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Underground Structures

Design and Instrumentation

Edited by

R.S. Sinha

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DEDICATION:

THIS BOOK IS DEDICATED TO THE
MEMBERS OF OUR FAMILIES, TO
OUR FRIENDS AND TO OUR READERS.

PREFACE

Underground Structures - Design and Instrumentation attempts to provide the most updated information on the design aspects of an underground structure. The structure may be a tunnel, a shaft, or a cavern, either alone or a combination thereof. In the preparation of the book, the philosophy was to include only items of utmost importance which will improve the quality of the final designed product, and will render a very cost effective design. A conscious decision was made not to include all the aspects of design or construction; items of minor importance were therefore left out. Emphasis was given to include, however, lessons learned from experiences which were of significant nature; mundane matters were excluded.

A separate book titled "Underground Structures - Construction and Investigation" covers important aspects of construction and subsurface investigation relevant to the design and construction of underground structures.

The book is an extension of international level short courses conducted in the years 1983, 1985, 1987, and 1989 in Colorado, U.S.A., where I served as a faculty director, course director and speaker. These short courses had participants, at one time or other, coming from Asia, Europe, South and North America. These courses won high ratings from the participants of the courses.

At the conclusion of the 1987 short course, it was clearly evident that there is a need in the technical community for a book of this kind that encompasses updated information on the design of tunnels, shafts, and caverns.

The goal of writing such a book seemed attainable when several speakers of the short courses concurred to contribute their efforts in coauthoring the book. I must admit that a single person's effort, like that of mine, would not have been that exhaustive and inclusive of all the important items that need to be considered and included in producing a most cost effective design for a buildable structure.

This book should serve the needs of Civil Engineers, Geotechnical Engineers, Geologists, Planners and Executives who are associated with the design and construction of underground structures.

The authors have obviously devoted considerable effort to chapter preparation and have shown extraordinary patience, cooperation, and courtesy to my editorial attempts and they have my sincere gratitude and appreciation.

VIII

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Enormous assistance in preparation of this book was provided by Lawrence Pedde, Mike Haverland, Joyce Steele, Sally Walker, Connie Megalong, my wife Pramila S. Sinha and son Neil S. Sinha. I thank them all very sincerely.

Finally, I express my appreciation to Jacques Kiebert and Robert L. Goodman of Elsevier Science Publishers for their very pleasant collaboration, and consideration in several matters and for a job well done.

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SELECTED SI CONVERSION FACTORS

| English unit | SI unit | Conversion factor F ($F \times$ English unit = SI unit) |
|--|--|--|
| inch (in) | metre (m) | 0.02540 |
| foot (ft) | metre (m) | 0.3048 |
| square inch (in ²) | square metre (m ²) | 6.452×10^{-4} |
| square foot (ft ²) | square metre (m ²) | 0.09290 |
| cubic inch (in ³) | cubic metre (m ³) | 1.639×10^{-5} |
| cubic foot (ft ³) | cubic metre (m ³) | 0.02832 |
| pound mass (lb) | kilogramme (kg) | 0.4536 |
| minute (min) | second (s) | 60 |
| degree (plane angle — °, deg) | radian (rad) | 1.745×10^{-2} |
| pound/cubic inch (lb/in ³) | kilogrammes/cubic metre (kg/m ³) | 2.768×10^4 |
| pound/cubic foot (lb/ft ³) | kilogrammes/cubic metre (kg/m ³) | 16.02 |
| pound force (lbf) | newton (N) | 4.448 |
| pound force/square inch (lbf/in ²) | newton/square metre (N/m ²) | 6895 |
| pound force/square inch (lbf/in ²) | bar (bar) | 0.06895 |
| foot pound (f) (ft-lbf) | joule (J) | 1.356 |

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Chapter 1

INTRODUCTION

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I-1 GENERAL

Underground structures such as tunnels, shafts, caverns and their appurtenances are structures completely encased and housed into the existing host ground medium. To a nonskeptical observer, such structures appear to possess the usual three dimensions of length, width, and height. But to an observant mind, the role of fourth dimension, time, is pertinent and important.

Unlike a surface or aerial structure, the interaction of the host medium with the underground structure plays a prominent role in the proper functioning of an underground structure. After an excavation is made, to accommodate an underground structure, the host medium undergoes a period of adjustment. The characteristics of the host ground require a period of time after excavation to come back to an equilibrium and stable condition. As such, the influence of these changing characteristics of the host ground, during the period of adjustment, must be accounted for in the design and construction of an underground structure.

Sometimes the ground characteristics change due to undesirable ground-water flow. For example at Shoshone Tunnel (USBR, 1987), with time the low-pH (pH = 4.0) water seeping through the limestone host ground created large cavities resulting in the invert collapse of the tunnel which required heavy resource investment to bring the tunnel back into operation. There are numerous other examples where, with time, the underground opening has suffered (1) basal heave of the tunnel invert, or (2) sides squeezed into the tunnel opening, or (3) the tunnel roof collapsed. These examples indicate that the effect of time must be accounted for in the design and construction of tunnels that traverse through somewhat poor host ground or where host ground shows characteristics of property changes with passage of time.

In the structural behavior of an underground structure, the host medium plays an active and important role. This requires that the constitutive relationship of the host medium must be accounted for during the design of an underground structure. But determination of ground characteristic is difficult to ascertain in a quantitative fashion. The difficulty is due to the fact that

ground is usually nonhomogeneous and therefore its characteristics cannot be predicted from pointwise observation of drill hole logging or other subsurface techniques. These techniques depend on homogeneity for extrapolation of ground parameters.

One way to get around this difficulty is to ignore the host media participation in the sharing of the load that otherwise would occur on an underground structure, but doing so will require that the support systems for the underground structure be hefty and thus the cost becomes high. Such designs are usually conservative and this practice should be discouraged. For sound engineering, the participation of the host media must be accounted for in the design which results in better engineering and reduced cost for the underground structure.

Certainly in locations of low cover or very poor ground conditions, the host medium will not participate as a structural member, but in most other situations, the contribution of the host medium as a load-sustaining member is a reasonable assumption.

When the host media acts as a load-participating member, average design stresses become meaningless. The concentration of stresses becomes predominant in the design. This happens because the majority of the host medium material does not yield before failure and, as such, the determination of peak stresses becomes very important. An exception to this will be an underground structure having low cover where the structural stability is more controlled by the geological discontinuities and where the movement of the host medium as a block is a more prominent factor than the peak stresses generated due to the excavation for the underground structure.

Another difficulty for the construction of an underground structure is the provision of access for construction. Very often, construction access is limited to the portals or from access shafts and/or access tunnels. This makes the transportation of men, materials, and equipment very difficult and restrictive. The construction activities, therefore, have to be cyclic and multi-activities of construction cannot be accommodated simultaneously. Thus, more time and resources are required for the construction of an underground structure as opposed to an above-ground or a surface structure.

In summary, the difficulties in assessing the constitutive relationship of the host medium; the possibility of the host medium material not yielding before failure; and the restricted space for conducting construction activities, make the final product, the underground structure, costly and time consuming. The design, construction, and instrumentation of an underground structure, therefore, require prudent planning, design, and construction sequencing and are more demanding than that required for surface and aerial structures.

1-2 COMMON TERMS

Common terms used in underground engineering are listed below:

"A" line. This is the dimensional line in an underground opening within which rock projections are not permitted. Initial structural steel rib supports may extend inside the "A" line. Also, hoop reinforcement for internal pressure or external load resistance may be placed inside "A" line.

Active supports. Those which impose predetermined loads at excavated rock faces at time of their installation.

Adit. A short tunnel connecting two main tunnels.

"B" line. This is the dimensional pay line for an underground opening excavation. The contractor is paid to "B" line dimensions no matter how much he over- or underexcavates, as long as he properly maintains the "A" line.

Bench. Part of an underground opening left temporarily unexcavated as the excavation or heading on top of it is advanced.

Boring. A subsurface investigation procedure for obtaining samples and studying ground-water conditions.

Breasting. Partial braced support of the face of the opening which helps in maintaining the stability of the ground during tunnel driving.

Burn cut. Pattern of relatively large "relief holes" drilled in the center of the face to provide space for the expansion of rock broken by a blasting agent.

California switch. Portable platform or siding, riding a rail track, used in a tunnel to allow passing of muck cars or material transportation trains.

Cover. Amount of rock and/or soil (or both) over the crown of a tunnel or a cavern.

Crosscut. A horizontal connection between two drifts or tunnels (adit).

Crown bars. Slender members of steel or wood installed in tunnel roof above sets.

Cut and cover. A shallow tunneling method in which ground is opened from the surface, the tunnel structure installed, and then the excavation is covered over.

Digger shield. A shield with means for mechanical excavation.

Discretionary support. Supports installed by contractor but not called for in the specifications.

Drift. A horizontal underground passage or a tunnel.

Drifting. Advancing of a drift or a tunnel.

Double heading. From one location, tunnel driven in two directions usually 180 degrees apart.

Double jack. Method of hand drilling using three men; two wielding heavy hammers and the third turning the steel.

Dry packing. Pea gravel or similar material forced in between lagging and the excavated surface to fill voids and furnish support.

Erector arm. Swing arm on boring machine or shield used for picking up supports and putting them in position.

Face. Nearly vertical wall at the farthest advance of a tunnel.

Far field stress. In situ stress.

Feeler hole. Hole driven ahead of the excavation for exploratory purposes.

Flowing ground. Ground flowing into the excavation like a viscous fluid under pressure of water.

Foot blocks. Blocks of material, wood, steel, or precast concrete or sacked concrete placed under ribs or posts to provide bearing.

Forepoling. Sharpened planks or steel sections driven ahead and over the top of supports into the tunnel heading as a protection against raveling in soft ground.

Gouge. Finely ground up rock material found in fault areas.

Grouting. The process of injecting grout into voids and discontinuities in the adjacent soil or rock to prevent or retard flow of water and to strengthen the surrounding ground.

Headframe. A tower built over a shaft to facilitate raising and lowering men, equipment, and material into the shaft.

Heading (top) and bench. A method of tunneling in which a top heading is excavated first followed by excavation of the bench.

High air. Compressed air used to activate pneumatic equipment and tools.

Initial support. Support placed immediately following excavation to maintain stability of the opening.

Invert struts. Structural compression members connecting the bases of the primary supports, installed across the invert to resist inward movement.

Jump set. Steel ribs or timber supports installed between overloaded supports.

Jumper. A steel bar used in manual drilling.

Lagging. Wooden planking or other structural material spanning between ribs.

Leachate. Suspended or dissolved materials transported into the tunnel by moving water.

Lifter. Holes drilled in the bottom of the invert and fired in the last firing cycle to fragment the invert rock and to lift the blasted muck.

Liner plate. Metal plates fastened together to support the ground behind the excavated face.

Lineation. A line on an aerial photograph indicating a linear geological feature.

- Mixed face. A face exposing simultaneously rock and unconsolidated materials.
- Mole. TBM (A tunnel boring machine).
- Muck. Broken rock or other material produced by the excavation process.
- Muck stick. A hand-held shovel.
- Mudsill. A horizontal longitudinal member installed to support sets or take load of the supports from the wall plate.
- Multiple drift. A method in which several interconnected small drifts are individually excavated and subsequently filled to form a continuous ring of support. The main tunnel is excavated from the inner side of the multiple drifts.
- Near field stress. Redistributed stress near the exposed faces of excavation.
- Open cut. A trench excavated from the surface.
- Open shield. A shield with no full face bulkhead.
- Packing. Filling the void between the rock and the support (backfilling).
- Panning. Sheet metal devices installed to deflect and divert the infiltrating ground water.
- Passive supports. They provide support only if the rock deforms and do not apply active pressure on excavated faces at the time of their installation.
- Pay line. "B" line.
- Permanent lining. Final lining which stays with the structure.
- Pilot tunnel. An exploratory tunnel of smaller diameter driven ahead of a prospective larger tunnel.
- Pipe jumbo. Traveling support for the discharge line of a concrete placer.
- Piping. A seepage phenomena in which locally concentrated flow causes erosion and void formation.
- Poling boards. Forepoling.
- Portal. An entrance to or exit from a tunnel.
- Posts. Vertical members of a tunnel support system.
- Powder factor. Number of pounds of powder or explosive per cubic yard of rock.
- Preplitting. A damage controlled blasting method in which the blast holes are closely spaced, lightly charged, and simultaneously blasted before the main blast.
- Prills. Water-resistant ammonium nitrate explosive.
- Primary lining. Initial lining which maybe used later as a part of the permanent structure.
- Pumping test. A pumped water field test to obtain information on ground-water conditions and permeability.

Raise. A shaft driven from bottom to top.

Raveling ground. Ground in which chunks of material begin to drop out after the excavated ground has been exposed for some time.

Rib. A curved (or straight) structural support member spaced longitudinally along the length of the excavated face.

Road header. A boom mounted excavating machine.

Rock bolts. See roof bolts.

Rock reinforcement. Internal structural member such as rock bolts, rock anchors and dowels embedded into rock mass, like concrete reinforcement, to structurally increase the internal strength of the ground mass.

Rock throw. The distance of throw of fragmented rock mass after blasting.

Roof. Overhead portion of an excavation.

Roof bolts. Structural members (usually long rods) installed in drilled holes, secured and tensioned to induce compression of rock zones.

RQD. Rock quality designation is an empirical way of determining the quality of rock.

$$RQD = \frac{\text{Cumulative total of core pieces larger than 4 inches in length}}{\text{Total length of bore hole}}$$

Running ground. Ground which on removal of support, runs like granulated sugar until the slope angle becomes equal to about 34 degrees.

Screed. Anything used to strike off a concrete placement.

Secondary lining. Permanent lining placed after the primary lining.

Segmental lining. Tunnel lining made of segmented prefabricated or precast units fitted together to conform to the tunnel shape.

Shaft. A vertical or near vertical excavation.

Shield. A structural enclosure to provide protection for construction personnel and provide space for excavation and support operations near the face of the tunnel.

Shifter. An underground equivalent to a foreman.

Shoring. A temporary support of excavation.

Shotcrete. Pneumatically applied concrete.

Shove. The act of advancing a tunnel shield with hydraulic jacks.

Skip. Hoist-operated mucking container.

Slick line. A section of the discharge line from a concrete placer.

Slickenside. A polished and striated surface within soil or rock mass resulting from relative displacement along the surface.

Sliding floor. A structural steel floor that is moved along over the excavated surface.

Slip. A minor fault (a geological feature).

Spall. To break off in small pieces.

Spiling. Wooden or steel support members driven in front of the second tunnel set and in the back of the first set from the face (similar to forepoling).

Spreader. Structural members (steel or timber) placed between the flanges of the structural steel supports.

Squeezing ground. A ground which squeezes or extrudes plastically into the excavation without any signs of fracturing or loss of continuity and without perceptible increase in water content.

Standup time. The amount of time after excavation that the ground can remain unsupported without local or general failure.

Stoppers. Drills for drilling overhead holes.

Struts. Compression members.

Super plasticizer. Water-reducing agents which increase workability of concrete and reduce water cement ratio.

Swell. Increase in volume of excavated material when water is added.

Swelling ground. Ground which undergoes volumetric expansion due to absorption of water.

Tail void. The annular space between the outside of the initial support and the outside of the tail of a shield.

TBM (tunnel boring machine). A full face rotating mechanical excavator.

Temporary lining. Timber or other supports which are not used permanently.

Tie rods. Tension members between ribs or sets.

Tights. Projections of rocks into a tunnel within the "A" line.

Tunnel. An underground stable opening of relatively uniform cross section and significant length.

Tunnel cycle. The cycle followed when excavating a tunnel by drilling and blasting method; the six parts of the cycle are drilling, loading, blasting, ventilating, mucking, and installing supports.

Tunnel excavating machine. Any mechanical tunnel excavating machine.

Turning the eye. The breaking out from a shaft to begin tunneling.

Unconformity. A surface which separates younger from older rock strata.

Value engineering. A procedure to suggest an alternative construction process other than specified in the original construction contract which will save cost or time or both.

Vent line. Pipeline to furnish fresh air and/or exhaust-polluted air.

Wall plate. Continuous horizontal structural member installed along the sides of tunnel near the spring line when a top heading or multiple heading is driven.

Working chamber. The space where construction work is being performed under compressed air.

Waterstop. A device placed and anchored across a joint to impede passage of water through the joint. Usually installed in concrete between lining placements.

1-3 PLANNING

Almost all construction work is governed by various planning acts. Certain special areas of interest falling under the preview of "conservation areas" require satisfying particular stringent criteria. Early and full consultation with the areas planning officer will later result in issuance of necessary permits for the construction. The project construction activity must satisfy the prevalent codes on various aspects of construction, including safety and health codes. Table 1-1 lists the initial planning consideration to select between the methods of construction: drill and blast, TBM, and road headers.

1-4 DATA COLLECTION

An adequate plan for design and construction requires collection of functional, geologic, environmental, hydraulic, material properties, and economic data.

Functional data provide necessary input for the determination of shape, size, grade, curves, and other related items for the actual dimensioning of the underground structure.

Geologic data provide information on the morphology, stratigraphy, petrography, geologic setting, discontinuity characteristics, ground water, and seismology of the host medium. These data provide input for the design and selection of the construction system.

Environmental data assist in evaluating the indirect costs, delays, and socioeconomic impacts of the project during and after the construction of the underground structures. Data on noise, vibration, dust, ground subsidence, utility line disruption, and traffic diversion constitute this kind of data.

Hydraulic data are required for the hydraulic analysis of underground structures to be used for transportation of water and/or liquid and for storage.

Material properties data are required for the design of the support system for both long- and short-term performance evaluation.

Economic analysis data include determining the applicable rate of interest, rate of return, and economic life of the underground structure in order to arrive at an acceptable cost and benefit ratio for the project.

These data need to be collected before, or concurrent with, the design of any underground structure.

TABLE 1-1

Planning considerations for selecting method of construction.

| Category | Drill and blast | TBM | Roadheader |
|--|--|--|--|
| Air blasts and slaps | Yes, but could be reduced by using delays in blasting | None | None |
| Average rate of progress (medium size tunnel)* | 30 feet per day | 100 feet per day | 40 feet per day |
| Borehole investigation | Not that important | Very important | Somewhat important |
| Boulder and glacial till | Drilling very difficult but most surely | Difficult for boulder but okay in till | Boulders not that difficult. Till okay |
| Clay gauges | Not that difficult | Very difficult to handle | Difficult |
| Comp. strength | All ranges | Not effective for rocks higher than 40 KSI comp. strength | Not effective for comp. strength greater than 14 KSI |
| Construction tolerance | Difficult to meet in bad ground | Will meet satisfactorily | Somewhat difficult to meet |
| Curvature of tunnel | Any curvature not less than 100-foot radius (controlled by muck removal equipment) | Not less than 575-foot radius | Can go through tighter curves (controlled by muck removal equipment) |
| Different ground conditions | Easy to accommodate | Very difficult to accommodate | Not that difficult |
| Drilling | Very sensitive to ground condition | Not required | Not required |
| Dust | Very dusty during smoking | Very much | Some dust |
| Exploratory drilling | No problem | Machine has to be specially designed | No problem |
| Faults | Caution required, difficulty in supporting but not in excavation | Cannot handle faults wider than 30 feet. 3- to 30-foot-wide faults are extremely difficult to handle | Medium difficulty |
| Flexibility | Very high | Very rigid | Intermittent |

TABLE 1-1 (continued)

Planning considerations for selecting method of construction.

| Category | Drill and blast | TBM | Roadheader |
|------------------------------------|--|--|--|
| Flowing ground condition | Not suitable | Special machines designed for the purpose | Not suitable |
| Fragmentation size | Controlled by type of explosive, its loading, and drilling pattern | Controlled by type of cutter, thrust of machine, spacing, and configuration of cutters | Controlled by type of bits and their spacing |
| Gaseous tunnels | Very difficult | Difficult | Medium difficulty |
| Gradient | Any gradient but not steeper than 18 degrees | Flatter gradient good. Not good for slopes exceeding 6 degrees (special machines up to 20 degrees) | Not exceeding 6 degrees |
| Ground-water problem | Can be controlled easily | Very difficult to control | Controllable |
| Initial cost | Not that much | Very high. Ratio of total cost/machine cost is 15 to 20 | Medium |
| Jet assisted excavation | Not required | Needed for very hard rocks, $\sigma_{uc} > 40$ KSI | Needed for $\sigma_{uc} > 14$ KSI* |
| Lead time for start of excavation | About a month | 3 months to 18 months to get a TBM | 1 month or so |
| Length of tunnel | Shorter lengths, up to 10,000 feet | Not less than 10,000 feet (unless used machines are available) | Up to 10,000 feet (in ideal conditions, longer lengths can be tried) |
| Mechanization | Not that high | Very much | Medium |
| Mixed face excavation | Not that difficult (most flexible) | Very difficult (least flexible) | Not that difficult |
| Monitoring direction of excavation | Not that important | Very important | Somewhat important |

TABLE 1-1 (continued)

Planning considerations for selecting method of construction.

| Category | Drill and blast | TBM | Roadheader |
|--|--|--|--|
| Muck removal | Very flexible (rails, trucks, etc.) | Needs conveyor belt to carry muck from face to back of machine and then on rails or trucks | Needs collecting arms and conveyor belt to carry muck from face to back and then on rails or truck |
| Multidrift excavation | Yes | Not used | Not usually used |
| Nearby ground deterioration | Very high for intensely charged holes | Not much | Medium |
| Noise | Too much (during blasting) | Not that much | Medium noise level |
| Number of manufacturers | Several | Three or four in the USA | Three or four in the USA |
| Partial face excavation | Always possible | Not possible | Possible |
| Portal accessibility | Not important | Very important else it will require shaft sinking for lowering of TBM | Somewhat important |
| Presupport and ground stabilization | Not difficult to perform ground presupports | Very difficult to do unless machine is designed for it | Not that difficult |
| Process | Cyclic, intermittent | Continuous | Continuous |
| Progress depends on: | Length, pattern, burden of hole, and type of explosive | Rock hardness, abrasion, machine's torque, horsepower, rotation, weight, and cutter types | Rock hardness, torque, horsepower, rotation, and cutter type |
| Required standup time (without modification of ground) | At least 3 hours (without presupports) | Earth balance or slurry shielded TBM may be used for zero standup time | 4 hours preferable |
| Rounds | About 12-foot drill takes out 10 feet round | None | None |
| RQD | Suitable for all ranges of RQD | Not good if RQD is between 25 to 45 percent | Good for all RQD |

TABLE 1-1 (continued)

Planning considerations for selecting method of construction.

| Category | Drill and blast | TBM | Roadheader |
|------------------------------|--|---|---|
| Running ground | Not suitable unless the ground is pregouted and stabilized | Specially designed machine can do the job | Not suited |
| Shape | Any shape | Only circular (except for special machines like mobile miner) | Circular, horse-shoe, modified horseshoe |
| Silty, sandy, clayey ground | Not suited | Shield TBM's are good | Not used |
| Size | Any size by heading and bench method | Currently from 6- to 40-foot diameter | Controlled by boom dimension. Generally 6 feet to 14 feet, but by benching any size |
| Squeezing ground | Some difficulty | Machine likely to be stuck unless it has walking blades or is capable of reducing its size | Some difficulty |
| Startup problems | Not severe | Severe and requires special considerations | Not severe |
| Steering problem | None | Depending on ground conditions, there is large propensity to steer out of alignment and grade | None |
| Surveying problems | More | Less | Medium |
| Uniformity of size of tunnel | Not necessary, can accommodate junctions, bifurcations, etc. | Extremely important | Not important |
| Utilization of equipment | 35 percent | 40 percent | 60 percent |
| Very firm clays | Yes | "Bit" type of cutters with shields | Not used |
| Vibration | Very much (during blasting) | Medium | High |

TABLE 1-1 (continued)

Planning considerations for selecting method of construction.

| Category | Drill and blast | TBM | Roadheader |
|----------|--|---|------------|
| Workers | Very skilled (supervisor of blasting optional) | Very skilled and specialized (opera- tor and mechanics) | Skilled |

*These are rough and average rates. Actual rates may vary.

