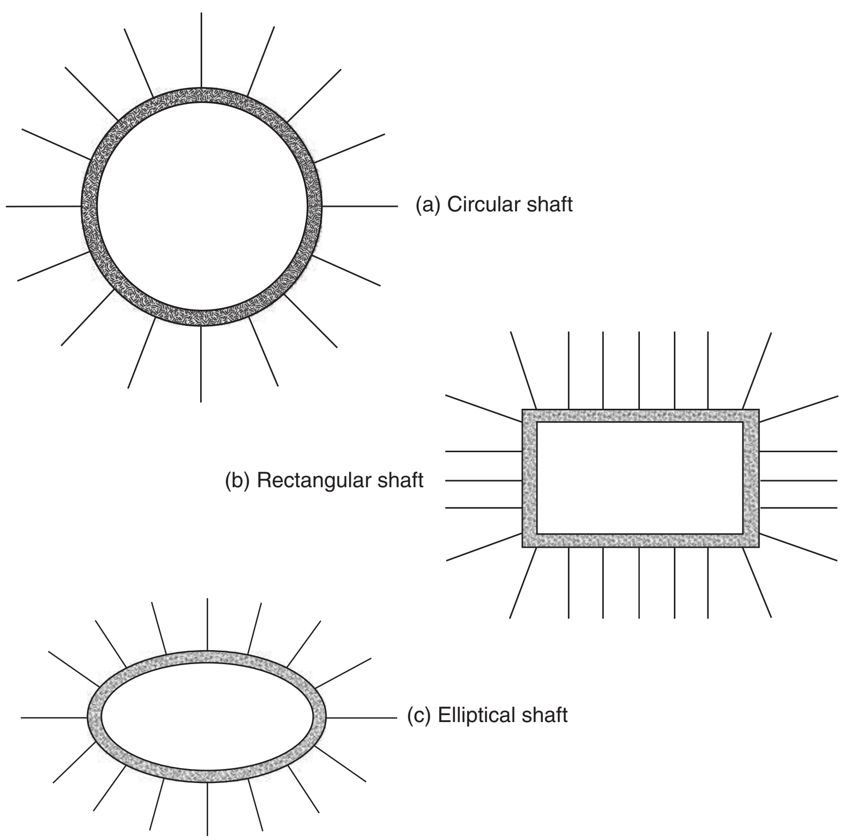

Fig. 24.1 Shapes of shafts in rock masses.

24.3 SELF-SUPPORTING SHAFT

Barton et al. (1974) obtained the following correlation for self-supporting shafts of diameter or width (B),

$$B = 2\,\mathrm{ESR}\ \mathrm{Q}_{\mathrm{w}}^{0.4}\ \text{meters} \tag{24.1}$$

It may be emphasized that $\mathrm{Q_w}$ is the actual rock mass quality for the walls in the case of shafts. The excavation support ratio (ESR) is as follows:

(i) Circular section – 2.5
(ii) Rectangular or square section – 2.0

General requirements for permanently unsupported shafts are,

$$J_\mathrm{n} < 9,\ J_\mathrm{r} > 1.0,\ J_\mathrm{a} < 1.0,\ J_\mathrm{w} = 1.0,\ \mathrm{SRF} < 2.5$$

Shafts up to 10 m depth generally do not require any supports. If depth of shaft H is less than $350\,\mathrm{Q}^{1/3}$ meters, non-squeezing ground condition is expected, same as in the case of tunnels. Fig. 24.2 presents the no-support size for the vertical shafts for any Q value directly. Thus, stability problems (or wedge failures, etc.) may be encountered only in poor to very poor rock qualities. It should be realized that $\mathrm{Q_w}$ is the actual rock mass quality for walls which can be altered significantly by stress conditions and deformations while excavating deep shafts (Kaiser & McCreath, 1994).

24.4 SUPPORT PRESSURES ON THE WALL OF SHAFT

The empirical theory of Barton et al. (1974) and its improvements suggested by Singh et al. (1992) is recommended for estimation of long-term support pressure (p_wall)

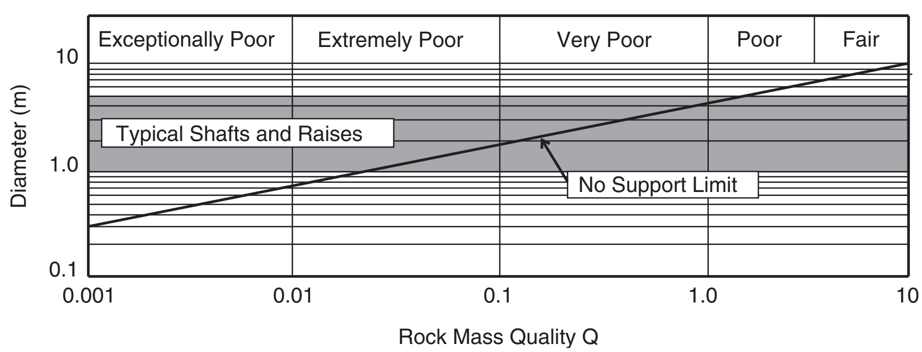

Fig. 24.2 No-support limit for vertical excavations adopted for wall conditions from Barton et al. (1974).

on the walls of a shaft as follows:

$$p_{\text{wall}} = \frac{0.2}{J_r} \left(\mathrm{Q_w}\right)^{-1/3} f \cdot f' \ \mathrm{MPa} \tag{24.2}$$

where

J_r = joint roughness coefficient,
f = correction factor for depth of shaft (H),
$= 1 + (H - 320)/800 \geq 1.0$, (24.3)
f' = correction factor for closure of wall of shaft in the squeezing rock-wall conditions and
= 1.0 for non-squeezing grounds ($H < 350\,\mathrm{Q}^{1/3}$ meters).

It is interesting to learn that p_{wall} is independent of width B and depth of the shaft less than 320 m (equation (24.3)). The rock mass quality in walls ($\mathrm{Q_w}$) is obtained after multiplying Q by a factor which depends upon the rock mass quality Q given below,

Range of Q	$\mathrm{Q_w}$
> 10	5 Q
0.1–10	2.5 Q
< 0.1	1.0 Q

Fig. 24.3 presents variation of wall support pressure with the rock mass quality directly (Kaiser & McCreath, 1994), for $H < 320$ m in non-squeezing grounds.

In the case of squeezing ground conditions under high overburden ($H > 350\,\mathrm{Q}^{1/3}$ meters and $J_r/J_a < 0.5$), the wall support pressure depends significantly on the wall closures. The correction factor f' for tunnels may also be adopted for shafts, as shown in Fig. 5.4b. It has been observed that p_{wall} decreases rapidly with increasing wall closures up to 4 percent of the diameter or width of the shaft. Therefore, stiff lining is not the solution for supporting shafts in the squeezing grounds like in the tunnels.

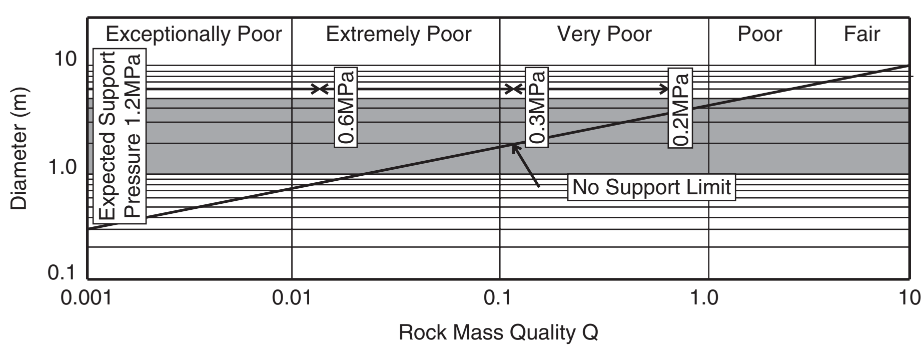

Fig. 24.3 Guide to estimate support pressures in vertical bored excavations (Golder Associates, 1976).

There may be slabbing of rock walls in the deep shaft in the over-stressed condition in good rocks where $H \gg 350Q^{1/3}$ meter but $J_r/J_a > 0.50$. Thus, in deep shafts ($H > 1000$ m), mild or moderate rock burst may take place. The wall support pressure is still governed by equation (24.2) with $f' = 1$. The ideal choice of support is full-column grouted rock bolts or un-tensioned resin anchors with SFRS. This will improve the ductility of reinforced rock arch. Thus, grouted rock anchors will tend to arrest the propagation of fractures, as cracks will not be able to open up due to effective rock reinforcement.

It has been experienced and which is logical also that the damage to rock mass due to blasting is less in shafts than in the roof of tunnels. Thus, overbreaks are of lesser magnitude in vertical shafts than in roof of tunnels in the same rock. Hence, blasting is done easily by vertical holes at the bottom of a shaft.

Shear zone in shafts does not create much problem as in tunnels. Needless to mention that proper average of Q values in shear zone and surrounding rock should be taken as mentioned in Section 28.7. Further, supporting water-charged strata is not much problem, as seepage water is collected at the bottom and pumped out of the shaft. Steel ribs may buckle in section of plastic gouge in a thick shear zone (>2 m) in a large shaft (>6 m diameter) even at shallow depths (<50 m) due to the squeezing of plastic clayey gouge. The buckling is found to stop after about a month. The buckled ribs should be replaced before concreting. In case the shaft is being excavated through the shear zone containing groutable material like river bed material, it is advisable to grout the area around the shaft before taking up the excavation. The grouting will help in increasing the stand-up time of the loose material and thereby reduce the excavation and supporting problems.

In case a shaft is bored by a machine, the support requirement would be much less. This is due to reduction in the damage to rock walls.

24.5 DESIGN OF SUPPORT SYSTEM

Chart of Grimstad and Barton (1993) in Fig. 10.2 should be used to recommend support system for rock mass quality in walls (Q_w). In hard rocks, rock bolts and/or anchors may be enough. Dip of boltholes should be 10° downward as in slopes. It will retain grout by force of gravity. Then anchors are pushed inside these holes. Weld mesh (6 mm diameter steel bars welded at center to center spacing of about 15 cm) should be tied in between the bolts to prevent rock falls in the shaft, if shotcrete is not used. Spot-bolting should be discouraged, as it does not form a structural system, except in a massive hard rock.

The length of pre-tensioned rock bolts is found as

$$l = 2 + (0.15B/\mathrm{ESR}) \tag{24.4}$$

The length of rock anchors is determined as follows:

$$l = 0.35B/\mathrm{ESR} \tag{24.5}$$

The thickness of shotcrete (t_{sc}) may be obtained by the following criterion:

$$t_{sc} = \frac{p_{wall} \cdot 0.6B}{2 \cdot q_{sc}} \tag{24.6}$$

where

$0.6B$ = distance between planes of maximum shear stresses in the shotcrete,
q_{sc} = shear strength of shotcrete,
= 3 MPa for conventional shotcrete and
= 5.5 MPa for steel fiber reinforced shotcrete (SFRS).

In modern times, shotcrete lining may be considered permanent support system. It is heartening to know that the thickness of concrete lining for permanent support system in mine shafts up to 4000 m depth is only 40 cm. The software package TM may also be used to design complete support system of rock bolts and shotcrete for complex geological conditions (Appendix II). Section 28.10.5 describes the use of steel ribs for supporting wide shafts.

The supports in the shaft and in the tunnel at the intersection of shaft with tunnel shall be designed after obtaining the Q value with modified J_n rating for intersections of two openings. In case of an unsupported tunnel, for long-term stability, as a rule of thumb, it is recommended that on either side of the shaft in the tunnel, the rock bolts (length and spacing of bolt as per the tunnel width) and 25 mm thick shotcrete support should be provided upto a distance equal to the diameter of the shaft. Similarly in the shaft, the rock bolt and shotcrete support system should be provided upto a distance equal to the width of the self-supporting tunnel.

24.6 SURGE SHAFT

The surge shafts are made above head race tunnel (HRT) to release energy of water hammer, when penstocks are shut down in a hydroelectric project. There is internal water pressure inside the surge shaft due to the effect of the water hammer when penstock is closed suddenly. There appears to be no harm if the concrete or shotcrete lining is cracked due to high hoop tensile stresses. The philosophy is the same as for the pressure tunnels (Chapter 23). However, the worst condition occurs when the shaft is empty and ground water pressure acts from outside on this lining. The required thickness of lining (t_c) works out to be approximately as follows.

$$t_c = \frac{B \cdot (p_w + p_s)}{2 \cdot f_c} \tag{24.7}$$

where

p_s = support pressure due to post-construction saturation,
p_w = ground water pressure,
≈ internal water pressure,
f_c = permissible compressive stress in the concrete and
B = span of opening of the shaft.

It is assumed that there is contact grouting of good quality between rock mass and concrete lining. So, no bending stresses will tend to develop inside the lining. It may be noted here that a good bond between rock and lining is the secret of success. Also, the contact grouting prevents damage of the concrete due to the vibrations from nearby blasting.

In water sensitive argillaceous rocks, the support pressure may increase after seepage of water into the rock masses as follows (Verman, 1993),

$$p_\mathrm{s} = \left(1 - \frac{E_\mathrm{sat}}{E_\mathrm{d}}\right) \cdot \gamma H - p_\mathrm{w} \tag{24.8}$$

where

E_sat = modulus of deformation of rock mass after saturation,
E_d = modulus of deformation of rock mass at natural moisture content,
γH = Overburden pressure at the point of consideration = horizontal in situ stress and
p_w = internal water pressure.

There may be construction difficulties in the excavation of shaft. Hill slope is cut or anchored for its stability so that a safe site is developed for excavation of (surge) shaft. In some projects, a pilot shaft of smaller diameter is made up to the level of HRT. Then full face of the surge shaft is excavated by drilling and blasting method. The muck falls down through pilot shaft inside the HRT. Then muck is transported out by the rail line. Of course manual shaft drilling should not be done from its bottom above a tunnel. Fatal accidents have taken place.

24.7 EXCAVATION

Excavation of shaft is usually done by drill and blast method in civil engineering projects. The tunnel shafts are generally less than 50 m deep. So machine boring is ruled out. Deeper shafts in hard rocks may be bored by machine. The drilling is done by hand-held drills to make vertical blast holes. The blasting system is designed for ease of drilling and minimizing overbreaks. The pyramid cut is used in circular shafts. The V-cut is preferred for excavating rectangular shafts (Jenny, 1982). After blast cycle is over, muck is taken up by cranes with bucket. The muck is dumped from the bucket on the ground or on the hopper. Prior to installation of support system, the loose rock pieces should be scaled down. In case ground water is seeping downwards, it should be pumped out by a pump of adequate capacity. Manual shaft drilling shall be avoided for safety in rainy season, as shear zone may be charged with rain water temporarily.

Recently, shafts are bored upwards from a tunnel to the ground level using a raise boring machine. The raise boring has been executed successfully in the hydroelectric projects for the following shafts of small diameter (Singh, 1993).

(i) Pressure shaft,
(ii) Spillway shaft,

(iii) Surge chamber,
(iv) Main inlet valve gallery relief,
(v) Draft tube valve gallery relief,
(vi) Ventilation and
(vii) Machine hall mucking, etc.

Singh (1993) has presented details of machines for raise boring and shaft sinking and their practical utility.

24.8 SELF-COMPACTING CONCRETE (KAUSHIK & KUMAR, 2004)

Normal cement concrete depends heavily on its degree of compaction for its performance, viz. strength and durability. In some locations, when either the thickness of the structural element is very small or congested reinforcement makes it difficult to facilitate proper compaction through internal vibrators, etc., the concrete may not be able to last for its designed life. Such situations are frequently encountered in the field, say in case of tunnel and shaft construction. In some situations, the concrete may be subjected to a humid environment (or sometimes, alternate wetting and drying conditions); resulting in rapid deterioration of the insufficiently compacted (porous) concrete.

Rheo-plastic concrete requiring very little vibration to get compacted can be advantageously used in such situations. Also, self-compacting concrete (SCC) was developed in Japan in late 80s. In tunnel construction and rehabilitation works, SCC has started gaining acceptance. In one notable project "Trans-Kawasaki Route" in Japan, a tunnel structure "Daishi-junction" is included, where SCC has been used with MMST method of construction. This tunnel has been constructed by connecting unit tunnels of steel segment construction through joint members and then filling with SCC to unify them into a large section tunnel. A tunnel rehabilitation project in Zurich, Switzerland in 2001 employed a concrete volume of 7000 cubic meters with SCC to get durable concrete in sections ranging from 10 to 6 cm thickness.

In SCC, full compaction of concrete is attained with its self-weight only. Such concrete fills spaces between the reinforcing bars and formwork completely without any vibration. The concrete is made up of usual ordinary Portland cement and normal coarse aggregates and sand. However, it needs a higher powder content, so either flyash, ground granulated blast furnace slag or limestone powder may be used as powder material. Super-plasticizer is required to get high flowability of concrete. For this polycarboxylate-ether-based super-plasticizers have been observed to work better than either naphthalene based or melamine based ones. The use of zero energy admixtures facilitates early hydration and stripping of formwork may be resorted to 8 h. To control segregation at high flowability, viscosity modifying agents (VMA) are used. VMA addition makes the concrete mix stable. The aggregates remain suspended in the viscous mix or mortar. SCC mixes are cohesive due to the large powder content.

Flyash is available widely and is an economical material. SCC mixes can be produced at site having a 28 day compressive strength in the range of 40–60 MPa. Due to flyash addition, the flexural strength of such concretes is generally 10 to 25 percent higher than the normal concrete with similar compressive strength. However, compatibility of the particular brand of cement with the super-plasticizer proposed to be used, needs to be established first. The concrete needs to be cured in a similar fashion as the normal concrete, but here, moist curing in the initial period is crucial for initiating the pozzolanic reaction of the flyash. Finishing of SCC is smooth and the permeability of the SCC is lower than the normal concrete of similar grades.

Thus use of SCC may result in better concrete practices in tunnel linings, shafts and related applications where the thickness is small and proper compaction of conventional concrete may be a nightmare.

A typical SCC mixture would be approximately as follows:

- Cement – 400 kg/m^3
- Flyash – 200 kg/m^3
- Fine aggregate (sand) – 800 kg/m^3
- Viscosity modifying agent – 3 to 6 kg/m^3
- Coarse aggregate – 700 kg/m^3
- Super-plasticizer – 8 to 10 kg/m^3
- Water – 180 kg/m^3

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25
Half tunnels

25.1 INTRODUCTION

Half tunnels are excavated as overhangs of hard rock slopes along the hill roads with one wall on the hill side and no wall on the valley side. These half tunnels have been existing since 1980, in spite of no support of the roof in the middle and higher Himalaya. Half tunnels, which are excavated as overhangs of hill slopes are superior to the conventional full tunnels or open road excavations, because they involve very less cost and time. However, due to lack of focus and their uncommon occurrence, the domain of half tunnel remained by and large unexplored. A photograph of half tunnel in Fig. 25.1 shows how exactly it looks like.

In addition to the economy of construction, half tunnels are attraction to tourists and help in the preservation of ecosystem due to minimum disturbance to the slopes. Anbalagan et al. (2003) have reported a detailed study which is summarized here.

25.2 APPLICATION OF ROCK MASS CLASSIFICATION

The rocks exposed at different sites have been studied and individual rock parameters are evaluated. The descriptions and corresponding ratings of the parameters and the final Q value for all the half tunnels are given in Tables 25.1A and 25.1B. The Q values for the rocks of the half tunnel vary from 18 to 38 indicating that the rocks fall under the category of good rock (Barton et al., 1974).

Similarly, the rocks are also studied for evaluating the parameters related to rock mass rating (RMR) (Bieniawski, 1989). The description and the corresponding rating of the individual parameters as well as the RMR_{basic}, i.e., RMR without adjustment for joint orientation, of rocks of half tunnels are given in Tables 25.2A and 25.2B. The RMR_{basic} values for half tunnels vary from 74 to 87 indicating that the rocks fall under the categories of good to very good rocks. The shear strength parameters were estimated according to the RMR_{basic}. These parameters were used in the stability analysis of the rock wedge if any.

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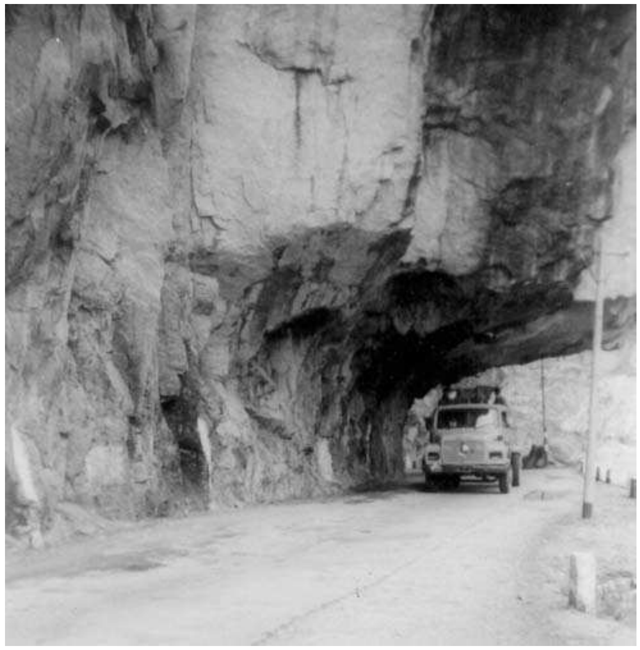

Fig. 25.1 Photograph of a half tunnel (Anbalagan et al., 2003).

The rock mass quality Q values have been plotted against span of the half tunnels on a log–log graph (Fig. 25.2). On the basis of the analysis of upper bound limit, the following relation is obtained between Q and span B_{ht} (in meters) of half tunnels.

$$B_{\mathrm{ht}} = 1.7\,\mathrm{Q}^{0.4} \tag{25.1}$$

This relation is comparable with the one given by Barton et al. (1974) between Q and maximum unsupported span (B) for ESR = 1.

$$B = 2 \times \mathrm{Q}^{0.4} \tag{25.2}$$

25.3 WEDGE ANALYSIS

Fig. 25.3 shows a typical rock wedge above half tunnels. The stability of half tunnels has been checked through wedge analysis. The analyses have been done both with the help of the computer program UWEDGE (Singh & Goel, 2002), considering the slope face as artificial frictionless joint and stereographically through Markland Test. Half tunnels M1, M2 and M3, where only one joint set was observed, were not analyzed by the program because the cases with number of joint sets less than two are automatically considered

Table 25.1A Descriptions, ratings of parameters and Q values of half tunnels in massive granite gneiss rocks (Anbalagan et al., 2003).

Half tunnel no.		T1	T2	T3	T4	T5	T6	T7
RQD	Description	Excellent	Excellent	Excellent	Good	Excellent	Excellent	Good
	Rating	95	95	98	98	98	98	95
J_n	Description	2 + Random	2 + Random	2 + Random	2 + Random	2 + Random	2 + Random	2 + Random
	Rating	6	6	6	6	6	6	6
J_r	Description	Irregular	Irregular	Rough	Rough	Irregular	Rough	Irregular
	Rating	3	3	3	3	3	3	3
J_a	Description	Unaltered	Unaltered	Stained	Unaltered	Stained	Unaltered	Stained
	Rating	1	1	1	1	1	1	1
J_w	Description	Minor inflow	Minor inflow	Minor inflow	Dry	Minor inflow	Dry	Minor inflow
	Rating	1	1	1	1	1	1	1
SRF	Description	Low stress	Low stress	Low stress	Low stress	Low stress	Low stress	Low stress
	Rating	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Q Value		18	19	19	20	20	20	19

Table 25.1B Descriptions, ratings of parameters and Q values of half tunnels (Anbalagan et al., 2003).

Half Tunnel No.		K1	K2	K3	M1	M2	M3	P1	P2
Rock type		Massive foliated micaceous quartzite			Massive white quartzite			Massive foliated slaty phyllite	
RQD	Description	Excellent	Excellent	Excellent	Excellent	Excellent	Excellent	Good	Good
	Rating	92	95	95	88	95	92	85	82
J_n	Description	2 + Random	2 + Random	2 + Random	1 + Random	1 + Random	1 + Random	2 sets	2 sets
	Rating	6	6	6	3	3	3	4	4
J_r	Description	Rough	Rough	Irregular	Rough	Rough	Irregular	Irregular	Irregular
	Rating	3	3	3	3	3	3	3	3
J_a	Description	Unaltered	Unaltered	Unaltered	Unaltered	Stained	Unaltered	Stained	Stained
	Rating	1	1	1	1	1	1	1	1
J_w	Description	Minor inflow	Dry	Dry	Dry	Dry	Dry	Minor inflow	Minor inflow
	Rating	1	1	1	1	1	1	1	1
SRF	Description	Low stress	Low stress	Low stress	Low stress	Low stress	Low stress	Low stress	Low stress
	Rating	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Q Value		18	19	19	35	38	37	25	24

Table 25.2A Descriptions, ratings of parameters and RMR values of half tunnels in massive granite gneiss (Anbalagan et al., 2003).

Half tunnel no.		T1	T2	T3	T4	T5	T6	T7
UCS	MPa	127	127	127	127	127	127	127
	Rating	12	12	12	12	12	12	12
RQD	%	92	95	95	98	98	98	95
	Rating	20	20	20	20	20	20	20
Spacing of disc.	Description	280 mm	300 mm	300 mm	280 mm	310 mm	300 mm	280 mm
	Rating	10	10	10	10	10	10	10
Condition of disc.	Description	Slightly rough surfaces, sep. <1 mm	Very rough surfaces, tight	Slightly rough surfaces, sep. <1 mm	Slightly rough surfaces, sep. <1 mm	Very rough surfaces, tight	Slightly rough surfaces, sep. <1 mm	Slightly rough surfaces, sep. <1 mm
	Rating	25	30	25	25	30	25	25
Ground water condition	Description	Damp	Damp	Damp	Dry	Damp	Dry	Damp
	Rating	10	10	10	15	10	15	10
RMR_{basic}		77	82	77	82	82	87	77

Note: "disc." stands for discontinuity and "sep." means separation.

Table 25.2B Descriptions, ratings of parameters and RMR values of half tunnels.

Half tunnel no.		K1	K2	K3	M1	M2	M3	P1	P2
UCS	MPa	205	205	205	212	212	212	151	151
	Rating	12	12	12	12	12	12	12	12
RQD	%	92	95	95	88	95	91	85	82
	Rating	20	20	20	17	20	20	17	17
Spacing of disc	Description	250 mm	270 mm	250 mm	300 mm	280 mm	300 mm	250 mm	250 mm
	Rating	10	10	10	10	10	10	10	10
Condition of disc.	Description	Slightly rough surfaces, sep. < 1 mm	Slightly rough surfaces, sep. < 1 mm	Slightly rough surfaces, sep. < 1 mm	Very rough surface, no spacing	Very rough surface, tight	Very rough surface, tight	Slightly rough surfaces, sep. < 1 mm	Slightly rough surfaces, sep. < 1 mm
	Rating	25	25	25	30	30	30	25	25
Ground water condition	Description	Damp	Dry	Dry	Dry	Dry	Dry	Damp	Damp
	Rating	10	15	15	15	15	15	10	10
RMR_{basic}		77	82	82	84	87	87	74	74

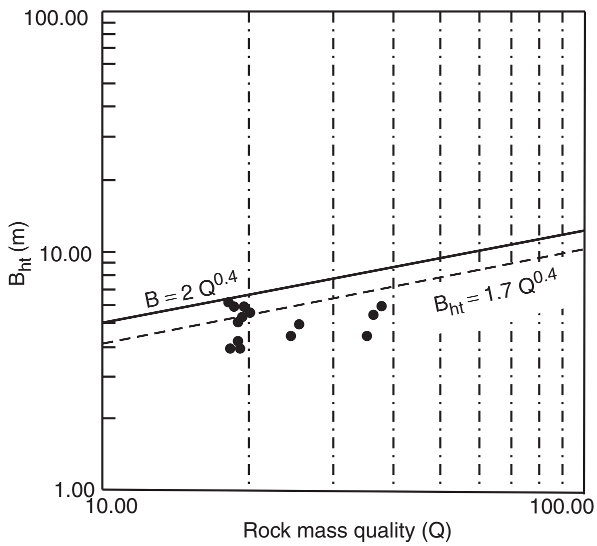

Fig. 25.2 Plot of rock mass quality Q vs span of half tunnel B_{ht}.

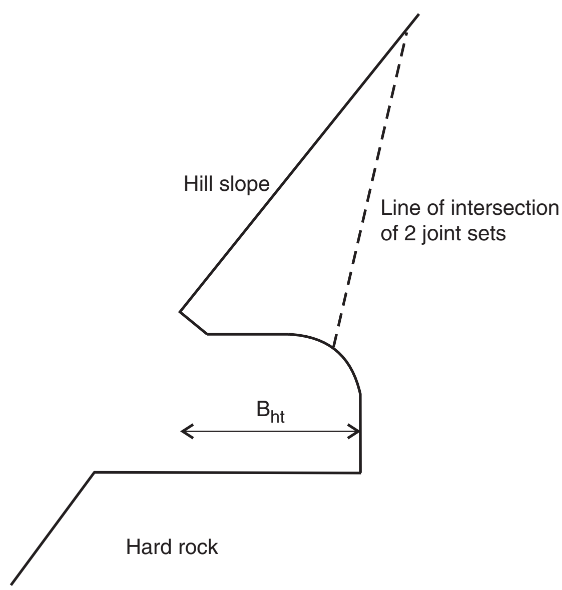

Fig. 25.3 Half tunnel along hill roads in hard rocks.

Table 25.3 Factors of safety for half tunnels based on wedge analysis.

Half tunnel no.	K1	K2	K3	P1	P2	T1	T2	T3	T4	T5	T6	T7
Factor of safety	25.8	23.0	29.8	9.44	62.0	8.65	7.73	3.72	4.55	3.25	3.12	3.25

stable wedges. The factors of safety of the wedge analysis for the half tunnels are shown in Table 25.3. The values range from 3 to 62.

25.4 STRESS DISTRIBUTION AROUND HALF TUNNEL

Results of the finite element analysis indicate the presence of compressive horizontal (normal) stress in the floor of the half tunnel. Magnitude of the horizontal (normal) stress increases along the floor of the half tunnel face towards the side wall. Horizontal stresses near the slope-floor junction vary from 0.01 to 0.06 MPa (compressive) for various cases. It varies between 0.31 to 0.93 MPa (compressive) near the wall-floor junction. On the roof, the horizontal (normal) stress decreases towards the crown and becomes tensile near the crown. On the roof, its magnitude varies from 0.06 to 0.47 MPa (compressive) near the roof-wall junction and it varies from 0.01 to 0.06 MPa (tensile) near the crown. On the wall the horizontal (normal) stress decreases towards the roof. It varies from 0.18 to 0.46 MPa (compressive) near the floor junction and it varies between 0.05 and 0.30 MPa (compressive) near the roof junction.