(1 ft. = 0.305 m and 1 psi = 6.89 kPa)

σ_{uc} = Unconfined compressive strength

KSI = KIPS per square inch

1-5 SHAPE

The shape of an underground structure is influenced by the geologic setting, magnitude and orientation of in situ existing ground stresses, selected construction method, strength of lining material to be used, and estimated ground load including its distribution. The common shapes, shown in figure 1-1, for tunnels are circular, horseshoe, modified horseshoe, trapezoidal, elliptical, and rectangular. In squeezing, swelling, and soft ground the common tunnel shape is circular. Tunnels excavated by tunnel boring machines are also circular. A tunnel excavated by drill and blast method and in somewhat competent ground could be of a modified horseshoe shape. In locations where side pressures are expected to be exerted by the host media, the shape will be a horseshoe or circular. Elliptical tunnels are common where the principal stresses of the host media are unequal and for sewer tunnels for

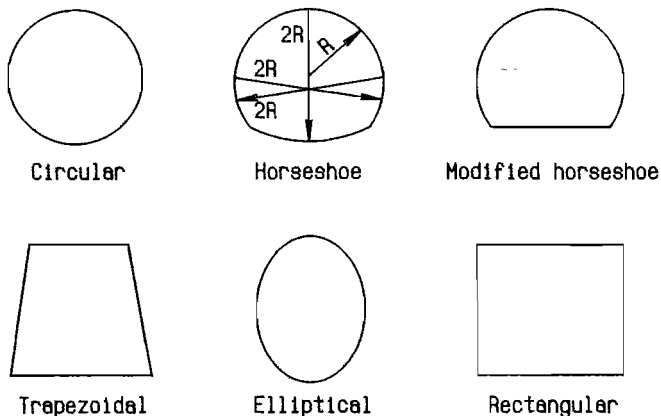


Fig. 1-1. Shapes of tunnels.

flow purposes. The major axis of the ellipse is usually parallel to the direction of major principal stress. Trapezoidal and rectangular tunnels are common in the mining industry.

The most common shapes for shafts are circular and rectangular. Circular shafts are suitable for machine and drilled excavation.

The most common shape for a large cavern is either a horseshoe or a modified horseshoe.

1-6 SIZE

The size of an underground structure is determined by the functional requirements of its capacity, geological setting, host media characteristics, and the selected construction methodology. It is prudent to allow some additional dimensions for future shotcreting or concreting of the tunnel in order to increase its service life. An additional 6 inches (150 mm) of dimensional allowance may extend the life of a tunnel by an additional 50 years.

From the point of view of construction convenience, the minimum desirable size of an opening should render a height of about 7 feet (2.1 m) and a width of about 5 feet (1.5 m). The largest soil tunnel built by multidrift method is 63 feet (19.2 m). In multidrift method, several tunnels of relatively small dimension are constructed parallel to and for the full length of the designed tunnel. These small tunnels or drifts are then filled with concrete, care being taken to fill the previous drift before excavating for the next adjacent drift. When all the drifts are filled in, thereby encircling the proposed tunnel's exterior extremities, the main tunnel is excavated under the protection of the filled in drifts. Eisenhower Tunnel in Colorado (Dutro and Patrick, 1982) and Mount Baker Ridge Tunnel in Washington (Johnson et al., 1985) are examples of multidrift construction.

The largest tunnel excavated in rock by a tunnel boring machine has a diameter of about 38 feet (11.5 m). The largest drill and blast tunnel has the dimension of about 50 feet (15.2 m).

The largest drilled shaft is presently limited to about 20 feet (6 m) in diameter, though a shaft as large as 116 feet (35.5 m) in diameter has been built (Tunnels and Tunnelling, 1986).

Presently, caverns are limited to a size of about 100 to 150 feet (30 to 46 m) in width. Larger caverns require special techniques of construction.

The spacings, dips, and characteristics of discontinuities in the host media determine the formation and size of movable rock blocks and thereby control the size of an opening and/or the underground structure.

1-7 GRADIENT

The grade of a tunnel is influenced by its functional requirement. For example, the maximum grade for a transit tunnel is 4 percent, the minimum being 0.3 percent. The maximum grade for a highway tunnel is 6 percent, minimum being about 0.5 percent for drainage purposes. The grade for a water conveyance or liquid transportation tunnel is usually dictated by hydraulic considerations and the permissible liquid velocity. The maximum permissible grade for muck handling is 18 percent for pneumatic tired vehicles and 4 percent for rail cars.

1-8 HORIZONTAL CURVES

Horizontal curves are uncommon for shafts and caverns. Sometimes, however, to avoid the severities in geology such as fault and shear zones or water bearing stratum, horizontal curves are provided in the alignment of some tunnels. Whenever a horizontal curve is required, an attempt should be made to provide as large a radius of curvature as possible. It is desirable not to use a radius of curve less than 200 feet (61 m), though in some tunnels curves as tight as 75 feet (22.9 m) in radius have been used. In tunnels excavated by tunnel boring machines, the minimum radius of curvature of a tunnel is dependent on the maneuverability of the tunnel boring machine. Railroad tunnels require a radius of curvature more than 1,500 feet (460 m).

1-9 VERTICAL CURVES

These curves are sometimes provided in highway tunnels. In all other underground structures, an attempt should be made to avoid the inclusion of such curves. Proper transition curves become essential for accommodating these vertical curves.

1-10 COVER REQUIREMENTS

For a free flow water or an air flow tunnel, the minimum cover over the crown of the tunnel should not be less than 50 percent of the largest dimension of the tunnel. For pressure tunnels, the minimum cover requirement should not be less than 50 percent of the operating head of the tunnel. For a pressure tunnel where there is less cover than the minimum, steel lining will be required to guard against the hydrofracturing of the host media if the pressure tunnel starts leaking. The vertical shafts obviously do not need any cover requirement. For caverns, the minimum cover should not be less than the largest dimension of the cavern.

In case of lower covers than those recommended, the support systems for the underground structures will have to be designed to take the full load of the overburden.

1-11 TUNNEL HYDRAULICS

Sudden changes in a tunnel cross section may create shock waves which may be structurally detrimental to the linings of water conveyance tunnels. The convergences and divergences should be designed to create minimal hydraulic impacts. Hydraulically, long radius bends are preferable over those with a short radius. Transitions, bends, grade changes, bifurcations, and manifolds for dividing flows should achieve a smooth change in velocity, an absence of swirls and vortices, and minimum head loss.

The freeboard or air space for gravity flow water conveyance tunnels is based upon a depth of flow equal to 0.82 times the internal diameter of a circular tunnel, but in no case should the freeboard be less than 1.5 feet (0.5 m). Frequently, flows which decrease this clearance have caused intermittent closure and consequent reduction of hydraulic radius. This results in a "gulping" cycle considered an annoyance unless it results in downstream overtopping of decreased freeboard.

The maximum design velocity for water conveyance mortar-lined, steel, or concrete-lined tunnels should be about 20 ft/s (6 m/s). Higher flow velocities, not to exceed the manufacturer's recommendation, can be used with epoxy-lined steel tunnel liners. Laboratory model tests for hydraulic flows are recommended for bifurcating or manifolded tunnels. The velocity for gravity flow can be calculated by using Manning's formula:¹

$$V = \frac{1.49 r^{2/3} s^{1/2}}{n}$$

1-1

where V = velocity in feet per second

r = hydraulic radius (wetted area in square feet/wetted perimeter in feet)

s = slope of invert of tunnel

n = roughness factor

$$(1 V = r^{2/3} s^{1/2} n^{-1} \text{ when units are in m/s and m}^2/\text{s})$$

Some of the most commonly used values of "n" are shown in table 1-2.

TABLE 1-2
Values of "n".

| Type of lining | Value of "n" |
|---------------------------|----------------|
| Concrete lining | 0.013 |
| Unfinished lining or rock | 0.14 to 0.017 |
| Very smooth lining | 0.018 |
| Segmented lining (core) | 0.017 to 0.018 |
| Cast-iron lining | 0.015 |
| Riveted steel | 0.015 to 0.018 |
| Brick | 0.016 |
| Uncoated | 0.013 |

Other acceptable formulas may be used for calculating the head loss.

The hydraulic loss in pressure tunnels is calculated by Darcy's formula:*

$$H_f = \frac{f l}{D} \frac{V^2}{2g}$$

1-2

where H_f = hydraulic loss (ft)

f = frictional coefficient

l = length of tunnel (ft)

D = diameter of tunnel (ft)

V = velocity (ft/s)

g = gravitational constant (ft/s²)

(* Use of units in meters gives head loss in meters)

The value of "f" changes with time because tunnel lining material deteriorates with time. For example, concrete tunnel linings get roughened by abrasion from water flows. A steel lining may corrode and be roughened by tuberculation. Vegetation and animal growth usually roughen the tunnel lining. The value of "f" ranges from 0.008 to 0.09 and is dimensionless.

In analyzing the pressure tunnels, one must consider the effects of all possible operational conditions to ensure that all critical design conditions have been studied. The effort taken to perform operation and surge studies in advance will serve as valuable prevention to eliminate costly problems in the future.

1-12 BRIEF HISTORICAL REVIEW

The art of constructing underground structures is nothing new to the human civilization (Szechy, 1973). In prehistoric days, tunnels and caverns were used for shelter, transportation, and escape from the attacks of animals and

enemies. During Biblical times, we notice descriptions of several tunnels connecting palaces and temples. The use of tunnels to carry water was well known to Roman rulers. The Greeks were the first to utilize advanced surveying techniques, about 500 B.C., to drive tunnels from both portals toward the middle of the tunnel which substantially reduced the total time of construction of the tunnel. This was a great achievement in an era when civil engineering works like tunnels required large amounts of cheap slave or prison labor and was very time consuming. The Romans, however, were the great tunnel builders. Because of the Roman philosophy that a civil engineering work had to last forever, the Roman tunnels are massive by today's standards. Today, we think 50 to 100 years life as adequate for tunnel structures.

The development of gunpowder during the Renaissance era, relating its application to the tunneling industry, very quickly superseded the old tunneling methods of cracking rock by shovels, picks, fire, and water. With the advent of blasting, ventilation systems had to be improved because the time taken to clear the smoke after blasting by natural processes was very long and unproductive. The cyclic process of drilling, loading, firing, smoking, scaling, and mucking is a very flexible system for tunneling and is applicable for very hard rock tunneling, short tunnels, and difficult excavations such as near the intersection of tunnels.

Marc Isambard Brunel, 1818 (Mayo, 1968), was the inventor of shield tunneling. Brunel tried his system in ground conditions that had very poor "standup time." Standup time is defined as the time during which the roof of an excavation will stand by itself without the help of external supports after the excavation is made. Since the time of Brunel, shield tunneling has undergone rapid improvements and today there are several types of tunnel boring machines which can efficiently excavate the most complex host medium.

Calladon and Cochrane, during 1818-30, established the importance of compressed air in tunneling through soft ground charged with water. Talbot, in about 1894, found the advantage of driving a tunnel on a 24-hour-per-day basis which provided ground stability during excavation through somewhat poor ground.

Modern tunneling activity received a boost during 1857 while tunneling through the Alps in Europe. Dynamite was used for the first time for the construction of St. Gotthard Tunnel and mechanical drills were also introduced at that time.

The sunken tube method of tunnel construction was introduced in America during the early 1900's for the railroad tunnel under the Detroit River. That was a major breakthrough in tunneling in aqueous environment.

That is a very quick overview in the history of tunneling. The basic difference between tunnels of olden times and today's tunnels lies in the sharing of the responsibilities for the tunnel construction. In olden times,

like any other big civil engineering endeavor, tunnel construction was in the domain of rulers and kings who had immense resources of time, capital, and labor force. Under that kind of sovereignty system, the underground construction took place. In modern days, there are usually several vested interests in the design and construction of underground structures and the benefits of individual sovereignty has completely disappeared. Today, owners, engineers, consultants, contractors, lawyers, material and equipment suppliers, geologists, planners, specialty contractors, administrators, and others team up to complete an underground project. The project becomes a team effort and that is the difference.

1-13 HIGHLIGHTS IN THE HISTORY OF TUNNELING (Bieniawski, 1979)

- 700 B.C. Siloam water tunnel in Jerusalem: 2 feet, 4 inches wide; 5 feet, 7 inches high; and 1,000 feet long. It is still in use.
- 41 A.D. Fucinus Tunnel in Rome: 19 feet wide, 9 feet high, and 3.5 miles long. Approximately 30,000 slaves worked for 11 years. The advance rate was about 3 inches (75 mm) per week.
- 1556 George Agricola wrote the first handbook on tunneling and mining.
- 1860 Invention of black powder.
- 1866 Machine drilling of blast holes was introduced.
- 1950 Advent of modern tunneling machine.

1-14 HISTORIC RATES IN TUNNELING

- 3 inches (75 mm) per week with 30,000 slaves.
- 10 feet (3 m) per week with hand drill and black powder.
- 200 feet per week with power drill and gelatin powder.
- 1,000 feet per week by machine boring.
- The world record of single day advance is 403 feet in shale for a tunnel bore of 10 feet, 7 inches in Oso Tunnel, U.S. Bureau of Reclamation, 1967.

1-15 FAMOUS AMERICAN TUNNELS

- 1820 Schuylkill Canal Tunnel, Auburn, Pennsylvania, is the first tunnel in the USA. It is 20 feet by 18 feet by 820 feet long.
- 1831 Allegheny Portage Railroad Tunnel, Altoona, Pennsylvania, 900 feet long.
- 1975 Eisenhower Tunnel, Colorado.
- 1979 World's largest diameter bored rock tunnel.
- 1988 TARP (Tunnel and Reservoir Project), Chicago. (Under construction)
- 1987 Mount Baker Ridge Tunnel, Washington, largest diameter soil tunnel. (Under construction)

1-16 SOME FAMOUS TUNNELS AROUND THE WORLD

| <u>Name</u> | <u>Country</u> | <u>Service</u> | <u>Length (km)</u> | <u>Year of construction</u> |
|---------------|-------------------|------------------|-------------------------|-----------------------------|
| Simplon | Italy/Switzerland | Railway | 19.8 | 1906 |
| Gotthard | Switzerland | Railway | 16.3 | 1881 |
| Montblanc | France/Italy | Highway | 12.6 | 1965 |
| Alva B. Adams | USA | Water conveyance | 19.5 | 1946 |
| Mersey | U.K. | Highway | 4.2 | 1886 |
| Seikan | Japan | Railway | 53.9 | 1985 |
| Moffat | USA | Railway | 9.9 | 1927 |
| Eurotunnel | U.K./France | Railway | Under construction 1988 | |

(1 Km. = 0.6 miles)

1-17 DIMENSIONS OF SOME COMPLETED CAVERNS

| <u>Name</u> | <u>Country</u> | <u>Dimension (ft)</u> | | |
|----------------------|----------------|-----------------------|--------------|---------------|
| | | <u>Length</u> | <u>Width</u> | <u>Height</u> |
| Helms | USA | 336 | 83 | 125 |
| North Field Mountain | USA | 328 | 70 | 120 |
| Dinorwic | North Wales | 588 | 77 | 196 |
| Okutataragi | Japan | 423 | 67 | 136 |
| Shintoyne | Japan | 459 | 72 | 151 |
| Okuyoshino | Japan | 518 | 67 | 136 |
| Numappara | Japan | 430 | 66 | 110 |
| Racoon Mountain | USA | 490 | 72 | 110 |
| Drakensburg | South Africa | 552 | 51 | 87 |

(1 ft. = 0.305 m)

1-18 CLASSIFICATIONS OF UNDERGROUND STRUCTURES

Underground structures can be classified by depth, use, ground type, methods of construction, type of lining, and flow conditions. Table 1-3 has the details of the classification:

TABLE 1-3
Classifications of underground structures.

| Classification | Tunnels | Shafts | Caverns |
|---------------------|---|---|--|
| A. By depth | Shallow Deep | Shallow Deep | Shallow Deep |
| B. By use | | | |
| B.1 Transportation | 1. Pedestrian 2. Highways 3. Railways 4. Subways 5. Navigational | 1. Man haulage 2. Material haulage 3. Equipment haulage | |
| B.2 Conveyance | 1. Water supply 2. Sewage 3. Storm water 4. Intake | 1. Water supply 2. Sewage 3. Storm water 4. Intake | |
| B.3 Utility | 1. Utility lines a. Telephones b. Cables c. Electrical lines d. Pipelines | 1. Access | 1. Vaults (manifolds, junction boxes, etc.) |
| B.4 Storage | | | 1. Liquid storage 2. Gas storage 3. Waste storage 4. Food storage 5. Refrigeration |
| B.5 Parking garages | | | Yes |
| B.6 Recreation | 1. Swimming pools 2. Play centers 3. Underground theaters 4. Gymnasiums | | |
| B.7 Mining | 1. Access 2. Ore recovery 3. Transportation | 1. Access 2. Ore recovery 3. Ventilation 4. Transportation | |
| B.8 Defense | 1. Shelters 2. Military command centers | | 1. Shelters 2. Military command centers |
| C. Ground type | Hard Soft Mixed | Hard Soft Mixed | Hard Soft Mixed |

TABLE 1-3 (continued)
 Classifications of underground structures.

| Classification | Tunnels | Shafts | Caverns |
|----------------------------|--|---|--|
| D. Methods of construction | Drill and blast Boring machines Roadheader Cut and cover Sunken tube Water jet assisted | Drill and blast Blind drilled Raise bored Down slashing Mechanical excavators | Drill and blast Mechanical excavators |
| E. Lining | Concrete lined Precast segmented liners Shotcreted tunnel Unlined Steel liners | Concrete lined Precast segmented liners Shotcreted Unlined Steel liners | Concrete lined Shotcreted |
| F. Primary support | Steel ribbed Shotcreted Rock bolted | Steel ribbed Shotcreted Rock bolted | Steel ribbed Rock bolted |
| G. Flow conditions | Free flow Pressure flow | Free flow | |
| H. Length | Long Intermediate Short | Long Short | Long Short |
| I. Grade | Flat Steep Gentle | | |
| J. Pattern | Single Multiple Stacked Bifurcating | Single | Single Multiple |
| K. Alignment | Straight Curved Spiral | Straight | Straight |

1-19 CODE REQUIREMENTS

Most codes are written for above-ground structures and, as such, they do not include the effects of ground/lining interaction. Even then, the relevant portions of codes such as AASHTO (American Association of State Highway and Transportation Officials), the railway codes, and State codes for clearances must be adhered to. The Uniform Building Code, Chapter 3, "Permits and Inspection", provides a valuable guide for obtaining the types of permits necessary to start construction of an underground structure. The relevant

codes for the design of steel, wood, and concrete structures are invaluable tools for the design of the support systems for underground structures.

1-20 SYSTEM ANALYSIS

System analysis, also known as investment analysis, rate of return analysis, benefit cost analysis, or payout analysis is essential to provide an insight into the economic justification and feasibility of an underground structure project. This is more important in an underground project than a surface or aerial project because an underground project is many times more expensive than a surface or aerial project. As in any engineering project, the selected underground project alternative must provide maximum benefits at the minimum expenditure of cost and time.

Benefits may be tangible or intangible. It is difficult to assign monetary value to an intangible benefit such as socioeconomic benefits that will accrue as a result of the construction of an underground structure. Sometimes, intangible benefits of an underground structure outweigh by several times the tangible benefits; for example, the advantage of an underground shelter against nuclear explosion may not become tangible unless there actually is a nuclear holocaust. Yet, it is essential that the public sector (Government agency) construct such shelters for the protection of their citizens to protect them from such possible dangers. How, then, can one assess any tangible benefits to the construction of such a shelter? It is very difficult to evaluate. Unlike a private sector undertaking, an underground structure project usually lies in the domain of the public sector where the nature of the decision making is a "group, not individual, action" and, as such, these decisions may not be profit oriented. Safety of citizens and welfare of the population become more important than obtaining a profit.