The maximum vertical tensile stress is observed near the crown of half tunnel about 0.4 m above the roof surface. Its variation ranges from 0.026 to 0.13 MPa for various cases. The variation of maximum tensile stress with span is shown in Fig. 25.4. Its magnitude is $1.1\gamma B_{ht}$ to $1.5\gamma B_{ht}$, where B_{ht} is half tunnel span. The value decreases with the flattening of the slope. The tensile stresses have also been observed at the junction of the floor and the slope face. However, it becomes insignificant with the flattening of the slopes. The maximum tensile stress in the roof is much less than the tensile strength of discontinuities (equation (8.21)). Therefore, it may be concluded that construction of half tunnels in hard and steep slopes is safe provided the detailed study of discontinuity confirm their stability. Also at the time of construction if any loose blocks are met with, they should be scaled down or spot-bolted.

These half tunnels had saved ecological disturbance because near vertical cut slopes would be very costly and ecologically unsound. Looking at the definite benefits that the half tunnels promise, exhaustive efforts should be made to plan them in narrow valleys with steep slopes characterized by massive and hard rocks.

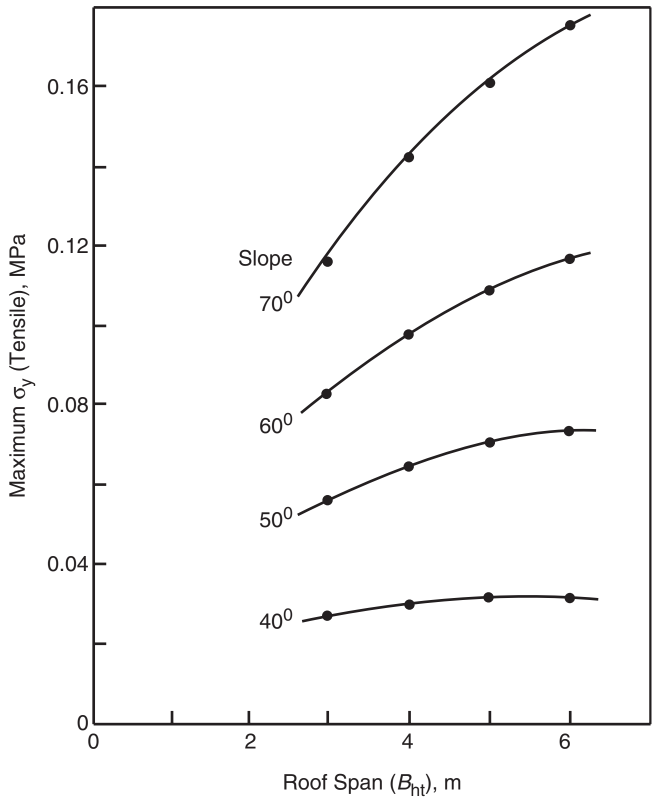

Fig. 25.4 Variation of maximum vertical tensile stress in roof with slope and span.

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26
Contractual risk sharing

"Engineers have to take a calculated risk, persons become wiser after an accident. If they were really wise, it was their duty to point out mistakes in the design to engineers."

Karl Terzaghi

26.1 THE RISK

Risk is a basic element of life. Life without risk is inconceivable and undesirable. If all risks were eliminated, the construction industry would cease to evolve. With the hope of greater profit, the contractor develops a new method accepting the risk that the method may not result in loss of anticipated profit. Without this element of risk he would lack the initiative and the incentive to select new techniques or execute them. The owner also assumes some risk when sponsoring a project, which may result in nullifying the projected benefits. It is well known, No risk! No gain!

While many risks in construction are inevitable, not all. Careful, thorough and detailed planning and engineering analysis will identify most of them and ways can be devised for avoiding some (the known) and lessening the trauma from those that are expected but cannot be foreseen clearly enough to avoid completely (the unknowns, the inherent uncertainties). The greatest need for sharing of risks is for occurrences that are not expected. The execution of these plans through the construction phase must be directed towards decisive action that will meet the planned objectives including the management of uncertainties and the risks. Contingency plans and the methods for managing risks must be kept up-to-date and revised to meet the actual situations/hazards which arise, in consultation with all contractors.

Earlier, risks in tunnelling were smaller and lesser and could be more easily classified and borne or handled by the various participants in a more equitable manner. But today's huge complex and imbalanced risks cannot be borne solely by one of the partners. Hence, means for allocating and sharing these risks should be evolved for the common good of the project construction organizations and its beneficiaries. The actual size and probability of the risk involving cost, time, credibility, reputation and ability to perform are unknown

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but real. The existence and impact of these risks should be appreciated and means of mutual benefit found. Most risks are evaluated, minimized or eliminated and the cost of doing this should be compared with an assessment of the original risk.

In too many instances the scramble to avoid risks by throwing them back and forth has made the construction scene a battlefield for lawyers, rather than an opportunity to accomplish useful and lasting works. The interests of the construction industry would be well served if more attention were directed to create a construction team composed of owners, engineers, contractors, geologists and insurers, each contributing their special expertise to solve common problems in the underground construction and each sharing risks related to their capabilities (Kuesel, 1979).

Contributions have been made by many countries towards the object of defining the sources of risk in a contract and in establishing how best, in the interest of the common good, these are shared among the parties concerned. The latter, sharing of risks, which builds upon practice in the UK and largely accepted in Austria and other European countries has a number of essential features, the most important of which are:

(i) Generally attribute acceptance of risk to the party best able to control its incidence (contractor) or, for minor risk, to make reasonable provision for its cost,
(ii) Provide appropriate encouragement to use methods of construction that show best prospects, in the available knowledge at any time, of an economic result,
(iii) Provide appropriate flexibility for change in construction methods to follow the range of variation in ground and other conditions foreseeable by a knowledgeable engineer and
(iv) Simple and equitable arrangements for disposal of disputes.

The US National Committee on Tunnelling Technology has given recommendations on better contracting for underground construction which include:

(i) Sharing of risks and their costs between the owner and the contractor. The risks are both construction risks and financial,
(ii) Handling of claims are required to be expedited,
(iii) Innovation in construction should be stimulated,
(iv) The award of work to the qualified contractor should be assured and
(v) Cost savings by other means should be realized (NAS, 1976).

The need for better management and better contracting in underground construction in US has been elaborated by Tillman (1981). Contracting practices for tunnelling have been discussed in fair detail by Bhat (1986). Muirwood and Sauer (1981) describe (i) the managerial principles for economic tunnelling resulting in a cheaper, faster and more reliable project to the owner; (ii) greater scope for the ingenuity of the engineer and (iii) the contractor with greater confidence for a fair return for his skill and resources.

Contracting practices in European countries and USA are compared by Ribakoff (1981). He concluded that a successful contract for both owner and contractor is the product of a marriage between good contracting practices and good management organization.

Samelson and Borcherding (1980) examined several barriers to productivity described by foremen from five different construction sites as:

(i) waiting for decisions,
(ii) waiting for materials and tools and
(iii) rework.

In Japan (Paulson & Akit, 1980), decisions are made by consensus approach. Though it takes time to achieve consensus; but once achieved it assures total commitment to the successful outcome of the decision, and implementation is almost assured. It is heartening to know that some corporations act like parents to their workers and their beloved children. May God prosper their love!

The International Tunnelling Association (ITA) Working Group on Contractual Sharing of Risks, in cooperation with the International Federation of Consulting Engineers (FIDIC) is preparing a standard contract for tunnelling work. The assessment of risk and its sharing in tunnelling has been brought out by Duddeck (1987). He discussed three categories of risks – functional, structural and contractual – and how they relate specifically to the design and construction of underground openings. He stressed for an urgent need for improved methods of risk assessment because the causes of functional and structural failures are complex and often interrelated. He proposed a number of recommendations concerning risk assessment.

Equitable sharing of risks means that the party bearing a greater part of the risk should be entitled to a greater share of the benefits or profits and the other parties should have no objection to it. If the sharing of risks is not equitable then there would be an imbalance between the risks actually borne and the profits made by the parties which may lead to disputes or litigation and consequently to delays and a higher project cost.

By including a clause covering adjustment in unit price for unknown conditions, the contractor is not tempted to escalate his item rates to cover the risk of adverse underground conditions. Full disclosure of all subsurface data available with the owner/department to the tenderers/bidders may lead to lower contract cost.

Disclaimer clauses relieving the owner of responsibility for the accuracy of the underground data furnished should be deleted. If the disclaimer clauses cannot be eliminated completely from a contract at least their number should be reduced to minimize malpractices. The absence of "changed conditions" provision in a contract will induce the contractor to put a contingency amount in his bid. So incorporation of this clause is beneficial to the owner (Ribakoff, 1981). ITA recommends that a changed condition clause be incorporated in all tunnelling contracts.

Departments should seek bids only from contractors having rigorous technical and financial pre-qualification. It has now been realized that pre-qualification of bidders is as much a part of the construction as selecting a suitable contractor. The practice of calling pre-qualification tenders by prospective bidders is being adopted in new projects. Authority to settle claims, commensurate with the scope of the project, should be delegated to both the representatives of the owner and of the contractors in the field. The decision of whether to use wrap-up insurance should remain with the owner.

A few sub-clauses under changed conditions clause as described below are suggested for the inclusion in tender/contract documents, of tunnelling contracts globally if they have not been considered by the owner.

> *"(a) The Contractor shall promptly, and before such conditions are disturbed, notify the engineer-in-charge in writing of: (i) subsurface or latent physical conditions at the site differing materially from those indicated in the contract, or (ii) unknown physical conditions at site, of an unusual nature, differing materially from those ordinarily encountered and generally recognised as inherent in work of the character provided for in this contract. The engineer-in-charge shall promptly investigate the conditions, and if he finds that such conditions do materially so differ and cause an increase or decrease in the contractor's cost of, or the time required for, performance of any part of the work under this contract, whether or not changed as a result of such conditions; an equitable adjustment shall be made and the contract modified in writing accordingly."*

> *"(b) No claim of the contractor under this clause shall be allowed unless the contractor has given the notice required in (a) above, provided, however, the time prescribed therefore may be extended by the government or the agency executing the contract."*

> *"(c) No claim by the contractor for an equitable adjustment hereunder shall be allowed if asserted after final payment under this contract."*

The Norwegian practice of risk sharing in tunnelling contracts has proved successful, in that 80 percent of their proposed 2600 km of tunnels have been driven with equivalent time risk sharing built into the contracts. No disputes with relevance to changed ground conditions have been reported in the period after the risk sharing provisions were accepted in their contracts (Kleivan & Aas, 1987).

Sharing of risks in tunnelling contracts and management of risks have been discussed by Badarinath et al. (1988, 1989). A survey of opinions of tunnelling experts in Himalayan projects indicated a low priority to sharing of risks whereas the ITA has realized its importance and brought out recommendations on sharing of risks. Crisis decision analysis is encouraged at the project site.

Risk is defined as "the possibility of loss, injury, disadvantage or destruction." That is, risk is an adverse chance. It is necessary to have information as to how the problems arise and with whom, what is the nature of the risks and how to alleviate them. Risks which are either undefined or unrecognized prior to the award of a contract cause much grief later. Owners or government departments should realize that a fair contract with equitable sharing of the risks according to the ability to assess and manage them would lead to earlier completion dates at lesser costs. The current contracting practices lead many tunnelling projects to wind up with tremendous increases in estimated cost, financial disasters, disputes and litigation. The situation is aggravated by the energy crisis, economic uncertainty, crisis of terrorism and shortage of materials and equipment. At the same time, if our industries are to develop their maximum technological potential, we must employ contracting practices which will encourage development.

Risks in underground construction are related to a number of factors listed below:

1. Acts of God
2. Accidents
3. Acceleration or suspension of work
4. Agencies involved
5. Allocation principles of risks
6. Costs
7. Construction and construction failure
8. Contract
9. Contractor/owner inherent
10. Changed conditions
11. Defective design/work
12. Decisions
13. Delays
14. Data
15. Disclosures of information
16. Disclaimers
17. Design of supports
18. Deductions
19. Economic disasters
20. Environmental
21. Evaluation
22. Escalation
23. Equipment
24. Funding and financial failure
25. Groundwater
26. Inflation
27. Innovation
28. Information
29. Insurance
30. Investigation
31. Labor
32. Materials
33. Management
34. Managerial competence
35. Physical risks
36. Political and social
37. Public disorder
38. Planning and scheduling
39. Pilot works
40. Quantity variations
41. Related to capability of individuals
42. Regulations
43. Reimbursements
44. Resolving problems
45. Responsibilities
46. Site access
47. Subsurface conditions
48. Subcontractor failure
49. Shared risks
50. Sociological problems
51. Support systems
52. Third party delays
53. Union strife, and
54. Water problems

26.2 MANAGEMENT OF RISK

Timely release of funds by the governments/departments would serve as a morale booster for the contractor. Delay in running payments to the contractor affects the workmen which certainly tell upon their efficiency. This is specially true in the dishonest poor societies. Total commitment of executives increases the confidence of contractors and reduces accidents. If payment to the contractor is made quickly, tunnelling will be faster naturally, due to quick reinvestment.

The subject of risk involves responsibility, liability and accountability. The basic principle of risk relationships is that the party taking the risk (contractor) should assume the liability and either suffer consequences or reap the benefits therefrom, depending on the outcome of the endeavor (Egbert, 1981). An instrumentation program to probe strata in advance of tunnelling in poor rocks and to study the adequacy of supports would result in safety and economy.

Team spirit is very much lacking in poor countries in the government departments, because of lack of mixing of top executives among junior staff and workers and a rigid hierarchy. Very few top officers associate themselves with the subordinates and their problems. *The spirit of mutual trust and benefit is very important in the risk management.*

In long tunnels, a number of contracts should be awarded for different reaches and lengths of tunnels to introduce an element of fair competition and encourage better performance. Another factor – energy management has nowadays become quite important in view of the monopoly of oil rich nations. Moreover, these days top security should be provided to all engineers and contractors against terrorists.

Fig. 26.1 represents the risks and risk sharing in tunnelling contracts. The risks in tunnelling contracts are related to the 19 factors recommended by the ITA. The risks inherent in these factors should be shared equitably between the contractor, owner, engineer, geologist and the insurer.

The engineers should be bold and try to take risk of new technologies (of internationally reputed corporations). The contract should include (i) clause for compensation to contractor for an unexpected geological conditions or surprises, (ii) clause on innovations by contractors and engineers on the basis of mutual agreements, (iii) clauses for first and second contingency plans for the preparedness and (iv) penalty for delays in construction. Obviously, contract is not a licence for injustice to any party. Injustice done should be corrected soon. The principle of crisis management is that one should not panic and one should not spread panic. The right persons at right places contribute to success of projects (according to Dr. V.M. Sharma).

The experience is that number of disputes increased rapidly, as the number of clauses increased in a contract beyond a certain limiting number according to Prof. J.J.K. Daemen. *There was increase in laws after every underground disaster but it was counter-productive. A judicious liberty is essential for increasing efficiency of an organisation.*

Our organising ability is increasing automatically and cyclically with time due to the law of negative bioentropy (Singh & Gupta, 2003). Conflicts are beneficial in increasing our inner strength.

Fig. 26.2 is a conceptual model of risk sharing. The clauses or provisions in a tender/contract document will either benefit or adversely affect the interests of the persons involved in any underground construction, namely, contractor, owner, engineer, geologist and insurer. The parties (involved) share the risks inherent in underground construction in different proportions. For equitable sharing of risks, the party taking the greater portion of the risk/s should be entitled to a greater share of the benefits or profits due to increased costs. If the profits to a party are not commensurate with the amount of

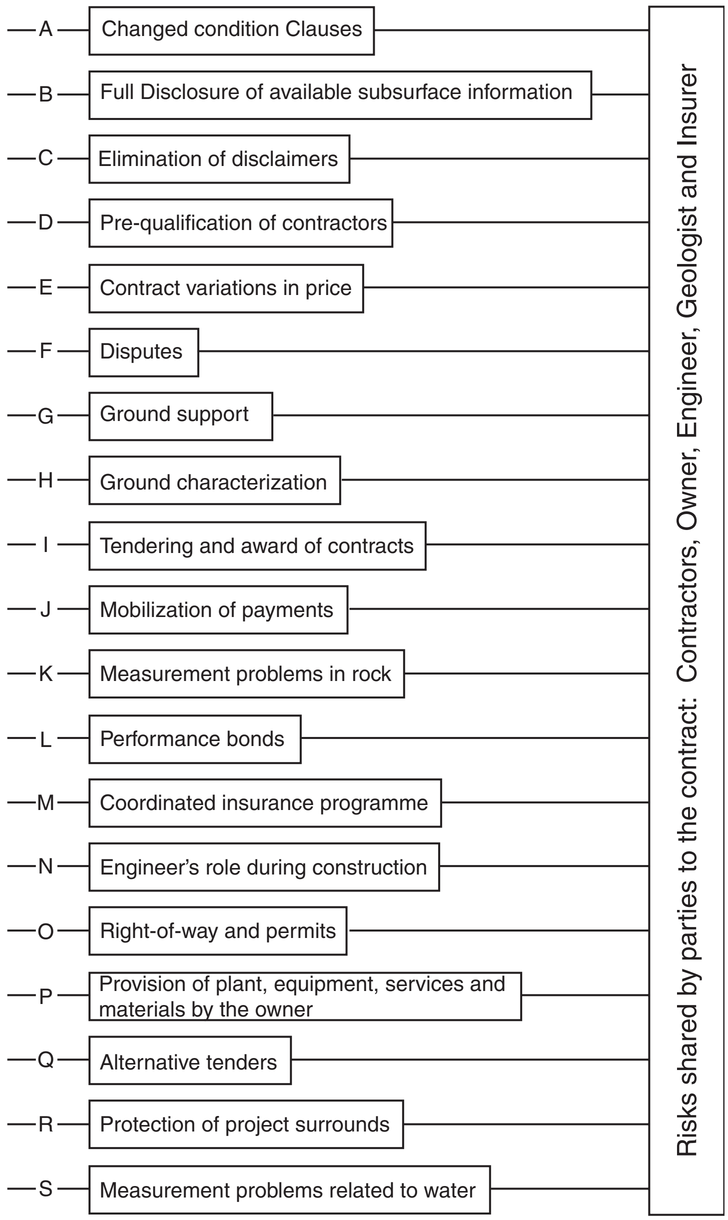

Fig. 26.1 Risks and risk sharing in tunnelling contracts (ITA, 1988).

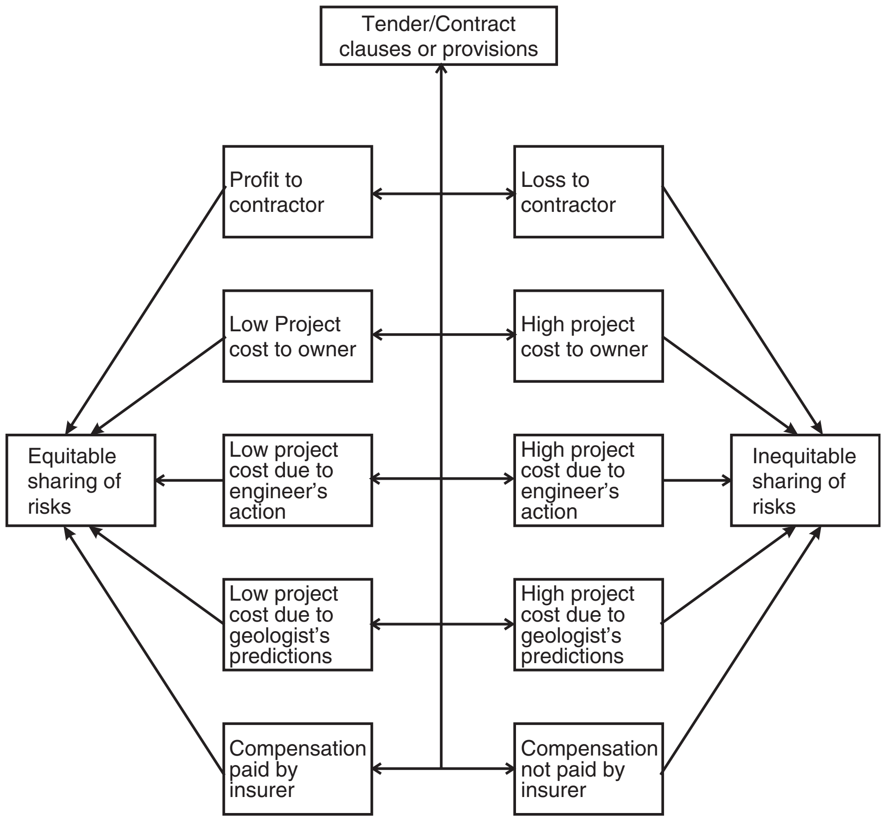

Fig. 26.2 Conceptual model of risk sharing (Badrinath, 1991).

risk taken by it, it will be inequitable sharing of risks. Badrinath (1991) has developed an expert system considering the five parties mentioned above. This software reveals whether a tunnelling contract has the risks of the project justly and fairly shared between the owner, engineer, contractor, insurer and the geologist responsible for execution of the project. This expert system may be used to educate construction engineers and managers to improve the contract documents for mutual benefit of all concerned. Mutual benefit and trust may decrease the heavy size of contract for tunnelling these days.

Fig. 26.3 shows the relation between risk sharing and contract types. The types of contracts could be turnkey, lump sum with fixed price, lump sum with price escalation, measurement of items, target amount and cost reimbursement contracts. Each of these types has the risks shared between the owner and the contractor complementary to each other (Kuesel, 1979). Another type of modern contract is BOT (build, operate and transfer to owner).

The features of minimizing project cost (Sutcliffe, 1972) are shown in Fig. 26.4. The total cost of the tunnel is a function of the economic factors and risk sharing. If the

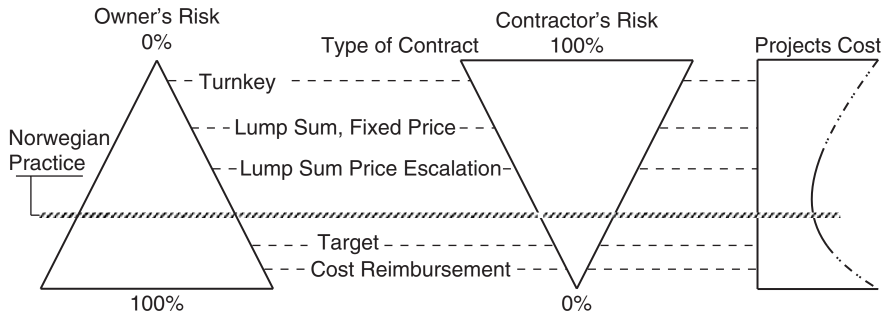

Fig. 26.3 Risk sharing and contract types (Kuesel, 1979; Barton et al., 1992).

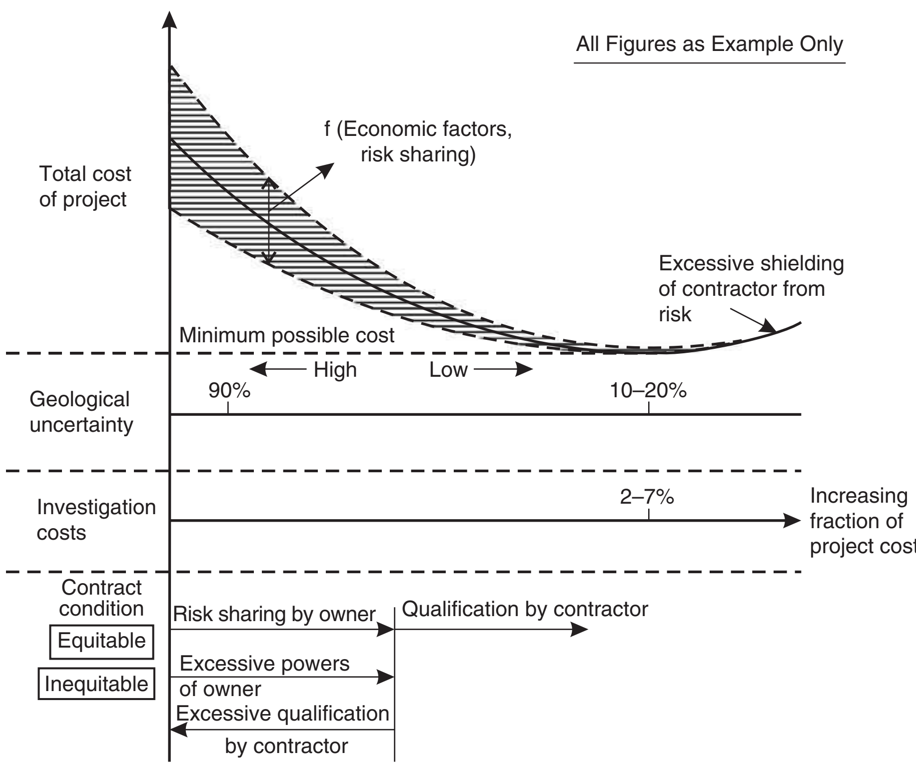

Fig. 26.4 Features of minimizing project cost (Sutcliffe, 1972).

investigations are thorough, the geological uncertainty is reduced as a result of the investigations, risks are shared by the owner and the contractor equitably and if the contractor is qualified then the project cost can be minimized.

In the court of law, denial of contract is not valid without detailed reasoning. Further, engineers owe contractors and sub-contractors an independent professional duty of care in the preparation of plans and specifications (ASCE, Civil Engineering, Vol. 73, No. 12, 2003).

26.2.1 Risk management tools – fault tree analysis

Fault tree analysis can be used to analyze a single or combined causal connection (relation) that precedes a negative event. Fault tree analysis is utilized either with or without quantifying probabilities for events. By using this tool, complex problems with many interacting events can be structured (Fig. 26.5). For further reading, refer Sturk (1998) and Ang and Tang (1984).

In tunnelling, the best approach is a strategic approach for management of risk and reduction of cost and time overruns, specially in the complex geological conditions. So tunnel design is basically a decision analysis problem of uncertainty management.

26.2.2 Recommendations of international tunnelling association (Eskesen et al., 2004)

(i) Tunnel failures have been the result of various reasons, such as insufficient site investigation, inadequate evaluation of risk at the planning stage, project understaffing, and mistakes during construction and operation phases.
(ii) Provision may also be made for revised risk management plan and time schedule as agreed between the parties when initial plan fails.

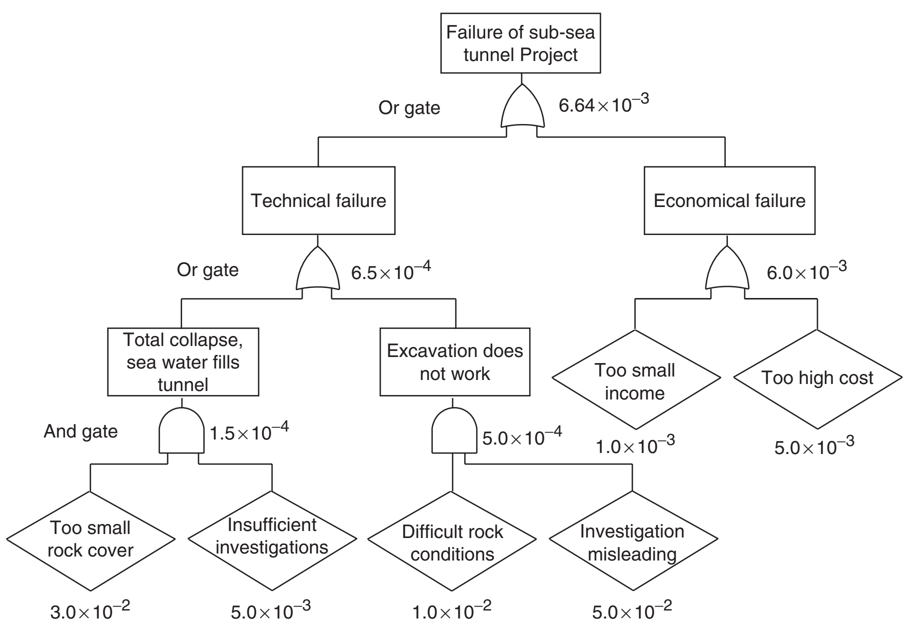

Fig. 26.5 Example of a fault tree with "and gates" and "or gates" and evaluated probabilities for an under-sea tunnel (Eskesen et al., 2004).

(iii) Due regard should be taken for common clauses for hazardous events such as,
   (a) Complexity and maturity of the applied technology,
   (b) Adverse unexpected ground and groundwater conditions (or geological surprises),
   (c) Subsidence on ground surface damaging structures and foundations,
   (d) Technical and/or managerial incompetence (Chapter 27),
   (e) Human factors and/or human errors,
   (f) Lack of sufficient communication and coordination between internal and external interfaces (workers) and
   (g) Combinations of several unwanted events that individually are not necessarily critical.

(iv) It should be stated in the tender documents that the contractor is responsible for effective risk management, regardless of the extent and details of the risk information deriving from the owner.

(v) The types of risk covered in contract are as follows.
   (a) Risk to the health and safety of workers and third party people, including personal injury and, in the extreme, loss of life;
   (b) Risk to third party property, specially normal buildings, cultural heritage buildings and infrastructure;
   (c) Risks to the environment including pollution and damage to flora and fauna;
   (d) Risk to employer (owner) in delay to completion; and
   (e) Risk of financial loss to the employer (owner).

(vi) Risk mitigation measures should be identified as long as the costs of the measures are not disproportionate with risk reduction obtained.

(vii) Tenderer may be allowed to modify his bid (technically and financially) after opening of tenders.

(viii) *The tender offering lowest cost, on the basis of sum of bid price + risk cost + upgradation of technology of firm and other costs (cost of delay + cost of litigation + cost of side effects, etc.), may be accepted by the owner from among a pre-selected list of pre-qualified firms.*

(ix) Information and training should be given as necessary to all personnel throughout the duration of a tunnelling project.

26.3 CONSTRUCTION PLANNING AND RISK

Owners should eliminate the known risk rather than try to transfer it. An active pre-contract construction planning would eliminate construction hurdles before they become sources of construction delays and disputes. This aspect is better done by the owner who has more time and is in a better negotiating position. By allocating the risk of negotiating all construction permits to the contractor, the owner would convert risk into a certainty rendering the negotiations to be more hurried, less effective and more costly than if he himself had done the homework before calling tenders. Vagueness in tender statements leads to all kinds of disputes. ITA Working Group on Health and Safety in Work had published "Guidelines for Good Tunnelling Practice" in November 1985. Tunnel engineers may find it useful practice.

26.4 TIME AND COST ESTIMATES

Usually the long tunnel projects take 5–10 years in complex geological conditions from the start of conceptual design till delivery of the scheme to the owners. A casual observer may consider it too long a period, but they are in most cases quite short accounting for the complexities of the project. In early optimistic days of the project, the owner must make estimates of time and cost stretching over many years, but actually based on little solid information. The early estimates are publicized and become frozen. Any subsequent changes even though based on more accurate data available later are suspect in the public eye and result in a loss of reputation of the engineer and his profession. The time of completion is affected by confusion in risk assessment. There is penalty clause for delay in completion of projects and reward for early completion of the same. Owners assess penalties for late completion, but contractors inflate their bids for unreasonable schedules and fight back through the courts for extra payments, much to the detriment of the owner. Thus, sufficient time should be allotted for the long tunnelling projects, after careful thought and based on the construction times of similar completed projects. While imposing a penalty on the contractor for late completion the opposite should be included, that is, payment of a bonus for early completion at a still higher rate to serve as an incentive. Delays and costs due to the owner's decision and approval processes and for his changeover on which the owner himself may have little control, should be allowed in the contract. Means for providing necessary reimbursement and time and for reducing or eliminating costly standby time should be found. The cost of prevention of loss and risk is far more than the cost of loss. The contractor should recognize them and provide measures for equitable risk sharing without including such risk factors into his bid.

The owner should work out the cost of time which is revenue earned per day on completion of the project. The financial incentives to the contractor for an early completion of tunnel should be proportional to the cost of time. The rules should not be rigid, but flexible and should be humanitarian. Failure is not a punishable offence. Justice should be to the satisfaction of all (*Mahatma Gandhi*). Let us work for glory of God within us.

"Better risk management leads to better rate of tunnelling."

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27
Rate of tunnelling

"Most human beings experience a certain amount of fear when confronted with change. The level varies from moderate dislike to intense hatred. One of the few things stronger than fear of change is love of money. Structure the change so that it provides a potential for profit and the change will happen."

"At some point in time the urgings of pundits, the theories of scientists and the calculations of engineers has to be translated into something that the miner can use to drive tunnel better, faster and cheaper. We shall call this change."

Excerpts of the report prepared by Baker, Robert, F. et al.

27.1 INTRODUCTION

Excavation of tunnels are affected by many uncertainties. The probable time for completion of tunnelling projects has been grossly underestimated in many cases. This is because proper evaluation of the factors that affect the rate of tunnel excavation is not made. The factors which affect tunnel excavation may be enumerated as:

(i) variation in ground/job conditions and geological problems encountered,
(ii) quality of management and managerial problems and
(iii) various types of breakdown or hold ups.

The first of these is very important, because for different types of ground conditions, the rate of tunnel driving is different. For example, the tunnelling rate is lower in poor ground conditions. Moreover, depending upon the ground conditions, different methods of excavation are adopted for optimum advance per round, so that the excavated rock could be supported within the bridge action period or the stand-up time. Frequent changes in ground conditions seriously affect the tunnelling rate because not only the support but also the excavation method needs to be changed. This is perhaps a major reason why the use of TBMs has not picked up for tunnelling in the Himalaya.

The second factor affects the rate of tunnelling differently due to different management conditions even in the same type of ground condition. The past experience has been that

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B. Singh and R. K. Goel

poor management condition affected tunnelling rate more adversely than poor rock mass condition.

The third factor pertains to the breakdowns or hold ups during various operations in tunnelling cycle. These hold ups cause delays which are random in nature. Based on the data collected from many projects, Chauhan (1982) proposed a classification for realistic assessment of rate of tunnelling presented in the following sections.

27.2 CLASSIFICATION OF GROUND/JOB CONDITIONS FOR RATE OF TUNNELLING

The rate of tunnelling is seriously affected by the ground conditions. The factors, under the ground condition, affecting the rate of tunnelling are (Terzaghi, 1946; Bieniawski, 1973, 1974; Barton et al., 1974).

(i) Geology, such as, type of rock, RQD, joint system, dip and strike of strata, presence of major fault or thrust zones and their frequencies and type and rock mass properties,
(ii) Method of excavation including blast pattern and drilling arrangement,
(iii) Type of support system and its capacity,
(iv) Inflow of water,
(v) Presence of inflammable gases,
(vi) Size and shape of tunnel,
(vii) Construction adits whether horizontal or inclined, their grade size and length and
(viii) High temperature in very deep tunnels ($H > 1000$ m).

On the basis of the above factors affecting the rate of tunnelling, the ground conditions are classified into three categories – good, fair and poor (Table 27.1). It means that for the good ground conditions the rate of tunnelling will be higher and for the poor ground conditions the rate of tunnelling will be lower. The job/ground conditions in Table 27.1 are presented in order of their weightage to the rate of tunnelling.

27.3 CLASSIFICATION OF MANAGEMENT CONDITIONS FOR RATE OF TUNNELLING

The rate of tunnelling may vary in the same ground condition depending upon management quality. The factors affecting management conditions are:

(i) Overall job planning, including selection of equipment and decision-making process,
(ii) Training of personnel,
(iii) Equipment availability including parts and preventive maintenance,

Table 27.1 Classification of ground/job condition (Chauhan, 1982).

		Job conditions		
S. No.	Parameter	Good	Fair	Poor
1.	Geologic structure	Hard, intact, massive stratified or schistose, moderately jointed, blocky and seamy	Very blocky and seamy squeezing at moderate depth	Completely crushed, swelling and squeezing at great depth
2.(a)	Point load strength index	>2 MPa	1–2 MPa	Index cannot be determined but is usually less than 1 MPa
(b)	Uniaxial compressive strength	>44 MPa	22–44 MPa	<22 MPa
3.	Contact zones	Fair to good or poor to good rocks	Good to fair or poor to fair rocks	Good to poor or fair to poor rocks
4.	Rock quality designation (RQD)	60–100 %	25–60 %	<25%
5.(a)	Joint formation	Moderately jointed to massive	Closely jointed	Very closely jointed
(b)	Joint spacing	>0.2 m	0.05–0.2 m	<0.05 m
6.(a)	Joint orientation	Very favorable, favorable and fair	Unfavorable	Very unfavorable
(b)	Strike of tunnel axis and dip with respect to tunnel driving	(i) Perpendicular 20 to 90° along dip, 45 to 90° against dip	(i) Perpendicular 20 to 45° against dip	(i) Parallel 45 to 90°
		(ii) Parallel 20 to 45°	(ii) Irrespective of strike 0 to 20°	–
7.	Inflammable gases	Not present	Not present	May be present
8.	Water inflow	None to slight	Moderate	Heavy
9.	Normal drilling depth/round	>2.5 m	1.2 m–2.5 m	<1.2 m
10.	Bridge action period	>36 h	8–36 h	<8 h

Note: The geologist's predictions based on investigation data and laboratory and site tests include information on parameters at S. Nos. 1 to 6. This information is considered adequate for classifying the job conditions approximately.

(iv) Operating supervision,
(v) Incentives to workmen,
(vi) Co-ordination,
(vii) Punctuality of staff,
(viii) Environmental conditions and
(ix) Rapport and communication at all levels.

These factors affect the rate of tunnelling both individually and collectively. Each factor is assigned a weighted rating (Table 27.2). The maximum rating possible in each subgroup has also been assigned out of 100 in Table 27.2 that represents ideal conditions. At a particular site the rating of all the factors is added to obtain a collective classification rating for management condition. Using this rating, the management condition has been classified into good, fair and poor as shown in Table 27.3. The proposed classification system for management is valid for tunnels longer than 500 m, which are excavated by conventional drilling and blasting method.

It may be noted that the rate of tunnelling can be easily improved by improving the management condition which is manageable unlike the ground conditions which cannot be changed. So, it is necessary to pay at least equal, if not more, attention to the management condition than to the ground condition. Hence, there is an urgent need for management consultancy for improving the tunnelling rate.

The key to success of tunnel engineers is evolution of a flexible method of construction of support system. On-spot strengthening of support system is done by spraying additional layers of shotcrete/SFRS or using long rock bolts in the unexpectedly poor geological conditions. This is a sound strategy of management in tunnelling within the complex geological situations. Affection is the key to success in the management. Young engineers love challenging works. There should be no hesitation in throwing challenges to young engineers. Otherwise these young engineers may loose interest in routine management.

27.4 COMBINED EFFECT OF GROUND AND MANAGEMENT CONDITIONS ON RATE OF TUNNELLING

A combined classification system for ground conditions and management conditions has been developed by Chauhan (1982). Each of the three ground conditions has been divided into three management conditions and thus nine categories have been obtained considering both ground and management conditions. The field data of six tunnelling projects in the Indian Himalayas have been divided into these nine categories for studying the combined effect. Each category has three performance parameters which are:

(i) Actual working time (AWT),
(ii) Breakdown time (BDT) and
(iii) Advance per round (APR).

Table 27.2 Ratings for management factors for long tunnels (Chauhan, 1982).

S. No.	Subgroup	Item	Maximum rating for		Remarks for improvement in management condition
			Item	Subgroup	
1.	Overall job planning	i) Selection of construction plant and equipment including estimation of optimal size and number of machines required for achieving ideal progress.	7		
		ii) Adoption of correct drilling pattern and use of proper electric delays.	6		
		iii) Estimation and deployment of requisite number of workmen and supervisors for ideal progress.	5		
		iv) Judicious selection of construction method, adits, location of portals, etc.	4		Horizontal adits sloping at the rate of 7% towards portal to be preferred to inclined adits or vertical shafts.
		v) Use of twin rail track	2		
		vi) Timely shifting of California switch at the heading	2	26	
2.	Training of personnel	i) Skill of drilling crew in the correct holding, alignment and thrust application on drilling machines	4		Proper control of drilling and blasting will ensure high percentage of advance from the given drilling depth and also good fragmentation of rock which facilitates mucking operation.
		ii) Skill of muck loader operator	4		

Continued

Table 27.2—Continued

S. No.	Subgroup	Item	Maximum rating for Item	Maximum rating for Subgroup	Remarks for improvement in management condition
		iii) Skill of crew in support erection	3		A skilled crew should not take more than 1/2 h for erection of one set of steel rib support.
		iv) Skill of blastman	2		
		v) Skill of other crews	2	15	
3.	Equipment availability and preventive maintenance	Time lost in tunnelling cycle due to breakdowns of equipment including derailments, etc.			
		i) upto 1 h.	12–15		
		ii) 1–2 h.	9–11		
		iii) 2–3 h.	6–8		
		iv) >3 h.	0–5	15	
4.	Operation supervision	i) Supervision of drilling and blasting (effectiveness depends on location, depth and inclination of drill holes, proper tamping and use of blasting delays)	7		Improper drilling may result in producing: i) unequal depth of holes which results in lesser advance per meter of drilling depth and ii) wrong alignment of hole which may lead to : a) overbreak due to wrong inclination of periphery holes and b) secondary blasting due to wrong inclination of other than periphery holes

				Item	Subgroup	
						Improper tamping of blast hole charge and wrong use of blasting delays result in improper blasting effects.
		ii)	Supervision of muck loading/hauling system	3		Especially in rail haulage system in which rapid feeding of mine cars to loading machine at the heading is essential for increasing productivity of loader.
		iii)	Supervision of rib erection, blocking and packing	3		
		iv)	Other items of supervision such as scaling, layout, etc.	2	15	
5.	Incentive to workmen	i)	Progress bonus	5		Define the datum monthly progress as that value which delineates good and fair management conditions for a particular job conditions. Introduce bonus slabs for every additional 5 m progress and distribute the total monthly bonus thus earned amongst the workmen on the basis of their importance, skill and number of days worked during the month. The amount for each slab should be so fixed that these are progressive and each worker should get about 50% of his monthly salary as progress bonus, if ideal monthly progress is achieved.
		ii)	Incentive bonus	2		This should be given for certain difficult and hazardous manual operations like rib erection/shear zone treatment, etc.

Continued

Table 27.2—Continued

S. No.	Subgroup	Item	Maximum rating for		Remarks for improvement in management condition
			Item	Subgroup	
		iii) Performance bonus	1		This should be given to the entire tunnel crew equally if the quarterly progress target is achieved.
		iv) Achievement bonus	1	9	It is to be given for completion of whole project on schedule. It should be given to the whole construction crew and may be equal to one year's interest on capital cost.
6.	Co-ordination	i) Co-ordination of activities of various crews inside the tunnel.	5		Co-ordination between designers and construction engineers should be given top priority. Designers should be boldly innovative.
		ii) Use of CPM for overall perspective and control of the whole job.	4	9	Safety saves money. Contingency and emergency plans should be ready before tunnelling.
7.	Environmental conditions and housekeeping	Proper lighting, dewatering, ventilation, provision of safety wear to workmen and general job cleanliness.	4	4	
8.	Punctuality of staff	i) Prompt shift change-over at the heading	4		
		ii) Loss of upto 1/3 h in shift change-over	3		
		iii) Loss of more than 1/3 h in shift change-over	0–2	4	
9.	Rapport and communication	Commitment, good rapport and communication at all levels of working including top management and government level including human relations.	3	3	Team spirit is the key to success in underground construction. The contractors have to be made to succeed.

Table 27.3 Rating for different management conditions (Chauhan, 1982).

S. No.	Management condition	Rating
1.	Good	80–100
2.	Fair	51–79
3.	Poor	$\leq$50

Table 27.4 Ground and management factors (Chauhan, 1982).

Ground conditions	Management conditions		
	Good	Fair	Poor
Good	0.78	0.60	0.44
Fair	0.53	0.32	0.18
Poor	0.30	0.21	0.13

A matrix of job and management factors has been developed from the data for evaluating tunnel advance rate as given in Table 27.4.

Ground and management factors in the matrix are defined as a ratio of actual monthly progress to achievable monthly progress under corresponding set of ground and management conditions. Knowing the achievable production for a tunnelling project, these factors could hopefully yield values of expected production under different management and geological conditions on the project.

Thus, in squeezing ground conditions, the rate of tunnelling would be only 13 percent of the theoretical rate for poor management condition. Past experience suggests that management tends to relax in good tunnelling conditions and becomes alert and active in poor rock conditions.

Further studies are needed to update Table 27.2 to 27.4 for modern tunnelling technology. Trends are expected to be similar.

Management of world bank-funded projects is an ideal example. They appoint international experts on rock mechanics on their hydroelectric projects. In major state-funded projects, international experts on rock mechanics should be appointed as the Board of Consultants, as in the past. The international experts help to achieve self-reliance.

27.5 TUNNEL MANAGEMENT (SINGH, 1993)

The management is the topmost art, demanding strength of character, intelligence and experience. Deficiencies in management are, therefore, difficult to remove. Experience is not what happens to you, it is what you do with what happens to you. Everyone is potentially a high performer and motivation comes from top. What glorifies self-respect automatically improves one's efficiency. Often interference by the manager mars the initiative of the young engineers. Feedback is essential to improve performance, just like feedback is very important for the stability of the governing system in electronics. Efficient clear communication of orders to concerned workers and their feedback is essential for success of management. Computer network and cell phones are used now-a-days for better informal rapport at a project site. The modern management is committed to visible management. The defeatist attitude should be defeated. The leader should have the willpower to complete the vast project. There should be respect for individual in the organization. Happier the individual, more successful he will be. *If you want to be happy for whole life, love your work.*

Tunnel construction is a complex, challenging and hazardous profession. It demands certainly a high skill in the leadership, technology and communication. On the spot decisions are needed in a crisis during tunnelling. Mutual respect between government engineers and contractors is need of the time. That is what privatization stands for. Usually bad news does not travel upwards to the executive management. Basic ingredient in management is trust. Quality consciousness should be the culture of a construction agency. Is quality work possible in government due to lack of creative freedom? Work of good quality is possible in fact by framing proper specifications in a contract document. Contractor's point of view is that payments should be made early for quick reinvestment. Unfortunately, construction industries are unorganized at present in many countries. With increasing trend for global organization, efficiency will go upwards in the future. No two construction jobs are alike. It is, therefore, very difficult to evolve a system (of stockpiles of materials, fleet of tunnelling machines, etc.) for a new project site. Construction problems vary so much from job to job that they defy tenders, machines and known methods. Then a contractor uses ingenuity to design tools and techniques that will lead to success in tunnelling. Machines may be used for various other purposes with slight modifications, beyond imagination. Excellent companies are really close to their customer (engineers) and pay them high regards. Their survival depends upon the engineer's satisfaction.

Critical path analysis, if properly applied and used, can be a great help to any construction agency, specially in a tunnelling job. Use of software for critical path analysis for cost control is most effective and economical. Then co-ordination among workers becomes easy. Naturally a management organization becomes more efficient during crisis. Cost consciousness must permeate all ranks of engineers and workers. Organization set-up is the back-bone of a long tunnelling project.

The completion of a hydroproject is delayed by the completion of long length of tunnels in weak and complex geological conditions. So, the idea of substantial bonus for early completion is becoming more widespread.

27.6 POOR TENDER SPECIFICATIONS

Tendering for tunnelling projects remains speculation, since actual ground conditions encountered during construction often do not match the conditions shown in the tender specifications, particularly in the Himalayas, young mountains and complex geological environment. The practice of adopting payment rates according to actual ground condition does not exist. Insufficient geological, hydrogeological and geo-technical investigations and poor estimates, etc. invariably lead to owner–contractor conflicts, delay in projects, arbitration and escalation of project cost, generally by three times. Following are some of the main reasons attributed to this poor tunnelling scenario in developing nations.

(i) Inadequate geological investigations and absence of rock mechanics appreciation before inviting a tender bid, resulting in major geological surprises during execution.
(ii) Lack of proper planning, sketchy and incompetent preparation of designs at pre-tender stage.
(iii) Unrealistic projection of cost estimates and cost benefit ratio and completion schedules at initial stages.
(iv) Inadequate infrastructure facilities at site.
(v) Unrealistic and unfair contract conditions and poor profit margins leading to major disputes and delays in dispute resolution.
(vi) Lack of motivation and commitment on the part of owners, especially government departments and public sector agencies.
(vii) Lack of specific provisions in the tender document itself with regard to modern technology.
(viii) Lack of teamwork between the owner, the contractor, the geologist and the rock mechanics expert.
(ix) Risk sharing between contractor and owner is generally not fair.
(x) Lack of indigenous construction technology in developing nations.

It is important here to emphasize that though sufficient expertise is available in the world in the tunnelling technology, the administration seldom takes advantage of the intellectual resources in the right perspective at the right time.

27.7 CONTRACTING PRACTICE

On some occasions, it is the inexperience or incompetence of the contractor that has delayed a project. Sometimes lack of strategy, weak project team and inadequate attention from the top management also result in delays and slippage. In some cases, contractors are found ill-equipped and starved of cash, besides lacking in professionalism. Just to grab the project deal, they compromise on rates. Finding very low profits when the work starts, they raise unreasonable claims and disputes to improve profit margin which results in disputes followed by arbitration, delays and time and cost over-runs in some developing countries.

Following measures are suggested to avoid delays in project schedules and cost escalation due to contractors.

(i) In the pre-bid meeting, an objective evaluation of potential contractors should be made and inefficient contractors should be eliminated at this stage itself.
(ii) Award of contract should be granted to a group of contractors, each expert in specific activities like design, tunnelling machines, construction, rock mechanics, geology, etc. By this process, the project authorities will have the benefit of the services of a team of competent contractors.
(iii) Contractors should induct trained and experienced staff and should undertake technology upgradation programmes on continuous basis. They should take active assistance during project commissioning from technical experts of R&D organizations. This will equip them to handle major geological surprises, substantiate their claims and economize their routine operations.

27.8 QUALITY MANAGEMENT BY INTERNATIONAL TUNNELLING ASSOCIATION

Oggeri and Ova (2004) have suggested the following principles of quality management for tunnelling.

(i) Quality in tunnelling means knowledge. Knowledge is necessary to answer correctly to the requirement of the design. Knowledge is necessary to "learn" and "copy" better what previous designers have done.
(ii) Experience, good contracts, professionalism, self-responsibility and simple rules are the basis to reach the objectives of design and perform properly.
(iii) Successful planning is the key to a successful project.
(iv) Transfer of information both upwards and downwards in an organization, in a format understood by all, is the key issue.
(v) *There is direct, linear relation between project quality and project cost.*
(vi) Design a strategy of tunnelling in all possible ground conditions at a project.
(vii) Tunnelling projects are well suited for "on-the-job training," since large projects use state-of-the-art technology. Engineers should participate in the international tunnelling conferences and meet the specialists and report their difficulties.
(viii) If a process is innovative, a testing program prior to the productions should be conducted.
(ix) All along the project a co-ordination of activities is necessary in order to achieve significant results for: (a) technical features, (b) economical results, (c) contractual agreements, (d) environmental effects and (e) safety standards.
(x) Correct choice is essential for the type of contract, conditions of contract, financing and procurement procedures for equipment.

(xi) Knowledge is transferred not only between parties during project phases, but also to parties after completion of a project, including the universities and other technical organizations.

An integrated approach of tunnelling is need of the time.

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28
Integrated method of tunnelling

"Excellence is not an act but a habit."

Aristotle

28.1 INTRODUCTION

Tunnelling is an art practiced by all engineers, geologists, planners and people. Failures should be regarded as challenges and opportunities for generating new knowledge and thereby increasing self-reliance in the tunnelling. The key to success is team spirit and love for rocks and nature.

The most challenging construction problem is the squeezing ground condition which is encountered in weak rock masses under high rock cover. Special treatment is necessary to support shear zones in the tunnels.

The philosophy of design of any underground excavation should be to utilize the rock mass itself as the principal structural materials, creating as little disturbance as possible during the excavation process and adding as little as possible in the way of shotcrete or steel supports. The extent to which this design aim can be met depends upon the geological conditions which exist at site and the extent to which the designer is aware of these conditions.

There are many difficult geological conditions and extraordinary geological occurrences (EGO) such as intra-thrust zones, very wide shear zones, geothermal zones of high temperature, cold/hot water springs, water charged rock masses, intrusions, etc. These are very difficult to forecast. Innovative methods of tunnelling will have to be invented and experts must be consulted.

In view of the difficulties in forecasting geological formations along deep and long tunnels particularly in complex geological environment, the suggested strategy of tunnelling is such that tunnelling could be done smoothly in usually all ground conditions. Authors recommend strongly, the adoption of the steel fiber reinforced shotcrete (SFRS) to cope up with even squeezing ground conditions. The use of steel ribs should be restricted to

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highly squeezing or swelling rock conditions only. This integrated method of tunnelling is accepted by Bureau of Indian Standards, India.

28.2 PROBE HOLES

Long tunnels, sometimes pass through the complex geological conditions particularly in case of deep tunnel. Geological predictions in deep tunnel are hard to make on the basis of surface observations. If the site conditions require, probe drill holes of about 75 mm diam. may be made at the face of the tunnel for about 20 m length. This probe hole will give reliable geological and geotechnical informations in advance of tunnelling. It will also help in suggesting the strategy of tunnelling.

28.3 EFFECT OF SEISMICITY

A tunnel in a seismic area, is likely to be affected near the portals and in neighborhood of faults and thrust. The effect is observed to be upto a distance along tunnel within $\pm B$ on both sides of the faults/thrusts, where $\pm B$ is span/size of the opening. The design support pressure in the affected length of the tunnel may be taken as 1.25 times of the ultimate support pressure.

28.4 TUNNEL INSTRUMENTATION

Instrumentation of tunnel openings should be done where squeezing ground condition is expected. The survival rate of tunnel instruments is generally as low as 30 percent, therefore many sections of the tunnel should be instrumented so that enough instruments survive and reliable data is obtained. The post-monitoring of support system in squeezing ground should also be carried out until support system has stabilized with time. In cases of squeezing ground conditions, observed vertical and horizontal tunnel closures should be less than 4 percent of tunnel width and height, respectively and *rate of deformation is less than 2–3 mm/month before concrete lining is built*.

Instrumentation may also be done at other locations as per need of the site conditions. It should be kept in mind that the psychology of construction engineers is such that they resist every effort, which reduces the momentum of enthusiasm of construction.

The displacements are measured by multipoint borehole extensometers. Extra long rock anchors may have to be installed where rate of displacement is not decreasing rapidly. The support pressures are determined by load cells and pressure cells. The tunnel closure is obtained by tape extensometers. Displacements across cracks in shotcrete and rock mass are monitored by 3D crackmeter. Grouting of cracks may be done after movements across cracks have stabilized. It is also essential to monitor the rate of seepage with the help of "V" notch at the end of the tunnel. If seepage is observed to increase with time, there is every danger of failure and flooding of tunnel within water-charged rock mass (crushed quartzite/sandstone/hard rocks, dolomite, shear zones, faults, etc.). Sometimes wide faults (>10 m) are met during tunnelling. They require attention of experts. In case of soft ground or soil like gouge within wide faults, the tunnel lining should be designed using design method of ITA (Duddeck and Erdmann, 1985). Chapter 14 discusses details of instrumentation.

28.5 SELECTION OF TYPE OF SUPPORT SYSTEM

Before taking up the design of supports, the rock load and pressure likely to act on the supports shall be estimated. The determination of rock load is complex problem. This complexity is due to the inherent difficulty of predicting the primary stress conditions in the rock mass (prior to excavation) and also due to the fact that the magnitude of the secondary pressure developing after the excavation of the cavity depends on a large number of variables, such as size and shape of cavity, depth of cover, strike and dip of rock formation in relation to alignment of tunnel, method of excavation, period of time elapsing before rock is supported and the rigidity of support. These pressures may not develop immediately after excavation but may take a long period due to the adjustment of displacements in the rock mass with time.

In major tunnels it is recommended that as excavation proceeds, load cell measurements and diametrical change measurements are carried out, so that rock loads may be correctly estimated. In the absence of any data of instrumentation, rock load or support pressure may be estimated by Q-system (see Chapter 5).

As the tunnels generally pass through different types of rock formations, it may be necessary to workout alternative cross sections of the tunnel depicting other acceptable types of support systems. These types may be selected to match the various methods of attack that may have to be employed to get through the various kinds of rock formations likely to be encountered. "A" and "B" lines shall be shown on these sections.

The support system shall be strong enough to carry the ultimate loads. For a reinforced concrete lining, it is economical to consider the (steel ribs) supports as an integral part of the permanent lining. Temporary support system must be installed within the stand-up time for safety of workmen but not too early.

The aim, in a nut shell, is to construct an inherently stable and robust yet ductile structural system (reinforced rock arch or ring and SFRS) to support a wide variety of ground conditions and weak zones, keeping in mind basic tunnel mechanics and the inherent uncertainities in the exploration, testing and behavior of geological materials.

28.6 STEEL FIBER REINFORCED SHOTCRETE

The steel fiber reinforced shotcrete is either alone or in combination with rock bolts, specially in large openings, provides a good and fast solution for both initial and permanent rock support. Being ductile, it can absorb considerable deformation before failure. The SFRS can withstand bending stresses, caused by faults.

There are two benefits of excellent bond between shotcrete lining and surface of opening in rocks, as follows:

- Support pressures are reduced effectively even in squeezing grounds and
- Bending stresses are not found to occur in the shotcrete lining due to bonding. As such it fails generally in shear only.

Controlled blasting should be used preferably. The advantage of fiber reinforced shotcrete is that a smaller thickness of shotcrete is needed, in comparison to that of conventional shotcrete (Fig. 28.1). Fiber reinforced shotcrete is required, specially in rock conditions where support pressure is high. Use of fiber-reinforced shotcrete along with the resin anchors is also recommended for controlling rock burst conditions because of high fracture toughness of shotcrete due to specially long steel fibers (Fig. 28.1b). This can also be used effectively in highly squeezing ground conditions. It ensures better bond with rock surface. With wire mesh, voids and pockets might form behind the mesh thus causing a poor bond and formation of water seepage channels as indicated in Fig. 28.1a in the case of normal shotcreting.

The major drawback of normal shotcrete is that it is rather weak in tensile, flexural and impact resistance strength. These mechanical properties are improved by the addition of

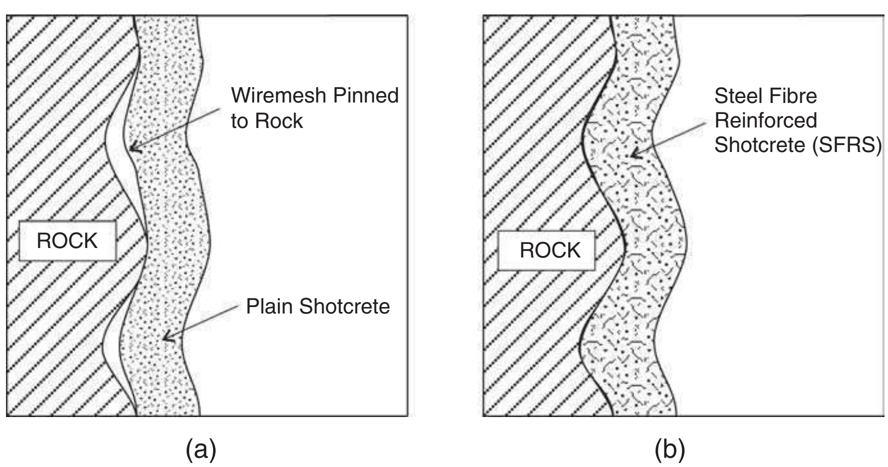

Fig. 28.1 Difference in application of shotcrete with (a) wire mesh and (b) steel fiber.

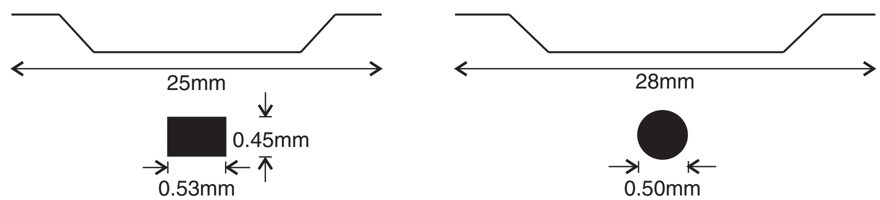

Fig. 28.2 Typical fibers used in shotcrete work.

steel fibers. Steel fibers are commonly made into various shapes to increase their bonding intimacy with the shotcrete. It is found that the hooked ends type of steel fibers behave more favorably than other types of steel fibers in flexural strength and toughness. Accelerators play a key role to meet the requirements of early strength.

Steel fibers make up between 0.5 and 2 percent of the total volume of the mix (1.5 to 6 percent by weight). Shotcrete mixes with fiber contents greater than 2 percent are difficult to prepare and shoot.

The steel fibers are manufactured by cutting cold drawn wires. Some of the important parameters of steel fibers are:

- Geometrical shape – as shown in Fig. 28.2. Length of the fibers may be 20 to 40 mm. Recommended sizes of the fibers are 25 to 35mm × 0.40 mm diameter,
- Aspect Ratio (length/equivalent diameter) – 60 to 75,
- Ultimate tensile strength > 1000 MPa,
- Shear strength of SFRS (long-term) – 8 to10 MPa.

28.6.1 Shotcrete ingredients

Shotcrete ingredients in fiber reinforced shotcrete are:

a) Cement,
b) Micro silica fumes (8–15 percent by mass of cement) for improving pumpability and strength and to reduce rebound,
c) Aggregate,
d) Water,
e) Hydration control agent (wet mix),
f) Super plasticizers (3–6 l/m^3) for slump increase and improvement in strength,
g) Accelerators (2–5 percent by mass of cement),
h) Curing agent,
i) Steel fibers.

Shotcrete ingredients and properties are listed in Table 28.1.

Table 28.1 Typical steel fiber reinforced shotcrete mix.