The parameters for performing a system analysis are (1) the objectives of the project, (2) acceptable rate of interest, (3) the constraints to the project, (4) economic life of the project, (5) level of risk which the project owners can withstand, and (6) the amount of underground geological uncertainty likely to be encountered during the construction of the project. Some of these parameters are very hard to evaluate realistically.

The rate of interest is influenced by the cycle of supply and demand of the money and also to the Government policies that regulate the movement of the money. It is axiomatic that the rate of interest will not remain constant throughout the economic life of the project. It is essential that a sensitivity analysis with different rates of interest be performed before deciding the soundness of an underground project.

In order to perform the sensitivity analysis, the interest formulas stated as equations 1-3 through 1-8 may be beneficial:

$$S = P(1+i)^n \quad 1-3$$

$$P = S(1+i)^{-n} \quad 1-4$$

$$R = P \frac{i(1+i)^n}{(1+i)^n - 1} \quad 1-5$$

$$R = S \frac{i}{(1+i)^n - 1} \quad 1-6$$

$$P = R \frac{(1+i)^n - 1}{i(1+i)^n} \quad 1-7$$

$$S = R \frac{(1+i)^n - 1}{i} \quad 1-8$$

In the equations above; P = Present value of money
 S = Future value of money
 i = Rate of interest over the period
 n = Number of periods
 R = Capital recovery factor

Constraints for an underground project run into several categories: physical, legal, administrative, political, and financial. These constraints regulate selection of an alternative. Physical constraints such as encountering an active fault during construction may raise the cost of the project substantially and in some cases may require its abandonment. Sensitivity analysis, with respect to different constraints, needs to be performed before deciding on an alternative as the most viable one to construct.

The economic life of an underground project usually runs from 50 to 100 years. However, the time when the project may become economically obsolete before that period cannot be predicted with surety. It is crucial during the system analysis process to assess a realistic, most probable economic life of the project.

The amount of risk that an owner will be able to sustain is somewhat subjective and varies from owner to owner. A definite percentage cannot be recommended. Ideally, an owner does not want to assume any risk for the underground project, but slowly owners are beginning to realize that assuring zero risk requires a large investment of capital.

Encountering uncertainties in a geological sense is more a rule than an exception. No owner has resources of time and monies large enough to authorize

a huge subsurface investigation program to rule out encountering of uncertainties during the construction of an underground project; as such, encountering of uncertainties during construction should be a design consideration.

Based on the information presented in the preceding paragraphs, it could be stated that performing a system analysis for an underground project is not an accurate science. It is really somewhat speculative and an experienced underground engineer will come closer to his or her predictions on the economic feasibility of the underground project.

Of all the different methods of performing system analysis such as present worth, rate of return, and benefit cost analysis, it appears benefit cost analysis is being used more frequently than the other methods of analysis.

In present worth analysis, the capital cost which includes cost of land, cost of construction, costs of material and labor, cost of equipment, and the operating and maintenance costs for future years over the economic life of the project are converted to the same base year with the use of equations 1-3 and 1-4, and the alternative which results in the minimum capital outlay is considered to be the best.

In the rate of return of analysis, the present capital outlay is annualized over the economic life of the project and the alternative which renders the maximum rate of return, after discounting the operating and maintenance costs, is considered to be the best.

The benefit cost ratio analysis is, however, much easier to understand because it gives the benefits in terms of cost spent on the project. In this analysis, the benefits, both tangible and intangible, are accounted for by assigning monetary values to them. The alternative which gives the maximum benefits per unit of dollar spent is considered to be the best.

As stated earlier, the performing of system analysis for an underground structure is not an exact science and as such in making decisions, case histories on similar projects will serve as an invaluable tool in making intelligent decisions on an underground project.

1-21 TOLERANCES AND SURFACE FINISHES

Tolerances are defined as allowable variations from specified lines, grades, and dimensions and the magnitude of surface irregularities. Tolerances must be consistent with modern construction practice and are governed by the effect they have on the final structure such that they do not impair the structural or operational functions of a specific structure. The allowable tolerances shown in table 1-4 are suggested as a guide.

TABLE 1-4

Allowable tolerances.

Hydraulic tunnels

1. Flow velocities less than 20 ft/s (6 m/s)
 - Departure from excavated alignment \pm 2 inches (50 mm)
 - Departure from specified grade \pm 1 inch (25 mm)
2. Flow velocities greater than 20 ft/s (6 m/s)
 - Departure from excavated alignment \pm 2 inches (50 mm)
 - Departure from specified grade \pm 1/2 inch (12 mm)

Other tunnels

3. Variation from specified inside diameter 0.5 percent
4. Variation in lining thickness + 1/2 inch (12 mm)
5. Surface irregularities (not subjected to high-velocity flow)
 - Abrupt irregularities 0 to 1 inch (0 to 25 mm)
 - Gradual irregularities 0.06-0.12 mm
6. Tunnels by pipe jacking
 - Outside diameter 1/4 to 1/2 inch (6 to 12 mm)
 - Inside diameter 1/4 to 1/2 inch (6 to 12 mm)
 - Out of squareness 1/8 to 1/3 inch (3 to 8 mm)
 - Departure from alignment 1 to 4 inches (25 to 100 mm)
 - Departure from grade 1 to 2 inches (25 to 50 mm)

1-22 QUANTITY ESTIMATES

The quantity estimates should be based on sound principles of estimating taking into consideration the adopted construction, contractual, and design methods for the completion of a specific project. To illustrate, the quantities estimated for a shotcreting job will not only consist of the geometrical volume of shotcrete but will be increased to include the rebound of shotcrete material. Of course, the rebound loss is dependent upon the process used to shotcrete (dry or wet process), the experience of operator, and the dryness of the surface to which the shotcrete will be applied. Sometimes, inclusion of as high as 70 percent rebound loss is considered justified for estimating purposes.

For concrete tunnel lining, the estimated quantity of cement per cubic yard of lining is not only dependent on the design strength of concrete but also on the method of excavation used. For example, one additional sack of cement per cubic yard of lining is allowed to fill in the over break area when the drill and blast method of excavation is used for tunneling. Tunnels excavated by

machines do not require the additional sack of cement.

An additional 5 percent, by weight, of reinforcing bars is required for estimating purposes for longitudinal rebars to reflect overlap of the reinforcements.

When calculating overall mucking out volume, the bulking volume of excavated material has to be accounted for which may be as high as 200 percent depending on the excavation method.

1-23 RIGHT-OF-WAY

Right-of-way is the total requirement of all real property interests and uses, both temporary and permanent, needed to construct, maintain, operate, and protect the underground structure. The right-of-way envelope is influenced by the topography, drainage, service roads, the nature of structures, and the appurtenant structures selected.

The specific right-of-way for a particular project may consist of one or more types of easements such as surface easement, underground easement, aerial easement, construction easement, and utility easement. The easements may be permanent or temporary.

The designer must prepare right-of-way plans and show the plats of recordation. The suggested underground easement for rock tunnels is an envelope containing two horizontal planes: one about 10 feet (3 m) above the crown and the other about 35 feet (10.5 m) below the elevation of the invert, and two vertical planes about 15 feet (4.5 m) from the tunnel sides. For earth tunnels, the easement may be reduced by 5 feet (1.5 m) for the horizontal and vertical planes.

1-24 SIZE OF STAGING AREA DURING CONSTRUCTION

The staging area must provide a sufficient work area for the contractor and the owner so that the facilities for construction, supervision, and inspection can be adequately performed. The requirements of the contractor's work area depend on the type of excavation method used. The following facilities shown in table 1-5 must be included in the work area.

The size and configuration of a staging area is dependent on construction and contractor's requirements and the topography of the terrain. Constrictions of topography such as steep gradients or proximity to steep slopes or narrow valleys tend to require an elongated configuration of the staging area. Normally, a 500- by 300-foot (152- by 92-m) space could adequately serve the requirements for a moderate sized staging area. An additional 150 by 100 feet (46 by 30 m) may be required for a waste water treatment plant site for treating the water recovered during tunneling before finally disposing the water to a natural drainage system.

TABLE 1-5
Staging area facilities.

| Facilities | Drill and blast method | TBM method* |
|-----------------------------|---------------------------|-------------|
| Contractor trailer office | Yes | Yes |
| Government trailer | Yes | Yes |
| First aid station | Yes | Yes |
| Maintenance and repair shop | Yes | Yes |
| Portal shed | Yes | Yes |
| Storage tank and pumps | | |
| Diesel | Yes | Yes |
| Gas | Yes | Yes |
| Butane | Yes | Yes |
| Storage shed | Yes | Yes |
| Explosive storage | Yes | Yes |
| Tool shed | Yes | Yes |
| Restroom | Yes | Yes |
| Garage | Maybe | Maybe |
| Water line | Yes | Yes |
| Sewer line | Yes | Yes |
| Power line | Yes | Yes |
| Telephone line | Yes | Yes |
| Waste water treatment plant | If required | If required |
| Settling ponds | If required | If required |
| Muck disposal area | | |
| Nearby | Yes | Yes |
| Far | If required | If required |
| Borrow area | If suitable | If suitable |
| Restricted area | Yes | If required |
| Barricades and signs | For storage of explosives | If required |

*TBM includes road headers.

The staging area must be accessible; if not, an access road has to be built before the start of construction.

The right-of-way for the staging area and other facilities must be obtained by the owners in the form of construction easements before the start of actual construction.

1-25 PROTECTION OF FACILITIES

It is desirable and usually necessary to protect the completed structures such as powerplants, pumping plants, dams, tunnels, and shafts so that they can continue to perform their intended services.

Protection of structures, installations, equipment, etc. from enemy attack, sabotage, subversive action, theft, malicious attack, riot, espionage, unauthorized use, entry, or other such action is very important to safeguard investments.

Protective barriers, protective lighting, intrusion detection systems,

access control, and security forces either alone or in combination are to be used for the protection of the facilities. Natural barriers, fences, walls, locked gates and doors, vehicle barriers, and limited approaches can constitute protective barriers. Protective lighting is usually high-intensity lighting on actuated switches. Intrusion detection systems are usually television monitors actuating an alarm system in case of any violation. Actual patrolling can be done by employees, guards, police, or trained dogs.

1-26 LIGHTING

Minimum illumination intensities as recommended in table 1-6 should be provided at all times. The selection and installation of lighting equipment should aim at providing illumination that is glare free and does not cast long dark shadows.

TABLE 1-6

The minimum illumination intensities in tunnels, caverns, and shafts.

| Particulars | Minimum intensities (foot candles) |
|--------------------------------------|---------------------------------------|
| Access ways | 5 |
| General working areas | 5 |
| Active headings (tunnels and shafts) | 10 |
| Welding areas | 30 |
| First aid stations and offices | 50 |
| Toilets and washrooms | 10 |
| Storage areas | 10 |
| Shops | 30 |

Explosion proof lighting and electrical systems shall be used for artificial illumination in areas where flammable liquids, vapors, fumes, dusts, or gases constitute a hazard.

Portable lighting devices including hand lamps, cap lights, and flash lights shall be approved as safe.

The national electrical code standards shall be maintained at all times.

1-27 VENTILATION

Ventilation in an underground structure should be a concern both during and after the construction. Natural ventilation systems in which the tunnel air is displaced longitudinally by fresh air may not be sufficient during or after the construction. The replacement of underground air by external fresh air in the right amount and velocity must ensure the acceptable level of air quality. The quantity of fresh air required depends on the amount of carbon monoxide,

methane, hydrogen sulfide, smoke, or other pollutants found in the air and also on the number of working persons, types of machinery used, the diameter of tunnel, and the excavation method used. In addition, actual requirement of the volume of fresh air is site dependent and depends on the atmospheric pressure differential at the portal and the tunnel face, altitude of tunnel, atmospheric temperature, and the seasonal variations. Table 1-7 provides rules of thumb for ventilation requirements.

TABLE 1-7

Minimum ventilation requirement for underground structures during construction.

| Category | Minimum volume of fresh air required |
|--------------------|---|
| Number of workers | 200 ft ³ (5.70 m ³)/minute/person |
| Diesel car | 75 ft ³ (2.20 m ³)/minute/brake horsepower |
| Diameter of tunnel | 1,000 x diameter of tunnel ft ³ (28.32 m ³)/minute |

The usual fan spacing may be 2,000 to 3,000 feet (600 to 900 m) and the fan should be usually reversible; i.e., it should be able to work on both "blow in" or "exhaust" modes.

The air line size for the usual methods of excavation is shown in table 1-8.

TABLE 1-8

Air duct size.

| Method of construction | Air duct/air line diameter in inches |
|------------------------|--------------------------------------|
| Drill and blast | 20 to 30 inches (500 to 750 mm) |
| Road header | 12 to 24 inches (300 to 600 mm) |
| Tunnel boring machine | 24 to 54 inches (600 to 1,350 mm) |

The velocity of air replenishment should lie between 50 to 100 feet (15 to 30 m) per minute and the quantity should be between 50,000 to 100,000 ft³ (1,415 to 2,830 m³) per minute.

To ensure an acceptable level of air quality, the upper limit of the following gases should not exceed the values shown in table 1-9.

TABLE 1-9
Desirable upper limit of concentration of harmful gases.

| | In parts per million |
|------------------|--|
| Carbon monoxide | 50 |
| Nitrogen oxide | 5 |
| Hydrogen sulfide | 10 |
| Methane | 1.5 |
| Other flammables | 40 percent of their respective lower explosive limits |

Methane gas in concentration between 5 to 15 ppm (parts per million) is highly explosive, concentration less than 5 ppm is not explosive, concentration more than 15 ppm cannot support explosion. Gas monitors may be used to ensure that the level of gases in the tunnel are within specified limits.

For dust control at the excavation face, water jetting may be used and the ventilation fan run in the exhaust mode.

In most cases, exhaust mode of air replenishment is more advantageous than the "blow in" mode. In the exhaust mode, air is sucked from the tunnel face through the inlet heading of the air duct and fresh air is allowed to rush into the tunnel from the portal area.

In all cases, the ventilation requirements must conform with the better of (1) the prevalent code regulations; or (2) the recommendations outlined in the latest edition of "Industrial Ventilation," published by the American Conference of Governmental Hygienists; or (3) the latest edition of "Threshold Limit Values for Chemical Substances and Physical Agents in Workroom Environment," also published by the American Conference of Governmental Industrial Hygienists.

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Chapter 2

DESIGN METHODS

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2-1 GENERAL

The first step in the design of an underground structure is to evaluate the functional requirements of the underground structure and its environmental conditions before and after the construction. It is imperative that the underground structure functions well and does not create any adverse impact on the environment.

The process of design inherently consists of selecting material and member sections which will not fail and will satisfactorily provide the required functional response of the proposed structure. Failure is the inability of the structure to function as designed.

Because underground structures are actual physical structures, it is necessary to analyze individually and collectively the different elements that constitute the structures. The final displacements of, or the developed forces in the element members should not exceed the allowable limits which will hamper the proper functioning of the structure as a whole. It becomes necessary, therefore, to model the structure in an analyzable format. To study the response of the structure, several types of models may be considered. Mathematical or computer models are generally less costly and less time consuming than a photoelastic model or an actual three-dimensional scaled physical model. Once a model is selected, the governing equations for equilibrium or motion are then established which are then solved and the solution tested for uniqueness, existence, sufficiency, and relevancy. The solution techniques for solving equations for equilibrium or motion could be simple, rigorous, numerical, or empirical, but the solution must exist, be unique, and relevant. It is occasionally impossible to get unique solutions such as those found on eigen value engineering problems on dynamic analysis. In those cases, one has to be satisfied with the most applicable eigen pair (frequency and mode shape) solutions. In steady-state problems, where response of structure does not change with time, and also in some propagation related problems where structural response changes with time, unique solutions are possible to find.

Any physical structure for analysis can be further divided and classified into different engineering systems according to geometry, loading, and material properties. Geometrically, a structure could be either continuous or noncontinuous. A continuous structure can be analyzed by using a set of differential or integral equations or by hybrid equations using both differential and integral equations. A discontinuous or discrete geometry will require numerical methods to solve the equations of equilibrium or motion.

The loading on a structure could be uniform or varying or may be pulsatic that changes with time. Again, the magnitude, direction, and point of application of the loading may be time dependent or independent. The material may be brittle, ductile, elastoplastic, or viscoelastic plastic. It may behave linearly or nonlinearly. In addition, the whole engineering system may be subjected to large displacements or large rotating.

Given such a wide variation in the range of loading, material properties, and effect of time, the design of underground structures is somewhat more complex than the design of other structures.

2-2 FUNCTIONAL REQUIREMENTS

The functional requirements of different underground structures are stated in table 2-1.

TABLE 2-1

Functional requirements of different underground structures.

| Underground structure | Minimum functional requirements |
|--------------------------|---|
| Water conveyance tunnels | Protect against host material fallout into tunnel, provide hydraulic capacity, carry expected flow without hydraulic adverse impact, hydraulic lining must be consistent with flow velocities, and protect against exfiltration, infiltration, and cavitation. |
| Power tunnels | Same as water conveyance tunnels and in addition must not have sharp bends or intersections. Lining material must ensure against power loss either due to adverse hydraulics or due to fluid loss. Must also ensure against landslides or hydraulic jacking due to water infiltrating into the host medium. |
| Storage caverns | Protect against host material fallout. Provide adequate storage without loss or contamination or property deterioration. Provide against infiltration or exfiltration. |

TABLE 2-1 (continued)
Functional requirements of different underground structures.

| Underground structure | Minimum functional requirements |
|-----------------------|---|
| Railway tunnel | Protect against host material fallout, provide adequate ventilation, lighting, and drainage. Should have gentle curves and grades consistent with the locomotive's capacity. |
| Highway tunnel | Provide proper ventilation and exhaustion of vehicle fuel fumes, provide proper ventilation and lighting, and provide grades and curves that are easy to communicate by the intended vehicles. Material lining should require less maintenance. |
| Shafts | Provide vertical and horizontal stability of the shaft opening. Provide against host material fallout. Provide ventilation, drainage, and lighting as required. |

2-3 LOADING

The loading mechanism of an underground structure is different from that of a surface or an aerial structure. For underground structures, the most important loading comes from the host ground itself. In competent host ground, the ground loading on the underground structure is quite insignificant and may be equal to zero where as in incompetent ground, it may be quite significant. The host ground pressures on the underground structure is quite complex. It is dependent on several factors such as the relative stiffness of the structure and the host ground, the elapsed time between the excavation and installation of support, the characteristics of the host ground, the in situ pressures, the size of the opening, the location of water table, and the adopted methods of construction.

If the support structures used to ensure the stability of the opening is relatively stiffer than the host ground, the support structure will attract more loading. In the same situation, a support system that is more flexible than the host ground will take lesser load than a stiffer support. In case of a flexible support, the ground by arching will take the major portion of the load and the support system will take a smaller share of load. A stiffer support attracts more load and a flexible support attracts more displacement. A steel support is more flexible than a concrete lining.

Figure 2-1 indicates a ground characteristic curve in which the ground pressure is plotted as an ordinate and radial displacement as an abscissa.

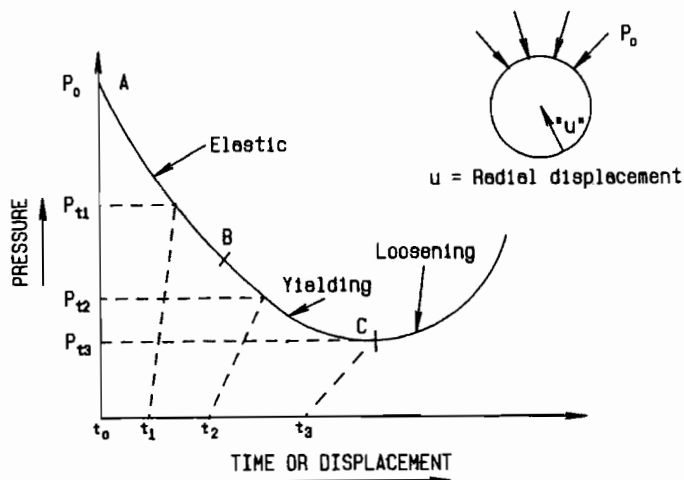


Fig. 2-1. Characteristic Curve.

At time " t_0 ," the theoretical pressure on support is " P_0 ," the in situ pressure and the radial deformation " u " is zero. Theoretical, because it is impossible to place a support without relaxing the ground and without reducing " P_0 ." When an opening is created, the excavation moves toward the opening and the value of " P_0 " starts to diminish. The portion "AB" of ground characteristic curve is purely elastic. From "B" to "C," the ground starts to yield, but by "arching," it can still take some load. From "C" onward, the ground starts to "loosen" and it can no longer sustain any load. At time " t_1 " when a support is placed to arrest the radial movement, it will have to sustain a pressure equal to " P_{t1} ." If the same support is placed at time " t_2 ," it will have to sustain a load equal to " P_{t2} ." As can be seen, " P_{t2} " is smaller than " P_{t1} ." A prudent design will be to place the support at time " t_3 " or just before the ground starts to loosen itself. At that time, the support will be required to sustain the least pressure " P_{t3} " to keep the opening stable. It is, however, very difficult to assess the exact time " t_3 " after which the ground starts to loosen up. This concept is further discussed in section 2-11.

Loosening load is a generic term and indicates the load that comes on the support structure immediately after the ground is excavated. In some cases, the final load coming on the support structure may ultimately exceed the loosening load with time due to the existence of "genuine ground pressure." The genuine ground pressure may be less than or equal to or be several times the in situ ground pressure that existed before the excavation.