		Mean aggregate size 6.35 mm	Mean aggregate size 10 mm
S. No.	Material	Quantity, kg/m^3	Quantity, kg/m^3
1.	Cement	446–558	>445
2.	Blended sand 6.35 mm maximum size	1483–1679	697–880
3.	10 mm aggregate	–	700–875
4.	Steel fiber	39–157	39–150
5.	Accelerator	Varies	Varies
6.	Water/cement (by weight)	0.40–0.45	0.40–0.45

Advantages of SFRS over shotcrete reinforced with welded mesh are as follows:

(i) Eliminates weld mesh resulting upto 5 percent savings in a typical cycle time. Every hour of shotcreting needs at least 3 h for mesh fixing.
(ii) SFRS allows fast shotcreting for quick supports, thus gives safer working conditions even within small stand-up time.
(iii) Ensures better bond with rock surface. With mesh, voids and pockets might form behind the mesh thus causing a poor bond and formation of water seepage channels as indicated in Fig. 28.1a.
(iv) Economical because of

a) Reduction of about 50 percent in shotcrete thickness
b) Less shotcrete consumption due to consistent thickness of SFRS layer.

Key to successful SFRS construction is the use of a well trained and experienced shotcrete application crew. *Pre-construction and post-construction testing of shotcrete shall be done for quality assurance.* Proper equipment should be used to avoid bunching of steel fibers and to ensure homogeneous mixing of fibers in the shotcrete.

To increase the stand-up time, for a full front tunnel profile in poor rock quality (or squeezing rock conditions), spiling dowels are provided as shown in Fig. 28.3.

To stabilize the broken zone in squeezing ground conditions more than one layers of SFRS is provided as shown in Fig. 28.4. The floor heaving problem in highly squeezing ground conditions can easily be solved by bolting the floor. Cutting the floor to maintain proper ground level is of no use, since heaving will redevelop. A minimum center to center spacing of tunnels of width B may be $6B$ for minimum interaction (Barton, 2002).

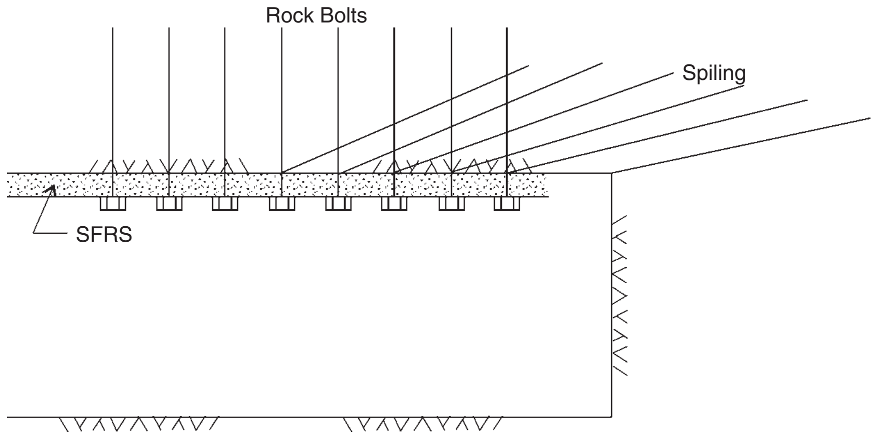

Fig. 28.3 Arrangement of spiling dowel with the advancement of tunnel face.

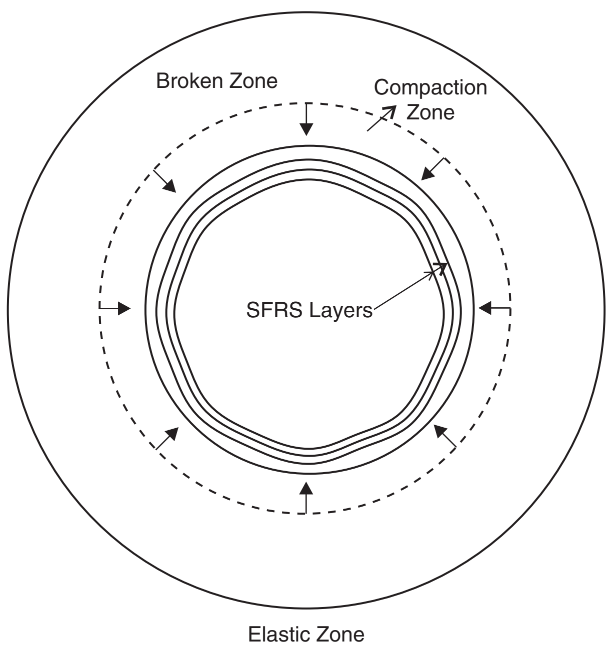

Fig. 28.4 Stabilization of broken zone in squeezing ground condition.

28.6.2 Capacity of fiber reinforced shotcrete

It is assumed that the fiber reinforced shotcrete is intimately in contact with the rock mass and having the tendency to fail by shearing.

Capacity of fiber reinforced shotcrete is given by

$$p_{\text{fsc}} = \frac{2 \cdot q_{\text{fsc}} \cdot t_{\text{fsc}}}{F_{\text{fsc}} \cdot B} \tag{28.1}$$

where

q_{fsc} = shear strength of steel fiber reinforced shotcrete (550 t/m^2 or 0.20 × UCS of SFRS),
t_{fsc} = thickness of fiber reinforced shotcrete (m),
B = size of opening (m),
$B \cdot F_{\text{fsc}}$ = distance between vertical planes of maximum shear stress in the SFRS (m) (Fig. AII.1a),
F_{fsc} = 0.6 ± 0.05 and
p_{fsc} = support capacity of fiber reinforced shotcrete lining (t/m^2).

The thickness of fiber reinforced shotcrete lining may be estimated by substituting ultimate support pressure (p_{roof}) in equation (28.1) in place of p_{fsc}. Additional layers of shotcrete should be sprayed to arrest tunnel closure if needed.

28.6.3 Drainage provision in shotcrete in transport tunnels within water-charged rock mass

Strips of about 50 cm width should not be shotcreted for free seepage of ground water, otherwise shotcrete is likely to crack due to building up of seepage pressure behind shotcrete in heavily charged formations (Zhidao, 1988). Drainage holes should be provided for proper drainage. The catch drain should have adequate capacity to carry seepage water or flood.

Very often one may observe that the seepage of water is concentrated to only one or just a few, often tubular openings in fissures and joints. It can be worthwhile to install temporary drainage pipes in such areas before applying the shotcrete. These pipes can be plugged when the shotcrete has gained sufficient strength. Further swellex (inflated tubular) bolts are preferred in water-charged rock masses. Cement grout bolts are not feasible here as grout will be washed out. Resin grout may not also be reliable. It may be mentioned that the seals used in the concrete lining for preventing seepage in the road/rail tunnels may not withstand heavy water pressure.

The pressure tunnels are grouted generally all round its periphery so that the ring of grouted rock mass is able to withstand heavy ground water pressure. Polyurethane should be used as a grout in rock joints under water as it swells 26 times and cements the rock mass.

28.7 TREATMENT OF SHEAR ZONE (BHASIN ET AL., 1995)

The mean Q-value may be determined, taking into consideration the breadth of weak/shear zone. The following formula may be employed in calculating the weighted mean Q-values from the Q-value for shear zone and surrounding rock mass (Fig. 28.5).

$$\log Q_m = \frac{b \cdot \log Q_{wz} + \log Q_{sr}}{b+1} \tag{28.2}$$

where

Q_m = mean value of rock mass quality Q for finding the support pressure,
Q_{wz} = Q value of the weak zone/shear zone,
Q_{sr} = Q value of the surrounding rock and
b = breadth of the weak zone in meter.

Similarly, weighted mean value of J_{rm} may be obtained after replacing log Q by appropriate value of joint roughness number in equation (28.2). In the same way, weighted mean of joint alteration number J_{am} may be calculated (Samadhiya, 1998).

The strike direction (θ) and thickness of weak zone (b) in relation to the tunnel axis is important for the stability of the tunnel and therefore the following correction factors (Table 28.2) have been suggested for the value of b in the above equation (28.2).

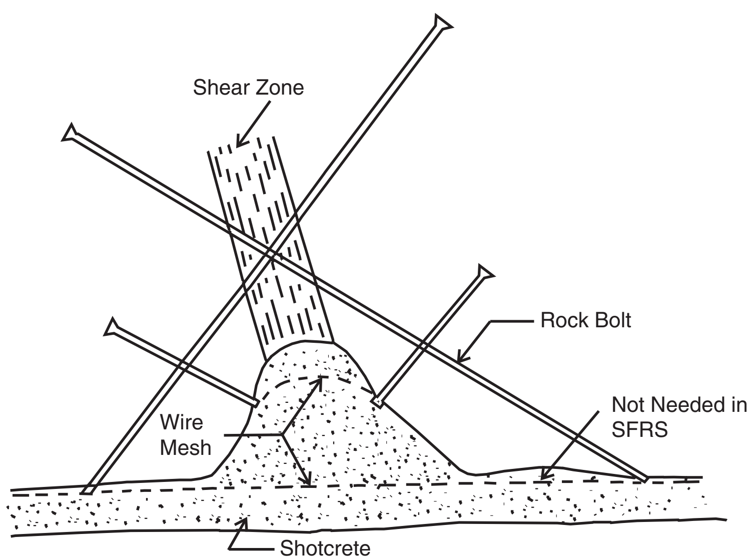

Fig. 28.5 Typical treatment of a narrow shear zone.

Table 28.2 Correction factors for thickness of weak zone (*b*).

Strike direction (θ) to the tunnel axis	"*b*" to be replaced by
90°–45°	*b*
45°–20°	2*b*
10°–20°	3*b*
<10°	4*b*

Special bolting system is required for supporting the weak shear zone, Fig. 28.5 shows a typical treatment of a thinner shear zone which is thicker than 50 cm. First, the gouge is cleaned out to the desired extent. Secondly, the rock bolts are installed across the shear zone and connected with the chain wire mesh. Finally, this "dental" excavation is back-filled with shotcrete or steel fiber reinforced shotcrete. In wide shear zone (>1 m), reinforcement has to be placed before shotcreting so that the reinforced shotcrete lining can withstand the heavy support pressure.

28.8 SHOTCRETE

28.8.1 General

Shotcrete for tunnel supports may be used as a thin skin type reinforcement or used in combinations with rock bolts, wire mesh and other more conventional tunnel reinforcements. Details are given below:

(i) All loose rock pieces shall be scaled out and the rock surface shall be washed by water-jet before applying shotcrete.
(ii) Shotcrete is forced into open joints, fissures, seams and irregularities in the rock surface and in this way serves the same binding function as mortar in a stone wall.
(iii) Initially a 25 mm thick shotcrete is sprayed immediately after the excavation.
(iv) Shotcrete hinders water seepage from joints and seams in the rock and thereby prevents piping of joint filing materials and air and water deterioration of the rock.
(v) Shotcrete's adhesion to the rock surface and its own shear strength provide a considerable resistance to the fall of loose rock blocks from the roof of a tunnel.
(vi) A thicker shotcrete layer (150 to 250 mm) provides structural support, either as a closed ring or as an arch type member.

(vii) The sound of hammer on shotcrete may indicate the hollow air gaps behind the weld mesh (Fig. 28.1a).

28.8.2 Mix

Shotcrete is a mixture of cement, sand and aggregate. The proportion of cement to aggregate in shotcrete may be normally 1:3 or 1:4, the aggregate being a mixture of sand and about 20 percent aggregate varying from 5 to 20 mm. The dry mixture of shotcrete shall be applied under pressure of about 3.5 kg/cm^2 by means of a nozzle through a concrete gun. To this nozzle, water shall also be added under pressure through a separate pipe. A special quick setting agent shall be added to the dry mixture to reduce time to less than 3 min only.

28.8.3 Thickness

The thickness of shotcrete required depends upon the type of rock, the extent of stratification and/or joints, blockiness and also the size of the tunnel. The thickness may normally range from 50 to 150 mm and whether it should be used plain or with wire mesh anchored to rock will depend upon the actual site conditions in each case (see Fig. 10.2).

28.8.4 Support capacity of shotcrete in roof

It is assumed that, shotcrete is intimately in contact with the rock mass and has the tendency to fail by shearing alone. Capacity of shotcrete (p_{sc}) is given by:

$$p_{sc} = \frac{2 \cdot q_{sc} \cdot t_{sc}}{F_{sc} \cdot B} \tag{28.3}$$

where

q_{sc} = shear strength of shotcrete (300 t/m^2 in most of the cases or $0.20 \times$ UCS of shotcrete),
t_{sc} = thickness of shotcrete (m),
B = size of opening (m),
$B \cdot F_{sc}$ = horizontal distance between vertical planes of maximum shear stress in the shotcrete (m) (see Fig. AII.1a),
F_{sc} = 0.6 ± 0.05 and
p_{sc} = support capacity of shotcrete lining (t/m^2).

The thickness of shotcrete may be estimated by substituting ultimate support pressure (p_{roof}) for p_{sc} in equation (28.3). Additional layers of shotcrete should be sprayed to arrest the rate of tunnel closures where needed.

28.9 ROCK/ROOF BOLTS

28.9.1 General

Roof bolts are the active type of support and improve the inherent strength of the rock mass which acts as the reinforced rock arch whereas, the conventional steel rib supports are the passive supports and supports the loosened rock mass externally. All rock bolts should be grouted very carefully in its full length. There are many types of rock bolts and anchors which may also be used on the basis of past experience and economy. The rock bolts may preferably be made out of thermomechanically treated (TMT) reinforcing steel bars. More details are given in Chapter 12.

28.9.2 Types of roof bolts

28.9.2.1 Wedge and slot bolt

These consist of mild steel rod, threaded at one end, the other being split into two halves for about 125 mm length. A wedge made from 20 mm square steel and about 150 mm long shall be inserted into the slot and then the bolt with wedge driven into the hole which will make the split end to expand and fit tight into the hole forming the anchorage. Therefore, a 10 mm plate washer of size 200 × 200 mm shall be placed and the nut tightened (Fig. 28.6). The efficiency of the splitting of the bolt by the wedge depends on the strata at the end of

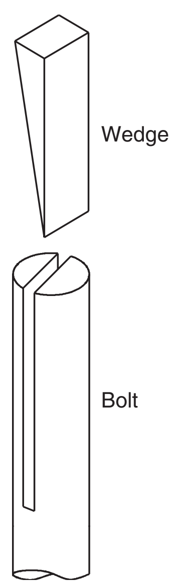

Fig. 28.6 Wedge and slot bolt.

the hole being strong enough to prevent penetration by the wedge end, and on the accuracy of the hole drilled for the bolt. The diameter of such bolts may be 25 or 30 mm. Wedge and slot bolts are not effective in soft rocks.

28.9.2.2 Wedge and sleeve bolt

This generally consists of a 20 mm diameter rod at one end which is a cold-rolled threaded portion. The other end of the rod is shaped to form a solid wedge forged integrally with the bolt and over this wedge a loose split sleeve of 33 mm external diameter is fitted (Fig. 28.7). The anchorage is provided in this case by placing the bolt in the hole and pulling it downwards while holding the sleeve by a thrust tube. Split by the wedge head of the bolt, the sleeve expands until it grips the sides of the tube. Special hydraulic equipment is needed to pull the bolts.
Note: All rock bolts should be grouted very carefully to its full length.

28.9.2.3 Perfo bolts

This method of bolting consists of inserting into a hole, a perforated cylindrical metal tube which has been previously filled with cement mortar and then pushing a plain or ridged bolt. This forces part of the mortar through the perforations in the tube and into intimate contact with the sides of the borehole thus cementing the bolt, the tube and the rock into one homogeneous whole (Fig. 28.8). The relation between the diameter of the borehole and the diameter of perfo sleeve and bolts is given in Table 28.3.

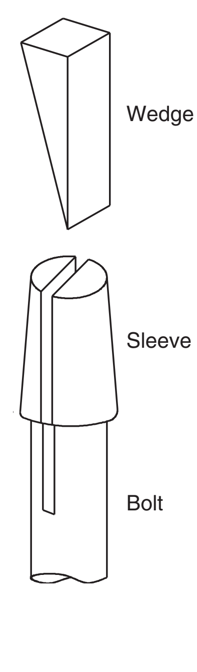

Fig. 28.7 Wedge and sleeve bolt.

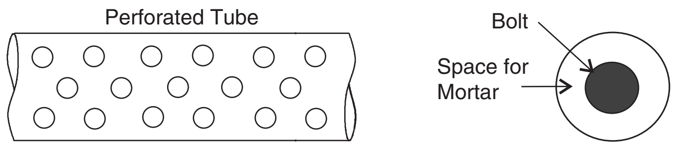

Fig. 28.8 Perfo bolt.

Table 28.3 Diameter of perfo sleeve and bolts.

Diameter of borehole (mm) (1)	Diameter of perfo sleeve (mm) (2)	Diameter of bolts (mm) (3)
40	36	30
38	31	25
31	27	18

Notes: (i) The bolts and anchors should be checked for their straightness within ±1 mm. (ii) Pull-out tests should be done on 5 percent of bolts and anchors to check their capacity (P_{bolt}). (iii) The cement : sand mortar should contain adequate expanding agent to avoid shrinkage cracks along the interface of boltholes and rock mass.

28.9.2.4 Swellex Bolts

These rock bolts are effective in weak rock masses charged with water.

28.9.3 Design

Immediately after a tunnel has been advanced by a length t (Fig. 12.13), the rock in this section expands and settles slightly developing a double arch effect. In the longitudinal direction of the tunnel, the arch rests on the still untouched rock at the front and on the already supported portion at the back (see arrows in Fig. 12.13). The second arch effect, perpendicular to the axis of the tunnel is given by the form of the roof, which is usually an arch in tunnels. The period to which this combined arch will stand without support depends on the geological conditions, the length t and the radius of the tunnel roof. But in most cases, even in badly disintegrated rock, it will be possible to maintain this natural arch for some time, at least a couple of hours. If the natural arch is not supported immediately after mucking, it will continue to sink down slowly until it disintegrates.

The portion that is liable (Fig. 12.13) to fall is generally parabolic in cross section having a depth $t/2$ though the loosening process will never go as deep as this, if the movement is stopped by timely support. Nevertheless, it is recommended that the bolts should not be made shorter than t that is twice the depth of the presumed maximum loosening. The natural surrounding rock of the cavity is in this way transformed into a protective arch. Its thickness is given by the length of the bolts l which should be bigger than t, also $l > B/4$ to $B/3$, as the arch also should have a certain relation to the width of tunnel (B).

The rock requires a pre-stress by bolting and the bolts should follow the static principles of pre-stressing in reinforced concrete as much as possible. As it is not possible to place bolts in the way of stress bars at the lower side of a beam, they should at least be given an oblique position in order to take the place of bent-up bars and stirrups (Fig. 12.12).

With an arch instead of a beam, the shear forces will be greatly reduced by the vault effect but even in arch shaped roofs, shear forces may be caused by joint systems, especially by system of parallel layers like sedimentary formations, schist, etc. Hence the bolts should not only be made to exert a strong pre-stress to the rock but also should be set in a direction which suits best to the static demands of the geological conditions as shown in Fig. 12.12.

Just as a static member of reinforced concrete has to be pre-stressed before receiving the load, the rock also may be pre-stressed by bolting before the load develops. This means that the space t in Fig. 12.13 shall be bolted immediately after blasting and at the same time again as the next round is being drilled. The spacing between bolts/anchors should be less than half the length of bolts/anchors. The pre-tension of the ungrouted bolts is lost after blasting, so rock bolts shall be pre-tensioned again.

28.9.4 Capacity of Rock Bolts/Anchors

The capacity of reinforced rock (p_{bolt}) arch is given by (see Fig. 28.17):

$$p_{\text{bolt}} = \frac{2 \cdot q_{\text{crm}} \cdot l'}{F_{\text{s}} \cdot B} \tag{28.4}$$

where

q_{crm} = minimum uniaxial compressive strength of reinforced rock mass (joint will be criitically oriented somewhere along the arch or tunnel axis),

$$= \left[\frac{P_{\text{bolt}}}{S_{\text{bolt}}^2} - u\right]\left[\frac{1+\sin\phi_{\text{j}}}{1-\sin\phi_{\text{j}}}\right] \geq 0 \tag{28.5}$$

$$\tan\phi_{\text{j}} = \frac{J_{\text{r}}}{J_{\text{a}}} \tag{28.6}$$

u = seepage pressure in the rock mass (t/m^2),
P_{bolt} = tension in bolt or anchor capacity (t),
S_{bolt} = center to center spacing of bolts/anchors (m),
l' = effective thickness of reinforced arch,

$$= l_{\text{bolt}} - \text{FAL}/2 - S_{\text{bolt}}/2 + S_{\text{rock}} \text{ (no shotcrete)}, \tag{28.7}$$

$$= l_{\text{bolt}} - \text{FAL}/2 - S_{\text{bolt}}/4 + S_{\text{rock}} \text{ (in case of shotcrete)}, \tag{28.8}$$

$\leq l_{\text{bolt}} - \text{FAL}/2$,

FAL = fixed anchor length (m) of anchors to develop pull-out capacity of P_{bolt},
= 100 × diameter of anchor bars,
= <1 m in case of mechanically anchored bolts,
S_{rock} = average spacing of fractures in rock (m),
l_{bolt} = length of bolt or anchor (m),
B = size of opening (m),
F_s = mobilization factor of bolt/anchor,
= $3.25 \cdot p_{roof}^{0.10}$ (for mechanically anchored and pre-tensioned bolts), (28.9)
= $9.5 \cdot p_{roof}^{-0.35}$ (for full-column-grouted anchors) and (28.10)
p_{roof} = ultimate support pressure in roof (t/m^2).

The spacing and length of bolts should be so chosen that the estimated capacity of rock bolts/anchors (p_{bolt}) is equal to ultimate support pressure (p_{roof}) or any desired value.

Full-column-grouted bolts are more efficient in poor rock conditions according to equation (28.10) than ungrouted pre-tensioned bolts. For permanent supports, all bolts should be grouted.

28.10 STEEL RIBS

Rock tunnel support systems of steel may be generally classified into the following principal types:

(a) Continuous ribs (Fig. 28.9A),
(b) Rib and post (Fig. 28.9B),
(c) Rib and wall plate (Fig. 28.9C),
(d) Rib, wall plate and post (Fig. 28.9D),
(e) Full circle rib (Fig. 28.9E) and
(f) Yielding arch steel rib with socketed joints.

Invert struts may be used in addition, with types (a) to (d) where mild side pressures are encountered (Fig. 28.9F) or squeezing ground is met, when shotcrete lining begins to fail again and again, despite addition of its extra layers.

28.10.1 Selection of the type of system

28.10.1.1 General

When choosing the type of support system, the following factors may be considered.

(a) Method of attack,
(b) Rock characteristics, its behavior and development of rock load and
(c) Size and shape of the tunnel cross section.

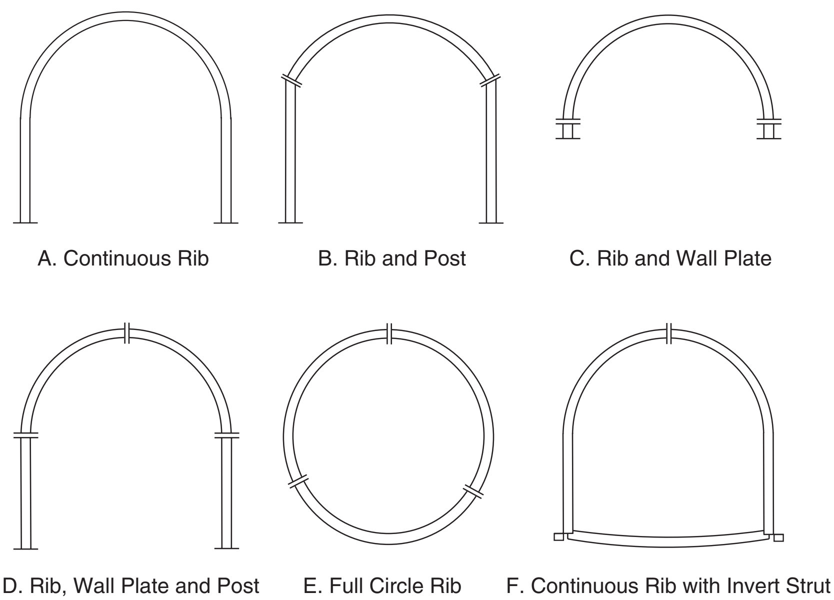

Fig. 28.9 Types of steel support system.

28.10.2 Selection of supports with reference to surrounding strata and shape of tunnel

28.10.2.1 Continuous ribs

This type can be erected more rapidly than the other types and is generally recommended for use in rocks whose bridge action period is long enough to permit the removal of gases and mucking. Invert strut may be used in addition where mild side pressures are encountered (Fig. 28.9F) and squeezing ground is met.

28.10.2.2 Rib and post

This type is generally recommended for use in tunnels whose roof joins the side walls at an angle instead of a smooth curve. It may also be used in large tunnels, such as double-track rail road or two-lane highway tunnels, to keep the size of the rib segments within handling and transporting limitations. Invert strut may be used in addition where mild side pressures are encountered (Fig. 28.9F) and squeezing ground is met.

28.10.2.3 Rib and wall plate

This type is generally recommended for use in tunnels with large cross sections with high straight sides through good rock or in large circular tunnels, where it is possible to support the wall plate by pins and where the strata below the wall plate does not require support. This type of support may also be used for tunnelling through spalling rock, provided spalling occurs only in the roof. However, in many cases it is extremely difficult to establish adequate support for the wall plate at any point above the floor-line due to the irregularity of the overbreak. Please ensure that wall plate does not cause bearing capacity failures of the side wall in the case of very wide openings.

28.10.2.4 Rib, wall plate and post

This type of support permits post spacing to be different from the rib spacing and is generally recommended for use in tunnels with high vertical sides. Invert strut may be used in addition, where mild side pressures are encountered (Fig. 28.9F) and squeezing ground is met.

28.10.2.5 Full circle rib

This type is recommended for use in tunnels in squeezing, swelling and crushed, or any rock that imposes considerable side pressure. The advantage of steel ribs is that the excavation by forepoling is easily done by pushing steel bars into the tunnel face and welding their opposite ends to the steel ribs (to support weak rocks before blasting).

28.10.3 Spacing of ribs

Spacing of ribs is given by:

$$S_{\text{rib}} = \frac{P_{\text{rib}}}{p_{\text{roof}} \cdot B} \tag{28.11}$$

where

P_{rib} = steel rib capacity (T),
S_{rib} = spacing of ribs (m),
B = size of opening (m) and
p_{roof} = ultimate support pressure in roof (t/m^2).

The support capacity of steel ribs (P_{rib}) should be so chosen that the minimum clear spacing between ribs in poor rock condition is 10 cm between flanges or more. Appendix III may be referred for the selection of steel rib capacity.

28.10.4 Selection of steel rib supports

All the types of supports mentioned above are suitable for the full face method of attack for rock where required bridge action period for providing supports is available. Detailed guidelines are suggetsed in Table 13.2.

Rib and wall plate or rib wall plate and post are suitable for heading and bench method. The rib, wall plate and post type may be supplemented by truss panels or crown bars, which are accessories developed to handle heavy loads that come quickly by supporting the intervening ribs while the bench is shot out.

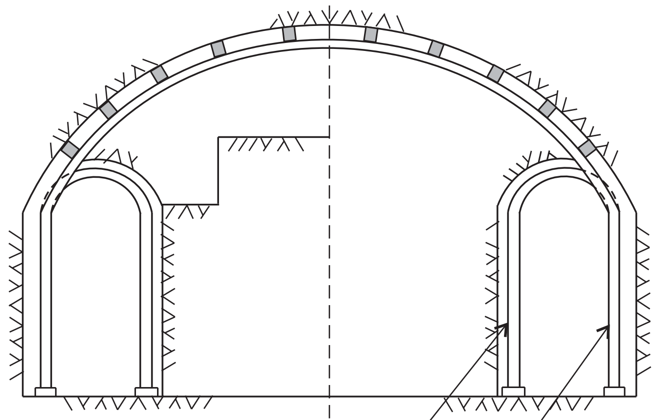

Fig. 28.10 Side drift method.

Where it becomes necessary to drive first the top heading only due to bad roof conditions, the rib and wall plate type of support is generally recommended for use in the heading, and post may or may not be used when the bench is taken out depending on rock conditions.

Where for driving a large size tunnel in poor rock conditions, the side drift method is used, the rib, wall plate and post type of supports are recommended; the wall plate, however, being flat. The posts and wall plates are erected in the drift which is driven ahead at each side at subgrade (Fig. 28.10).

Where extreme conditions are encountered, however, breakups to the crown may be made, leaving a central core. Temporary posts may be quickly placed between the core and the roof at dangerous spots, and crown bars may be slid forward to quickly catch up the roof. The roof ribs should then be placed on the wall plates and securely blocked to take the roof load, after which the temporary posts may be removed.

The side drifts themselves usually need support which should be removed just prior to shooting out the core of the main tunnel and re-used ahead. The support system used for the drifts is hybrid. The outer side consists of the posts and wall plates which later becomes a part of the support for the main tunnel, whereas the inner side is a continuous rib (Fig. 28.10).

28.10.5 Type of support for shafts

For shafts, usually the full circle rib or segmental ribs are recommended depending upon the slope and rock conditions. In vertical shafts, ribs may be hung from top by hanger rods and blocked and packed. The spacing of hanger rods may be worked out as in the case of tie rods keeping in view that they shall be strong enough to support the weight of ribs.

28.10.6 Components of tunnel supports

Design of various components of tunnel supports shall be done in accordance with existing national codes of practice.

28.10.6.1 Ribs

Ribs may be made of structural beams. H-beams or wide flange beams may be preferred to I-beams, as the wider flanges provide more surface for blocking and lagging, and the section has greater resistance against twisting. Channel sections are not recommended as their unsymmetrical section is prone to twisting, and their flanges are narrow. In small tunnels, however, channel bent about their minor axis may be used under small rock loads. When choosing the profiles with different rock loads, it is advisable to select beams of equal depth.

28.10.6.2 Posts

The spacing between the posts may be normally equal to that of the ribs. However, by inserting a wall plate between the ribs and the posts, the spacing of the posts can be made independent of ribs. The posts may be made of H-section. The depth of these should normally be the same as that of the ribs, though in many cases they may be of lighter sections as long as no side pressure is present.

28.10.6.3 Invert struts

Where the wall pressures are present and tunnel section has not been converted to a full circle, it is necessary to prevent the inward movement of the rib or posts feet and in such cases, invert struts should be provided at tunnel subgrade. They should be so attached to the vertical members that they receive the horizontal pressure. They may be curved to form an inverted arch where there is an upthrust from the floor.

28.10.6.4 Wall plates

The following three types of wall plates are commonly used:

(a) Double beam,
(b) Single beam and
(c) Flat wall plate.

The double and single beam wall plates which are intended to resist bending in vertical planes are recommended for use to transmit the loads from the ribs on to block or posts with a spacing different from that of the ribs. Flat wall plates merely serve as an erection expedient and a convenient surface for horizontal blocking. Their resistance to bending in vertical planes being very small, whenever flat wall plates are used, a post shall be placed under each rib.

Double beam wall plates may be made of two I-beams placed side by side, webs vertical with about a 100 mm space between flanges to give access to the clamping bolt and admit concrete (Fig. 28.11). The beams should be spaced by vertical diaphragms welded under each rib seat. Ribs and posts should be clamped by toggle plates and bolts, thus avoiding the time required for matching the boltholes. This method of attachment also permits variable spacing of either or both the ribs and the posts. This type of beam provides a broad surface of contact for blocking and to engage ribs and posts. Its box section makes it stable with respect to rolling and twisting.

Single beam wall plates may be H-beams, with webs vertical. To enable them to transmit vertical loads from rib to post, they may be reinforced at each rib seat with vertical T-shaped plates, if necessary (Fig. 28.12). Attachment of ribs and posts may be made by bolting through the flanges.

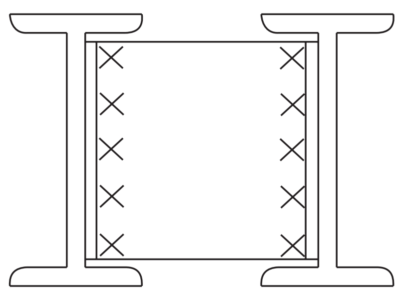

Fig. 28.11 Double beam wall plate.

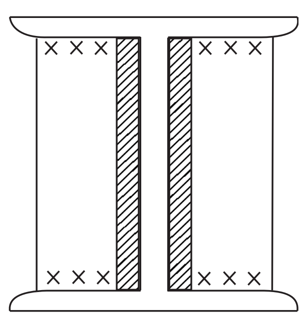

Fig. 28.12 Single beam wall plate.

Flat wall plates may be I-beams or wide flange beams used with their webs horizontal. They function merely as a cap for the posts and a sill for erecting roof ribs. The web should be punched with vent holes also to pass reinforcing rods if the concrete is reinforced.

28.10.6.5 Crown bars

Crown bars may be built up of double channels as shown in Fig. 28.13 or may be H-beams or square timber beams. They are located parallel to the axis of the tunnel either resting on the outer flanges of the ribs already erected or attached to the ribs in hangers. Crown bars are an accessory, a construction expedient intended to carry loads till the rib sets are erected and the rock loads permanently transferred to them. They have one of the two functions to perform (i) to support the roof immediately after ventilation and thereby gain time for the installation of ribs; and (ii) to support the roof or roof ribs over the bench shot thereby relieving or supplementing the wall plates.

28.10.6.6 Truss panels

These are accessories for use with the combination of rib and post types of support, for the heading and bench or top-heading methods of attack and heavy roof loads. Their purpose is to form, in combination with the ribs, a truss to span the gap produced by the bench shot.

The truss panels may be attached to the inside face of the ribs for a distance of one or more ribs ahead of the bench shot as shown in Fig. 28.14 and should be left there until posts are installed, at which time they should be removed and sent up ahead. Attachment may be by means of only two bolts at each rib. The truss thus formed may even be designed to carry the roof over two bench shots making it more convenient to get in the post.

When truss panels are used, no wall plate is required although the flat wall plate may be used to keep the lower ends of the ribs lined up laterally if it is difficult to block the

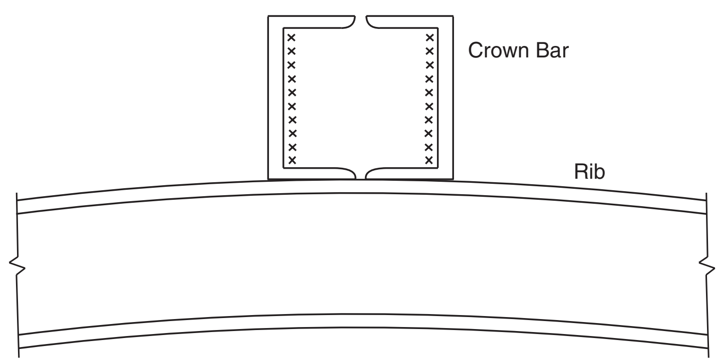

Fig. 28.13 Crown bar.

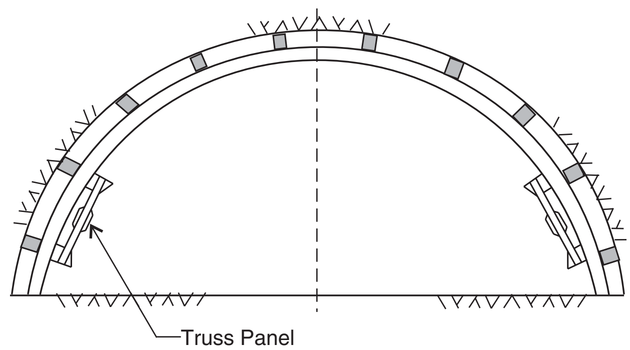

Fig. 28.14 Truss panel.

individual ribs against the rock. The truss panels eliminate the need for wall plate for drifts.

28.10.6.7 Bracing

Longitudinal bracing serves to increase the resistance of ribs and posts to buckling about their minor axis and to prevent a displacement of these set members during blasting. If the space between the ribs or post is bridged by lagging which is firmly attached to the webs, no such bracing is required. The most common types of bracing which is known as the rods and collar braces are shown in Fig. 28.15. The braces may, however, be placed as most convenient.

Tie rods usually may be of 15 to 20 mm diameter, with thread and two nuts on each end. The length shall be at least 100 mm more than the spacing of the ribs. The spacing of the rods may be kept such that the slenderness ratio l/r for ribs is not greater than 60, where l is the spacing of the rods, and r is the least radius of gyration of the ribs. Collar braces may be usually pieces of timber, 75×100 mm, 100×150 mm, 150×150 mm or any conventional size. Holes in pairs shall be provided in the web of ribs and posts for the tie rods. Collar braces should be set in the line between ribs, tie rods inserted and the nuts tightened. Wooden collar braces should be removed before placing final lining.

Spreaders which are additional braces may be angles, channels, or I-beams with a clip angle or plate either bolted or welded on each end to the ribs. These are left in the concrete. In tunnels having steep slopes, tie rods may be replaced by spreaders.

28.10.6.8 Blocking

It is generally done by using timber pieces tightly wedged between the rock and the rib.

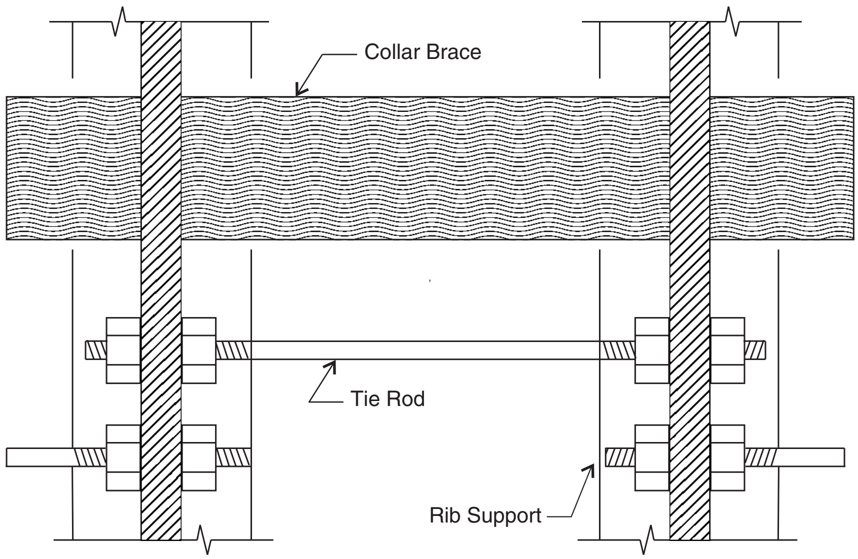

Fig. 28.15 Collar brace and tie rod.

28.10.6.9 Lagging

It performs one or more of the following functions:

a) To provide protection from falling rock or spalls;
b) To receive and transfer loads to the rib sets;
c) To provide a convenient surface against which to block in case it is not convenient to block directly against the rib, because of irregular overbreak;
d) To provide a surface against which to place back packing;
e) To serve as an outside form for concrete lining, if concrete is not to be poured against the rocks and
f) To divert water, and to prevent leaching and honeycombing of concrete.

Lagging may be made either of steel, precast concrete or timber. Steel laggings may be made out of channels, beams, beams and plates and liner plates. Liner plates, which are pressed steel panels may also be used with or without ribs depending upon the rock conditions. It is recommended that use of timber in underground work should be minimized as far as practicable, since timber once fixed can be rarely removed safely and likely to deteriorate and prove a source of weakness. Total prohibition of timber is, however, not practicable.

The spacing of lags should be closest at the crown, increasing down to spring line. On the side only an occasional lag should be used, if necessary. Close lagging should be employed where rock conditions make it necessary.

28.10.6.10 Packing

The function and type of packing depends on the rock condition. In dry tunnels through jointed rock, packing is only used to fill large cavities produced by excessive overbreak. In broken, crushed or decomposed rock it serves to transfer the rock load to the lags, thereby acting as a substitute for excessive blocking. In squeezing rock it provides continuous contact through the laggings with the rib sets. In jointed water-bearing rock it has primarily the function of a drain.

Dry pack, which usually consists of tunnel spoil (hard) shoveled or hand packed into the space between the lagging and the rock, is recommended for use only where excessive rock loads are not likely to develop. It may be placed simultaneously with the erection of the lagging. Starting at the lowest point, a few lags may be placed and tunnel spoil (hard rock) shoveled in behind. This procedure may be carried upto the crown at which point it is necessary to pack endwise.

Concrete packing It is recommended for use where considerable rock loads are anticipated. However, its use is not recommended in case the tunnel supports are designed as yielding supports. Concrete packing may be M10 concrete. It may be placed by manual labor or by pneumatic placer to the possible extent. Where excessive loads are anticipated, concrete packing should start from the inner flanges of the steel support so as to embed the whole steel supports in concrete. In such cases, it is recommended that precast concrete may be used as additional lagging between two adjacent ribs so as to serve the purpose of form work.

Pea gravel packing Pea gravel packing by blowing of gravel is recommended in shield-driven tunnels to fill the annular space around the lining left by the advancing tail of the shield. Gravel should be blown through the grout holes provided in the liner segments as the shield is shoved forward.

Note: Pea gravel packing is considered highly desirable kind of packing for most purposes but it has not been practiced so far in rock tunnels.

28.10.6.11 Grouting

Grouting to fill any space outside the concrete lining may be usually done after the main concrete lining is in place. But there are occasions where it is desirable to do grouting at low pressure soon after concrete packing. When the main concrete lining is likely to be delayed considerably it is desirable to do grouting at low pressure after concrete packing.

28.10.7 Factors determining spacing and layout of supports

The strength and spacing of rib system should be determined by the rock load. For a given rock load and cross section of tunnel, the spacing between the ribs and whether the ribs shall be in two or more pieces shall be worked out. The spacing of the ribs should be so chosen that the sum of the cost of ribs and lagging is minimum. For preliminary designs in ordinary rocks, the depth or rib section may be taken as 60 to 75 mm for every 3 m of bore diameter with ribs spaced at about 1.2 m for moderate loads, 0.6 to 1.0 m for heavy loads and 1.6 m for very light loads. Equation (28.11) is recommended for this purpose.

For junctions, plugs and control chamber, etc. supports should be designed to suit special features of the work and its construction procedures. Wooden or concrete blocks of suitable size and thickness may be provided, if necessary, in the bottom portion to provide adequate bearing area to the rib.

In tunnels, where supports are not to be used as reinforcement, they may be installed plumb (vertical) or perpendicular to the axis of the tunnel depending on tunnel slope and as found convenient. However, where supports are to be used as reinforcement in pressure tunnels, they may be installed at right angles to the tunnel axis, if practicable.

For speed of erection of supports it is essential to:

(a) design a support system with the minimum number of individual members, consistent with construction convenience;
(b) design the joints with utmost simplicity and absolute minimum number of bolts and
(c) fabricate the members with simple bolt and wrench clearances. Time consuming close fits shall be avoided.

28.10.8 Design

28.10.8.1 Stresses

Permissible stress in steel shall be in accordance with the code of practice in a nation. However, if the ribs are bent cold, the maximum permissible fiber stress in steel should be 165 MPa ($= 0.66 \times$ yield strength).
Note: The yield strength of steel ribs and steel structures is 250 MPa. The yield strength of (TMT) rock bolts and reinforcement bars in the concrete is 415 MPa. The yield strength of thin steel fibers in SFRS is 1000 MPa.

28.10.8.2 Ribs

Rock load may be assumed to be transmitted to the ribs at blocking points, each blocking point carrying the load of the mass of rock bounded by four planes, namely, the longitudinal planes passing through mid-points between the blocks and transverse planes passing through mid-points between the ribs to a height equal to the acting rock load. The blocking points may be assumed to be held in equilibrium by forces acting on it in the same manner as panel points in a truss. Values of thrust in the rib may be computed by drawing the force polygon. Ribs should be designed for the thrust thus computed taking into account

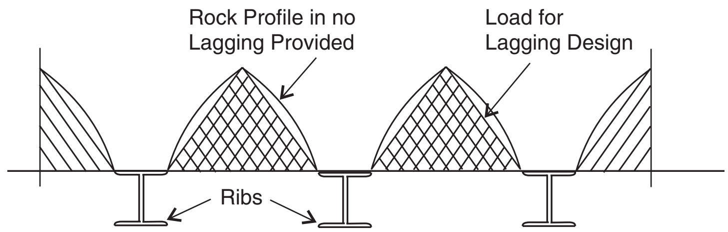

Fig. 28.16 Loading diagram for lagging.

the eccentricity of this thrust with reference to the rise of the arc between the blocking points which will cause flexural stresses in addition to direct stresses.

28.10.8.3 Lagging

Lagging may be designed for the load of rock mass as shown in Fig. 28.16.

28.10.8.4 Linear plates

Where only linear plates are used for support, their cross-sectional area and joints should be designed to transmit the thrust. It should be ensured that linear plates are thoroughly in contact with ground so that the passive resistance is developed and no bending moments are induced. For tunnels with more than 3 m diameter linear plates may be reinforced by I-beams. Where linear plates do not form a ring and are used in top half ribs they should be designed as lagging. The thickness of linear plates may vary from 3 to 10 mm depending upon the size of bore and loads encountered. The size of bolts may vary from 12 to 15 mm diameter.

28.10.8.5 Joints

Butt joints should be preferred to spliced joints. In soft grounds and poor rock, welding of joints in the field should be avoided as far as possible.

28.11 GROUTING IN PRESSURE TUNNELS

28.11.1 General

Grouting is carried out to fill discontinuities in the rock by a suitable material so as to improve the stability of the tunnel roof or to reduce its permeability or to improve the properties of the rock. Grouting is also necessary to ensure proper contact of rock face of the roof with the lining. In such cases, the grouting may be done directly between the two surfaces or the process of grouting may be used to fill the voids in the rubble packing where used. All the three types of grouting may not be required in all cases. The grouting procedures should aim at satisfying the design requirements economically and in conformity with the construction schedules. The basic design requirement generally involve the following:

(a) Filling up the voids, cavities, between the concrete lining and rock and/or between the concrete and steel liner;
(b) Strengthening the rocks around the bore by filling up the joints in the rock system;
(c) Strengthening the rock shattered around the tunnel;
(d) Strengthening the rock, prior to excavation by filling the joints with cementing material and thus improving its stability;
(e) Closing up water-bearing passages to prevent the flow of water into the tunnel and/or to concentrate the area of seepage into a channel from where it can be easily drained out and
(f) To increase the capacity of grouted rock arch.

Before drawing up the specifications for grouting, the design requirements should be established. In general for all underground structures, grouting is an universal requirement for all concrete lined tunnels. Design requirements are only to establish the maximum allowable pressure at which this grouting is to be carried out and the zone in the cross section and the spacing of grout holes, both in the direction of the tunnel. For consolidation grouting, the design requirement to be established is the thickness of the rock stratum around the tunnel that is to be strengthened and made impermeable, the pressure and the spacing pattern of holes. This will determine the depth to be grouted.

For tunnels, the commonly used procedures are to continue grouting to refusal at the design pressure in each hole or to interrupt the grouting if there is heavy intake with little or no pressure build up, indicating very open structures and escape of grout to a long distance.

28.11.2 Pattern, depth and arrangement of holes

28.11.2.1 Backfill grouting

The purpose of backfill grouting is to fill the space left unfilled (with concrete) between the concrete lining and the rock surface in the arch portion of any tunnel or cavity.

Backfill grouting should be done after the concrete lining has gained strength. The period of waiting may be from 21 to 28 days. In case of precast lining segments, this restriction of waiting will not apply and the grouting may be done immediately after the segments are erected. Backfill grouting is limited to the arch portion of a tunnel or cavity and may not be required in case of shafts if the concrete is poured vertically.

The grout holes at the crown should be placed 5 to 10° from the crown, alternately in the left and right of the crown. In addition to the crown hole there should be two more holes, one on either side of the crown. These holes will be 90° apart and will be located such that one of these two holes are at 22.5° from the crown alternately on the right and left of the crown. Such sections shall be normally 3 m apart. The exact location of the holes may be varied or additional holes provided depending upon the actual excavation profile at any section. The actual spacing of sections may also be varied on similar considerations. It should, however, be also adjusted to suit the length of the arch shutter used in such a way that there is no hole at the joint and the normal pattern of holes is more or less uniform in the shutter length.

In the case of circular or horse-shoe tunnels, in addition to these holes, two holes (one on either side), located roughly at 45° on either side of the invert should be used. The location should be such that the holes are about 45 to 60 cm, above the junction of the invert and arch. The mortar used for backfill grout may normally consist of cement, sand and water mixed in the proportion of 1:1:1 by weight. It may, however, be suitably modified if its conditions so warrant. The size of grouting sand should be determined for each job by actual experimentation as it would depend on the type of sand, equipment available and fracture spacing. Backfill grouting should normally be done at a pressure of 2 kg/cm^2 (0.2 MPa).

28.11.2.2 Contact grouting

The aim of contact grouting is to fully pack up the space between the concrete lining and the rock surface or the space between the steel liner and concrete lining caused by shrinkage or left unfilled even after backfill grouting. This is required for fulfilling the design assumption of the rock/concrete taking part of the load along with the lining of water pressure tunnels and to prevent local accumulation of water, if any, and building up local pressure (Chapter 23).

Contact grouting may be done after the concrete lining has gained strength to withstand the pressure and shrinkage, if any, has taken place. The usual minimum period of 25 to 28 days of waiting should be allowed.

The contact grouting should be limited to only the top arch (90° on either side of the crown) of pressure tunnels. In case of vertical shafts and steel liner, contact grouting should be done along the full periphery. In case of steel liners, the grouting should also be done all round its periphery in the case of penstocks.

The holes at the crown should be placed 5 to 10° from the crown, being alternately to the left and right of the crown. In addition to the crown hole, there should be two more holes one on either side of the crown in each section. These holes will be 90° apart and will be located such that one of the two holes is at 22.5° from the crown, being alternately on the right and left of the crown. Such holes may normally be 3 m apart.

In the case of circular or horse-shoe tunnels, in addition to these holes, two holes (one on either side), located roughly at 45° on either side of the invert should be used. The location should be such that the holes are about 45 to 60 cm above the junction of the invert and arch as in the case of backfill grouting. The depth of holes for contact grouting should be such that at each location, the holes extend 30 cm beyond the concrete lining into rock.

28.11.2.3 Consolidation grouting

The aim of consolidation grouting is to fill up the joints and discontinuities in the rock up to the desired depth. Consolidation grouting should always be done after the backfill grouting is completed in a length of at least 60 m behind the chainage of grouting (towards the portal).

Consolidation grouting should be usually done all round the tunnel, and for a uniform radial distance from the finished concrete face. The grout may be determined by the designer based on the design of the concrete lining and the extent to which cracks are assumed to extend in rock when the lining is stressed by internal water pressure. Usually the depth should be between $0.75D$ and D where D is the finished diameter of the tunnel, except in special reaches where it could be more. The maximum depth so far used in the Himalaya is about 15 m.

The pattern of grout holes for consolidation may be a set of holes in one vertical plane, such a plane being called the grout plane. The spacing of the grout planes will depend upon the structural formation of rock and the travel of grout at the specified pressure. The actual spacing as in the case of contact grouting should also be adjusted in the field to suit the length of the shutter used for concreting. In this plane, the number of holes may normally be four for small size tunnels and six for large size tunnels. The arrangement should be staggered in alternate grout planes, by about half the spacing between the holes along the periphery in the plane. In special locations the number of holes may be increased. The top three holes in grout pattern may be used for both backfill, contact and consolidation grouting.

Around shafts and large opening like powerhouse, the grout pattern may be similar, but the number of holes in the plane may be increased depending on the size, but the spacing should generally exceed the depth of the hole.

Contact grouting would not generally be necessary where consolidation grouting had been done. However, it should be decided by actual contact grouting in some holes after consolidation grouting. Depending upon the rock formations and the grout intake, the consolidation grouting should be done in one or more stages with increasing pressures. Maximum grout pressure should not normally exceed twice the design load on lining or support pressure systems as the case may be.

28.11.3 Pressure to be used for grouting

The pressures to be used for grouting will depend on the rock characteristics, the design requirements and the rock cover. With adequate rock cover (more than three times the diameter of the tunnel), the first two parameters will govern. For backfill grouting the maximum pressure recommended is 5 kg/cm^2 (0.5 MPa). For consolidation grouting, a maximum pressure of 7.0 kg/cm^2 (0.7 MPa) is normally recommended but this may be increased up to 20.0 kg/cm^2 (2.0 MPa) in special cases, provided that there is adequate cover and the joints in the rock are not likely to open up by this pressure.

The pressure gauge should be watched constantly so that the pressure on the grout is regulated as long as grouting is in progress. Any desired increase or decrease in the grouting pressure is obtained by changing the speed of the grout pump. When the grout in the supply line becomes slugging, the grout hole valve should be closed and the blow off valve is opened so that the supply line may be flushed or washed.

28.11.4 Testing for efficacy of grouting

This testing may be done by drilling the holes in between the grout planes and by testing water intake in these test holes. If this is compared with the water test made before the grouting, this water intake will give an indication of the efficacy of grouting. Further grouting of this test hole and intake in this hole will give further indications. It is only after these tests that the engineer-in-charge may decide on increasing the number of grout planes if required.

28.11.5 Capacity of grouted rock arch (Figs 28.17 and 28.18)

$$p_{gt} = \frac{2 \cdot q_{gt} \cdot l_{gt}}{F_{gt} \cdot B} \tag{28.12}$$

where

q_{gt} = uniaxial compressive strength of grouted rock mass (t/m^2),
l_{gt} = thickness of grouted arch (m),
B = size of the opening (m),
F_{gt} = mobilization factor of grouted arch,
= $9.5 \cdot p_{roof}^{-0.35}$ and
p_{gt} = ultimate support pressure in roof (t/m^2).

In case of water-charged rock mass or post-construction saturation of rock mass, heavy support pressure and seepage pressure may develop on the concrete lining or shotcrete lining. The extra high support pressures/seepage pressures may be taken care off by grouted rock arch (equation (24.8)).

28.12 DESIGN OF INTEGRATED SUPPORT SYSTEM

28.12.1 General

A semi-empirical approach is used to determine the capacity of support system consisting of shotcrete, reinforced rock arch, steel rib and grouted rock arch. In Fig. 28.17, the dotted

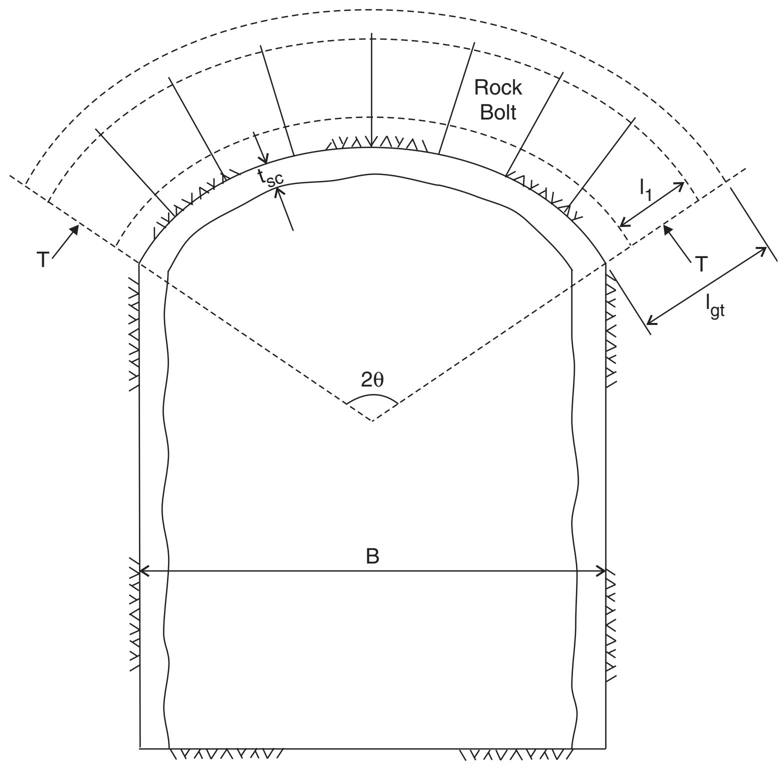

Fig. 28.17 Capacity of reinforced rock arch.

line shows the effective width of the reinforced rock arch. The load carrying capacity of the reinforced rock arch is dependent on the minimum uniaxial compressive strength of the reinforced rock arch.

Fig. 28.17 shows that the total support pressure ($p_{roof} + u$) will be equal to the sum of the capacities of shotcrete, reinforced rock arch, steel rib and grouted rock arch. Simple hoop action is assumed as illustrated in Fig. 28.17. The arch subtends an angle of 2θ at its center (=180° for tunnels).

Ultimate support pressure = Total capacity of support system (Singh et al., 1995)

$$(u + p_{roof}) = p_{sc} + p_{bolt} + p_{rib} + p_{gt} \tag{28.13}$$

$$\text{or} \quad (u + p_{roof}) = \frac{2 \cdot q_{sc} \cdot t_{sc}}{F_{sc} \cdot B} + \frac{2 \cdot q_{crm} \cdot l'}{F_s \cdot B} + \frac{P_{rib}}{S_{rib} \cdot B} + \frac{2 \cdot q_{gt} \cdot l_{gt}}{F_{gt} \cdot B} \tag{28.14}$$

In the above simple equation, it is assumed that shotcrete will shear along a length of arch approximately equal to ($F_{sc} \cdot B$). All the notations in the above equation have the same meanings as described earlier. In the case of SFRS, notation "sc" is replaced by "fsc."

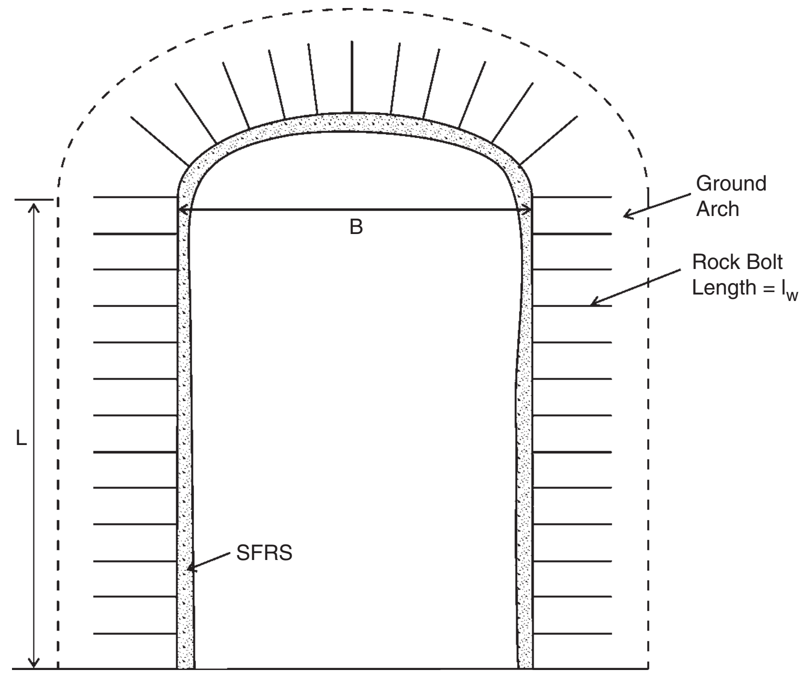

Fig. 28.18 Design of wall reinforcement of caverns.

The following trends have been obtained:

(i) Pre-tensioned bolt is more effective in good rock conditions.The efficiency of pre-tensioned bolt decreases slightly for poor rocks due to creep and loss of tension.
(ii) The full-column-grouted untensioned anchors are more effective in poor rock conditions than in good rock conditions. The reason may be that the anchors are subjected to large radial strains in poor rock masses leading to more tension induced in anchors. Fig. 28.18 illustrates better performance of grouted anchors.

28.12.2 Application of semi-empirical design approach

For tunnels located near faults/thrusts (with plastic gouge) in seismic areas, the ultimate support pressures may be increased by about 25 percent to account for accumulated strain in the rock mass along the fault. If the tunnel is away from the fault by a distance *B*, the seismic effect is negligible (Chapter 21).

The support pressure, due to squeezing out of gouge from the shear zone, may be estimated by applying Terzaghi's theory of arching which indicates that the support pressure, with gouge, will increase with the width of the shear zone. As such, the treatment of shear zone is essential to bear the high support pressure as shown in Fig. 28.5. However, in thin shear zone (approx < 50 cm), the support pressure is very small and hence, there is no cause of worry.

The capacity of shotcrete and reinforced rock arch is calculated by trial and error. The design parameters are selected, so that the ultimate pressure is equal to the design capacity. If support pressure is high, say more than 5 kg/cm^2 (0.5 MPa), steel ribs may be used and embedded in shotcrete. The spacing of steel ribs may be estimated until equation (28.14) is satisfied. The design philosophy is illustrated in Fig. 28.19. In case of water-charged rock mass, experience shows that the NGI design tables do not give useful parameters as they neglect the seepage pressure. Equation (28.12) may then be used to find out the extent upto which the rock mass should be grouted. Grouting is possible generally where thick shotcrete has been provided to take high grouting pressure.

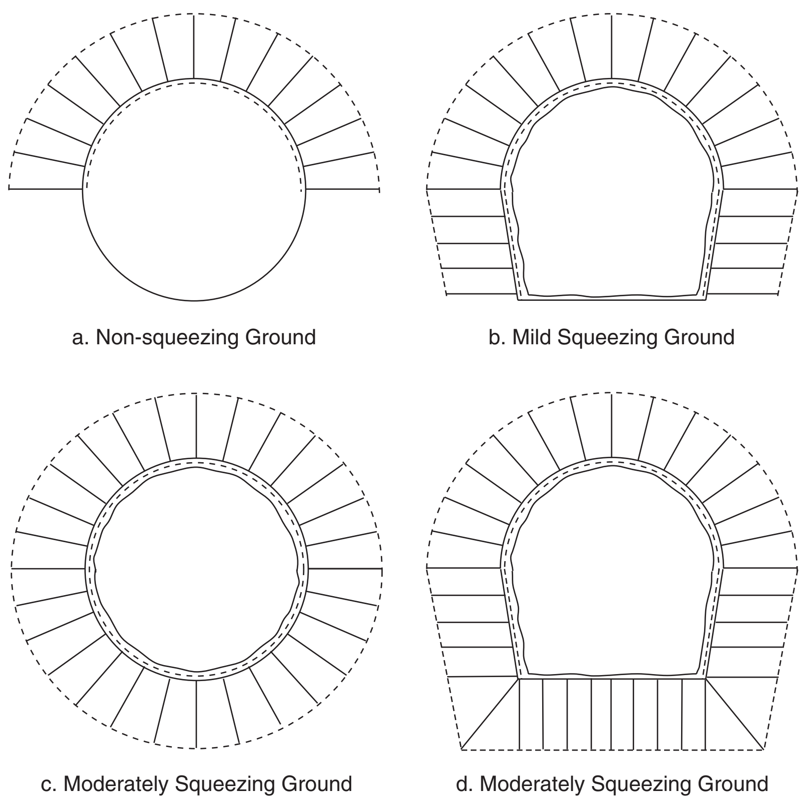

Fig. 28.19 Design philosophy of rock reinforcement in tunnelling.