For horizontal beddings, the height of rock " H_R " that is loosened and that exerts loosening load on the crown is given by equation 2-1(a).

$$H_R = 0.5 W \quad 2-1(a)$$

where W is the width of opening.

For vertical beddings

$$H_R = 0.25 W \quad 2-1(b)$$

and for inclined bedding

$$H_R = 0.25 \text{ to } 0.5 W \quad 2-1(c)$$

The pressure due to loosening is

$$P_L = H_R \times \gamma_r / \text{unit area} \quad 2-2$$

where γ_r is the unit weight of the rock.

The formations like igneous, sedimentary, and metamorphic rocks possess different characteristics. Usually, igneous rocks are heavier and more competent than sedimentary rocks. Competent rocks show very little amount of loosening load but substantial amounts of genuine rock pressures. Genuine rock pressure is the inherent rock pressure which the rock is capable of exerting as a consequence of the excavation and is somewhat dependent on the geological history of the rock.

Discontinuities such as bedding planes, joints, faults, folds, shear zones, seams, gauges, dykes, and fractures influence the magnitude of loosening load. The effect of bedding direction has been pointed out in equations 2-1. A more jointed, folded, faulted, seamy, and fractured rock will generally exert more loosening load than a competent rock which contains less discontinuities. Three or more sets of discontinuities will form distinct rock blocks which need to be supported by the support structure.

Several researchers and practitioners have tried to characterize the rock and thereby assess the rock loads that they are capable of exerting on the support structures. Enumerating all those efforts to determine the rock load is considered unnecessary. Discussed will be only the important ones such as those by Terzaghi, Barton et al., Wickham et al., and Bieniawski's method of determining rock loads. Empirical methods of design are discussed in sections 2-4, 2-5, 2-6, and 2-7. The role of in situ pressures on support loading is discussed in section 2-9.

The larger the opening, the larger is the loosening load on the crown of the

support system (see sec. 2-4). The location of the water table is important in determining the strength characteristics of a rock. An intact rock that is fully saturated by water loses 50 percent of its inherent strength. Instead of reducing the inherent strength, the loading is increased by 100 percent to take care of water submergence.

Methods of excavation such as drill and blast have a tendency to create more loosening of the immediate zone that surrounds the excavation. Drill and blast method creates more loosening and more rock loads than excavation by TBM (tunnel boring machines). The exact quantitative comparison of loosened zone between drill and blast and TBM methods is not possible because the amount of loosening during drill and blast is influenced by several factors such as powder factor, pattern of drilling, sequence of loading, use of delay system, type of explosive used, and characteristics of the rock. The loosened zone during tunnel boring machine excavation depends on the thrust of the machine, the type of cutter used, the rotational speed of the tunnel boring machine, and the characteristics of the rock. However, it could be stated that a drill and blast excavation will disturb a zone about three to six times the disturbed zone of a tunnel boring machine. Of course, the disturbed zone during blasting can be reduced by using controlled blasting techniques. A zone of rock which has received a peak particle velocity of more than 4 inches per second (100 mm/sec) during blasting should be considered to be disturbed.

A full face excavation will create more load on supports than a partial face excavation. A heading and bench method of construction will create lesser load on supports than a full face excavation.

The loads on the side walls of the support structure is usually one-third of the load on the crown and the invert is only subjected to 50 percent of the load on the crown.

The squeezing and swelling loads and their treatment are discussed in chapter 6.

2-4 TERZAGHI'S ROCK LOAD (Terzaghi, 1946)

In 1946, Terzaghi, working on steel set supported railroad tunnels, developed a simplified type of rock load on roofs of tunnels as shown in table 2-2.

Terzaghi's rock load is based on only nine types of rock and the width and height of the opening. It has been used extensively in the USA. However, the rock loading thus provided has a larger factor of safety and results in overly conservative design. For rock conditions 1, 2, and 3 (see table 2-2), the height of opening does not enter into consideration, and in type 9 rock condition (see table 2-2), the loading is arbitrary up to 250 feet of rock

load, irrespective of the value of $(B + H_t)$. This method is very simplistic to use if the definitions of rock types are clearly understood. A generalized definition of rock types is provided in table 2-3.

TABLE 2-2

Rock load H_R in feet of rock on roof of support in tunnel with width B (ft) and height H_t (ft) at depth of more than $1.5 (B + H_t)$.

| Rock condition | Rock load H_R in feet | Remarks |
|--|--|---|
| 1. Hard and intact | zero | Light lining, required only if spalling or popping occurs. |
| 2. Hard stratified or schistose | 0 to 0.5 B | Light support. Load may change erratically from point to point. |
| 3. Massive, moderately jointed | 0 to 0.25 B | Light support. Load may change erratically from point to point. |
| *4. Moderately blocky and seamy | 0.25 B to 0.35 $(B + H_t)$ | No side pressure. |
| *5. Very blocky and seamy | (0.35 to 1.10) $(B + H_t)$ | Little or no side pressure. |
| *6. Completely crushed but chemically intact | 1.10 $(B + H_t)$ | Considerable side pressure. Softening effect of seepage toward bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs. |
| 7. Squeezing rock, moderate depth | (1.10 to 2.10) $(B + H_t)$ | Heavy side pressure, invert struts required. Circular ribs are recommended. |
| 8. Squeezing rock, great depth | (2.10 to 4.50) $(B + H_t)$ | Heavy side pressure, invert struts required. Circular ribs are recommended. |
| 9. Swelling rock | Up to 250 ft. irrespective of value of $(B + H_t)$ | Circular ribs required. In extreme cases use yielding support. |

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 by () can be reduced by 50 percent.

NOTE: Some of the most common rock formations contain layers of shale. In an unweathered state, real shales are no worse than other stratified rocks.

TABLE 2-3

Definitions of rock types.

| Type of rock | Definition |
|---|--|
| 1. Hard and intact | The rock is unweathered. The unconfined compressive strength is equal or above 30,000 lb/in ² (200 MPa). It has long standup time. After the excavation, the rock may have some popping and spalling failures. |
| 2. Hard stratified or schistose | The rock is hard but is layered. The layers are usually widely separated. The rock may or may not have planes of weakness. |
| 3. Massive, moderately jointed | This is a jointed rock. The joints are widely separated. The joints may or may not be cemented. The rock mass between joints is huge. |
| 4. Moderately blocky and seamy | The joints are less separated. Blocks are about 3 feet (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 there is no side pressure exerted. |
| 5. Very blocky and seamy | The joints are pretty close. Sizes of blocks are less than 3 feet (1 m). The interlocking is not as good as type 4 rock. Some side pressure of low magnitude is expected. |
| 6. Completely crushed but chemically intact | The rock is almost like a crusher run aggregate. There is no interlocking. Considerable side pressure is expected. The rock size could be few inches (several mm) to up to 1 foot (30 mm). |
| 7. Squeezing rock, moderate depth | Squeezing is a mechanical process in which the rock advances toward the opening but no volume change occurs. Moderate depth is a relative term and could be up to 150 feet (50 m). |
| 8. Squeezing rock, great depth | The depth may be more than 150 feet (50 m) to thousands of feet. |
| 9. Swelling rock | Swelling is associated with volume change and is due to chemical change of the rock, usually in presence of water. Some shales absorb moisture from air and swell. Rocks containing swelling minerals such as montmorillonite, illite, and others can swell and exert heavy pressures on the rock support. |

The definitions in table 2-3 of the rock types are more qualitative than quantitative. This rock classification is somewhat subjective and is heavily influenced by the experience of the designers. However, the method is very simplistic, does not require elaborate geotechnical investigation, and the supports so designed are usually sturdy. This rock load is usually for long tunnels and basically considers loosening load. If genuine rock pressures exist that are much larger than the loosening loads, then this method will not be applicable.

Once the rock load has been estimated, the support structure can be designed by using methods of analysis shown in chapter 5.

2-5 THE "Q" SYSTEM

In 1974, N. Barton, R. Lien, and J. Lunde introduced a "Q" system and recommended a value of roof load in kg/cm^2 ($1 \text{ kg}/\text{cm}^2 = 14.22 \text{ lb}/\text{in}^2$).

$$P_{\text{roof}} = \left[\frac{2.0}{J_r} \right] Q^{-\frac{1}{3}} \quad 2-3$$

or

$$P_{\text{roof}} = \frac{2.0 J_n^{\frac{1}{2}}}{3 J_r} Q^{-\frac{1}{3}} \quad 2-4$$

where Q = rock quality value and is given by equation 2-5

NOTE: TABLE 2-2 (continued)

However, the term shale is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so-called shale may behave in the tunnel like squeezing or even swelling rock.

If a rock formation consists of a sequence of horizontal layers of sandstone or limestone and of immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the so-called shale and rock is likely to reduce very considerably the capacity of the rock located above the roof to bridge. Hence, in such rock formations, the roof pressure may be as heavy as in a very blocky and seamy rock.

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$$

- 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

Table 2-4 shows the suggested values of J_n and J_r , table 2-5 shows the values of J_a and J_w , and table 2-6 shows the values of SRF.

TABLE 2-4

Descriptions and ratings for the parameters RQD, J_n , and J_r (reproduced by permission of Springer Verlag).

| | | | |
|---|-----------|--|--------------|
| ROCK QUALITY DESIGNATION | | (RQD) | Note: |
| A. Very poor | 0 - 25 | (i) Where RQD is reported or measured as ≤ 10 (including 0) a nominal value of 10 is used to evaluate Q in Eq. 2-5. | |
| B. Poor | 25 - 50 | (ii) RQD intervals of 5; i.e., 100, 95, 90, etc., are sufficiently accurate. | |
| C. Fair | 50 - 75 | | |
| D. Good | 75 - 90 | | |
| E. Excellent | 90 - 100 | | |
| JOINT SET NUMBER | | (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 | Note: (i) For intersections use $(3.0 \times J_n)$. | |
| J. Crushed rock, earthlike | 20 | (ii) For portals use $(2.0 \times J_n)$. | |
| JOINT ROUGHNESS NUMBER | | (J_r) | |
| (a) Rock wall contact and (b) Rock wall contact before 10 cms shear | 4 | | |
| A. Discontinuous joints | 4 | Note: (i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m. | |
| B. Rough or irregular, undulating | 3 | | |
| C. Smooth, undulating | 2 | | |

TABLE 2-4 (continued)

Descriptions and ratings for the parameters RQD, J_n , and J_r (reproduced by permission of Springer Verlag).

| JOINT ROUGHNESS NUMBER | (J_r) | |
|---|---------------|--|
| D. Slickensided, undulating. | 1.5 | (11) $J_r = 0.5$ can be used for planar slickensided joints having lineations, provided the lineations are favorably oriented. |
| 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 (nominal) | |
| J. Sandy, gravelly, or crushed zone thick enough to prevent rock wall contact | 1.0 (nominal) | |

TABLE 2-5

Descriptions and ratings for the parameters J_a and J_w (reproduced by permission of Springer Verlag).

| JOINT ALTERATION NUMBER | (J_a) | ϕ_r (approx.) | |
|--|---------|--------------------|--|
| (a) Rock wall contact | | | |
| A. Tightly healed, hard, nonsoftening, impermeable filling, i.e., quartz or epidote. | 0.75 | (-) | Note: (i) Values of (ϕ_r) are intended as an approximate guide to the mineralogical properties of the alteration products, if present. |
| B. Unaltered joint walls, surface staining only. | 1.0 | (25°-35°) | |
| C. Slightly altered joint walls. Nonsoftening mineral coatings, sandy particles, clay-free disintegrated rock, etc. | 2.0 | (25°-30°) | |
| D. Silty- or sandy-clay coatings, small clay-fraction (nonsoftening). | 3.0 | (20°-25°) | |
| 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.) | 4.0 | (8°-16°) | |

TABLE 2-5 (continued)

Descriptions and ratings for the parameters J_a and J_w (reproduced by permission of Springer Verlag).

| JOINT ALTERATION NUMBER | (J_a) | ϕ_r (approx.) | |
|--|-------------------------------|--|--|
| (b) Rock wall contact before 10 cms shear | | | |
| F. Sandy particles, clay-free disintegrated rock, etc. | 4.0 | (25°-30°) | |
| G. Strongly overconsolidated, nonsoftening clay mineral fillings. (Continuous, < 5 mm in thickness.) | 6.0 | (16°-24°) | |
| H. Medium or low overconsolidation, softening, clay mineral fillings. (Continuous, < 5 mm in thickness.) | 8.0 | (12°-16°) | |
| 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. | 8.0-12.0 | (6°-12°) | |
| (c) No rock wall contact when sheared | | | |
| K. Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition). | 6.0, 8.0 or 8.0, 12.0 | (6°-24°) | |
| N. Zones or bands of silty or sandy clay, small clay fraction (nonsoftening). | 5.0 | | |
| O. Thick, continuous zones or bands of clay (see G, H, J for description of clay condition). | 10.0, 13.0 or 13.0-20.0 | (6°-24°) | |
| JOINT WATER REDUCTION FACTOR | (J_w) | Approx. water pressure (kg/cm ²) | |
| A. Dry excavations or minor inflow, i.e., < 5 l/min. locally. | 1.0 | < 1 | Note: (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. |
| B. Medium inflow or pressure occasional outwash of joint fillings. | 0.66 | 1.0-2.5 | |
| C. Large inflow or high pressure in competent rock with unfilled joints. | 0.5 | 2.5-10.0 | |
| D. Large inflow or high pressure, considerable outwash of joint fillings. | 0.33 | 2.5-10.0 | |

TABLE 2-5 (continued)

Descriptions and ratings for the parameters J_a and J_w (reproduced by permission of Springer Verlag).

| JOINT WATER REDUCTION FACTOR | (J_w) | Approx. water pressure (kg/cm ²) |
|---|-----------|--|
| E. Exceptionally high inflow or water pressure at blasting, decaying with time. | 0.2-0.1 | > 10.0 |
| F. Exceptionally high inflow or water pressure continuing without noticeable decay. | 0.1-0.05 | > 10.0 |

TABLE 2-6

Descriptions and ratings for the parameter SRF (reproduced by permission of Springer Verlag).

| STRESS REDUCTION FACTOR | (SRF) | |
|--|---|--|
| (a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated. | | Note: |
| | | (i) Reduce these values of SRF by 25 - 50% if the relevant shear zones only influence but do not intersect the excavation. |
| 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 \leq 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 \leq 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 | |
| (b) Competent rock, rock stress problems. | | |
| | σ_c/σ_1 σ_t/σ_1 | |
| H. Low stress, near surface. | > 200 > 13 | 2.5 (ii) For strongly anisotropic stress field |

TABLE 2-6 (continued)

Descriptions and ratings for the parameter SRF (reproduced by permission of Springer Verlag).

| STRESS REDUCTION FACTOR | σ_c/σ_1 | σ_t/σ_1 | (SRF) | |
|---|---------------------|---------------------|---------|--|
| J. Medium stress. | >200-10 | 13-0.66 | 1.0 | (if measured): |
| K. High stress, very tight structure. (Usually favorable to stability, may be unfavorable to wall stability.) | 10-5 | 0.66-0.33 | 0.5-2.0 | when $5 < \sigma_1/\sigma_3 < 10$, reduce σ_c and σ_t to $0.8 \sigma_c$ and $0.8 \sigma_t$; when $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6 \sigma_c$ and $0.6 \sigma_t$ where: σ_c = unconfined compression strength, σ_t = tensile strength (point load), σ_1 and σ_3 = major and minor principal stresses. |
| L. Mild rock burst (massive rock). | 5-2.5 | 0.33-0.16 | 5-10 | |
| M. Heavy rock burst (massive rock). | < 2.5 | < 0.16 | 10-20 | |
| (c) Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressures. | | | | |
| N. Mild squeezing rock pressure. | | | 5-10 | (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). |
| O. Heavy squeezing rock pressure. | | | 10-20 | |
| (d) Swelling rock; chemical swelling activity depending on presence of water. | | | | |
| P. Mild swelling rock pressure. | | | 5-10 | |
| R. Heavy swelling rock pressure. | | | 10-15 | |

Once the value of P_{roof} is found, the structural support system can be designed by use of any of the structural analysis methods shown in chapter 5. Alternatively, Barton et al. suggest an empirical design method.

2-5.1 Empirical design method, the "Q" system

Once the value of "Q" has been determined, another parameter called "excavation support ratio" (ESR) is determined by the use of table 2-7. With this known value of ESR, determine the "equivalent dimension" of the opening as follows:

$$\text{Equivalent dimension} = \frac{\text{Span, diameter or height of opening}}{\text{ESR}}$$

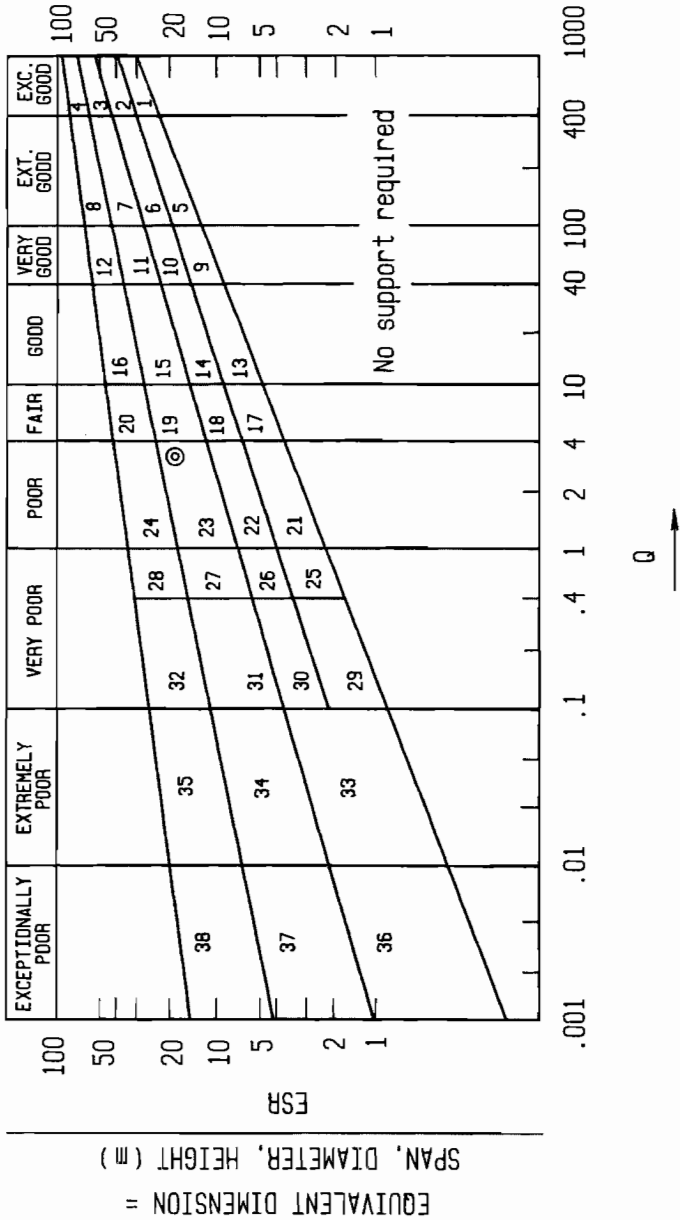
With the known values of equivalent dimension, enter figure 2-2 and read the numerical value on the figure 2-2 against the calculated value of "Q." This numerical value is the support type number (see table 2-8) which Barton et al. suggest for supporting the opening.

For example, if $Q = 50$ and Equivalent Dimension = 30, on figure 2-2 the number read is 11 which means support category 11 is to be used. According to table 2-8, support category 11 corresponds to a systematic bolting at a spacing of 6 feet to 9 feet (2 m to 3 m).

TABLE 2-7

The excavation support ratio (ESR) approximate to a variety of underground excavations (reproduced by permission of Springer Verlag).

| Type of excavation | ESR |
|---|-----|
| A. Temporary mine openings, etc. | 3-5 |
| B. Vertical shafts: (i) circular section | 2.5 |
| (ii) rectangular/square section | 2.0 |
| C. Permanent mine openings, water tunnels for hydropower (exclude high pressure penstocks), pilot tunnels, drifts, and headings for large excavations, etc. | 1.6 |
| D. Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels, etc. (cylindrical caverns?) | 1.3 |
| E. Power stations, major road and railway tunnels, civil defense chambers, portals, intersections, etc. | 1.0 |
| F. Underground nuclear power stations, railway stations, sports and public facilities, factories, etc. | 0.8 |



$$\text{Calculated Rock Mass Quality } Q = \left(\frac{RQD}{J_n}\right) \times \left(\frac{J_r}{J_a}\right) \times \left(\frac{J_w}{SRF}\right)$$

Fig. 2-2. Support System (Reproduced by permission of Springer Verlag).

TABLE 2-8

Support category (reproduced by permission of Springer Verlag).

| Support category | Q | Conditional factors $\frac{RQ^2/J_n}{J_r/J_n}$ | SPAN/ ESR (m) | P kg/cm ² (approx.) | SPAN/ ESR (m) | Type of support |
|------------------|----------|---|------------------|--------------------------------------|------------------|---|
| 1 | 1000-400 | - | - | <0.01 | 20-40 | sb (utg) |
| 2 | 1000-400 | - | - | <0.01 | 30-60 | sb (utg) |
| 3 | 1000-400 | - | - | <0.01 | 46-80 | sb (utg) |
| 4 | 1000-400 | - | - | <0.01 | 65-100 | sb (utg) |
| 5 | 400-100 | - | - | 0.05 | 12-30 | sb (utg) |
| 6 | 400-100 | - | - | 0.05 | 19-45 | sb (utg) |
| 7 | 400-100 | - | - | 0.05 | 30-65 | sb (utg) |
| 8 | 400-100 | - | - | 0.05 | 48-88 | sb (utg) |
| 9 | 100-40 | >20 ≤20 | - | 0.25 | 8.5-19 | sb (utg) B (utg) 2.5-3 m |
| 10 | 100-40 | >30 ≤30 | - | 0.25 | 14-30 | B (utg) 2-3 m B (utg) 1.5-2 m + c/m |
| 11 | 100-40 | >30 ≤30 | - | 0.25 | 23-48 | B (tg) 2-3 m B (tg) 1.5-2 m + c/m |
| 12 | 100-40 | >30 ≤30 | - | 0.25 | 40-72 | B (tg) 2-3 m B (tg) 1.5-2 m + c/m |

TABLE 2-8 (continued)
Support category (reproduced by permission of Springer Verlag).

| Support category | Q | Conditional factors $\frac{R007J_n}{J_r/J_n}$ | SPAN/ ESR (m) | P kg/cm ² (approx.) | SPAN/ ESR (m) | Type of support |
|------------------|-------|--|------------------|--------------------------------------|------------------|-------------------|
| 13 | 40-10 | >1.5 | - | 0.5 | 5-14 | sb (utg) |
| | | ≥1.5 | - | | | B (utg) 1.5-2 m |
| | | <1.5 | - | | | B (utg) 1.5-2 m |
| 14 | 40-10 | <1.5 | - | 0.5 | 9-23 | B (utg) 1.5-2 m |
| | | - | >15 | | | + S 2-3 cm |
| | | >10 | - | | | B (tg) 1.5-2 m |
| | | <10 | >15 | | | + c1m |
| 15 | 40-10 | - | <15 | 0.5 | 15-40 | B (tg) 1.5-2 m |
| | | >10 | - | | | + c1m |
| | | <10 | - | | | B (tg) 1.5-2 m |
| 16 | 40-10 | >15 | - | 0.5 | 30-65 | + S (mr) 5-10 cm |
| | | <15 | - | | | B (tg) 1.5-2 m |
| | | - | - | | | + c1m |
| | | | | | | B (tg) 1.5-2 m |
| | | | | | | + S (mr) 10-15 cm |

TABLE 2-8 (continued)
Support category (reproduced by permission of Springer Verlag).

| Support category | Q | Conditional factors $\frac{RQD}{J_n}$ | $\frac{J_r}{J_n}$ | SPAN/ ESR (m) | P kg/cm ² (approx.) | SPAN/ ESR (m) | Type of support |
|------------------|------|--|-------------------|------------------|--------------------------------------|------------------|-------------------|
| 17 | 10-4 | >30 | - | - | 1.0 | 3.5-9 | sb (utg) |
| | | >10, <30 | - | - | | | B (utg) 1-1.5 m |
| | | <10 | - | >6 | | | B (utg) 1-1.5 m |
| | | <10 | - | <6 | | | + S 2-3 cm |
| 18 | 10-4 | >5 | - | >10 | 1.0 | 7-15 | S 2-3 cm |
| | | >5 | - | <10 | | | B (tg) 1-1.5 m |
| | | <5 | - | >10 | | | + c/m |
| | | <5 | - | <10 | | | B (utg) 1-1.5 m |
| | | <5 | - | <10 | | | + c/m |
| | | <5 | - | <10 | | | B (tg) 1-1.5 m |
| 19 | 10-4 | - | - | >20 | 1.0 | 12-29 | + S 2-3 cm |
| | | - | - | <20 | | | B (tg) 1-2 m |
| | | - | - | >35 | | | + S (nr) 10-15 cm |
| | | - | - | <35 | | | B (tg) 1-1.5 m |
| 20 | 10-4 | - | - | >35 | 1.0 | 24-52 | + S (nr) 5-10 cm |
| | | - | - | <35 | | | B (tg) 1-2 m |

TABLE 2-8 (continued)
Support category (reproduced by permission of Springer Verlag).

| Support category | Q | Conditional factors $\frac{RQD}{\gamma_n}$ | $\frac{J_r}{\gamma_n}$ | SPAN/ ESR (m) | P kg/cm ² (approx.) | SPAN/ ESR (m) | Type of support |
|------------------|---------|---|----------------------------------|------------------|--------------------------------------|------------------|--|
| 21 | 4-1 | ≥ 12.5 | ≤ 0.75 | - | 1.5 | 2.1-6.5 | B (utg) 1 m + S 2-3 cm S 2.5-5 cm B (utg) 1 m |
| 22 | 4-1 | $\geq 10, < 30$ | > 1.0 > 1.0 ≤ 1.0 | - | 1.5 | 4.5-11.5 | B (utg) 1 m + c/m S 2.5-7.5 cm B (utg) 1 m + S (mr) 2.5-5 cm B (utg) 1 m |
| 23 | 4-1 | ≥ 30 | - | ≥ 15 | 1.5 | 8-24 | B (tg) 1-1.5 m + S (mr) 10-15 cm B (utg) 1-1.5 m + S (mr) 5-10 m |
| 24 | 4-1 | - | - | ≥ 30 | 1.5 | 18-46 | B (tg) 1-1.5 m + S (mr) 15-30 cm B (tg) 1-1.5 m + S (mr) 10-15 cm |
| 25 | 1.0-0.4 | > 10 < 10 | > 0.5 > 0.5 < 0.5 | - | 2.25 | 1.5-4.2 | B (utg) 1 m + mr or c/m B (utg) 1 m + S (mr) 5 cm B (tg) 1 m + S (mr) 5 cm |

TABLE 2-8 (continued)
Support category (reproduced by permission of Springer Verlag).

| Support category | Q | Conditional factors RQD/J_n | J_r/J_n | SPAN/ ESR (m) | P kg/cm ² (approx.) | SPAN/ ESR (m) | Type of support |
|------------------|---------|----------------------------------|-----------|------------------|--------------------------------------|------------------|---------------------------|
| 26 | 1.0-0.4 | - | - | - | 2.25 | 3.2-7.5 | B (tg) 1 m |
| | | | | | | | + S (mr) 5-7.5 cm |
| 27 | 1.0-0.4 | - | - | - | 2.25 | 6-18 | B (utg) 1 m + S 2.5-5 cm |
| | | | | | | | B (tg) 1 m |
| | | | | | | | + S (mr) 7.5-10 cm |
| | | | | | | | B (utg) 1 m |
| | | | | | | | + S (mr) 5-7.5 cm |
| | | | | | | | CCA 20-40 cm |
| 28 | 1.0-0.4 | - | - | - | 2.25 | 15-38 | + B (tg) 1 m |
| | | | | | | | S (mr) 10-20 cm |
| | | | | | | | + B (tg) 1 m |
| 29 | 0.4-0.1 | - | - | - | 3.0 | 1.0-3.1 | B (tg) 1 m |
| | | | | | | | + S (mr) 30-40 cm |
| | | | | | | | B (tg) 1 m |
| | | | | | | | + S (mr) 20-30 cm |
| | | | | | | | B (gt) 1 m |
| | | | | | | | + S (mr) 15-20 cm |
| 29 | 0.4-0.1 | - | - | - | 3.0 | 1.0-3.1 | CCA (sr) 30-100 cm |
| | | | | | | | + B (tg) 1 m |
| | | | | | | | B (utg) 1 m + S 2-3 cm |
| 29 | 0.4-0.1 | - | - | - | 3.0 | 1.0-3.1 | B (utg) 1 m + S (mr) 5 cm |
| | | | | | | | B (tg) 1 m + S (mr) 5 cm |
| | | | | | | | B (tg) 1 m + S (mr) 5 cm |

TABLE 2-8 (continued)
Support category (reproduced by permission of Springer Verlag).

| Support category | Q | Conditional factors $\frac{RQD/J_n}{J_r/J_n}$ | SPAN/ ESR (m) | P kg/cm ² (approx.) | SPAN/ ESR (m) | Type of support |
|------------------|----------|--|------------------|--------------------------------------|------------------|---|
| 30 | 0.4-0.1 | ≥ 5 | - | 3.0 | 2.2-6 | B (tg) 1 m + S 2.5-5 cm S (mr) 5-7.5 cm |
| | | < 5 | - | | | |
| | | - | - | | | |
| 31 | 0.4-0.1 | > 4 | - | 3.0 | 4-14.5 | B (tg) 1 m + S (mr) 5-12.5 cm S (mr) 7.5-25 cm CCA 20-40 cm + B (tg) 1 m CCA (sr) 30-50 cm + B (tg) 1 m |
| | | $\leq 4, > 1.4$ | - | | | |
| | | < 1.5 | - | | | |
| 32 | 0.4-0.1 | - | ≥ 20 | 3.0 | 11-34 | B (tg) 1 m + S (mr) 40-60 cm B (tg) 1 m + S (mr) 20-40 cm CCA (sr) 40-120 cm + B (tg) 1 m |
| | | - | < 20 | | | |
| | | - | - | | | |
| 33 | 0.1-0.01 | ≥ 2 | - | 6 | 1.0-3.9 | B (tg) 1 m + S (mr) 2.5-5 cm S (mr) 5-10 cm S (mr) 7.5-15 cm |
| | | < 2 | - | | | |
| | | - | - | | | |

TABLE 2-8 (continued)
Support category (reproduced by permission of Springer Verlag).

| Support category | Q | Conditional factors RQD/J_n | J_r/J_n | SPAN/ ESR (m) | P kg/cm ² (approx.) | SPAN/ ESR (m) | Type of support |
|------------------|------------|----------------------------------|-------------|------------------|--------------------------------------|------------------|--|
| 34 | 0.1-0.01 | ≥ 2 | ≥ 0.25 | - | 6 | 2.0-11 | B (tg) 1 m + S (mr) 5-7.5 cm |
| | | < 2 | > 0.25 | - | | | S (mr) 7.5-15 cm |
| | | - | < 0.25 | - | | | S (mr) 15-25 cm |
| | | - | - | - | | | CCA (sr) 20-60 cm + B (tg) 1 m |
| 35 | 0.1-0.01 | - | - | > 15 | 6 | 6.5-28 | B (tg) 1 m + S (mr) 30-100 cm |
| | | - | - | ≥ 15 | | | CCA (sr) 60-200 cm + B (tg) 1 m |
| | | - | - | < 15 | | | B (tg) 1 m |
| | | - | - | < 15 | | | + S (mr) 20-75 cm |
| | | - | - | < 15 | | | CCA (sr) 40-150 cm + B (tg) 1 m |
| 36 | 0.01-0.001 | - | - | - | 12 | 1.0-2.0 | S (mr) 10-20 cm S (mr) 10-20 cm + B (tg) 0.5-1 m |
| | | - | - | - | | | |
| | | - | - | - | | | |
| 37 | 0.01-0.001 | - | - | - | 12 | 1.0-6.5 | S (mr) 20-60 cm S (mr) 20-60 cm + B (tg) 0.5-1 m |
| | | - | - | - | | | |

TABLE 2-8 (continued)
Support category (reproduced by permission of Springer Verlag).

| Support category | Q | Conditional factors $\frac{RQD}{J_n}$ | $\frac{J_r}{J_n}$ | SPAN/ ESR (m) | P kg/cm ² (approx.) | SPAN/ ESR (m) | Type of support |
|------------------|------------|--|-------------------|------------------|--------------------------------------|------------------|--|
| 38 | 0.01-0.001 | - | - | >10 =10 | 12 | 4.0-20 | CCA (sr) 100-300 cm CCA (sr) 100-300 cm + B (tg) 1 m S (mr) 70-200 cm S (mr) 70-200 cm + B (tg) 1 m |

Note: The type of support to be used in categories 1 to 8 will depend on the blasting technique. Smooth wall blasting and thorough barring-down may remove the need for support. Rough-wall blasting may result in the need for single applications of shotcrete, especially where the excavation height is > 25 m.

Key to support tables:

- sb = spot bolting
B = systematic bolting
(utg) = untensioned, grouted
(tg) = tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rockmasses)
S = shotcrete
- (mr) = mesh reinforced
c/m = chain link mesh
CCA = cast concrete arch
(sr) = steel reinforced
Bolt spacings are given in meters (m).
Shotcrete, or cast concrete arch thickness is given in centimeters (cm).

2.6 ROCK STRUCTURE RATING

Wickham et al. in 1972 came up with the relationship shown in equations 2-6 and 2-7:

$$W_r = \frac{D}{302} \left[\frac{6,000}{RSR + 8} \right] - 70 \quad 2-6$$

$$W_r = \frac{D}{302} (RR) \quad 2-7$$

where RR = rib ratio

$$= \left[\frac{6,000}{RSR + 8} \right] - 70$$

W_r = rock load in kips/ft²

D = diameter of opening in feet

RSR = rock structure rating

The rock structure rating is defined by Wickham et al. as the sum of three parameters "A," "B," and "C." The values of A, B, and C are shown in tables 2-9, 2-10, and 2-11.

Once the value of W_r is defined, the support system can be designed by using methods described in chapter 5.

Parameter A (table 2-9) is a general appraisal of rock structures through which the tunnel is to be driven. Geological information needed to define the limits of measure and describe the structure is available in the preconstruction period. It is usually presented in terms compatible to all disciplines, such as "massive granite" or "intensely folded serpentine" formation. The assigned weighted value for parameter A in the first instance would be 30; in the second, 9.

TABLE 2-9

Rock structure rating - parameter "A", general area geology (reproduced by permission of Society of Mining Engineers, Inc.).

| Basic rock type | Massive | Geologic Structure | | |
|-----------------|---------|----------------------------|------------------------------|-----------------------------|
| | | Slightly faulted or folded | Moderately faulted or folded | Intensely faulted or folded |
| Igneous | 30 | 26 | 15 | 10 |
| Sedimentary | 24 | 20 | 12 | 8 |
| Metamorphic | 27 | 22 | 14 | 9 |

Parameter B (table 2-10) relates to the joint pattern (strike, dip, and joint spacing) and the direction of drive. Most surface geology surveys or maps give an indication of the strike and dip of various formations. Consequently, approximations as to limits of measure for these two factors can be made. Corresponding direction of drive is determined from project planning. There are usually several sources of information that can be used in determining the anticipated average joint spacing of the rock structure. Geological terms such as "closely jointed" or "blocky," driller's logs, core analysis, or RQD indices are examples. Geology reports usually give some description of anticipated joint spacing. Defining this factor is difficult but it is felt that a reasonable approximation can be made by considering all available information. For purposes of the RSR method of evaluation, five numerical limits of measure are given for joint spacing. The respective bracketed words in the left-hand column of table 2-10 are used to show intended correlation or equivalency between the given numerical limits and common geological terminology. The value to be assigned to parameter B can be obtained from the table by considering appropriate limits of measure determined for joint spacing with respect to the strike and dip of the formation and direction of drive.

TABLE 2-10

Rock structure rating - parameter "B," joint pattern - direction of drive (reproduced by permission of Society of Mining Engineers, Inc.).

| Average joint spacing (feet) | <u>Strike perpendicular to axis</u> | | | | | <u>Strike parallel to axis</u> | | |
|---------------------------------|-------------------------------------|----------|-----------------|----------|--------------------|--------------------------------|----------|----------|
| | Direction of drive | | | | | Both | | |
| | <u>Both</u> | | <u>With dip</u> | | <u>Against dip</u> | | | |
| | Dip of prominent joints* | | | | | | | |
| | <u>1</u> | <u>2</u> | <u>3</u> | <u>2</u> | <u>3</u> | <u>1</u> | <u>2</u> | <u>3</u> |
| <0.5 (closely jointed) | 14 | 17 | 20 | 16 | 18 | 14 | 15 | 12 |
| 0.5-1.0 (moderately jointed) | 24 | 26 | 30 | 20 | 24 | 24 | 24 | 20 |
| 1.0-2.0 (moderate to blocky) | 32 | 34 | 38 | 27 | 30 | 32 | 30 | 25 |
| 2.0-4.0 (blocky to massive) | 40 | 42 | 44 | 36 | 39 | 40 | 37 | 30 |
| >4.0 (massive) | 45 | 48 | 50 | 42 | 45 | 45 | 42 | 36 |

*1 = <20°, 2 = 20° to 50°, 3 = 50° to 90°

Parameter C (table 2-11) is a general evaluation of ground water inflow effect on support requirements. It takes into consideration the following: (1) the overall quality of the rock structure as indicated by the numerical sum of values assigned to parameters A and B, (2) the condition of joint surfaces, and (3) the anticipated amount of inflow. Establishing limits of measure or estimating possible occurrence of the last two factors is normally left to the discretion of the contractor. Data pertaining to pump tests, local wells, ground water levels, surface hydrology, topography, and rainfall should be considered in conjunction with the anticipated geological formation in estimating ground water inflows. Condition of joint surfaces would be appraised from surface or historical geology, driller's logs, or inspection of core samples. The RSR method allows for three types or conditions of joint surfaces which are described as: (1) tight or cemented, (2) slightly weathered and, (3) severely weathered or opened; and four quantitative estimates of water inflow. The value assigned to parameter C is obtained from the table by using the limits of measure determined for the different factors.

TABLE 2-11

Rock structure rating - parameter "C" ground water, joint condition (reproduced by permission of Society of Mining Engineers, Inc.).

| Anticipated water inflow (gpm/1,000 ft) | Sum of parameters A + B | | | | | |
|--|-------------------------|----|----|------------------|----|----|
| | 20-45 | | | 46-80 | | |
| | Joint condition* | | | Joint condition* | | |
| | 1 | 2 | 3 | 1 | 2 | 3 |
| None | 18 | 15 | 10 | 20 | 18 | 14 |
| Slight (200 gpm) | 17 | 12 | 7 | 19 | 15 | 10 |
| Moderate (200-1,000 gpm) | 12 | 9 | 6 | 18 | 12 | 8 |
| Heavy (1,000 gpm) | 8 | 6 | 5 | 14 | 10 | 6 |

*1 = tight or cemented, 2 = slightly weathered, 3 = severely weathered or open.

The RSR value of the particular geological section under consideration is the numerical sum of parameters A, B, and C. These values will range from 25 to 100, and reflect quality of the rock structure regardless of size of tunnel opening or method of excavation. Each distinct formation penetrated by the tunnel would be separately analyzed with respect to RSR values.

2-6.1 Empirical design, rock structure rating

Wickham et al. (1972) suggest an empirical method of design based on the value of RSR. Table 2-12 provides their suggested recommendations for steel sets against openings of 16 and 20 feet (5 and 6 m).

TABLE 2-12

Rib spacing (ft) based on RSR values and tunnel diameter (reproduced by permission of Society of Mining Engineers, Inc.).

| RSR value | Tunnel diameter | | | | | |
|--------------|-----------------|------|------------------------------|---------|-------|-------|
| | 16 feet | | | 20 feet | | |
| | 6H15.5 | 6H25 | Steel rib - spacing 8WF31 | 6H20 | 8WF31 | 8WF48 |
| 27 | 1.4 | 2.2 | 3.2 | 1.2 | 2.1 | 3.3 |
| 30 | 1.6 | 2.6 | 3.7 | 1.4 | 2.4 | 3.8 |
| 35 | 2.0 | 3.2 | 4.6 | 1.7 | 3.1 | 4.8 |
| 40 | 2.6 | 4.1 | 5.9 | 2.2 | 3.9 | 6.1 |
| 45 | 3.3 | 5.2 | 7.5 | 2.8 | 5.0 | 7.8 |
| 50 | 4.3 | 6.8 | | 3.6 | 6.5 | |
| 55 | 5.7 | | | 4.8 | | |
| 60 | 7.9 | | | 6.7 | | |

According to them, the spacing for systematic bolting based on 1-inch (25-mm) bolt diameter with a working load of 24,000 pounds (11,000 kg) is given by equation 2-8.

$$\text{Spacing of pattern bolting} = \sqrt{\frac{24}{W_r}} \quad 2-8$$

(For W_r see equations 2-6 and 2-7.)

In case shotcrete is used to support the opening, the thickness of shotcrete layer is given by equation 2-9:

$$t = 1 + \frac{W_r}{1.25} \quad 2-9$$

where t = thickness of shotcrete layer.

Wickham et al. (1972) realized the beneficial effect of the use of the tunnel boring machine in reducing the extent of loosening zone surrounding an excavation. They suggest to increase the RSR rating for a TBM excavated opening by the use of figure 2-3. An increased RSR will render decreased value of W_r and therefore will require less support.

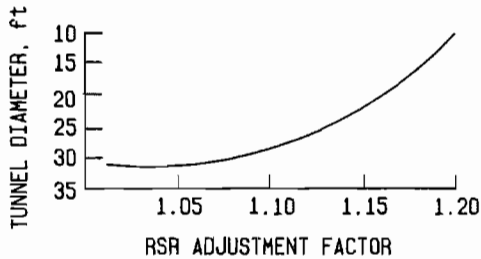


Fig. 2-3. RSR Adjustment for TBM Operation (reproduced by permission of Society of Mining Engineers, Inc.).

2-7 EMPIRICAL DESIGN, ROCK MECHANICS RATING

Bieniawski (1974) developed a rock mechanics rating system based on six parameters: (1) the uniaxial compressive strength of intact rock, (2) rock quality designation, (3) spacing, (4) orientation, (5) condition of discontinuities, and (6) ground water conditions. He assigned numerical rating values to all these parameters. The rock mechanics rating is the summation of the individual ratings of the six parameters. Based on the value of the rock mechanics rating, Bieniawski divides the whole universe of rock mass into five classes, I through V as shown in table 2-13.B, and then assigns standup time to each class as shown in table 2-13.C. Table 2-14 shows the effects of orientation. Based on rock mass class, Bieniawski then recommends a support system shown in table 2-15.

In a later publication, Rutledge (1978) shows a correlation of Bieniawski's "RMR" to Wickham et al. "RSR" and Barton et al. "Q" system as shown in the following equations.

$$\text{RMR} = 9 \log_e Q + 44$$

$$\text{RSR} = 0.77 \text{ RMR} + 12.4$$

$$\text{RSR} = 13.3 \log_e Q + 46.5$$

In 1979 Bieniawski developed a correlation of his RMR with the in-situ modulus of rock deformation as:

$$E_m = 2 \text{ RMR} - 100 \quad \text{For values of RMR greater than 50.}$$

TABLE 2-13

Geomechanics classification of rock masses (reproduced by permission of Z.T. Bieniawski).