In very poor rock conditions, assumptions are generally invalid. Hence, special specifications need to be followed to treat thick shear zones, rock burst conditions and highly squeezing conditions.

Appendix II presents a software TM for design of support system for the tunnels and caverns in the rock mass with and without shear zone on the basis of semi-empirical method.

In case of water and power tunnels, seepage pressure may be assumed equal to the internal water pressure and the worst case is when the tunnel is empty and seepage water pressure acts on the shotcrete lining. If required, rock mass may be grouted to take high support and seepage pressure. Alternatively, concrete lining may be designed according to the design criteria.

28.12.3 Tunnel through intra-thrust zone

In Himalaya and other young mountains, the tunnel has to pass through intra-thrust zone in some complex geological and tectonic situations. The faults and thrust are subjected to slip over a long period of time due to very slow tectonic movement of Indian plate with respect to Chinese plate. It is therefore essential to build a segmented concrete lining so that segments of the concrete lining can slip with respect to each other with time. The design decision should be taken on the basis of instrumentation data within the intra-thrust zone (Chapter 20). Duddek and Erdmann (1985) have suggested a design approach for tunnel in soil.

28.12.4 Experience in poor rock conditions

Fiber reinforced shotcrete has proved very successful in 6.5 km long tunnel in Uri Hydel Project in India. The main advantage is that lesser thickness of fiber reinforced shotcrete is needed. No weld mesh is required to reinforce shotcrete. Its rebound is less due to steel fibers provided shotcrete is graded properly and sprayed properly. It is costly but would become cost-effective on the extensive demand for the steel fiber reinforced shotcrete.

The experience with the use of mesh (weld mesh, etc.) has been unsatisfactory where there were overbreaks in the tunnel after blasting. The weld mesh was spread between bolts and shotcrete was sprayed. Soon mesh started rebounding shotcrete and could not penetrate inside the mesh and fill the gap between mesh and the overbreak. Consequently, gaps were left out above shotcrete and sound to hammering could indicate hollow gaps above. Further, weld mesh started vibrating due to blast vibrations and it caused loosening of the shotcrete. As such, the experience with mesh reinforced shotcrete has been unsatisfactory in case of overbreak.

It is recommended that mesh should not be used where even surface of tunnel is not available due to overbreak, provided shotcrete is used. However, the thickness of shotcrete should be increased by 1 cm (approx).

28.12.5 Flowing ground

The crushed rock/sand may flow down like a fluid rapidly when the tunnel passes through a water-charged wide shear zone (>1 m) unexpectedly. Naturally it is squeezing ground condition also. The broken zone expands further with time and the process of flow is repeated again and again until the water gushes out and the tunnel is filled by rock debris upto its roof.

Fig. 28.20 shows a strategy to tackle the flowing ground condition. First the water should be drained out by making advance probe hole within the tunnel face. The rate of discharge should be monitored with the help of "V" notch at the end of the tunnel. Experience suggests that shear zone should be drained and grouted well before tunnelling. Fortunately crushed rock is groutable in most cases. The face of the tunnel should also be grouted as the face is unstable.

The serious flowing ground conditions were met in Maneri Bhali Stage I Head Race tunnel, Satluj–Beas Link tunnel, Dulhasti Hydroelectric project and Tala Hydroelectric Project (Bhutan) in upper Himalaya. The tunnelling work has to be stopped for many months. The discharge was so unprecedented that workers were washed away. In all cases, wide shear zones were punctured unexpectedly.

The experience is that the rock ring of thickness of about radius of tunnel should be grouted well in advance of puncturing a possible shear zone. Then the tunnel face should be excavated by smooth blasting. The rock mass should be obviously further grouted until shear zone is crossed safely. It is easy to say but challenging to achieve. In field, the grout holes are inclined and cone of grouted rock mass may be formed ahead of tunnel face. Grouting is done such that successive cones are formed with minimum thickness l_{gt} equal to a.

The analysis also proves that suggested thickness of grouted rock ring should be able to withstand support pressure which is equal to the overburden pressure.

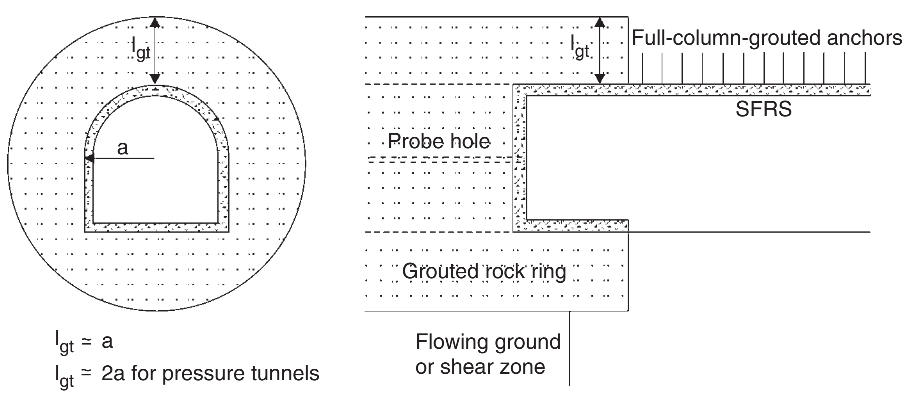

Fig. 28.20 Grouting ahead of flowing ground.

28.13 SPECIAL REQUIREMENTS

(i) Scaling of loose pieces of rock should be done thoroughly first of all.

(ii) The first layer of 25 mm thick shotcrete is sprayed immediately to make opening reasonably smooth. Then weld mesh is spread and nailed into this shotcrete layer. Next rock anchors (or rock bolts) are provided. Finally, additional layers of shotcrete are sprayed according to the design. Perfect bond between shotcrete and rock mass is the trick of the trade. A strong bond results in more effective thickness of shotcrete depending on the size of rock blocks.

(iii) Sometimes the grouting of long bolts is not done satisfactorily because of lack of supervision and difficulties with the expanding agent (aluminum powder or expanding agent is seldom added). Therefore, pull-out tests should be conducted on at least 5 percent of the bolts to check their quality. If required, extra bolting should be done to strengthen the support system. (Pre-tensioned bolts must be tensioned again due to loss of pre-tension after initial round of blasting to excavate the tunnel face. Then they are grouted with cement mortar for long life).

(iv) For deep and long tunnels in complex geological conditions, a 20 m long probe hole of 75 mm diam. should be drilled inside the tunnel face in order to obtain an accurate picture of geological conditions in advance of the tunnelling. The probe hole will also dissipate seepage pressure slowly within the water-charged rock mass, which is likely to be punctured during tunnelling. This technique will also avoid flash floods soon after blasting and consequent loss of life and damage to the support system, provided the side drain is of adequate capacity.

(v) In poor rock masses, spiling bolts (inclined towards the tunnel face) may be installed before blasting to increase the stand-up time of the tunnel roof (Bischoff et al., 1992). Shotcrete is then sprayed on roof, and then the spiling bolts are installed. In the final cycle, roof bolts are installed.

(vi) In cases involving argillaceous rocks and swelling rocks, where the bond with shotcrete is poor, the thickness of the shotcrete should be increased by about 30 percent.

(vii) At the intersection of the approach tunnel and the main tunnel, the support capacity of the reinforced rock arch of the approach tunnel should be strengthened by 100 percent, up to a distance equal to three times the bolt length of the main tunnel. This will help the reinforced rock arch of the approach tunnel to bear the thrust from the reinforced arch of the main tunnel.

(viii) In highly squeezing ground conditions ($H \gg 350\ Q^{1/3}$ m and $J_r/J_a < 0.5$), steel ribs with struts should be used when the SFRS fails repeatedly despite the addition of more layers. With steel ribs, excavation by forepoling is easily accomplished by pushing steel rods into the tunnel face and welding the opposite ends to the ribs. Floor heaving can be prevented by rock bolting of the floor. Some delay, but less than the stand-up time, is necessary to release the strain energy of the broken zone. Smaller blast holes will also be helpful and broken zones should be instrumented. In such cases, the tunnel closure must be arrested before it reaches 4 percent of the width of the tunnel.

(ix) For very poor rock masses, steel ribs should be installed and embedded in the shotcrete to withstand the high support pressures.

(x) In the case of steel ribs in large tunnels and caverns, haunches should be strengthened by installing more anchors to help withstand the heavy thrust due to the ribs.

(xi) For treatment of shear zones, crossed rock anchors/bolts should be provided across the shear zones. After the gouge has been cleaned to the desired extent, anchors are connected to the weld mesh and, finally, dental shotcrete is back-filled. In wide shear zones, shotcrete reinforcement is also placed to help withstand high support pressure, except in case of SFRS. The anchors should be inclined according to the dip of the shear zone to stop squeezing of the gouge, and thereby stabilize the deformations (Fig. 28.5).

(xii) In the case of an unstable portal, horizontal anchors of equal length should be provided inside the cut slope, so that it acts as a reinforced rock breast wall.

(xiii) In rockburst-prone regions, resin anchors and steel fiber reinforced shotcrete should be used to increase the ductility of the support system and thus convert the brittle mode of failure into the ductile mode of failure. Rock anchors should be installed immediately on the rock burst side of the tunnel.

(xiv) The concrete lining for water/pressure tunnels should be laid far away from the tunnel face within the squeezing ground where the broken zone is stabilized (i.e., about four times the radius of the broken zone). In addition, the concrete lining should be segmented within an active thrust zone to allow relative movement along the faults/thrust. The concrete/RCC lining may be built after the rate of deformation has reduced to less than 2 mm/month.

(xv) The thickness of SFRS should not be less than 70 mm in the under sea tunnels.

Example

Two parallel road tunnels are being constructed for 6 lanes in the basalts. The tunnels are D-shaped with diameter (B) of around 16 m and 2 m high side walls with clear spacing of 20 m. The maximum overburden (H) is 165 m. The rock mass parameters are, RMR = 73, Q = 10, $J_a = 1.0$, $J_r = 3.0$ and $J_w = 1.0$ (minor seepage from side walls). The construction engineers want rapid rate of tunnelling and life of support system should be 100 years. The uniaxial compressive strength of SFRS is 30 MPa and its flexural strength is 3.7 MPa.

The short-term support pressure in roof may be assessed by the following correlation (equation (4.6)) for arch opening given by Goel and Jethwa (1991).

$$p_{\text{roof}} = \frac{7.5B^{0.1}\,H^{0.5} - \text{RMR}}{20\ \text{RMR}} = \frac{7.5 \times 16^{0.1} \times 165^{0.5} - 73}{20 \times 73} = 0.037\,\text{MPa}$$

The ultimate support pressure is read by the chart (Fig. 5.2) of Barton et al. (1974) as follows (the dotted line is observed to be more reliable than correlation).

$$p_{\text{roof}} = 0.9 \times 1 \times 1\ \text{kg/cm}^2 \quad \text{or} \quad 0.09\,\text{MPa}$$

(The rock mass is in non-squeezing ground condition ($H < 350\ \text{Q}^{1/3}$) and so $f' = 1.0$. The overburden is less than 320 m and so $f = 1.0$.)

It is proposed to provide the steel fiber reinforced shotcrete (SFRS) [and no rock bolts for fast rate of tunnelling]. The SFRS thickness (t_{fsc}) is given by the following correlation (equation (28.1)).

$$t_{\text{fsc}} = \frac{0.6Bp_{\text{roof}}}{2q_{\text{fsc}}} = \frac{0.6 \times 1600 \times 0.09}{2 \times 5.5} = 8\,\text{cm}$$
$$= 16\,\text{cm (near portals)}$$

The tensile strength of SFRS is considerd to be about one-tenth of its UCS and so its shear strength (q_{sc}) will be about, $2 \times 30/10 = 6.0$ MPa, approximately 5.5MPa (UTS is generally lesser than its flexural strength). Past experience is also the same.

The life of SFRS may be taken same as that of concrete in the polluted environment that is about 50 years. Life may be increased to 60 years by providing extra cover of SFRS of 5 cm. If SFRS is damaged latter, corroded part should be scrapped and new layer of shotcrete should be sprayed to last for 100 years. So recommended thickness of SFRS is

$$t_{\text{fsc}} = 13\,\text{cm}$$
$$= 21\,\text{cm (near portals)}$$

The width of pillar is more than the sum of half-widths of adjoining openings in the non-squeezing grounds. The width of pillar is also more than the total height of the larger of two caverns (18 m), hence proposed separation of 20 m is safe.

The following precautions need to be taken:

(i) The loose pieces of rocks should be scrapped thoroughly before shotcreting for better bonding between two surfaces.
(ii) Unlined drains should be created on both the sides of each tunnel to drain out the ground water and then should be covered by RCC slabs for road safety.

(iii) The tunnel exits should be decorated by art and arrangement should be made for a bright lighting to illuminate well the tunnels to generate happy emotions among road users.

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29

Critical state rock mechanics and its applications

"All things by immortal power near or far, hiddenly to each other are linked."

Francis Thompson
English Victorian Post

29.1 GENERAL

Barton (1976) suggested that the critical state for initially intact rock is defined as the stress condition under which Mohr envelope of peak shear strength reaches a point of zero gradient or a saturation limit. Hoek (1983) suggested that the confining pressure must always be less than the unconfined compression strength of the material for the behavior to be considered brittle. An approximate value of the critical confining pressure may, therefore, be taken equal to the uniaxial compressive strength of the rock material.

Yu et al. (2002) have presented a state-of-the-art on strength of rock materials and suggested a unified theory. The idea is that the strength criterion for jointed rock mass must account for the effect of critical state in the actual environmental conditions.

The frictional resistance is due to the molecular attraction of the molecules in contact between smooth adjoining surfaces. It is more where molecules are closer to each other due to the normal stresses. However, the frictional resistance may not exceed the molecular bond strength under very high confining stresses. Hence, it is no wonder that there is a saturation or critical limit to the frictional resistance (Prasad, 2003). There should be limit to everything in the nature.

Singh and Singh (2005) have proposed the following simple parabolic strength criterion for the unweathered dry isotropic rock materials as shown in Fig. 29.1.

$$\sigma_1 - \sigma_3 = q_c + A\sigma_3 - \frac{A\sigma_3^2}{2q_c} \quad \text{for } 0 < \sigma_3 \leq q_c \tag{29.1}$$

Tunnelling in Weak Rocks
B. Singh and R. K. Goel

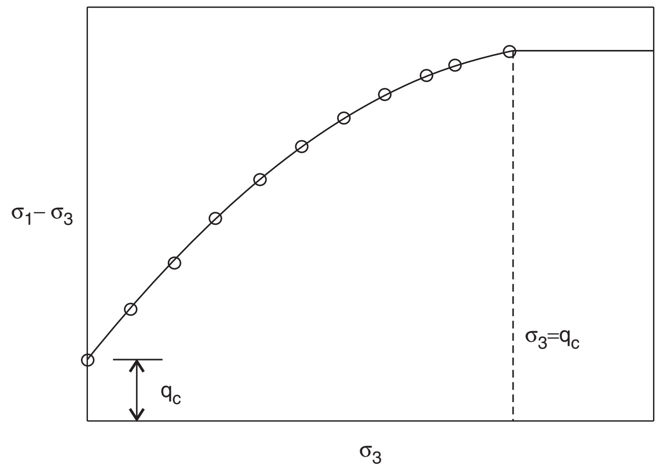

Fig. 29.1 Parabolic strength criterion.

where

$A \;=\; \frac{2\sin\phi_p}{1-\sin\phi_p}$

ϕ_p = the peak angle of internal friction of a rock material in nearly unconfined state ($\sigma_3 = 0$) and

q_c = average uniaxial compressive strength of rock material at $\sigma_3 = 0$.

It may be proved easily that deviator strength (differential stress at failure) reaches a saturation limit at $\sigma_3 = q_c$ that is,

$$\frac{\partial(\sigma_1 - \sigma_3)}{\partial\sigma_3} = 0 \quad \text{at}\, \sigma_3 = q_c \tag{29.2}$$

Unfortunately, this critical state condition is not met by the other criteria of strength. It is heartening to note that this criterion is based on single parameter "*A*" which makes a physical sense. Sheorey (1997) has compiled the triaxial and polyaxial test data for different rocks which are available from the world literature. The regression analysis was performed on 132 sets of triaxial test data in the range of $0 \le \sigma_3 \le q_c$.

The values of the parameter *A*, for all the data sets were obtained. These values were used to back-calculate the σ_1 values for the each set for the given confining pressure. The comparison of the experimental and the computed values of σ_1 is presented in Fig. 29.2. It is observed that the calculated values of σ_1 are quite close to the experimental values. An excellent coefficient of correlation, 0.98, is obtained for the best fitting line between the calculated and the experimental values.

For comparing the predication of the parabolic criterion with those of the others, Hoek and Brown (1980) criterion was used to calculate the σ_1 values. The coefficient of correlation (0.98) for Hoek–Brown predictions is observed to be slightly lower and poor for weak rocks, when compared with that obtained for the criterion proposed in this chapter.

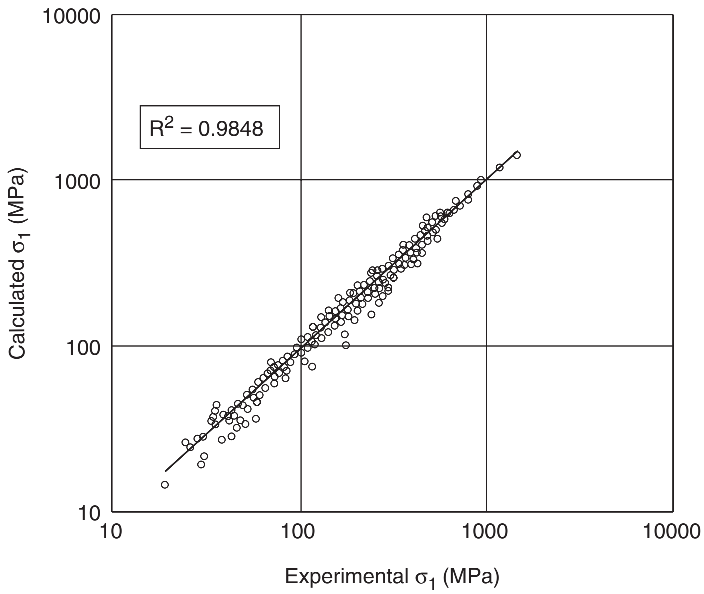

Fig. 29.2 Comparison of experimental σ_1 values with those calculated through the proposed criterion (equation (29.1)) (Singh & Singh, 2005).

In addition to the higher value of coefficient of correlation, the real advantage of proposed criterion, lies in the fact that only one parameter, A is used to predict the confined strength of the rock and A makes a physical sense.

A rough estimate of the parameter A may be made without conducting even a single triaxial test. The variation of the parameter, A, with the uniaxial compressive strength (UCS), q_c is presented in Fig. 29.4. A definite trend of A with UCS (q_c) is indicated by this figure and the best fitting value of the parameter A may be obtained as given below:

$$A \cong \frac{7.94}{q_c^{0.10}} \quad \text{for } q_c = 7 - 500\,\text{MPa} \tag{29.3}$$

Fig. 29.3 compares experimental σ_1 values with those predicted by using equation (29.3) without using the triaxial data. A high coefficient of correlation of 0.93 is obtained. Thus, the proposed criterion appears to be more faithful to the test data than Hoek and Brown (1980) criterion. This criterion is also better fit for weak rocks as the critical state is more important for rocks of lower UCS. The law of saturation appears to be the cause of non-linearity of the natural laws.

29.2 SUGGESTED MODEL FOR ROCK MASS

The behavior of jointed rock mass may be similar to that of the rock material at critical confining pressure, as joints then cease to dominate the behavior of the rock mass.

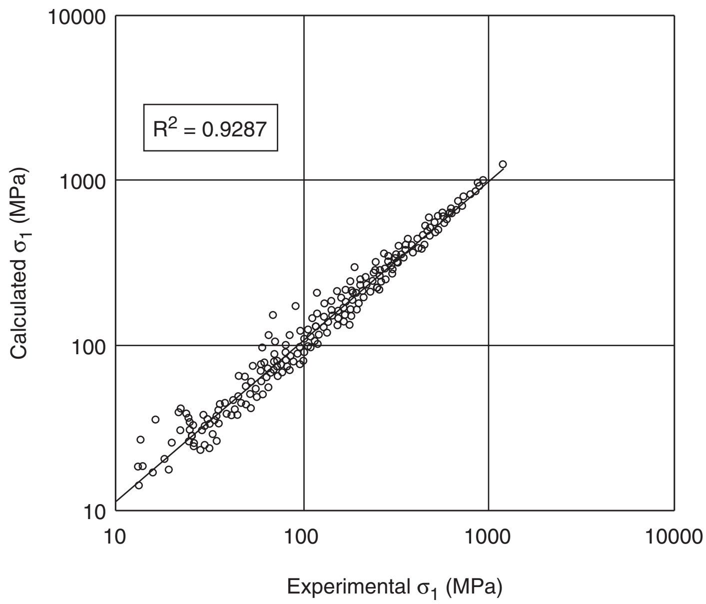

Fig. 29.3 Comparison of experimental σ_1 values with those predicted using present criterion without using triaxial test data (Singh & Singh, 2005).

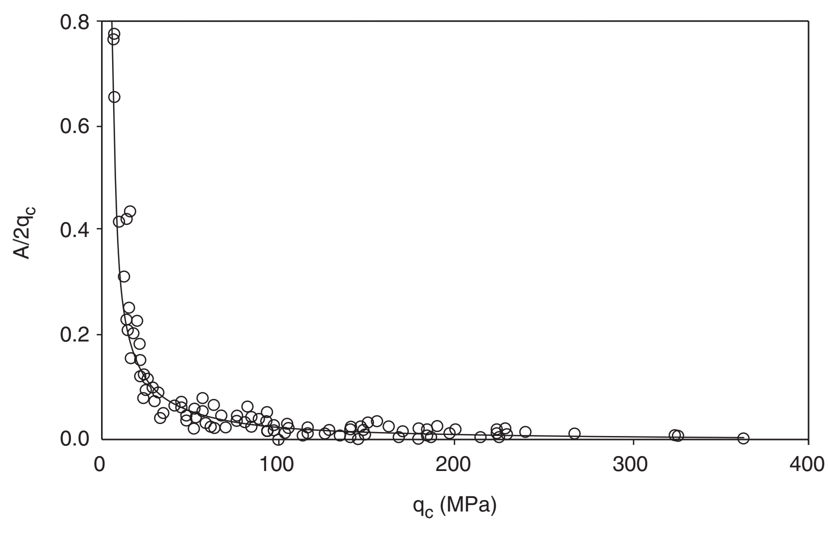

Fig. 29.4 Variation of parameter $A/2q_c$ with UCS of the intact rocks (q_c).

Therefore, one may assume that the deviator strength will reach the critical state at $\sigma_3 = q_c$. As such, this critical confining pressure may be independent of the size of specimen. Perhaps the deviator strength may also achieve a critical state when intermediate principal stress $\sigma_2 \cong q_c$ (equation (8.2)). Thus an approximate simple parabolic and polyaxial criterion is suggested for the underground openings as follows,

$$\sigma_1 - \sigma_3 = q_{\text{cmass}} + \frac{A(\sigma_2 + \sigma_3)}{2} - \frac{A}{4q_c}(\sigma_2^2 + \sigma_3^2) \tag{29.4}$$

$$\text{for } 0 < \sigma_3 \le q_c \text{ and } 0 < \sigma_2 \le q_c$$

where

q_{cmass} = uniaxial compressive strength of the rock mass,
= $7\gamma Q^{1/3}$ MPa, (29.5)
γ = unit weight of rock mass in gm/cc or t/m^3,
Q = post-construction Barton's rock mass quality just before supporting a tunnel,
A = $\frac{2\sin\phi_p}{1-\sin\phi_p}$, (29.6)
ϕ_p = the peak angle of internal friction of a rock mass and
q_c = average uniaxial compressive strength (UCS), upper bound of UCS for anisotropic rock material of jointed rock masses under actual environment.

It may be verified by differentiating equation (29.4) that

$$\frac{\partial(\sigma_1 - \sigma_3)}{\partial\sigma_3} = 0 \text{ at } \sigma_2 = \sigma_3 = q_c$$

$$\frac{\partial(\sigma_1 - \sigma_3)}{\partial\sigma_2} = 0 \text{ at } \sigma_2 = q_c \text{ for any value of } \sigma_3$$

The triaxial test data ($\sigma_2 = \sigma_3$) on models of jointed rocks was collected from Brown (1970), Brown and Trollope (1970), Ladanyi and Archambault (1972), Einstein and Hirshfeld (1973), Hoek (1980), Yazi (1984), Arora (1987) and Roy (1993). The parameter A was computed by the least square method as was done for the rock materials. An approximate correlation between A and q_c was found as given below.

$$A = 2.46\, q_c^{0.23} \tag{29.7}$$

The polyaxial tests on cubes of jointed rocks at IIT Delhi suggest that the mode of failure at high σ_2 is brittle and not ductile as expected. This is seen in tunnels in medium to hard rocks.

The angle of internal friction (ϕ_p) in equation (29.6) may be chosen from the correlation of Mehrotra (1993) (cited by Singh & Goel, 2002), according to RMR both for the nearly dry and saturated rock masses (Fig. 29.5). It is based on the extensive and carefully conducted block shear tests at various project sites in the Himalaya. It may be seen that ϕ_p is significantly less for the saturated rock mass than that for the nearly dry rock mass for the same final RMR. So the parameter A will be governed by the degree of

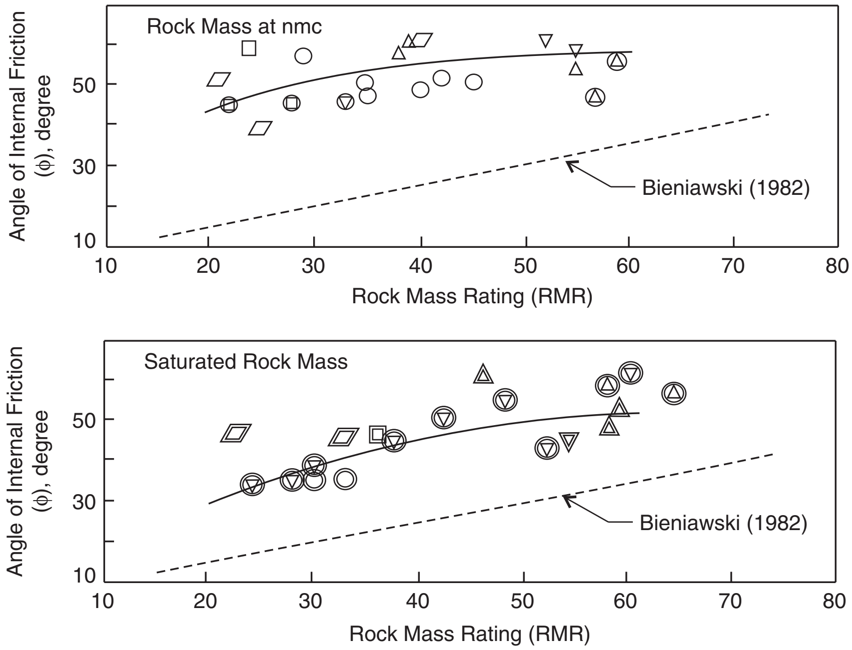

Fig. 29.5 Relationship between rock mass rating (RMR) and angle of internal friction (ϕ_p) (Mehrotra, 1992) [nmc: natural moisture content].

saturation in equation (29.7). Hoek and Brown (1997) have developed a chart between friction angle ϕ_p and geological strength index (GSI $=$ RMR $-$ 5 for RMR $\geq$ 23) for the various values of rock material parameter m_r. It is seen that ϕ_p increases significantly with increasing value of m_r for any GSI. In the case of rock mass with clay-coated joints, equation (29.13) may be used to estimate ϕ_p approximately. Equation (29.13) takes into account approximately the seepage erosion and piping conditions in the weak rock masses (Barton, 2002). Seepage erosion (flow of soil particles from joints due to the seepage, especially during rainy seasons) rapidly deteriorates the rock mass quality (Q) with time. Seepage may be encountered at great depths even in granite unexpectedly, due to the presence of a fault. *“Uncertainty is the law of nature.”*

It should be mentioned that Murrell (1963) was the first researcher who predicted that major principal stress (σ_1) at failure increases with σ_2 significantly, but it reduces when σ_2 is beyond $\sigma_1/2$ and $\sigma_3 = 0$. The three sets of polyaxial test data cited by Yu et al. (2002) shows a negligible or small trend of peaking in σ_1 when $\sigma_2 \gg q_c$. The attempt was made to fit in the proposed polyaxial strength criterion (equation (29.4)) in the above test data for rock materials (Dunham dolomite, trachite and coarse gained dense marble). The recent polyaxial test data on tuff (Wang and Kemeny, 1995) was also analyzed. The equation (29.4) was found to be fortunately rather a good fit into all the polyaxial test data at $\sigma_2 < q_c$ and $\sigma_3 < q_c$.

Kumar (2002) has collected data of 29 km NJPC tunnel in gneiss in Himalaya as mentioned in Table 29.1. It may be noted that the rock mass strength (q_{cmass}) is too less than the expected tangential stress (σ_θ) along the tunnel periphery. The q_{cmass} from linear criterion is some what less than σ_θ. It is interesting to know that the parabolic polyaxial criterion predicts the rock mass strength (q''_{cmass} in equation (29.8)) in the range of 0.64 to 1.4 σ_θ generally. In one situation, the rock mass was found to be in the critical state locally. It matches with the failure conditions in the tunnel beyond overburden of 1000 m where (mild rock burst or) spalling of rock slabs was observed. So the proposed simple polyaxial strength criterion (equation (29.4)) fits in the observations in the tunnels in the complex and fragile geological conditions in a better way than other criteria. Better fit also suggests that the peak angle of internal friction of rock mass may be nearly the same as that for its rock material in the case of unweathered rock mass.

The equation (29.4) suggests the following criterion of failure of rock mass around tunnels and openings ($\sigma_3 = 0$ on excavated face and $\sigma_2 = P_o$ along tunnel axis),

$$\sigma_\theta > q''_{cmass} = q_{cmass} + \frac{A \cdot P_o}{2} - \frac{A \cdot P_o^2}{4 \cdot q_c} \leq q_{cmass} + \frac{A \cdot q_c}{4} \qquad (29.8)$$

where q''_{cmass} is the biaxial compressive strength of rock mass, corrected for greater depths.