A. Classification parameters and their ratings

| | | | | |
|--|--|---|---|---|
| 1 <u>Uniaxial compressive strength of intact rock</u> (Rating) | | | | |
| <u>>200 MPa</u> (10) | <u>100 - 200 MPa</u> (5) | <u>50 - 100 MPa</u> (2) | <u>25 - 50 MPa</u> (1) | <u><25 MPa</u> (0) |
| 2 <u>Drill core quality RQD</u> (Rating) | | | | |
| <u>90 - 100%</u> (20) | <u>75 - 90%</u> (17) | <u>50 - 75%</u> (14) | <u>25 - 50%</u> (8) | <u><25% or highly weathered</u> (3) |
| 3 <u>Spacing of joints</u> (Rating) | | | | |
| <u>>3 m</u> (30) | <u>1 - 3 m</u> (25) | <u>0.3 - 1 m</u> (20) | <u>50 - 300 mm</u> (10) | <u><50 mm</u> (5) |
| 4 <u>Strike and dip orientations of joints</u> (Rating) | | | | |
| <u>Very favorable</u> (15) | <u>Favorable</u> (13) | <u>Fair</u> (10) | <u>Unfavorable</u> (6) | <u>Very unfavorable</u> (3) |
| 5 <u>Condition of joints</u> (Rating) | | | | |
| <u>Very tight: separation <0.1 mm. not continuous.</u> (15) | <u>Tight: <1 mm and continuous. No gouge.</u> (10) | <u>Open: 1 - 5 mm Continuous Gouge < 5 mm</u> (5) | <u>Open > 5 mm Continuous Gouge > 5 mm</u> (0) | |
| 6 <u>Ground water inflow (per 10 m of tunnel length)</u> (Rating) | | | | |
| <u>None</u> (10) | <u><25 liters/min</u> (8) | <u>25 - 125 liters/min</u> (5) | <u>>125 liters/min</u> (2) | |

TABLE 2-13 (continued)

Geomechanics classification of rock masses (reproduced by permission of Z.T. Bieniawski).

| B. Rock mass classes and their ratings | | | | | |
|--|----------------|-----------|-----------|-----------|----------------|
| Class No.: | I | II | III | IV | V |
| Description of class: | Very good rock | Good rock | Fair rock | Poor rock | Very poor rock |
| Total rating: | 100 - 90 | 90 - 70 | 70 - 50 | 50 - 25 | <25 |

C. Meaning of rock mass classes in tunneling

| Class No.: | I | II | III | IV | V |
|-----------------------|----------|----------|--------|---------|------------|
| Unsupported span: | 5 m | 4 m | 3 m | 1.5 m | 0.5 m |
| Average standup time: | 10 years | 6 months | 1 week | 5 hours | 10 minutes |

TABLE 2-14

The effect of joint strike and dip orientations in tunneling (reproduced by permission of Z.T. Bieniawski).

| Strike perpendicular to tunnel axis | | | | Strike parallel to tunnel axis | |
|-------------------------------------|-----------|-------------------|-------------|--------------------------------|----------|
| Drive with dip | | Drive against dip | | | |
| Dip | Dip | Dip | Dip | Dip | Dip |
| 45 - 90° | 20 - 45° | 45 - 90° | 20 - 45° | 45 - 90° | 20 - 45° |
| Very favorable | Favorable | Fair | Unfavorable | Very unfavorable | Fair |

Dip 0° to 20°: Unfavorable, irrespective of strike

TABLE 2-15

Guide for selection of primary support in 5 m to 12 m diameter tunnels at shallow depth (reproduced by permission of Z.T. Bieniawski).

| Rock mass class | <u>Alternative support systems for drilling and blasting construction</u> | | |
|-----------------|--|---|--|
| | Mainly rockbolts* | Mainly shotcrete | Mainly steel ribs |
| I | GENERALLY NO SUPPORT IS REQUIRED | | |
| II | Rockbolts spaced 1.5 to 2.0 m plus occasional wire mesh in crown. | Shotcrete 50 mm in crown. | Uneconomic. |
| III | Rockbolts spaced 1.0 to 1.5 m plus wire mesh and 30 mm shotcrete in crown where required. | Shotcrete 100 mm in crown and 50 mm in sides plus occasional wire mesh and rockbolts where required. | Light sets spaced 1.5 to 2 m. |
| IV | Rockbolts spaced 0.5 to 1.0 m plus wire mesh and 30 to 50 mm shotcrete in crown and sides. | Shotcrete 150 mm in crown and 100 mm in sides plus wire mesh and rockbolts, 3 m long spaced 1.5 m. | Medium sets spaced 0.7 to 1.5 m plus 50 mm shotcrete in crown. |
| V | Not recommended. | Shotcrete 200 mm in crown and 150 mm in sides plus wire mesh, rockbolts and light steel sets. Close invert. | Heavy sets spaced 0.7 m with lagging. Shotcrete 75 mm as soon as possible. |

*Resin bonded bolts 20 mm diameter, length 1/2 tunnel width.

2-8 EVALUATION OF EMPIRICAL DESIGN APPROACHES

The empirical methods of design stated in sections 2-4, 2-5, 2-6, and 2-7 are basically for the design of initial support systems because they deal only with loosening loads. In cases where loosening loads exceed the genuine rock pressures, squeezing or swelling loads, the empirical design methods can adequately serve the purpose; otherwise they fall short.

Effective use of empirical design approaches requires evaluations of both the creator's and user's judgments. In order to judge the applicability of the use of empirical design methods, it is important to realize the sampling conditions and biases of the creator in developing the particular empirical methods. Similarly, the user's judgment in determining the applicable geological parameters into the empirical models is crucial in the validity of the design.

Naturally, all facets of design factors are not all inclusive in the empirical methods. For example, the role of construction progress cannot be modeled in the existing empirical models. Similarly, the actual excavation conditions may be quite different to the prototype model and then empirical methods could not be that applicable. The empirical methods usually provide overdesigned structures but the models are simple to use and do not require elaborate subsurface evaluation. It is advantageous to use more than one empirical method to arrive at a design recommendation.

2-9 RATIONAL METHODS OF DESIGN

Rational methods of design are based on theories of elasticity and plasticity and are approached through the concepts of strain and stress. They include the consideration of in situ stresses and the loss of inherent strength of rock due to removal of confining pressure because of excavation. In an existing geological environment, before excavation, the rock element is in a triaxial state of stress. An excavation reduces the triaxial state to either uniaxial or biaxial states, which considerably reduces the carrying capacities of the rock elements. Figure 2-4(a) shows rock strength reductions due to reduction in confining pressure. Reduced strength of rock elements is seen on figure 2-4(b) and effect of rock moisture content is seen on figure 2-4(c).

2-9.1 In situ stresses

The magnitude and directions of in situ stresses can be estimated by conducting in situ measurements. In absence of in situ testing, the gravitational in situ stress due to the weight of overlying strata can be derived by the application of theory of elasticity. Using a rectangular coordinate system, the principal stresses σ_{zz} , σ_{xx} , and σ_{yy} on a rock element in a homogeneous rock medium are shown on figure 2-5.

The principal gravitational major stress $\sigma_{zz} = \gamma Z$ where γ is the unit weight of rock and Z is depth of overlying strata.

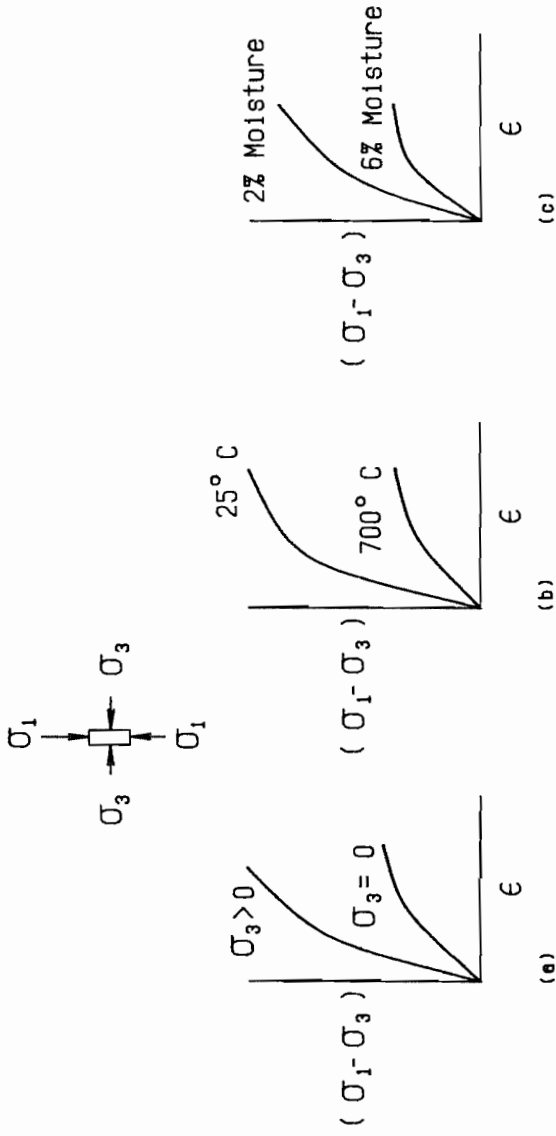
Strain in x direction:

$$\epsilon_{xx} = \frac{1}{E} [\sigma_{xx} - \nu(\sigma_{yy} + \sigma_{zz})] \quad 2-10(a)$$

where ν is Poisson's ratio.

Strain in y direction:

$$\epsilon_{yy} = \frac{1}{E} [\sigma_{yy} - \nu(\sigma_{zz} + \sigma_{xx})] \quad 2-10(b)$$



(a) Strength Increases With Increasing Confining Pressure
 (b) Strength Loss Due to Higher Ambient Temperatures
 (c) Strength Loss Due to Increased Moisture Content

Fig. 2-4. Rock Strength Curves.

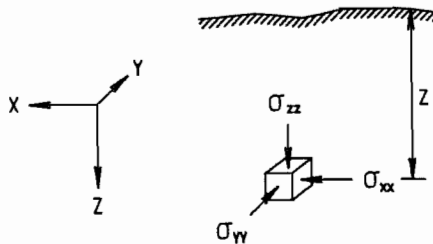


Fig. 2-5. Principal Stresses on a Rock Element.

Due to confinement, the rock element is not free to displace either in the "x" or "y" direction and because only gravitational forces are assumed to act, one can conclude:

$$\epsilon_{xx} = \epsilon_{yy} = 0 \quad \text{and} \quad \sigma_{xx} = \sigma_{yy}$$

Substituting these values in either of equations 2-10 one can find

$$\sigma_{xx} - \nu\sigma_{xx} - \nu\sigma_{zz} = 0 \quad \text{or} \quad \sigma_{yy} - \nu\sigma_{yy} - \nu\sigma_{zz} = 0$$

from which

$$\sigma_{xx} = \sigma_{yy} = \frac{\nu}{1 - \nu} \sigma_{zz} = k\sigma_{zz}$$

$$\text{where } k = \frac{\nu}{1 - \nu}$$

2-11

If measured in situ stresses differ from these calculated values, as shown in equation 2-11, one will have to conclude that the in situ stresses have component stresses from other than gravitational sources such as tectonic stresses (active or remanent), residual stresses, or induced stresses (from manmade or geological structures).

Hoek and Brown (1980) report to have observed "k" values to range from as low as 0.48 to as high as 5.56.

2-9.2 Stresses and strains

A rock mass subjected to vertical pressure " σ_v " and horizontal pressure " σ_h ," both applied at boundaries at great distances from the center of the circular opening of radius "R," is shown in figure 2-6.

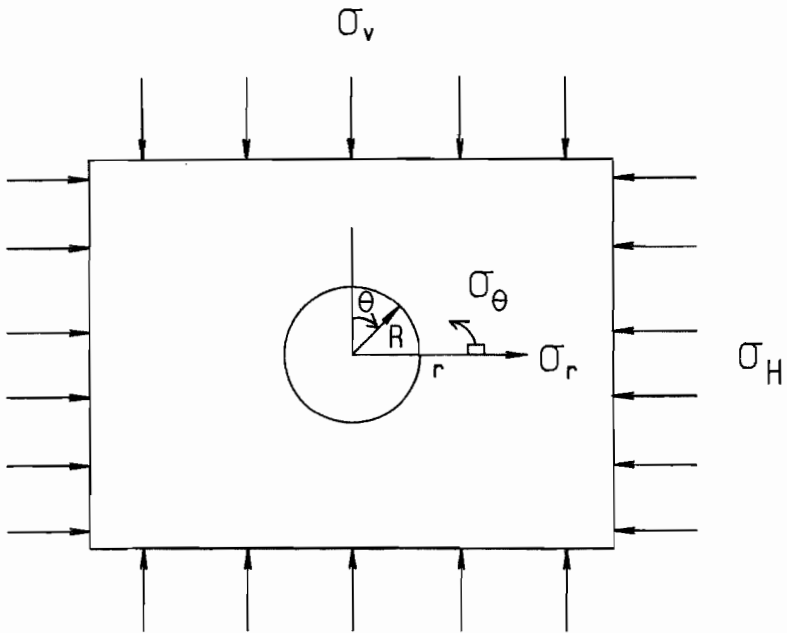


Fig. 2-6. Rock Mass With a Circular Opening.

The stresses and strains due to creation of an opening in such a homogeneous and isotropic rock mass under biaxial pressure loading and plane strain conditions can be found out by invoking Airy's stress function (Volterra and Gains, 1971) and resorting to Kirsch's solution (1898) as shown in equations 2-12.

The radial stress at point "P"

$$P_{\sigma_R} = \sigma_v \left[\left[\frac{1+k}{2} \right] \left[1 - \frac{r^2}{R^2} \right] + \left[\frac{1-k}{2} \right] \left[1 - 4\frac{r^2}{R^2} + 3\frac{r^4}{R^4} \right] \cos 2\theta \right] \quad 2-12(a)$$

$$\text{where } k = \frac{\sigma_h}{\sigma_v}$$

and the tangential stress at point "P"

$$P_{\sigma_{\theta}} = \sigma_v \left[\left(\frac{1+k}{2} \right) \left[1 + \frac{r^2}{R^2} \right] - \left(\frac{1-k}{2} \right) \left[1 + 3\frac{r^4}{R^4} \right] \cos 2\theta \right] \quad 2-12(b)$$

and the shear stress

$$\sigma_{r_{\theta}} = \sigma_v \left[\left(\frac{1-k}{2} \right) \left[1 + 2\frac{r^2}{R^2} - 3\frac{r^2}{R^4} \right] \sin 2\theta \right] \quad 2-12(c)$$

In plane strain condition,

$$(i) \text{ applying strain, } \epsilon_r = \left(\frac{1-\nu^2}{E} \right) \sigma_r - \frac{\nu(1-\nu)}{E} \sigma_{\theta}$$

Radial strain at point "P"

$$P_{\epsilon_r} = \frac{\sigma_v(1+\nu)}{E} \left[\left(\frac{1+k}{2} \right) \left[(1-2\nu) - \frac{r^2}{R^2} \right] + \left(\frac{1-k}{2} \right) \left[1 - 4\frac{r^2}{R^2} (1-\nu) + 3\frac{r^4}{R^4} \right] \cos 2\theta \right] \quad 2-12(d)$$

$$\text{and (ii) applying strain, } \epsilon_{\theta} = \left(\frac{1-\nu^2}{E} \right) \sigma_{\theta} - \frac{\nu(1-\nu)}{E} \sigma_r$$

we get tangential strain at point "P"

$$P_{\epsilon_{\theta}} = \frac{\sigma_v(1+\nu)}{E} \left[\left(\frac{1+k}{2} \right) \left[(1-2\nu) + \frac{r^2}{R^2} \right] - \left(\frac{1-k}{2} \right) \left[1 - 4\frac{r^2}{R^2} + 3\frac{r^4}{R^4} \right] \cos 2\theta \right] \quad 2-12(e)$$

and radial displacement at point "P"

$$P_{U_R} = \frac{\sigma_v(1+\nu)}{E} \left[\left(\frac{1+k}{2} \right) \left(R(1-2\nu) + \frac{r^2}{R} \right) + \left(\frac{1-k}{2} \right) \left[R + 4\frac{r^2}{R} (1-\nu) - \frac{r^4}{R^3} \cos 2\theta \right] \right] \quad 2-12(f)$$

If $\sigma_{\theta} = 0$ in equation 2-12(b) and $R \geq 10r$, then at $r = 10R$,

$$\text{Stress } (\sigma_{\theta})/\theta = \pi/2, 3\pi/2 = \frac{\sigma_v}{2} \left[2 + \frac{r^2}{R^2} + \frac{3r^4}{R^4} \right] = 1.013 \sigma_v \equiv \sigma_v, \text{ the original}$$

stress, which indicates that the creation of opening, practically, has no effect at points distant five times diameter from the center of the opening. However, Barla (1974) demonstrated that in a stratified rock mass, the effect of opening on the existing environment is significant only up to one diameter of the opening measured from the edge of the opening.

By substituting $k = 1$ and $\theta = 0^\circ$ (crown) and $\theta = 90^\circ$ (spring line) in equation 2-12, we get values of

$$\sigma_R = \sigma_v \left[1 - \left(\frac{r}{R} \right)^2 \right] \quad 2-13(a)$$

$$\sigma_\theta = \sigma_v \left[1 + \left(\frac{r}{R} \right)^2 \right] \quad 2-13(b)$$

$$\sigma_{R\theta} = 0 \quad 2-13(c)$$

If a uniform internal pressure " P_i " is applied for $k = 1$ and $\theta = 0^\circ$ and 90° (Deere, et al., 1969), the values are modified to

$$\sigma_R = \sigma_v \left[1 - \left(\frac{r}{R} \right)^2 \right] + P_i \left(\frac{r}{R} \right)^2 \quad 2-14(a)$$

$$\sigma_\theta = \sigma_v \left[1 + \left(\frac{r}{R} \right)^2 \right] - P_i \left(\frac{r}{R} \right)^2 \quad 2-14(b)$$

Table 2-16 shows the approximate ranges of internal support pressure " P_i " which could be provided by different support systems.

TABLE 2-16
" P_i " by different support systems.

| Support system | P_i |
|-----------------|--------------------------------------|
| Steel sets | 200 to 500 psi (1.38 to 3.44 MPa) |
| Steel lining | 500 to 3,000 psi (3.44 to 20.69 MPa) |
| Concrete lining | 200 to 500 psi (1.38 to 3.44 MPa) |
| Shotcrete | 50 to 200 psi (0.39 to 1.37 MPa) |
| Rockbolts | 20 to 75 psi (0.14 to 0.52 MPa) |

Bray (1967) demonstrated that if the rock contains 10 or more sets of discontinuities (joints), then its behavior can be approximated to the behavior of a homogeneous and isotropic mass with only 5 percent of error due to assumed homogeneity and isotropy condition. Also, if a rock is massive and contains very little discontinuity, it could be approximated to behave as a homogeneous medium. Hoek and Brown (1980) indicated that homogeneity is a matter dependent on the sample size. If the sample size is considerably reduced, the most heterogeneous rock will become a homogeneous rock. Deere et al. (1969)

indicated that if fracture spacing and opening size ratio is equal to or less than 1/100, the rock should be considered discontinuous and beyond this range should be considered continuous and possibly anisotropic. It is realized rock is hardly homogeneous or isotropic and, as such, the values obtained at equations 2-12 through 2-14 may not be applicable and should be used with caution.

2-9.3 Plastic zone created due to opening

The extent of plastic zone surrounding an underground opening is dependent on the rheology and the constitutive relationship of the host rock. The fundamental Hookean, Newtonian, or Saint Venant's rheological models are shown in figure 2-7.

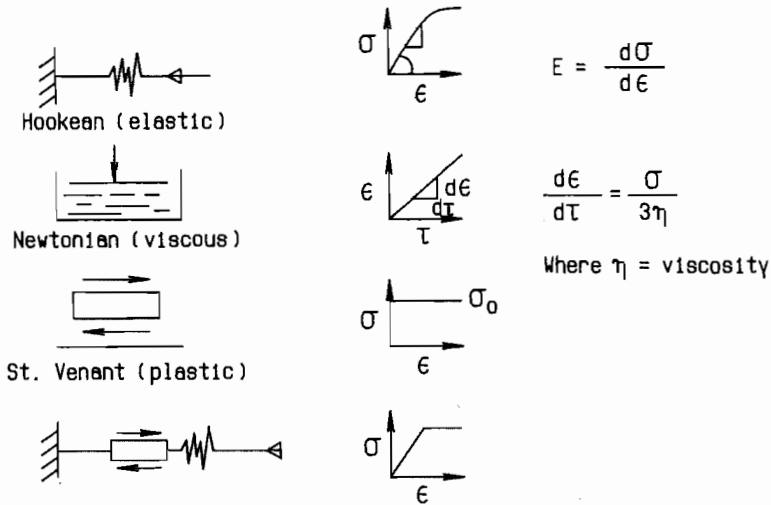


Fig. 2-7. Fundamental Rheological Models and Constitutive Relationship.

A rock behavior could be idealized as a combination of any of the fundamental rheological models. Some of the rock models and their constitutive behavior are shown in figure 2-8.

The next step is to define the failure mechanism of a rock sample. The simplest of the failure mechanism theories are attributable to Rankine and Saint Venant. According to Rankine, the failure will occur when the induced stresses have exceeded the maximum stress which the material can withstand. Saint Venant recommended that failure occurs when the induced strains exceed the maximum strain which the material can withstand.

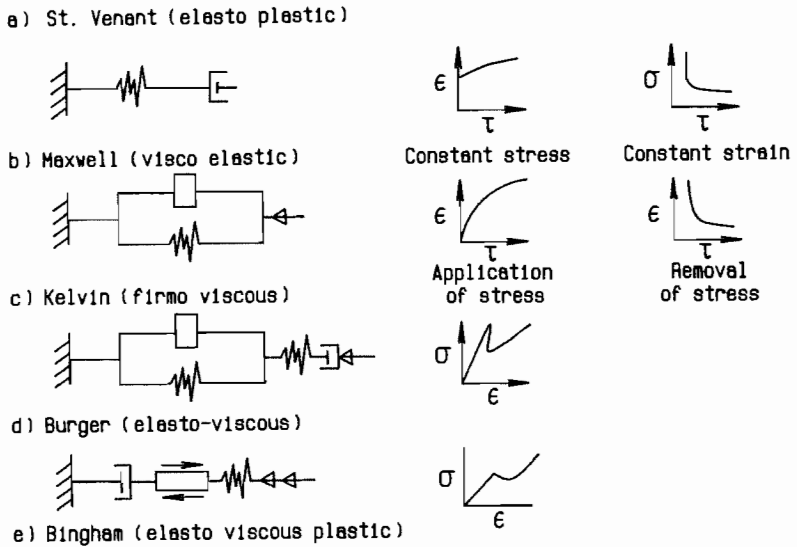


Fig. 2-8. Rock Rheological Model and its Constitutive Behavior.

Based on the questionable applicability of such simple theories to rock failures, other theories of failures have been formulated which are stated below.

(a) Tresca's theory: According to him, failure will ensue when

$$\frac{\sigma_1 - \sigma_3}{2} \geq \tau \quad 2-15(a)$$

where σ_1 is maximum principal stress
 σ_3 is minimum principal stress
 τ is the shear stress of the material

(b) Mohr-Coulomb's theory:

$$\tau = C + \rho \tan \phi \quad 2-15(b)$$

where C = cohesion
 ρ = induced normal stress
 ϕ = angle of friction

(c) Mohr's theory:

$$\sigma_1 = \sigma_3 \left[\frac{1 + \sin \phi}{1 - \sin \phi} \right] + 2c \frac{\cos \phi}{1 - \sin \phi} \quad 2-15(c)$$

(d) Griffith's theory:

$$(\sigma_1 - \sigma_3)^2 = 8 \sigma_t (\sigma_1 + \sigma_3) \text{ if } (\sigma_1 + \sigma_3) > 0 \quad 2-15(d)$$

$$\text{or } \sigma_3 = \sigma_t \text{ if } (\sigma_1 + 3\sigma_3) < 0$$

where σ_t = safe tensile stress in uniaxial tension

(e) Huber-Von Mises-Hencky theory:

$$(\sigma_{11} - \sigma_{22})^2 + (\sigma_{22} - \sigma_{33})^2 + (\sigma_{33} - \sigma_{11})^2 + 6 (\sigma_{12}^2 + \sigma_{23}^2 + \sigma_{31}^2) = 2\sigma_t^2 \quad 2-15(e)$$

where $\sigma_{11}, \sigma_{22}, \sigma_{33}$ are normal stresses

$\sigma_{12}, \sigma_{23}, \sigma_{31}$ are tangential stresses

(f) Octahedral shear stress theory:

$$(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_2)^2 = 2 \sigma_t^2 \text{ or } 2 \sigma_c^2 \quad 2-15(f)$$

(g) Hoek and Brown theory:

$$\sigma_1 = \sigma_3 + \sqrt{m \sigma_c \sigma_3 + s \sigma_c^2} \quad 2-15(g)$$

where m and s are constants

σ_c = uniaxial compressive strength

Kastner (1949) based on three assumptions, namely (1) $\sigma_1 = \sigma_\theta$ and $\sigma_3 = \sigma_r$, (2) failure mode is represented by

$$\sigma_\theta - \sigma_r \left[\frac{1 + \sin \phi}{1 - \sin \phi} \right] - 2c \frac{\cos \phi}{1 - \sin \phi} = 0, \text{ and}$$

(3) external uniform hydrostatic pressure of P_0 ($\sigma_v = \sigma_h = P_0$) is acting, performed a two-dimensional analysis. He found that the maximum tangential stress in the plastic zone at a distance "R" from center is

$$\sigma_{\theta P} = \frac{\sigma_{UC}}{k_p - 1} \left[k_p \left(\frac{R}{r} \right)^{(k_p - 1)} - 1 \right] \quad 2-16(a)$$

and the maximum radial stress is

$$\sigma_{RP} = \frac{\sigma_{UC}}{k_p - 1} \left[\left(\frac{R}{r} \right)^{(k_p - 1)} - 1 \right] \quad \text{and} \quad \tau_{R\theta} = 0 \quad 2-16(b)$$

where σ_{UC} = unconfined compressive stress

$$k_p = \frac{1 + \sin\phi}{1 - \sin\phi}$$

ϕ = angle of friction

r = radius of the circular opening

$\tau_{R\theta}$ = shear stress

It is worthwhile to notice that both $\sigma_{\theta P}$ and σ_{RP} are independent of the value of P_0 and depend on the material's unconfined uniaxial compressive strength and angle of friction.

The variation of σ_{θ} and σ_r with plastic zone formation are shown in solid line on figure 2-9. If no plastic zone is formed, the variations of σ_{θ} and σ_r are shown in dotted line on figure 2-9.

The radius " R_p " of plastic zone is determined by considering that the values of σ_{θ} and σ_r for plastic and elastic zones are same at this radius R_p where elastic zone begins.

The value of

$$R_p = r \left[\frac{2}{k_p + 1} \left\{ \frac{\sigma_{UC} + P_0(k_p - 1)}{\sigma_{UC}} \right\} \left(\frac{1}{k_p - 1} \right) \right] \quad 2-17$$

Value of R_p thus calculated should be increased to include the effects of weakening of the rock if blasting methods of excavation is used. The increase for normal blasting should be 100 percent, and 25 percent for controlled blasting. For tunnel boring machine excavation, a 20-percent increase is recommended. A higher margin is recommended for drill and blast against tunnel boring machine because Masterton (1981) reported a 15-to-20 percent overbreak for drill and blast and 3-to-5 percent overbreak by TBM.

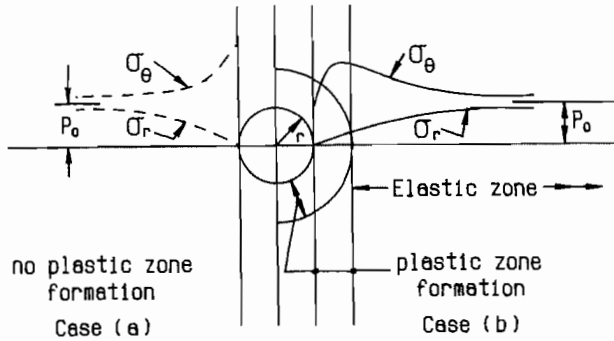


Fig. 2-9. Variations of σ_θ and σ_r .

2-10 FLEXIBILITY AND STIFFNESS METHODS

A flexible support system will attract more deformation and lesser load than a stiffer system. The flexibility ratio

$$F = \frac{E_m}{\frac{(1 + \nu_m)}{6 E_s I_s} (1 - \nu_s^2) r^3} \quad 2-18$$

where E = modulus of elasticity
 ν = Poisson's ratio
 subscript m = medium
 s = support
 r = radius of opening
 I = moment of inertia

Compressibility ratio

$$C = \frac{\frac{E_m}{(1 + \nu_m)(1 - 2\nu_m)}}{\frac{E_s t_s}{r (1 - \nu_s^2)}} \quad 2-19$$

where t = thickness

Rock tunnels have the compressibility ratio greater than 1, and for soil tunnels " C " $<$ 1. Schwartz and Einstein (1980) recommend the limiting

conditions for the applicability of solutions of stiffness and flexibility methods. These limitations are (1) the ground mass is considered homogeneous, isotropic, and linearly elastic; (2) the underground structure is located at depths greater than two times the diameter or width of the opening; (3) the ground stresses do not significantly vary in the zone of underground structures; (4) plane strain conditions are applicable; and (5) the lining system is close to an elastic cylinder.

At the support and medium interface, two possible conditions can exist: (1) full shear or no slip or (2) no shear transfer or full slip. The relationship for thrust moment, external force, and displacements in radial and tangential directions are given by the following equations.

Full Slip Case (Schwartz and Einstein, 1980)

$$\frac{T}{PR} = \frac{1}{2} (1 + k) (1 - a_0) + \frac{1}{2} (1 - k) (1 - 2a_2) 2 \cos 2\theta \quad 2-20(a)$$

$$\frac{M}{PR^2} = \frac{1}{2} (1 - k) (1 - 2a_2) 2 \cos 2\theta \quad 2-20(b)$$

$$\frac{U_s E}{PR (1 + \nu)} = \frac{1}{2} (1 + k) a_0 - (1 - k) [(5 - 6\nu) a_2 - (1 - \nu)] 2 \cos 2\theta \quad 2-20(c)$$

$$\frac{V_s E}{PR (1 + \nu)} = \frac{1}{2} (1 - k) [(5 - 6\nu) a_2 - (1 - \nu)] \sin 2\theta \quad 2-20(d)$$

where T = thrust

P = vertical pressure

k = horizontal pressure/vertical pressure

R = radius of opening

M = moment

ν = Poisson's ratio of host medium

ϕ = angle measured from spring line

U_s = radial deformation of support

E = modulus of elasticity of host medium

$$a_0 = \frac{C_1 F_1 (1 - \nu)}{C_1 + F_1 + C_1 F_1 (1 - \nu)}$$

$$a_2 = (F_1 + 6) (1 - \nu) / [2F_1 (1 - \nu) + 6 (5 - 6\nu)]$$

$$\text{where } C_1 = ER (1 - \nu_s^2) / E_s A_s (1 - \nu^2)$$

$$F_1 = ER^3 (1 - \nu_s^2) / E_s I_s (1 - \nu^2)$$

where ν_s = Poisson's ratio of support
 A_s = area of support
 E_s = modulus of elasticity of support
 I_s = moment of inertia of support

For No Slip Case (Schwartz and Einstein, 1980)

$$\frac{I}{PR} = \frac{1}{2} (1 + k) (1 - a_0) + \frac{1}{2} (1 - k) (1 + 2a_3) \cos 2\theta \quad 2-21(a)$$

$$\frac{M}{PR^2} = \frac{1}{4} (1 - k) (1 - 2a_2 + 2b_2) \cos 2\theta \quad 2-21(b)$$

$$\frac{U_s E}{PR (1 + \nu)} = \frac{1}{2} (1 + k) a_0 + \frac{1}{2} (1 - k) [4 (1 - \nu) b_2 - 2a_3] \cos 2\theta \quad 2-21(c)$$

$$\frac{V_s E}{PR (1 + \nu)} = -(1 - k) [a_3 + (1 - 2\nu) b_2] \sin 2\theta \quad 2-21(d)$$

where symbols are as before and

$$a_3 = b_1 \cdot b_2$$

$$b_1 = \frac{(6 + F_1) (C_1) (1 - \nu) + 2F_1 \nu}{3F_1 + 3C_1 + 2C_1 F_1 (1 - \nu)}$$

$$b_2 = \frac{C_1 (1 - \nu)}{2 [C_1 (1 - \nu) + 4\nu - 6b_1 - 3b_1 C_1 (1 - \nu)]}$$

With the calculated values of thrust, moment, and radial and tangential deformations, the support system can be designed by methods discussed in chapter 5.

2-11 CONVERGENCE - CONFINEMENT METHOD

When an opening is excavated for an underground structure, the existing stresses prior to excavation redistribute and adjust themselves to a new equilibrium condition. These stress changes require displacements to occur and the excavated ground tries to converge toward the opening. The amount of convergence depends on the host ground characteristics, method of construction, and the size of opening used. It is possible to conceive of a characteristic

curve shown as curve "G" on figure 2-10 which represents the radial convergence of a point in the roof of the opening. At point "A" on the "G" curve, the ground stress equals that existing prior to excavation σ_0 and the convergence is equal to zero. As σ_0 reduces due to creation of opening, the ground converges elastically up to point "B" on the "G" curve. The radial convergence at the face of opening, applying theory of elasticity is

$$U_b = \frac{(\sigma_0 - \sigma_b)}{E} (1 + \nu) r \quad 2-22(a)$$

$$\text{using } 2G = \frac{E}{(1 + \nu)}$$

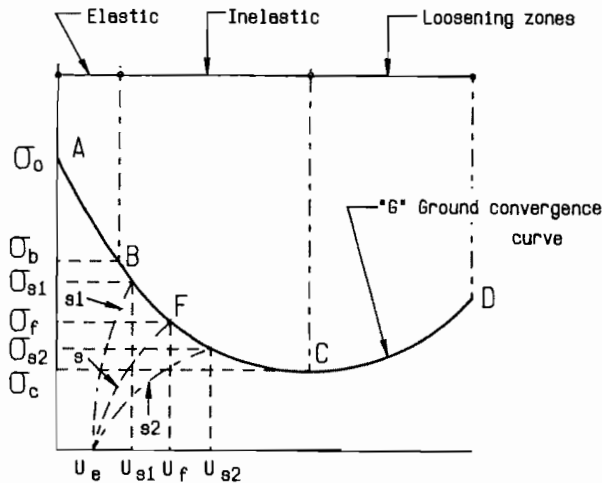


Fig. 2-10. Ground Characteristic and Support Confinement Curves.

The convergence at any other radius "R" is

$$U_{bR} = \frac{(\sigma_0 - \sigma_b)}{2Gr} \cdot R^2 \quad 2-22(b)$$

The further reduction of σ_b to σ_c will bring more radial convergence into existence. The determination of the value of U_f , i.e. the radial displacement of inelastic zone, requires nonlinear analysis preferably using finite element methods with realistic ground parameters (which are usually difficult to assess). An approximate evaluation of U_f after Brady and Brown (1985) is shown in equation 2-22(c).

$$U_f = r \frac{(\sigma_0 - \sigma_f)}{G(1 + E_p)} \left[\frac{(E_p - 1)}{2} + \left\{ \frac{r_p}{r} \right\} (1 + E_p) \right] \quad 2-22(c)$$

where r = radius of the opening

σ_f = stress after relaxation

E_p = modulus of deformation in inelastic zone

r_p = boundary of the relaxation zone as provided in equation 2-17

Beyond point "C," the material starts to loosen and it is important to provide confinement before the material reaches the point "C."

The confinement provided by a support system has its own characteristic curves shown as graphs "s," "s₁," and "s₂" on figure 2-10. These curves "s" are easier to determine than curve "G." This is because the constitutive relationship of support material is easily determinable.

Supposing the support was installed after the ground has suffered initial convergence of "U_e" with the support confinement curve of "s." The point "F" is the intersection of curves "G" and "s" and the support must be able to provide confinement pressure of σ_f to arrest the convergence of the opening at the value of "U_f." A stiffer support shown with support characteristic "s₁" installed at the same time as support with curve of "s" will have to share more support pressure σ_{s1} and result in less convergence U_{s1} than U_f. A more flexible support shown as s₂ will provide less confinement as σ_{s2} and more convergence of U_{s2} than σ_f and U_f, respectively.

The convergence confinement curves for roof, wall, and floor of the opening have to be different as shown on figure 2-11.

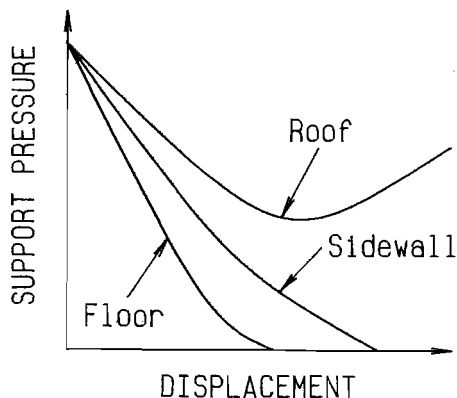


Fig. 2-11. Support Curves for Roof, Sidewall, and Floor of an Opening.

The ratio of convergences of crown and spring line could be up to 11 times that of roof, as observed at Washington Metro tunnels.

Though the concept of convergence and confinement is very interesting indeed, it is difficult to develop the ground convergence curve for very complicated geotechnical material.

2-12 NATM METHOD

The New Austrian Tunneling Method (NATM), introduced by Rabcewicz (1964) was slow in getting acceptability in the USA. But the real breakthrough came when an Austrian contractor, using NATM, successfully drove a twin single track railway tunnel at Mt. Lebanon in Pittsburgh in 1984 (Martin, 1987). Then followed the value engineering change proposal to construct, by using NATM methods, the Wheaton subway station and the associated tunnels. At this project an estimated cost saving of \$36 million was demonstrated by using NATM. The proposal was accepted and the project completed at substantial savings. This second successful completion of the project by NATM and great cost savings caught the attention of American engineers and, now, several other projects using NATM are being contemplated.

The NATM is a method by which the host ground surrounding an excavation for an underground structure is made into an integral part of the support structure. The host ground and the external support structure together take the full load. The host ground takes a major share of the load and the support takes a much smaller share of the ground load. This results in saving costs of external support systems.

Recalling equations 2-12 and 2-13, one will notice that the tangential stresses are always higher than radial stresses when an opening is created. Thus, if a support system can provide tangential resistance in the form of increased frictional resistance at the support and host interface, then the further relaxation of stresses due to excavation can be adequately resisted. Shotcrete provides strong frictional resistance. The ideal resistance will be provided by a closed ring of a very thin shotcrete membrane. But many times it is not practical to close the invert of the opening by shotcreting. Thus, the shotcrete in the roof and the sidewalls have to provide the tangential resistance. In order to help the resisting capability of this open shotcrete ring thus formed, use of rockbolts become necessary.

Rabcewicz (1964) found that a 5.9-inch- (150-mm)-thick shotcrete layer applied to a 32.8-foot- (10-m)-diameter tunnel could sustain a loosening load of 75 feet (23 m) of rock. Use of steel or timber support system for the same situation had to be much more expensive.

The NATM is an observational method and requires (1) application of a thin layer of shotcrete with or without rockbolts, wire mesh fabric, and lattice

girder; and (2) monitoring and observing the convergence of the opening.

If the observed convergence exceeds the acceptable limits, then subsequent applications of next layers of shotcrete are required until the convergence has stopped or is within the acceptable range. The shotcrete thickness is, thereby, optimized according to the admissible deformations.

The geometry of the opening is very crucial so that adequate ground arching action can develop. Straight reaches are carefully substituted by curved configurations.

The thickness of shotcrete layer required to sustain the equilibrium of an opening is discussed in chapter 9.

2-13 DISCONTINUITY ANALYSIS METHOD

It is well known that host rock contains discontinuities in the forms of bedding planes, joints, faults, folds, shear zones, seams, gauges, dykes, and fractures. These discontinuities together with the planes of excavation can form a block which is unstable and may fall into the opening. Stability can be ensured if the unstable block can be held into its original position by rockbolting or by providing an external support or by injection grouting which will increase the interlocking and shear resistances of the block at its interfaces with other blocks. Figure 2-12 shows an excavation with two sets of joint system.

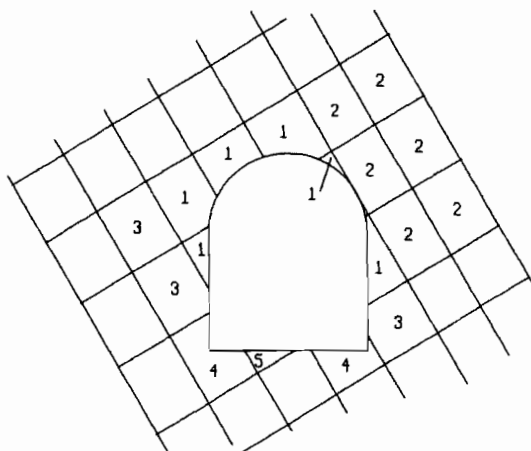


Fig. 2-12. Opening and Joints.

If we do not have a third set of joint system existing normal or subnormal to and running parallel to the plane of this paper, then all the blocks will be infinite and be stable. For infinite blocks, stability is never a problem. In figure 2-12, the finite blocks marked 1, formed by the intersections of at least three or more sets of joints, will fall into the opening if friction at their interfaces become incapable of resisting the movement. Blocks 2 will not slide unless blocks 1 already have fallen out. Blocks 3, 4, and 5 will not fall into the opening unless hydrostatic pressures move them upward.

The geological survey with oriented core can describe the various existing joints and discontinuities and then the analysis of those joints and discontinuities together with the planes of excavation should result into the identification of blocks which are likely to move. For ensuring stability, they must remain in their position. This is usually done by providing external support. Once the weight of the movable block is determined, a rockbolt system with adequate factor of safety, usually 2, can be designed to ensure stability.

Goodman (1988) indicated that it is only necessary to provide stability to the "key block." He defined "key block" as the single block, the removal of which will bring uncontrollable movement of several other blocks that are supported by the "key block." In soft rocks, each block that has the potential of moving must be restrained by using rockbolts or any other external support system. Soft rocks under stress can generate new fractures and thus generate several new key blocks.

2-14 NUMERICAL METHODS

Analysis of stresses and strains, of complicated geometrical shapes of openings, intersections of tunnels with tunnels or shafts or galleries, and complex geological environment require discretization of elements and materials. These analyses are very complex and more conducive to numerical methods than performing longhand calculations. Another alternative is to perform analysis by physical or photoelastic methods. Physical modeling is very expensive and time consuming. Photoelastic modeling is becoming a dying art in face of the availability of powerful computers for numerical analysis.

Numerical methods of analysis are discussed in chapter 3.

2-15 REFERENCES

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Chapter 3

NUMERICAL METHODS

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3-1 INTRODUCTION

This chapter follows up on Chapter 2, Design Methods. Numerical methods, like other design methods, have been briefly described. The special attention given to numerical methods through this chapter has two primary reasons: First, numerical methods represent the most versatile and complex group of computational methods used for tunnel engineering. Second, the use of numerical methods by tunnel and geotechnical engineers is growing every day. This is not amazing considering the rapid increase of computerization of engineering offices on one hand, and the fast advance of software development on the other hand.