29.3 RESIDUAL STRENGTH

Mohr's theory will be applicable to residual failure as a rock mass would be reduced to non-dilatant soil like condition. Thus, residual strength $(\sigma_1 - \sigma_3)_r$ of rock mass is likely to be independent of the intermediate principal stress. So, the following criterion is suggested.

$$(\sigma_1 - \sigma_3)_r = q_{cr} + A_r \cdot \sigma_3 - \frac{A_r \cdot \sigma_3^2}{2 \cdot q_c} \quad \text{for } 0 < \sigma_3 \leq q_c \qquad (29.9)$$

where

q_{cr} = UCS of rock mass in the residual failure,

$= \frac{2c_r \cos\phi_r}{1-\sin\phi_r}$ (29.10)

$A_r = \frac{2\sin\phi_r}{1-\sin\phi_r}$, (29.11)

c_r = residual cohesion of rock mass,

= 0.1 MPa

= 0 if the deviator strain exceeds 10 percent,

$\phi_r = \phi_p - 10° \geq 14°$ (29.12)

$\tan\phi_p = (J_r \cdot J_w/J_a) + 0.1$ (based on Barton, 2002). (29.13)

Singh and Goel (2002) have analyzed 10 tunnels in the squeezing ground condition considering linear criterion ($J_r/J_a < 0.5$, $J_w = 1.0$ and $\gamma H \ll q_c$). There is a rather good

Table 29.1 Comparison of tangential stress (σ_θ) and rock mass strength (q''_{cmass}) considering intermediate principal stress.

S. No.	Overburden H(m)	Q after Tunnelling	UCS, q_c (MPa)	ϕ_p (deg)	$A = \frac{2\sin\phi_p}{1-\sin\phi_p}$	$q_{\text{cmass}} = 7\gamma Q^{1/3}$ (MPa)	$P_o = \gamma H$	$q'_{\text{cmass}} = q_{\text{cmass}} + \frac{AP_o}{2}$	$q''_{\text{cmass}} = q_{\text{cmass}} + \frac{AP_o}{2} - \frac{AP_o^2}{4q_c} \leq q_{\text{cmass}} + \frac{Aq_c}{4}$ (MPa)	$\sigma_\theta = 2\gamma H$ (MPa)
1	1430	4.7	50	45	4.8	31.6	38.6	124.2	88.4	77.2
2	1420	4.0	32	37	3.0	30.0	38.3	87.5	53.1	76.6
3	1420	4.5	50	45	4.8	31.1	38.3	123.0	87.8	76.6
4	1320	1.8	32	37	3.0	23.1	35.6*	76.5	47.1	71.2
5	1300	3.5	50	45	4.8	28.7	35.1	112.9	83.4	70.2
6	1300	2.0	60	45	4.8	23.8	35.1	108.0	83.4	70.2
7	1300	1.8	55	45	4.8	23.1	35.1	107.3	80.5	70.2
8	1300	3.3	50	45	4.8	28.0	35.1	112.2	82.7	70.2
9	1230	2.2	50	45	4.8	24.7	33.2	104.4	77.9	66.4
10	1180	4.7	42	55	9.1	31.6	31.9	176.1	121.0	63.8
11	1180	2.0	34	30	2.0	23.8	31.9	55.7	40.7	63.8
12	1180	3.4	42	45	4.8	28.3	31.9	104.9	75.8	63.8
13	1100	7.5	42	45	4.8	37.0	29.7	108.3	83.1	59.4
14	1090	7.0	50	45	4.8	36.2	29.4	106.8	86.0	58.8
15	1060	3.8	50	45	4.8	29.4	28.6	98.0	78.4	57.2

Note: In NJPC tunnel, no rock burst was observed except slabbing and noises due to cracking at overburden (H) above 1000 m. The angle of internal friction ϕ_p for rock mass was assumed same as that for the rock material (gneiss) approximately.

*Rock mass is in the critical state locally as in situ stress along tunnel axis (P_o) is more than UCS.

cross-check between the proposed theory and the observed support pressures in the squeezing ground conditions except in a few cases. Thus residual cohesion was back-analyzed to be 0.1 MPa approximately and zero where deviatoric strain exceeded 10 percent. Similarly, the residual angle of friction was inferred to be about 10° less than the peak angle of internal friction but more than 14°.

This model explains the likely mode of failure of rock mass in the deep tunnels. For example, severe rock burst condition may be encountered where A or ϕ_p is high (where $J_r/J_a > 0.5$). It is because the peak deviator strength or differential stress $(\sigma_1 - \sigma_3)$ is very high compared to the residual deviator strength $(\sigma_1 - \sigma_3)_r$. The locked-up strain energy may be dissipated in the form of seismic waves. On the other hand, the squeezing condition or plastic failure may develop where A or ϕ_p is very low, as there may not be any significant difference in the peak and residual deviatoric strengths.

As such, the following criteria for the heavy rock burst in deep tunnels in the hard rocks is suggested.

$$\frac{\sigma_\theta}{q''_{cmass}} > 2 \tag{29.14}$$

$$\frac{J_r \cdot J_w}{J_a} > 0.5 \tag{29.15}$$

and

$$H > H_{cr} = \frac{2.5 q_c}{\gamma} \tag{29.16}$$

Thus severe rock burst conditions may develop in hard rocks which has entered into the critical state ($P_o > q_c$) and where the overburden (H) exceeds the limit of equation (29.16). It is assumed that the ratio of in situ minimum principal stress and overburden pressure (K) is about 0.4 ± 0.10 at great depths in equation (29.16).

The thermic zones (of high temperatures) are also likely to be encountered at greater depths. So the tunnel face may have to be air-conditioned like in very deep mines (Kolar Gold Field, India). The efficiency of the workers is very low and they cannot work for more than a few hours under high temperatures. In addition, the thermic zone may also be in critical state and the tunnelling hazard may be doubly serious. So it will be better to realign the tunnel to by-pass the rock mass in at least the critical state or in thermic zone or both, to be on the safe side and avoid severe tunnelling hazards.

In case of rock mass in the critical state, pre-tensioned rock bolts or resin grouted bolts may be effective, as radial stresses will be released. It is also suggested that one may try to use thick SFRS (steel fiber reinforced shotcrete) lining which has good bond with the rock mass and its compressive strength is higher than that of the rock material. Resin anchors may make rock mass ductile.

The understanding of critical state rock mechanics is essential for (i) deep tunnels for the underground nuclear waste disposal and (ii) also in the petroleum engineering.

Very deep drill-holes may be unstable in the weak rock layers (shale) in the critical state ($H > H_{cr}$).

29.4 EFFECT OF CONFINING PRESSURE ON FRICTION ANGLE

Sometimes a fault or thrust passes through deep tunnels. A non-linear analysis is preferred by the incremental method. The Mohr's envelope based on equation (29.1) gives the slope of angle of friction (ϕ) at any initial stress condition. The shear strength criterion for fault in terms of increase in shear strength ($\Delta\tau$) for expected rise in the effective normal stress ($\Delta\sigma$) is simply as follows:

$$\Delta\tau = (\Delta\sigma - \Delta u)\tan\phi \tag{29.17}$$

where Δu is increase in seepage pressure.

If $\Delta\tau$ exceeds the incremental shear strength of weaker rock adjoining to a fault under high confining stresses, earthquakes may occur in that area.

The friction angle (ϕ) along any point of deep-seated fault may be predicted accurately from the following empirical equation for $\tan\phi_o < 2$ and any value of UCS > 10 MPa which is derived from equation (29.1) (Singh et al., 2004).

$$\begin{aligned} \tan\phi &= \tan\phi_o\left[1 - (\sigma_3/q_c)^{\pi/2}\right] \quad \text{for } \tan\phi_o < 2 \quad \text{and } q_c > 10\,\text{MPa} \qquad (29.18) \\ &= 0 \quad \text{for } \sigma_3 > q_c \end{aligned}$$

Where ϕ_o is the friction angle in the nearly unconfined state ($\sigma_3 = 0$). It is interesting to know that ϕ will be negligible along a boundary between earth-plates beyond a great depth due to the critical state conditions. Shankar et al. (2002) have back-analyzed a friction angle of only 5° beyond a depth of 40 km below the ground surface along the plate boundary in the Tibet Himalayan plate. It is interesting to note that lesser the frictional resistance along colliding inter-plate boundaries, lesser will be the locked-up strain energy in the large earth plates and so lesser are the chances of great earthquakes in that area. Infact, highest earthquake of only M7 on Richter's scale had taken place in the Tibetan plateau. Thus there is a balancing mechanism in the nature to avoid too high intensity of earthquakes in a planet.

In weak rocks, the high confining stress may reduce porosity of rock material and so increase its UCS (q_c in equation (29.18)). For all practical purposes, the coefficient of friction ($\Delta\tau/\Delta\sigma$) may be assumed to be negligible beyond a confining stress of UCS at $\sigma_3 = q_c$.

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Appendix I

Tunnel mechanics

"Experiments should be reproducible – they should all fail in the same way."

Murphy Law

AI.1 ELASTIC STRESS DISTRIBUTION AROUND CIRCULAR TUNNELS

Study of rock mechanics has fascinated many researchers. One of the reasons for fascination is that distribution of stresses and displacement gives valuable insight on the stability of underground openings.

There is a difference in the elastic solutions for displacements around tunnels and circular holes in the steel plate. In the case of tunnels, the opening is made in the stressed state of the medium, whereas hole is first made in the stress-free state of steel plate and then loaded along its boundaries. It is interesting to note that solutions for stresses are the same in both the cases, but displacements are different.

The following assumptions are made in the analysis (Fig. AI.1):

(i) The rock mass is homogeneous, isotropic, dry, linearly elastic and infinite medium.
(ii) The tunnel is circular in shape having radius a.
(iii) The in situ stress field is homogeneous and non-hydrostatic. The vertical stress is P and the horizontal stress is λP. The vertical stress P is generally assumed equal to the overburden pressure (γH). The horizontal in situ stress is higher than vertical stress in most of the cases in civil engineering projects.
(iv) The modulus of elasticity (or deformation E_d) is the same in loading and unloading condition. In other words, there is no stress induced anisotropy.

Terzaghi and Richart (1952) have derived the solution with respect to the stressed state. The compressive stresses are positive. The radial displacement is positive in the direction of radius vector. The following solution is for plane stress case.

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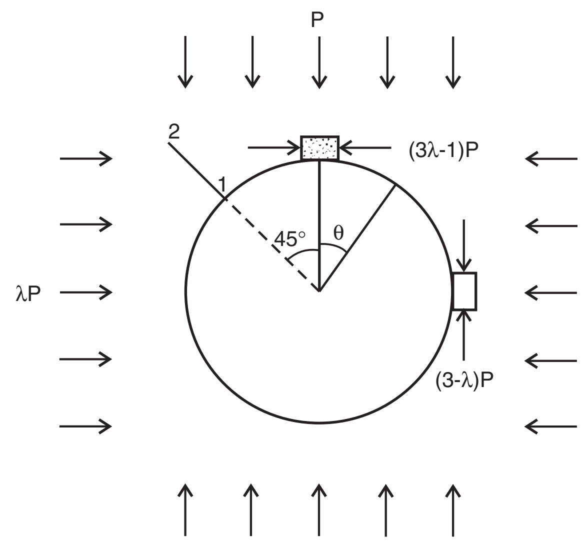

Fig. AI.1 Stress concentration around circular tunnel in non-hydrostatic state of in situ stress.

$$\sigma_r = \frac{P}{2}\left[(1+\lambda)\left(1-\frac{a^2}{r^2}\right)+(1-\lambda)\left(1+\frac{3a^4}{r^4}-\frac{4a^2}{r^2}\right)\cos 2\theta\right] \tag{AI.1}$$

$$\sigma_\theta = \frac{P}{2}\left[(1+\lambda)\left(1+\frac{a^2}{r^2}\right)-(1-\lambda)\left(1+\frac{3a^4}{r^4}\right)\cos 2\theta\right] \tag{AI.2}$$

$$\tau_{r\theta} = \frac{P}{2}(1-\lambda)\left(1+\frac{2a^2}{r^2}-\frac{3a^4}{r^4}\right)\sin 2\theta \tag{AI.3}$$

$$\begin{aligned} u_r = &-\frac{Pa^2}{2E_d r}\left[(1+\nu)\left(1-\frac{a^2}{r^2}\cos 2\theta\right)+4(1-\nu^2)\cos 2\theta\right] \\ &-\frac{P\lambda a^2}{2E_d r}\left[(1+\nu)\left(1+\frac{a^2}{r^2}\cos 2\theta\right)-4(1-\nu^2)\cos 2\theta\right] \end{aligned} \tag{AI.4}$$

$$u_{12} = -\frac{P(1+\lambda)}{2E_d}(1+\nu)a\left[1-\frac{a}{r^2}\right] \tag{AI.5}$$

It is interesting to note that the stress distribution is independent of modulus of deformation (E_d), as expected. It should be noted that the radial displacement u_r is half of that in opening in stress-free state. Sometimes extensometers are installed to monitor relative displacements u_{12} between point 1 at $r = a$ and point 2 at $r = r_2$ (Fig. AI.1). Equation (AI.5) gives the expression for u_{12}.

Let us suppose that the tunnel is internally pressurized by radial stress p_i. The solution for induced additional stresses is simple as follows:

$$\sigma_r = p_i \frac{a^2}{r^2} \tag{AI.6}$$

$$\sigma_\theta = -p_i \frac{a^2}{r^2} \tag{AI.7}$$

$$u_r = \frac{(1+\nu)\, a\, p_i}{E_d} \tag{AI.8}$$

It may be seen that tangential stresses are tensile in nature.

Since the principle of superposition is valid in the elastic body, the final solution for internally pressurized tunnel is the sum of equations (AI.1) and (AI.6) for radial stresses; and equations (AI.2) and (AI.7) for tangential stresses.

In the special case of hydrostatic in situ stresses, the final solution is simple as shown in Fig. AI.2. It is fascinating to note that the sum of radial and tangential stresses is equal to twice the vertical in situ stress everywhere in the rock mass.

$$\sigma_r = P\left(1 - \frac{a^2}{r^2}\right) + p_i \frac{a^2}{r^2} \tag{AI.9}$$

$$\sigma_\theta = P\left(1 + \frac{a^2}{r^2}\right) - p_i \frac{a^2}{r^2} \tag{AI.10}$$

$$\sigma_r + \sigma_\theta = 2P \tag{AI.11}$$

$$u_a = \frac{(1+\nu)(P - p_i)\, a}{E_d} \tag{AI.12}$$

The above equations have been derived for plane stress state (stresses along the tunnel axis is zero). The solution would be modified slightly for displacements for plane strain case by substituting Poisson's ratio in the above equations (Jaeger & Cook, 1969),

$$\nu \rightarrow \frac{\nu}{(1-\nu)} \tag{AI.13a}$$

$$E_d \rightarrow \frac{E_d}{(1-\nu^2)} \tag{AI.13b}$$

Fortunately, equations (AI.5), (AI.8) and (AI.12) remain unaltered in the plain strain case. These laws of transformations may be easily deduced by comparing the strains in a cube subjected to the principal stresses for plane stress ($\sigma_2 = 0$) and plain strain case [$\varepsilon_2 = 0$ or $\sigma_2 = \nu(\sigma_1 + \sigma_3)$].

Recently, Carranza and Fairhurst (2000) presented the following empirical equation for estimation of radial displacement u_r at a distance x ahead of tunnel face ($x \leq 0$) and

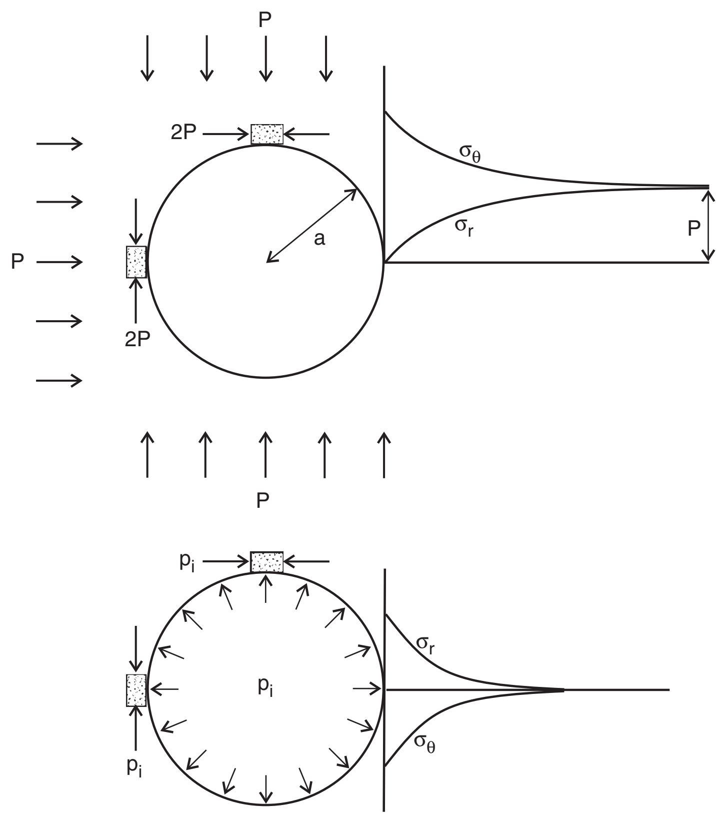

Fig. AI.2 The stress distribution around circular tunnel in hydrostatic stress condition and plain strain condition.

behind the face ($x > 0$) towards portal in the case of circular tunnel within a hydrostatic in situ stress field.

$$\frac{u_r}{u_{r\infty}} = \left[1 + \exp\left\{\frac{-x/a}{1.10}\right\}\right]^{-1.70} \tag{AI.14}$$

$$= 0.31 \text{ at } x = 0 \text{ (tunnel face)}$$

$$= 1.0 \text{ at } x \to \infty$$

$$= 0 \text{ at } x \to -\infty$$

The radial displacement $u_{r\infty} = u_a$ (for $p_i = 0$) in equation (AI.12). Thus, significant displacement may be lost during monitoring even near the face.

It can be noted that above equations and elasto-brittle-plastic solutions in subsequent articles are also applicable for the nearly vertical shafts in rock masses.

AI.2 PROPOSED ELASTO-PLASTIC THEORY OF STRESS DISTRIBUTION IN BROKEN ZONE IN SQUEEZING GROUND

Fig. AI.3 shows a concentric circular broken zone or plastic zone within squeezing ground. The following assumptions are made to get approximate solutions for the stress distribution.

(i) Rock mass is isotropic and homogeneous and dry. Tunnel is circular having radius *a*.

(ii) Rock mass follows the polyaxial strength criterion within the elastic zone (Section 8.4)

$$\sigma_1 - \sigma_3 = q_{\text{cmass}} + \left(\frac{\sigma_2 + \sigma_3}{2}\right) \cdot A \tag{AI.15}$$

(iii) Rock mass strength is reduced to residual state within the broken zone and obeys Mohr's theory as follows:

$$\sigma_1 - \sigma_3 = q_{\text{cr}} + \sigma_3 \cdot \alpha \tag{AI.16}$$

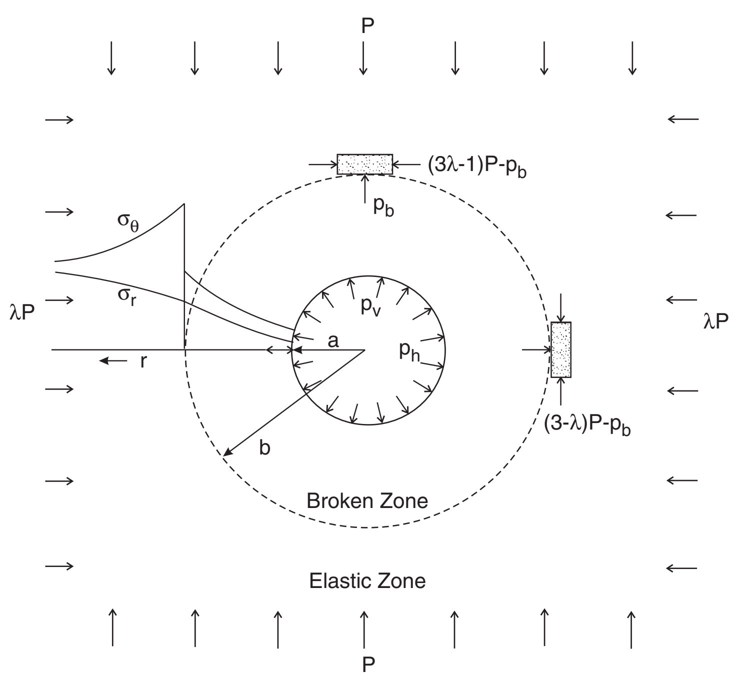

Fig. AI.3 Stress distribution in squeezing ground.

$$\alpha = \frac{2 \sin \phi_r}{1 - \sin \phi_r}$$

$$q_{cr} = \frac{2\, c_r \cos \phi_r}{1 - \sin \phi_r}$$

$$\tan \phi_p = (J_r/J_a) + 0.1$$

$$c_r = 0.1\,\text{MPa}$$

$$\phi_r = \phi_p - 10^\circ \geq 14^\circ$$

(iv) Broken zone is circular and concentric with the tunnel, and the gravity is assumed to act radially for simplifying analysis. The tunnel is supported uniformly and there are no rock bolts.
(v) In situ principal stress along tunnel axis is P_o. The vertical in situ stress is P and horizontal in situ stress is λP.
(vi) Tunnel supports are provided and they exert support pressure p_v and p_h in the vertical and horizontal directions, respectively.
(vii) There is no rock burst or brittle failure $[(J_r/J_a) < 0.5]$

The proposed analysis is more rigorous than those suggested by other researchers to the best knowledge of the authors. It is partially verified by case histories in Himalaya.

Elastic Zone

The tangential stress in horizontal direction is

$$\begin{aligned} \sigma_1 &= \sigma_\theta = (3 - \lambda)P - p_b \\ \sigma_3 &= p_b \end{aligned} \tag{AI.17}$$

The radial stress p_b at $r = b$ is obtained from equations (AI.15) and (AI.17) as follows:

$$(3 - \lambda)P - p_b - p_b = q_{cmass} + \left(\frac{P_o + p_b}{2}\right) \cdot A$$

$$\therefore\ p_b = \frac{(3 - \lambda) \cdot P - q_{cmass} - P_o \cdot (A/2)}{\big(2 + (A/2)\big)} \tag{AI.18}$$

Similarly the radial stress at $r = b$ along vertical axis is given by

$$p_b = \frac{(3\lambda - 1) \cdot P - q_{cmass} - P_o \cdot (A/2)}{\big(2 + (A/2)\big)} \tag{AI.19}$$

Broken Zone

The equilibrium equation within the broken zone is as follows:

$$\frac{d\sigma_r}{dr} - \frac{\sigma_\theta - \sigma_r}{r} = -\gamma \tag{AI.20}$$

where γ is the unit weight of rock mass. Substituting $\sigma_\theta - \sigma_r$ from equation (AI.16) in equation (AI.20), one gets ($\sigma_1 = \sigma_\theta$, $\sigma_3 = \sigma_r$),

$$\frac{d\sigma_r}{dr} - \frac{q_{cr} + \alpha\sigma_r}{r} = -\gamma \tag{AI.21}$$

The assumption of radial gravity is not serious, as the influence of gravity on support pressure is insignificant.

The solution of equation (AI.21) is

$$\alpha \cdot \sigma_r + q_{cr} = c \cdot r^\alpha - \frac{\alpha \cdot \gamma \cdot r}{1 - \alpha} \tag{AI.22}$$

$$\sigma_r = -c_r \cdot \cot\phi_r + \frac{c}{\alpha} \cdot r^\alpha - \frac{\gamma \cdot r}{1 - \alpha} \tag{AI.23}$$

At $r = b$, $\sigma_r = p_b$. So this solution becomes

$$p_b = -c_r \cdot \cot\phi_r + \frac{c}{\alpha} \cdot b^\alpha - \frac{\gamma \cdot b}{1 - \alpha}$$

$$\frac{c}{\alpha} = \left[p_b + c_r \cot\phi_r + \frac{\gamma b}{1 - \alpha}\right] \cdot \frac{1}{b^\alpha}$$

$$\text{or} \quad \sigma_r = \left[p_b + c_r \cot\phi_r + \frac{\gamma \cdot b}{1 - \alpha}\right] \cdot \left(\frac{r}{b}\right)^\alpha - c_r \cot\phi_r - \frac{\gamma \cdot r}{1 - \alpha} \tag{AI.24}$$

The wall support pressure is thus derived as follows ($r = a$, $\gamma = 0$)

$$p_h = [p_b + c_r \cot\phi_r] \left(\frac{a}{b}\right)^\alpha - c_r \cot\phi_r \tag{AI.25a}$$

$$p_h = \left[\frac{(3 - \lambda)P - q_{cmass} - AP_o/2}{2 + (A/2)} + c_r \cot\phi_r\right] \left(\frac{a}{b}\right)^\alpha - c_r \cot\phi_r \tag{AI.25b}$$

Thus squeezing will take place where hydrostatic in situ stress field exists and

$$A < 4 - \left(\frac{2\, q_{cmass}}{P}\right) \tag{AI.26}$$

$$\text{A} < 1 \text{ or } 2 \tag{AI.27}$$

Similarly, the roof support pressure is obtained from equations (AI.19, AI.24) ($r = a$):

$$p_v = \left[\frac{(3\lambda - 1)P - q_{cmass} - P_o \cdot A/2}{2 + (A/2)} + c_r \cot\phi_r\right]\left(\frac{a}{b}\right)^{\alpha} - c_r \cot\phi_r + \gamma M\gamma(b - a) \tag{AI.28}$$

$$= p_h + \gamma M\gamma\,(b - a) \quad \text{for } \lambda = 1 \tag{AI.29}$$

$$M_\gamma = \frac{a}{(b - a)} \cdot \frac{1 - \sin\phi_r}{1 - 3 \cdot \sin\phi_r}\left[\left(\frac{a}{b}\right)^{\alpha - 1} - 1\right] \tag{AI.30}$$

The equations (AI.25b) and (AI.28) are same as derived by Daemen (1975) for $\lambda = 1$ except the expression for radial stress p_b at the outer boundary of the broken zone. It is easy to derive the complicated expression for p_b using Hoek and Brown's criterion. The same may be substituted in equations (AI.24), (AI.25a) and (AI.29) to get support pressures. It may be noted that γ is negative at the bottom of a tunnel.

AI.3 SHORT-TERM SUPPORT PRESSURE ON CLOSELY SPACED TUNNELS IN SQUEEZING GROUND CONDITION

Sometimes several tunnels are required in a hydelproject to carry a given quantity of water because the size of a single tunnel becomes too large to be economical. Besides the considerations of economy, these tunnels may have to be closely spaced due to constraints of topography on a river side. It is, therefore necessary to analyze stresses on the rock pillar between two adjacent tunnels (or road tunnels). Jethwa (1981) derived the following closed form elasto-plastic solution.

The following assumptions have been made:

(i) The tunnels are circular in shape and their center to center spacing is $2b$ (Fig. AI.4).
(ii) The rock mass around the tunnels has failed due to overstressing and the thickness of the broken zone of rock mass is equal to half the pillar width (Fig. AI.4). The stress distribution has been assumed to be axi-symmetric around each tunnel.
(iii) The short-term support pressure on the support is p_{ho} in the horizontal direction.
(iv) The effect of gravity is neglected.
(v) There are no cross openings connecting the tunnels.

The relationship between the tangential stress $\left(\sigma_\theta^{bro}\right)$ and radial stress $\left(\sigma_r^{bro}\right)$ within the broken zone according to Coulomb's law is given by,

$$\sigma_\theta^{bro} = \sigma_r^{bro}\frac{1 + \sin\phi_r}{1 - \sin\phi_r} + 2 \cdot c_r \frac{\cos\phi_r}{1 - \sin\phi_r} \tag{AI.31}$$

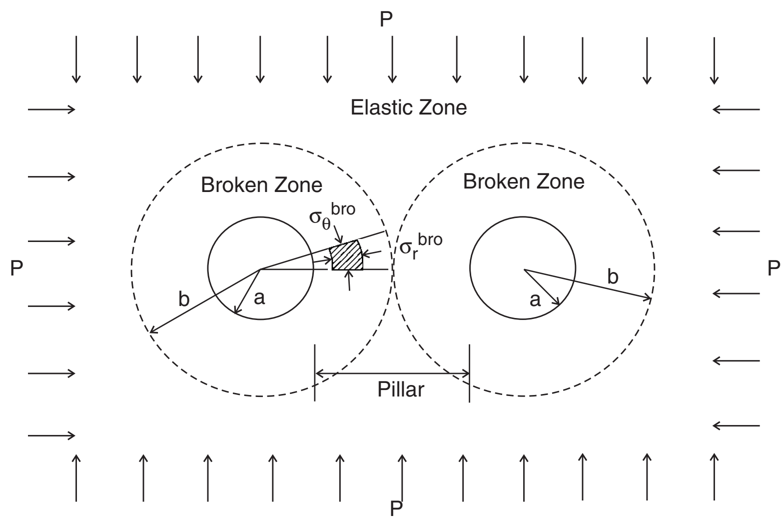

Fig. AI.4 Stresses on the pillar between two adjacent tunnels in squeezing ground condition.

The radial stress within the broken zone (Daemen, 1975) is given by equation (AI.23) $\left(\sigma_r^{bro} = p_{ho} \text{ at } r = a \text{ and } \gamma = 0\right)$,

$$\sigma_\theta^{bro} = \frac{1}{\sin\phi_r}\left[(p_{ho}\cdot\sin\phi_r + c_r\cdot\cos\phi_r)(r/a)^\alpha - c_r\cdot\cos\phi_r\right] \tag{AI.32}$$

The expression for tangential stress $\left(\sigma_\theta^{bro}\right)$ may be obtained from equation (AI.31) after substituting for σ_r^{bro} from equation (AI.32). So,

$$\sigma_\theta^{bro} = \frac{1+\sin\phi_r}{1-\sin\phi_r}\cdot\frac{1}{\sin\phi_r}\left[(p_{ho}\cdot\sin\phi_r + c_r\cos\phi_r)(r/a)^\alpha - c_r\cos\phi_r\right] + \frac{2c_r\cos\phi_r}{1-\sin\phi_r} \tag{AI.33}$$

The strength (S_p) of the pillar per unit length may be determined by integrating the tangential stress $\left(\sigma_\theta^{bro}\right)$ along the pillar width ($2b$) as follows:

$$S_p = 2\int_a^b \sigma_\theta^{bro}\,dr \tag{AI.34}$$

Substituting for σ_θ from equation (AI.33) one gets,

$$S_p = \frac{1+\sin\phi_r}{1-\sin\phi_r}\cdot\frac{2}{\sin\phi_r}\int_a^b\left[(p_{ho}\cdot\sin\phi_r + c_r\cos\phi_r)(r/a)^\alpha - c_r\cos\phi_r\right]dr$$
$$+\frac{2c_r\cos\phi_r}{1-\sin\phi_r}\int_a^b 2\,dr \tag{AI.35}$$

or

$$S_p = \frac{2 \cdot J}{\sin \phi_r} \left[(p_{ho} \cdot \sin \phi_r + c_r \cos \phi_r) \cdot \frac{a}{J} \cdot \left\{ (b/a)^J - 1 \right\} - (b-a) \cdot c_r \cos \phi_r \right]$$
$$+ \frac{4(b-a) \cdot c_r \cos \phi_r}{1 - \sin \phi_r} \tag{AI.36}$$

where $J = \dfrac{1 + \sin \phi_r}{1 - \sin \phi_r} = \alpha + 1$

On simplification, equation (AI.36) could be reduced to,

$$S_p = \frac{2 \cdot J}{\sin \phi_r} \left[(p_{ho} \cdot \sin \phi_r + c_r \cos \phi_r) \cdot \frac{a}{J} \cdot \left\{ (b/a)^J - 1 \right\} - 2 \cdot (b-a) \cdot c_r \cot \phi_r \right] \tag{AI.37}$$

The actual load L_p acting on the pillar is given by,

$$L_p = 2\,P\,(b-a) \tag{AI.38}$$

Finally, the factor of safety (F_p) of the pillar is obtained as the ratio between the pillar strength and the actual load on it, that is

$$F_p = \frac{S_p}{L_p} \tag{AI.39}$$

Substituting for S_p and L_p from equations (AI.37) and (AI.38) respectively, equation (AI.39) yields,

$$F_p = \frac{(J/\sin \phi_r) \cdot [(p_{ho} \cdot \sin \phi_r + c_r \cos \phi_r)] \cdot (a/J) \cdot \left\{ (b/a)^J - 1 \right\} - (b-a) \cdot c_r \cot \phi_r}{P(b-a)} \tag{AI.40}$$

$$\geq 3$$

For the failed rock mass, c_r may be assumed to be zero. Equation (AI.40) then simplifies to,

$$F_p = \frac{p_{ho} \cdot (b^J - a^J)}{P \cdot a^{\alpha}(b-a)} \tag{AI.41}$$

Solving for p_{ho}

$$p_{ho} = \frac{F_P \cdot P \cdot a^{\alpha}(b-a)}{(b^J - a^J)} \tag{AI.42}$$

Equation (AI.42) may be used to determine the short-term horizontal support pressure (P_{ho}) for the design of supports in the case of two adjacent tunnels using an appropriate factor of safety (about 3) for the pillar. Alternately, the factor of safety may be calculated if P_{ho} is known. Equation (AI.42) has actually been adopted in the design of the main roadways of the Noonidih-Jitpur Colliery for improving the factor of safety of shaft pillar which was practically crushed (Jethwa et al., 1979). The short-term vertical support pressure "p_{vo}" may be predicted approximately by equation (AI.28) ($\lambda = 1$). The implicit assumption in the above derivation is that stresses at the broken zone boundary remain practically unaffected by the adjacent tunnels.

AI.4 SEISMIC SUPPORT PRESSURES

Shotcrete lining is found to fail due to horizontal seismic support pressure within a clayey fault zone. Equation (AI.28) suggests that the horizontal body force $\alpha_h \cdot \gamma$ may act towards tunnel center and a vertical body force may act vertically downwards during earthquake. Thus the additional seismic support pressures may be of the following order,

$$p_{hseismic} = \alpha_h \cdot \gamma \cdot M_\gamma \cdot (b - a) \tag{AI.43}$$

$$p_{vseismic} = \alpha_v \cdot \gamma \cdot M_\gamma \cdot (b - a) \tag{AI.44}$$

where

$p_{hseismic}$ = horizontal seismic support pressure for squeezing ground,
$p_{vseismic}$ = vertical seismic support pressure for squeezing ground,
α_h = coefficient of horizontal peak ground acceleration at level of tunnel during earthquake and
α_v = coefficient of vertical peak ground acceleration at level of tunnel during earthquake.

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Appendix II

Software TM for empirical design of support system for caverns and tunnels

"Realistic solutions to rock mechanics problems must involve a combination of general principles of mechanics with a mature physical insight developed through intelligent observations and experiment both in laboratory and the field."

Charles Fairhurst (1968)
University of Minnesota, USA

AII.1 GENERAL

The aim of supporting underground opening is to reinforce the rock mass so that it can act as an inherently stable, ductile and robust structural system to support unstable zones in the rock mass. This objective may be achieved by constructing a reinforced rock arch, i.e., an array of (perfo/resin) rock anchors in both the roof and the side walls, according to the overall ground conditions. Weld mesh should also be used to provide local stability to rock blocks hanging in between the rock anchors, together with normal shotcrete.

Steel fiber reinforced shotcrete (SFRS) should be provided where squeezing ground conditions are likely to be encountered. However, in highly squeezing ground conditions ($H \gg 350\,Q^{1/3}$ m and $J_r/J_a \ll 0.5$; H is the overburden in meters and Q is Barton's rock mass quality), steel ribs with struts should be installed when the shotcrete lining begins to fail again and again, despite the addition of extra layers of shotcrete. Another advantage of steel ribs is that excavation by forepoling is easily done by pushing iron bars into the tunnel and welding their opposite ends to the ribs. The floor heaving problem in highly squeezing grounds may be easily solved by bolting the floor. Cutting the floor to maintain proper level is of no use, since heaving will redevelop.

However, it would be wiser to realign the tunnel to pass through safer zones ($H < 350\,Q^{1/3}$ m). Alternatively, a larger tunnel may be divided into smaller tunnels in the squeezing ground to reduce construction problems.

In the case of openings within a water-charged rock mass, steel ribs may be used to support the rock mass after it has been reinforced by Swellex rock bolts. Continuous

Tunnelling in Weak Rocks
B. Singh and R. K. Goel

shotcrete should not be used because it will act as a barrier to underground water. Generally when shotcrete is applied to rock masses with well-defined water bearing joints, it is important to provide drainage through the shotcrete layer in order to relieve high water pressure. Drain holes, fitted with plastic pipes are commonly used for this purpose. Water-charged rock masses may also be grouted to form a grouted arch with drainage holes.

Advantages of an integrated NATM–NMT approach to support rock masses are: a faster rate of tunnelling; flexibility with regard to construction and general strategy for a wide variety of ground conditions and weak zones; built-in stability and greater economy. Further, this type of support system can always be strengthened easily by adding more rock anchors and shotcrete layers as and when required.

In potentially unstable portals, horizontal rock anchors of equal length should be provided inside the cut slope of the portal so that it acts as a reinforced-rock-breast wall. The rock cover should be at least equal to the width of the tunnel.

The aim, in a nutshell, is to construct an inherently stable, ductile and robust structural system to support a wide variety of ground conditions and weak zones, keeping in mind basic tunnel mechanics and the inherent uncertainties in the exploration, testing and behavior of geological materials.

It has been difficult to convince Asian engineers to adopt modern methods of tunnel support. Failure of the rock bolt support system in caverns of the Sardar Sarovar (Narmada) Project (Gujrat, India) and the tunnel of the Chamera Dam Project (Himachal Pradesh, India) have also undermined the confidence of engineers. Their hesitation is also due, in part, to confusion about the function of rock bolts.

AII.2 SOFTWARE TM

The advantage of SFRS is that a thinner layer of shotcrete needs to be applied, in comparison to conventional shotcrete. Furthermore, fiber-reinforced shotcrete is especially necessary in poor rock conditions where support pressure is high.

The use of fiber-reinforced shotcrete together with resin anchors is also recommended for controlling rock burst conditions because of the high fracture toughness which especially long steel fibers provide (NGI, 1993). It appears that this type of system may also be successful in highly squeezing ground conditions, such as those encountered in the lower Himalaya. Therefore, a simple semi-empirical theory is proposed by Singh et al. (1995) to illustrate how rock bolts and shotcrete/SFRS resist the support pressures. The correlations for various types of support system have been deduced from extensive tables and charts of Norwegian Geological Institute (NGI). Thakur (1995) evaluated critically this semi-empirical method on the basis of over 100 case histories and found it satisfactory. Park et al. (1997) used this design method for four food storage caverns in Korea. Samadhiya (1998) has verified the semi-empirical theory for shotcrete support by three-dimensional stress analysis of cavern with a shear zone in the Sardar Sarovar Project. Bureau of Indian Standards (BIS) has adopted this design method for code on Tunnelling in Rocks. Kumar (2002) has found that the proposed semi-empirical theory is applicable upto support capacity of 27 t/m^2. The method is conservative beyond this capacity. Generally support pressures are much less than this limit in rock masses. Support pressures may be estimated according to Section 5.6.2.

As such the software TM is developed on the basis of above semi-empirical method for tunnels and large caverns. It also takes into account the adverse effect of shear zones on the stability of support system (Grimstad & Barton, 1993; Bhasin et al., 1995). The program TM has been used in three case histories. It gives realistic designs of support systems even in complex geological conditions. This program has also been used successfully for the design of rock bolt and shotcrete system at Ganwi mini Hydelproject, H. P., India, and many other very wide tunnels.

The software TM can be copied from CD in the book of Singh and Goel (2002). The file names are Software/Tunnel/Source/TM.FOR and Software/Tunnel/TM/*.*. Source program is TM.FOR in Fortran.

The length of anchors/rock bolts in the wall of cavern is designed to prevent the buckling of reinforced rock wall column. This length depends also upon the depth of damage (d) due to blasting (Fig. AII.1). The following correlation between d and weight of charge W (kg/m);

$$d = 1.94\, W^{1.23} \text{ meters} \tag{AII.1}$$

The output of TM is (i) the optimum angle of roof arch in cavern, (ii) thickness of shotcrete/SFRS and (iii) design of rock bolt system for both roof and the walls, and suggested special specifications.

The thickness of shotcrete (t_{sc}) may be estimated rather easily from equations (28.1) and (28.3). It needs to be realized that shotcrete lining of adequate thickness and quality is a long-term support system. This is true for rail tunnels also. It must be ensured that there is a good bond between shotcrete and rock surface. Tensile bending stresses are not found to occur even in the irregular shotcrete lining in the roof due to a good bond between shotcrete and rock mass in an arched roof opening. Rock bolts help in better bonding. Similarly, contact grouting is essential behind the concrete lining to develop a good bond between the lining and rock mass to arrest its bending. However, bending stresses can develop within the fault zones. The SFRS can bear bending stresses due to steel fibers.

In the over-stressed brittle hard rocks, resin anchors should be installed to make the reinforced rock arch a ductile arch. Thus, mode of failure is designed to be ductile from the brittle failure (rock burst). Hence, failure would be slow giving enough time for local strengthening (or retrofitting) of the existing support system.

AII.3 EXPERIENCE IN POOR ROCK CONDITIONS

Steel fiber reinforced shotcrete (SFRS) has proved very successful in the 6.5 km long tunnel for the Uri Hydelproject and desilting underground chambers of NJPC in Himalaya.

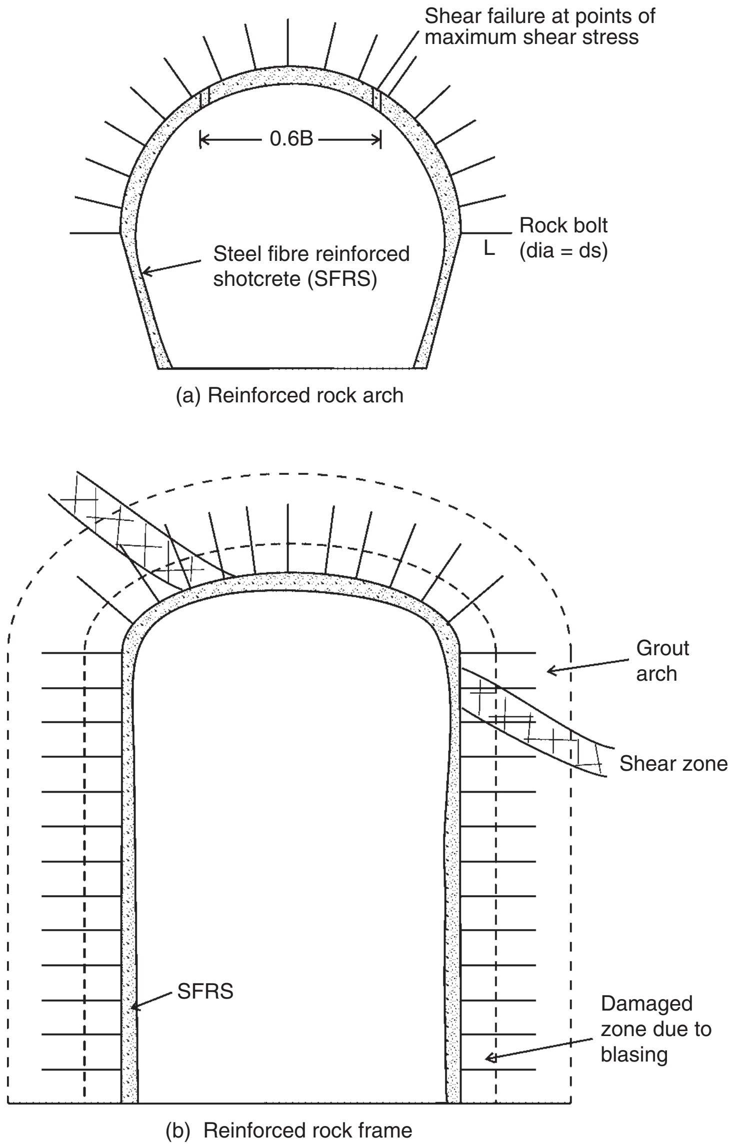

Fig. AII.1 Design of support system for underground openings.

The main advantage is that a smaller thickness of SFRS is needed. No weld mesh is required to reinforce the shotcrete. If the shotcrete is graded and sprayed properly, there is less rebound, thanks to the steel fibers. This method is now economical, safer and faster than the conventional shotcrete. Controlled blasting technique is adopted to excavate the tunnel where SFRS is to be used. Further, selection of right ingredients and tight quality control over application are key to the success of SFRS.

AII.4 CONCLUDING REMARKS

The following conclusions are offered on the basis of modelling and design experiences of support systems in poor rock masses.

1. In a poor rock mass, the support pressure resisted by the rock bolts is small in comparison to that taken by shotcrete, which is generally the main element of the long-term support system for resisting heavy support pressure in tunnels.
2. The proposed semi-empirical method and TM software are based on an integrated approach in the design of shotcrete, rock bolts/anchors, steel ribs and grouted arch, taking into account both seepage pressure and support pressure. The mobilization factors for each member have been derived from NGI tables and charts of support systems for both shotcrete and fiber-reinforced shotcrete.
3. The data analysis suggests that untensioned full-column grouted bolts are likely to be more effective than the pre-tensioned bolts in supporting poor rock masses.

AII.5 USERS MANUAL – TM

PLEASE TYPE THE FOLLOWING PARAMETERS IN THE CASE OF NO SHEAR ZONE (SEE FIG. AII.1)

1. TITLE (<80 CHARACTERS)
2. SPAN OF UNDERGROUND OPENING (M)
   SLIDING ANGLE OF FRICTION ALONG JOINTS = ATAN(Jr/Ja) (DEG)
   AVERAGE SPACING OF JOINTS APPROXIMATELY (M)
   SEEPAGE PRESSURE IN ROOF (T/SQ.M)
   MINIMUM UNIAXIAL COMPRESSIVE STRENGTH OF GROUTED ROCK MASS (T/SQ.M)
   SHEAR STRENGTH OF SHOTCRETE/FIBRE REINFORCED SHOTCRETE (T/SQ.M)
   (SAY 300 FOR SHOTCRETE & 550 FOR FIBRE REINFORCED SHOTCRETE)
   ESTIMATED ULTIMATE SUPPORT PRESSURE IN ROOF (T/SQ.M)
   {Increase support pressure by 25 % for seismic region}
3. LENGTH OF ROCK BOLT/ANCHOR IN ROOF (L) (M)
   FIXED ANCHOR LENGTH (100Ds/ACTUAL< L & < 1 m FOR PRE-TENSIONED BOLT)
   SPACING OF BOLT/ANCHOR (< L/2 AND 2.25 M) (M)
   {SQUARE ROOT OF AREA OF ROCK MASS SUPPORTED BY ONE ROCK BOLT}

BOLT/ANCHOR CAPACITY (REDUCE PROPORTIONATELY IF L < ACTUAL FAL) (T)
CAPACITY OF STEEL RIB (T)
SPACING OF STEEL RIBS (T)
THICKNESS OF GROUTED ROCK MASS (M)

{PLEASE OMIT THIS LINE No. 4 OF DATA IF SPAN OF OPENING < 10M}

4. HEIGHT OF WALL OF CAVERN (M)
MODULUS OF DEFORMATION OF ROCK MASS (T/SQ.M)
AVERAGE VERTICAL STRESS ABOVE HAUNCH ALONG BOLT (T/SQ.M)
SEEPAGE PRESSURE IN WALL (T/SQ.M)
ESTIMATED ULTIMATE WALL SUPPORT PRESSURE (T/SQ.M)
SHEAR STRENGTH OF GROUTED ROCK MASS
THICKNESS OF GROUTED ROCK MASS IN WALL (M)
DEPTH OF DAMAGE TO ROCK MASS DUE TO BLASTING (1-3 M)

5. SAFE BEARING PRESSURE OF ROCK MASS IN WEAK ZONE (T/SQ.M) (see Table 4.3)

6. TYPE NN EQUAL 1 FOR REDESIGN AT SAME LOCATION, 2 FOR REDESIGN OF WALL SUPPORT AT SAME LOCATION, 0 FOR STOP, -1 FOR NEW LOCATION AND -2 FOR AREA OF SHEAR ZONE

{RESTART INPUT DATA FROM THE BEGINNING FOR NN = -1 OR PARA 3 FOR NN = 1 AND 2
OR
PLEASE TYPE THE FOLLOWING PARAMETERS IN CASE OF LOCATION NEAR THE SHEAR ZONE WITH NN = -2}

TITLE

ROCK MASS QUALITY IN SHEAR ZONE
JOINT ROUGHNESS NUMBER IN SHEAR ZONE
JOINT ALTERATION NUMBER IN SHEAR ZONE
MODULUS OF DEFORMATION OF SHEAR ZONE (T/SQ.M)
WIDTH OF SHEAR ZONE (M)
STRIKE DIRECTION OF SHEAR ZONE (DEG.)
{WITH RESPECT TO AXIS OF TUNNEL/CAVERN}

ROCK MASS QUALITY IN SURROUNDING ROCK MASS
JOINT ROUGHNESS NUMBER IN SURROUNDING ROCK MASS
JOINT ALTERATION NUMBER IN SURROUNDING ROCK
MODULUS OF DEFORMATION OF ROCK MASS (T/SQ.M)
HEIGHT OF OVERBURDEN OF ROCK MASS (M)

{PLEASE CONTINUE FROM PARA 2}

(*a*) ***Input File – TM.DAT***

```
DESIGN OF SUPPORT SYSTEM FOR CAVERN OF SARDAR SAROVAR
PROJECT (SSP)
23. 57. 0.8 0. 0. 300. 8.8
6. 2. 1.75 32. 0. 0. 0.
52. 750000. 250. 0. 1. 0. 0. 3.
200.
0
```

(*b*) ***Output File: TM.OUT***

```
DESIGN OF SUPPORT SYSTEM FOR CAVERN OF SARDAR SAROVAR PROJECT
SPAN OF UNDERGROUND OPENING (M)                          = 23.000
SLIDING ANGLE OF FRICTION ALONG JOINTS (DEG)             = 57.000
AVERAGE SPACING OF JOINTS (M)                            = 0.800
AVERAGE VALUE OF SEEPAGE PRESSURE IN ROOF (T/SQ.M)       = 0.000
COMPRESSIVE STRENGTH OF GROUTED ROCK (T/SQ.M)            = 0.000
SHEAR STRENGTH OF SHOTCRETE (T/SQ.M)                     = 300.000
ESTIMATED ULTIMATE ROOF SUPPORT PRESSURE (T/SQ.M)        = 8.800
LENGTH OF ROCK BOLT/ANCHOR IN ROOF (M)                   = 6.000
FIXED ANCHOR LENGTH (M)                                  = 2.000
SPACING OF BOLT/ANCHOR IN ROOF (M)                       = 1.750
BOLT/ANCHOR CAPACITY (T)                                 = 32.000
CAPACITY OF STEEL RIB (T)                                = 0.000
SPACING OF STEEL RIBS (M)                                = 0.000
THICKNESS OF GROUTED ROCK MASS (M)                       = 0.000
--------------------------------------------------------------------
CAPACITY OF SHOTCRETE LINING IN ROOF (T/SQ.M)            = 0.000
CAPACITY OF REINFORCED ROCK ARCH (T/SQ.M)                = 9.167
CAPACITY OF STEEL RIBS (T/SQ.M)                          = 0.000
CAPACITY OF GROUTED ROCK ARCH (T/SQ.M)                   = 0.000
THICKNESS OF SHOTCRETE IN ROOF (M)                       = 0.026
RECOMMENDED ANGLE OF ARCH AT CENTRE (APPROXIMATE)        = 104.

HEIGHT OF WALL OF CAVERN (M)                             = 52.000
MODULUS OF DEFORMATION OF ROCK MASS (T/SQ.M)             = 750.000
AVERAGE VERTICAL STRESS ABOVE HAUNCH (T/SQ.M)            = 250.000
AVERAGE VALUE OF SEEPAGE PRESSURE IN WALL (T/SQ.M)       = 0.000
ESTIMATED WALL ULTIMATE SUPPORT PRESSURE (T/SQ.M)        = 1.000
SHEAR STRENGTH OF GROUTED ROCK (T/SQ.M)                  = 0.000
```

THICKNESS OF GROUTED ROCK MASS IN WALL (M)	= 0.000
DEPTH OF DAMAGE TO ROCK MASS DUE TO BLASTING (M)	= 3.000
SUGGESTED LENGTH OF BOLT/ANCHOR FOR WALL (M)	= 6.000
SPACING OF BOLTS/ANCHORS FOR WALL (M)	= 1.750
THICKNESS OF SHOTCRETE IN WALL (M)	= 0.052

---

* THE REINFORCED ROCK WALL COLUMN MAY NOT BUCKLE, SO THE SPACING OF ROCK BOLTS/ANCHORS MAY BE INCREASED AND THICKNESS OF SHOTCRETE/SFRS REDUCED IN WALLS ON THE BASIS OF PAST EXPERIENCE AND UNDERGROUND WEDGE ANALYSIS.

SAFE BEARING PRESSURE OF ROCK MASS (T/SQ.M)	= 200.000
SIDE OF BASE PLATE OF BOLT/ANCHOR (M)	= 0.40000

SPECIAL SPECIFICATIONS

1. FOR TREATMENT OF SHEAR ZONES, CROSSED ROCK ANCHORS/ BOLTS SHOULD BE PROVIDED ACROSS SHEAR ZONES. FIRST GOUGE SHOULD BE CLEANED TO DESIRED EXTENT. ANCHORS ARE PROVIDED AND CONNECTED TO WELD MESH. FINALLY DENTAL SHOTCRETE IS BACK-FILLED. IN WIDE SHEAR ZONE, REINFORCEMENT IN SHOTCRETE IS ALSO PLACED TO WITHSTAND HIGH SUPPORT PRESSURE. ANCHORS SHOULD BE INCLINED ACCORDING TO DIP OF SHEAR ZONE TO STOP SQUEEZING OF GOUGE AND THEREBY STABILISE DEFORMATIONS.
2. IN CASE OF POOR ROCK MASS, SPILING BOLTS (INCLINED TOWARDS TUNNEL FACE) SHOULD BE INSTALLED BEFORE BLASTING TO INCREASE THE STAND-UP TIME OF TUNNEL ROOF. SHOTCRETE IS THEN SPRAYED ON ROOF. NEXT SPILING BOLTS ARE INSTALLED. IN FINAL CYCLE, ROOF BOLTS ARE INSTALLED.
3. IN CASES OF ARGILLACEOUS ROCKS AND SWELLING ROCKS WHERE ITS BOND WITH SHOTCRETE IS POOR, THICKNESS OF SHOTCRETE MAY BE INCREASED BY ABOUT 30%.
4. IN CASE OF UNSTABLE PORTALS, HORIZONTAL ANCHORS OF EQUAL LENGTH SHOULD BE PROVIDED INSIDE THE CUT SLOPE, SO THAT IT ACTS AS A REINFORCED ROCK BREAST WALL.
5. ALL BOLTS SHOULD BE GROUTED AS PROTECTION FROM CORROSION. AT LEAST 5% BOLTS SHOULD BE PULLED OUT TO CHECK THEIR PULL OUT CAPACITY PARTICULARLY NEAR SHEAR/ FAULT ZONES.

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Appendix III

Capacity of blocked steel beam sections in the roof of tunnel

Tunnelling in Weak Rocks
B. Singh and R. K. Goel

Capacity in pounds per foot of tunnel width of various blocked continuous steel beam sections in the roof of tunnel (Proctor & White, 1946).

Beam				B = Width of tunnel to outside design concrete line (in feet)										
				14′–0″	16′–0″	18′–0″	20′–0″	22′–0″	24′–0″	26′–0″	28′–0″	30′–0″	32′–0″	34′–0″
				S = Maximum blocking point spacing (in inches)										
Nominal depth and flange width	Type	Weight per foot (lbs)	A_s (in^2)	40″	42″	44″	46″	48″	50″	52″	54″	56″	58″	60″
4″	I	7.7	2.26	2750	2470									
4″ × 4″	Stanchion	10.0	–	3750	3370									
4″ × 4″	H	13.0	3.83	4780	4310	3910	3570							
5″	I	10.0	2.94	4030	3620	3280	2990							
5″ × 5″	Stanchion	13.5	–	5760	5180	4690	4270							
5″ × 5″	Stanchion	16.0	4.68	6920	6220	5630	5130							
5″ × 5″	H	18.9	5.54	7860	7060	6390	5820							
6″	I	12.5	3.67	5590	5030	4540	4130	3790						
6″	I	17.25	5.07	7100	6380	5770	5260	4830	4450					
6″ × 4″	Light beam	12.0	3.55	5510	4940	4460	4060	3730						
6″ × 4″	Light beam	16.0	4.74	7540	6760	6110	5570	5100	4470					
6″ × 6″	Stanchion	15.5	–	7450	6670	6030	5490	5033	4650					
6″ × 6″	Stanchion	18.0	–	8690	7780	7040	6410	5870	5420					
6″ × 6″	Stanchion	27.5	–		12010	10860	9890	9060	8370					
6″ × 6″	H	20.0	5.87	9550	8560	7740	7050	6460	5960					
6″ × 6″	H	25.0	–	11800	10570	9570	8710	8040	7360	6830	6350	5930		
7″	I	15.3	4.50			5990	5450	4990	4610					
8″	I	18.4	5.41			7640	6950	6370	5860	5450				
8″	I	23.0	6.77			8640	8290	7600	7010	6500	6040	5640	5280	4960

8″ × 4″	Light beam	15.0	4.44	6320	5750	5270	4860					
8″ × 5.25″	Wide flange	17.0	5.00	7310	6680	6120	5650	5240				
8″ × 5.25″	Wide flange	21.0	6.16	9210	8390	7680	7090	6570	6100	5700		
8″ × 6.5″	Wide flange	24.0	7.08	10600	9650	8400	8150	7560	7020	6560	6150	5770
8″ × 6.5″	Wide flange	27.0	7.95	11920	10850	9930	9160	8500	7900	7380	6910	6480
8″ × 8″	Wide flange	31.0	9.13	13820	12590	11530	10640	9860	9160	8560	8020	7530
8″ × 8″	Wide flange	35.0	10.3	15640	14250	13110	12040	11160	10370	9690	9070	8530
8″ × 8″	Wide flange	40.0	11.7	17870	16270	14890	13750	12740	11840	11050	10360	9740
8″ × 8″	Wide flange	48.0	14.1		18150	17990	16600	15390	14290	13360	12510	11760
8″ × 8″	Wide flange	58.0	17.1			21700	20030	18560	17240	16110	15110	14210
8″ × 8″	Wide flange	67.0	19.7			25100	23190	21840	19970	18650	17470	16420
10″	I	25.4	7.46			9610	8860	8210	7630	7130	6680	6280
10″	I	35.0	10.3			12520	11540	10740	9940	9290	8700	8170
10″ × 5.75″	Wide flange	21.0	6.4			8220	7580	7020	6520	6090	5710	5370
10″ × 5.75″	Wide flange	26.0	7.6			10320	9500	8820	8180	7640	7160	6740
10″ × 8″	Wide flange	33.0	9.71			13130	12080	11210	10430	9740	9130	8580
10″ × 8″	Wide flange	41.0	12.4			16460	15170	14070	13060	12210	11440	10760
10″ × 10″	Wide flange	49.0	14.4				18290	16970	15770	14730	13820	12990
10″ × 10″	Wide flange	54.0	15.8				20200	18750	17410	16270	15260	14340
10″ × 10″	Wide flange	66.0	19.4				24950	22900	21280	19870	18630	17520
10″ × 10″	Wide flange	77.0	22.6					26760	24840	23230	21760	20480
10″ × 10″	Wide flange	89.0	26.0					30930	28710	26850	25160	23650

Conversion Factors: 1 lb/in^2 = 0.00703 MPa; 1 lb = 0.4536 kg; 1 ft. = 0.3048 m.

Notes:

(i) The above table will be helpful in making a preliminary selection of steel rib size. It shows the allowable vertical load, in pounds per linear foot of rib, projected on the horizontal, when applied to the rib at blocking points spaced as indicated. Thus, the product of vertical load shown in the table above and spacing of rib in ft. should be approximately equal to ultimate support pressure in the roof for selection of the steel rib section.

(ii) Data given is for tunnels with a semi-circular roof. Ribs for tunnels of other shapes should be investigated separately.

(iii) In the table, values are based on a maximum fiber stress of 24,000 p.s.i.

(iv) Deductions have been made for tie rod holes in the web. If additional holes in the web or flanges are desired, the allowable loads should be reduced accordingly.

(v) It is assumed that all wedges and blocks will be kept tight. No side loads are considered to be acting on the vertical legs.

(vi) Blocking point spacing cannot be exceeded without reducing the rib capacity. Capacity will decline in proportion to the square of the blocking point spacing as it is increased; hence the capacity falls off sharply if the spacing shown is exceeded. Conversely, closer spacing resulting from additional blocking points will increase the capacity rapidly.

(vii) If the blocking point spacing is reduced to zero by concrete packing, there will be no bending stresses in the curved portion of the rib and the capacity of arch rib (p_{rib}) in lbs/ft^2 can be approximately estimated by $p_{\mathrm{rib}} = (2A_s \cdot \sigma_y / F_s \cdot B \cdot S_{\mathrm{rib}}) = p_{\mathrm{roof}}$, where A_s is the cross-sectional area of the steel rib in square inch; σ_y is the yield strength of steel in p.s.i $= 24{,}000$; B is the width of the tunnel in feet; S_{rib} is the spacing of steel ribs in feet and F_s is factor of safety $= 1.5$.

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