A large body of literature is available on the various aspects of numerical methods and their applications. Reference is made where appropriate.

The goal of this chapter is to demonstrate how closely numerical methods relate to their practical applications to tunnel design. Developments of mathematical approaches of the various methods are limited but still useful for engineers with various backgrounds. No in-depth knowledge of mathematics or computer science is pre-requisite to follow this discussion.

3-2 COMPUTATIONAL METHODS: NUMERICAL METHODS AND THEIR ALTERNATIVES

Numerical methods are the subject of this chapter. Alternatives to numerical methods are also available as tools of tunnel engineering. The three alternatives most commonly used are:

- o Closed Form Methods (see chapter 2)
- o Analytical Methods (see chapter 2)
- o Numerical Methods.

3-2.1 Applicability and Comparison of Computational Methods

Closed form methods, analytical methods, and numerical methods differ in terms of their capability to simulate actual conditions. Also, different costs are associated with each method. A computational method should be used that best satisfies the specific need (Schiffman, 1972). If a simple problem is to be solved, a simple computational method may be sufficient. For a simple problem, the use of a numerical method might mean an inefficient utilization of computational resources. If a complex problem is to be solved, the use of numerical methods is most likely necessary. Sometimes, more than one approach may be suitable if consecutively employed in different phases of the design for one tunnel project. For instance, a closed form or analytical method may be sufficient during preliminary design of a tunnel in order to establish feasibility or basic geometrical or lining criteria. A numerical method may be necessary during final design to verify the preliminary assumptions and perform a detailed design analysis.

One distinctive characteristic of numerical methods is a discretization of the problem to be solved. By comparison, closed form and analytical methods do not require such discretization. Discretization is necessary if the problem to be analyzed is very complex or if true conditions shall be modeled with high accuracy. Discretization typically requires a large number of equations to describe the individual elements and their interrelations. Consequently, computers are used. The use of computer to solve discretized problems is implied in the term "Numerical Method".

The number of computers used in engineering offices is growing steadily. Also, more complex and yet user friendly software is continuously developed. Thus, numerical methods are increasingly becoming a popular engineering tool for tunnel design.

3-3 APPLICABILITY AND USE OF NUMERICAL METHODS TO TUNNEL ENGINEERING

Numerical methods are applicable and used throughout engineering disciplines. Prevalent applications to civil engineering problems are the analysis of stress, strain, and deformations. Also, the analysis of fluid flow and heat transfer through porous

media is often performed through numerical methods. Tunnel engineering, for both civil and mining purposes, may involve all of these applications.

Computer codes available for these applications are numerous, and new codes are constantly being developed. As of 1981, the U. S. National Committee for Rock Mechanics compiled a list of 15 codes considered to represent state-of-the-art for tunnel and mining engineering (Bieniawski, 1984). Today the number of computer codes available to solve tunnel-related problems is certainly much higher. The number and quality of publications and conferences on this subject worldwide suggests that several hundred applicable codes have been developed to date.

The purpose and goal of numerical computations in tunnel engineering varies. A distinction can be made by whether the analysis aims at obtaining qualitative or quantitative results. These results are discussed below.

3-3.1 Qualitative Analyses

Qualitative results are not expressed in absolute numbers, i.e. quantities. Instead, quantitative analyses enhance the conceptual understanding of the engineering principles that govern the solution of the problem. For instance, the understanding of the impact that certain parameters describing the tunnel and the surrounding medium can have on stress, strain or deformation, is classified as qualitative understanding.

For the purpose of discussing qualitative approaches to understanding, numerical analyses are divided in two groups: comparative studies and basic principles studies. Comparative studies in turn are divided in parameter studies and sensitivity studies.

(i) Parameter Studies. Parameter studies account for the unknowns inherent to subsurface conditions. The studies aim to analyze the impact that a possible range of subsurface conditions has on the civil structures below or above ground surface. The uncertainty about certain subsurface parameters, or the variability of these parameters evaluated in a testing program, may be overcome by performing an analysis for the extremes and the expected values of the assumed range.

(ii) Sensitivity Studies. Unlike for parameter studies, subsurface parameters are known or at least held constant for sensitivity studies. Rather, the impact from a possible variation of civil structure parameters is analyzed. Such parameters include the tunnel geometry, the relative location of underground structures, and the size and depth of the tunnel. Similarly, parameters related to the construction of civil structures may be varied to analyze their impact. For instance, the excavation sequence at the face or the relative advance of two adjacent tunnel headings can have significant impact on stress, strain and displacement around each tunnel and also impact on other close by structures. The purpose of the sensitivity studies is to optimize the civil structure parameters to the given subsurface conditions.

(iii) Basic Principle Studies. Basic principle studies are performed with both the parameters of the tunnel structure and the surrounding medium held constant. These studies aim at enhancing the understanding of engineering principles that determine the design requirements. For instance, the analysis and understanding of the stress flow in ground pillars between adjacent tunnels may help to optimize the tunnel layout. The study of earthquake effects or blast impacts on a tunnel structure is another application. Also, the study of kinematic mechanisms that conceivably may lead to tunnel failure may be beneficial.

3-3.2 Quantitative Analyses

Generally, qualitative results are commonly accepted as useful outcome of numerical methods applied in design. By comparison, quantitative results are often viewed more skeptically.

Quantitative results are expressed in absolute values. For the purposes of discussion, the two quantitative analyses options are described as design analysis and back analysis as follows:

(i) Design Analysis. Design requirements of the tunnel excavation support and lining are determined from design analyses. The anticipated strains in the surrounding medium, surface settlement, and the impact on other structures may also be the object of this analysis. Another result possibly obtained from the design analysis, though delicate and controversial, is the evaluation of the maximum permissible deformation of the

tunnel walls that is critical for the tunnel integrity (Wagner and Schullter, 1988). For this purpose, considerable interpretation experience is necessary to obtain valid information.

The problems associated with and the experience required to effectively use numerical methods are more thoroughly discussed later in this chapter. One problem relates to the difficulty to validate or calibrate quantitative results from numerical computations. One means to validation of quantitative results is the performance of a back analysis.

(ii) Back Analysis. For this analysis the calculation input parameters are obtained from measurements during the construction of the tunnel to be analyzed. Back analyses may be performed for two purposes: to validate the quantitative results obtained from a numerical analysis previously performed; and to obtain realistic input parameters for a numerical analysis to be performed in the future. One such back analysis case history is described by Gens et al. (1988). For example, the design of a main tunnel based on the displacements measured in its pilot tunnel may rely on a back analysis approach. In this case a numerical analysis would be first made for the pilot tunnel. Subsequently, deformations measured in the pilot tunnel are used to calibrate the numerical computation. The resulting "true" ground parameters are then used for the numerical analysis of the main tunnel.

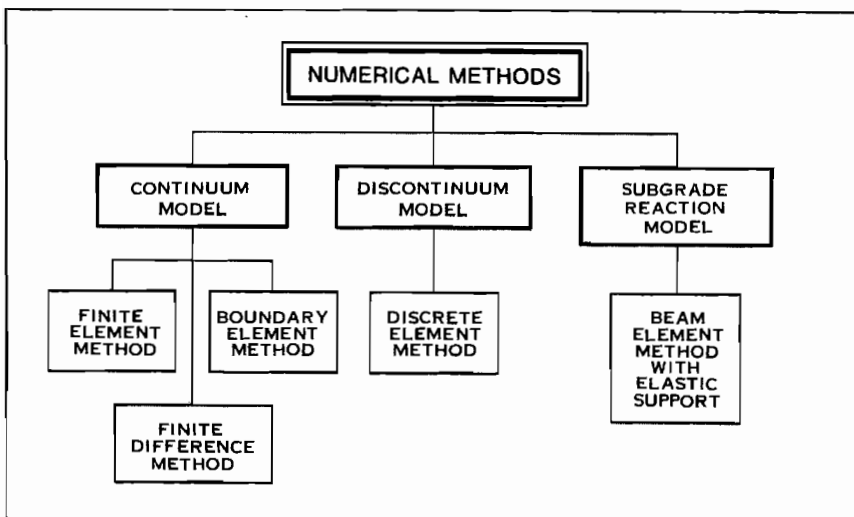
Another scenario may involve measurements taken during construction of the main tunnel that are then compared with the corresponding deformations predicted from the numerical calculations done for the same section. If the two values deviate, the measured value can be used to calibrate the analysis. The calibrated model is then used to adjust or further the tunnel design.

In general, a back analysis is of particular benefit when ground parameters follow a complex constitutive law that cannot be described easily (Zeng et al., 1988). Special applications of back analyses are for instance the determination of in situ stresses from instrumented rock burst occurrences (Jiayou et al, 1988).

3-4 DESCRIPTION AND COMPARISON OF NUMERICAL METHODS

Numerical methods used for tunnel engineering are listed in Table 3-1. Each method listed involves a discretization of the problem domain, which is facilitated by a computer-assisted analysis. Three different models are identified in the Table as the basis for the numerical methods discussed below. These models are: Continuum Model, Discontinuum Model, and Subgrade Reaction Model.

TABLE 3-1
Numerical methods and models for tunnel engineering.



The numerical methods associated with these models are: Beam Element Method with Elastic Support, Finite Element Method (FEM), Finite Difference Method (FDM), Boundary Element Method (BEM), and Discrete Element Method (DEM). In addition, hybrid methods have evolved by combining two or more of these individual methods. The methods are discussed individually in the following.

3-4.1 Beam Element Method with Elastic Support

The Beam Element Method is also referred to as "Coefficient of Subgrade Reaction Method", and is illustrated in Figure 3-1a. The tunnel lining is simulated by beam elements. The surrounding ground, that provides the embedment of the lining, is simulated by spring elements. Spring elements are typically oriented perpendicular to the lining, simulating the normal stresses induced to the ground from outward lining deflection. In addition, tangential spring elements can simulate shear stresses induced between the lining and the ground. The stiffness of the spring elements is determined from the stiffness, i.e. the modulus, of the ground and the curvature of the lining. To simulate actual conditions, spring elements under tension must be eliminated from the calculation. This is done through an iterative process.

The strengths and weaknesses of the method are:

Strengths: A large number of structural computer programs can be used to analyze a tunnel lining by means of the Beam Element Method with Elastic Support. The required computer processing and storage capacity is typically small compared with that required for other numerical methods.

Weaknesses:

- o The model used for the Beam Element Method with Elastic Support can only simulate simple or very simplified ground and tunnel conditions.
- o Each spring element simulates the embedment that is provided by the ground area it represents. Unlike in real conditions, the spring elements, i.e. supporting ground areas, are not connected with each other.

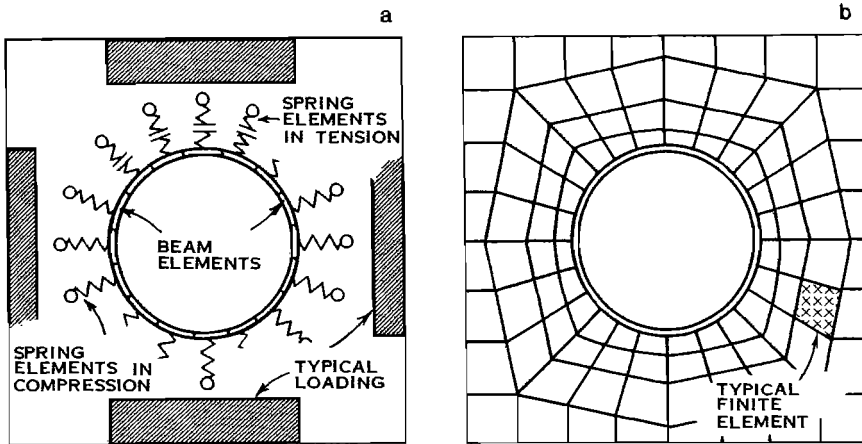


Fig. 3-1. Models for tunnel engineering - examples. (a) Beam element model with elastic support. (b) Finite element model.

3-4.2 Finite Element Method (FEM)

In the Finite Element Method (Fig. 3-1.b), the subsurface is predominantly modeled as a continuum. Discontinuities can be modeled individually. The problem domain, i.e. host ground, is discretized into a limited number of elements that are connected at nodal points. Each element is finite, i.e. geometrically defined and limited in size. This characteristic makes for the name of the method, Finite Element Method. The stress-strain relationship of the ground is described by an appropriate constitutive law. The stress, strain, and deformation to be analyzed are caused by changing the original (primary) subsurface condition. Such change is, for instance, induced by the tunneling process. Stress, strain and deformation induced in one element impacts the behavior of its neighboring elements, and so forth.

The complex interrelation between the interconnected elements makes for a highly complex mathematical problem. The analysis is performed by solving the equation matrix that models the mesh made up of the limited number of elements. That is, a system of equations is set up which relates unknown quantities to

known quantities via a global stiffness matrix. For instance, the relationship of nodal forces to displacements is analyzed this way throughout the finite element mesh. The concept to solve for unknown values at all points at one time is referred to as implicit approach. Some mathematical treatment is provided in Section 3-5. For additional selected references on mathematical concepts of the finite element method see Zienkiewicz (1971) and Bathe (1982).

The strengths and weaknesses of the method are:

Strengths: Highly complex underground conditions and tunnel characteristics can be analyzed. The capability of the Finite Element Method includes the simulation of complex constitutive laws, non-homogeneities, and the impact of advance and time dependent characteristics of the construction methods.

Weaknesses:

- o Solving of the complex mathematical problem requires a large computer processing and storage capacity.
- o Most Finite Element programs require more program and computer knowledge from the user than other methods do. Also, extensive output is typically generated that makes comprehension of the results more difficult. As a minimum, some graphical display capability should be included with the program. For very complex problems, for instance three-dimensional computations, a pre- and post-processing program is indispensable to facilitate data handling.
- o Unless a hybrid model is formed, arbitrary external boundary conditions of the Finite Element Model must be defined. In order to avert any impact from these boundaries on the analysis of stress, strain and deformation close-by and along the tunnel circumference, the boundaries are set at a sufficient distance away from the tunnel. Consequently, a large mesh is required that relates to a large required computer capacity.

3-4.3 Finite Difference Method (FDM)

The method is similar to the Finite Element Method in that the subsurface is modeled as a continuum that is divided into a number of elements which are interconnected at their nodes. The primary difference lies in the approach used to solve the unknown

parameters. In contrast to the implicit approach of the Finite Element Method, the Finite Difference Method is based on the explicit approach discussed in the following.

The explicit method builds on the idea that for a small enough time step, a disturbance at a given mesh point is experienced only by its immediate neighbors. This implies that the time step is smaller than the time that the disturbance takes to propagate between two adjacent points. For most Finite Difference programs this time step is automatically determined such that numerical stability is ensured. Initially conceived as a dynamic, i.e. time related, computation approach the Finite Difference method can be used to solve static problems by damping the dynamic solution. Then, "time step" does not refer to a physical but rather to a problem solution (time) step. Analyzed velocities relate to displacement in length per time step.

The separate solution for individual mesh points implies that no matrices need to be formed. For each time step an individual solution is obtained for each mesh point. The calculation cycle leading to the solution involves Newton's law of motion and the constitutive law of the in situ material. The acceleration solved for a mesh point is integrated to yield the mesh point velocity, which in turn is used to determine the strain change. Subsequently, strains determine the corresponding stress increments which in turn generate forces on the surrounding mesh points. These are summed to determine the resulting out-of-balance force which relates to the acceleration that started the calculation cycle. The method is described in more detail by Cundall and Board (1988).

The strengths and weaknesses of the method are:

Strengths:

- o The explicit approach facilitates analysing the behavior of the problem domain as it evolves with time. This allows for a step-by-step analysis of possible failure mechanisms.
- o Because no matrices are formed the required processing and storage capacity of the computer is relatively small.
- o The solution without matrices also allows for the analysis of large displacements without significant additional computer effort.
- o Most efficient for dynamic computations.

Weakness:

- o If used for static problems the method may require more computation time than most other numerical methods.

3-4.4 Boundary Element Method (BEM)

This method has only recently gained on popularity in the engineering community. Today, the Boundary Element Method is increasingly used for the linear and non-linear static, dynamic and thermal analysis of solids. Likewise, transient heat transfer and transient thermal visco-plasticity is simulated with the method. The use of the Boundary Element Method for tunnel engineering is also growing (Banerjee and Dargush, 1988).

Like the Finite Element Method and Finite Difference Method, the Boundary Element Method models the ground as a continuum. Some of the differences to those methods are:

- o Unless singularities of the ground mass shall be modeled, a discretization of the problem domain is necessary for the excavation boundary only. A numerical calculation is confined to these boundary elements. The medium inside those boundaries is typically described and simulated by partial differential equations. These equations are most often linear and represent approximate formulations of the actual conditions.
- o Contrary to the Finite Element Method and Finite Difference Method, the problem is solved by integration of the partial differential equations. This approach gives the Boundary Element Method the alternative name "Integral Method". For more detail on the boundary element method see Crouch and Starfield (1983).

The excavation boundaries are also referred to as "external boundaries". If discontinuities between the external boundaries shall be analyzed, "internal boundaries" are introduced. "Internal boundaries" model the interfaces between different material types or discontinuities. The method involving the analysis of internal boundary elements is referred to as "Displacement Discontinuity Method" and represents a specific type of the Boundary Element Method.

The strengths and weaknesses of the method are:

Strengths:

- o The system of equations to be solved is small compared

with that required for the Finite Element Method. Hence, a comparably small computer capacity is sufficient.

- o Data input and output are comparably simple and are easily processed.

- o The Boundary Element Method is very efficient and economical for two- or three-dimensional problems when the defined boundaries are of greatest concern.

Weaknesses:

- o Today the capacity of most boundary element programs is, with few exceptions, limited to linear constitutive ground behavior. Even so, much progress is currently under way with program developments.

- o Complex construction procedures and time dependency of material characteristics cannot be modeled easily.

3-4.5 Discrete Element Method (DEM)

The Discrete Element Method is also referred to as "Distinct Element Method" or "Rigid Block Method". In contrast to the methods discussed above, the ground mass is not modeled as a continuum. Rather, the ground mass is modeled by individual blocks that are rigid in themselves. The method is applicable if the joint displacements so overshadow the internal block deformation that the latter can be neglected. In this case, the deformation of the ground mass is governed by the movement along the joints between rigid blocks.

The Discrete Element Analysis begins with the computation of incremental forces acting in the joints. The resulting accelerations of the rigid blocks are integrated to give new positions and orientations of the block centroids. This in turn yields new increments of joint forces, which continue the calculation cycle. See Cundall (1976) for more details.

The strengths and weaknesses of the method are:

Strengths:

- o The method is especially useful for kinematic studies of large block systems, e.g., where highly jointed rock masses around the tunnel are modeled.

- o The magnitude of block movements that can be analyzed is large compared with that obtained from most continuum models. The required computer capacity is comparably small.

Weaknesses:

- o The computation requires the input of joint location and orientation. This information is not normally known prior to construction of the tunnel. Even so, parameter studies can be performed by assuming various joint configurations.

3-4.6 Hybrid and Complementary Methods

Each numerical method may be used most efficiently if combined with other numerical methods. The purpose of coupling individual numerical methods is typically twofold. First, the strengths of each method can be preserved while its weaknesses may be eliminated. Secondly, the combination of individual methods and their associated models can create a model that best describes the specific problem.

Several forms of model combinations are:

- (i) The problem domain is divided into two or more areas that are analyzed simultaneously. Different models are used for each area.

Example: Continuum model combined with Discrete Element Method. Figure 3-2a shows the division of the problem domain into two areas. The far field area, away from the tunnel opening, is modeled as a continuum. The near field, i.e., close to the tunnel opening, is modeled with Discrete Elements. This reflects the anticipated ground displacement if jointed rock is encountered and movements are not restrained by support and construction measures. Since the far field area is of less concern to the engineer and the ground mass is more confined, a continuum model is justified.

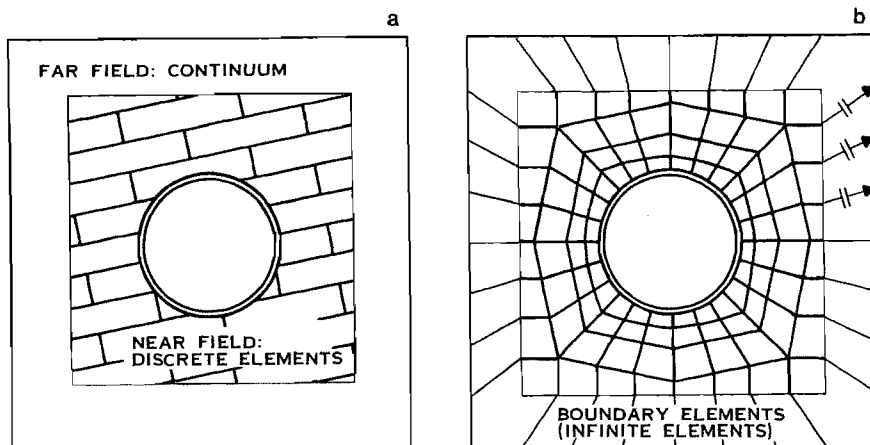


Fig. 3-2. Hybrid Methods. (a) Continuum model combined with discrete elements. (b) Boundary element method combined with finite element method.

Example: Boundary Element Method combined with Finite Element Method. Figure 3.2b depicts the two areas that are analyzed differently. The purpose of surrounding the Finite Element mesh with boundary elements is to eliminate the need for arbitrary and rigid boundary conditions. Hence, the size of the Finite Element mesh can be reduced, which allows for significant reduction of the required computer capacity. The boundary elements used for this purpose are also referred to as "Infinite Elements". The name implies that any disturbance that reaches the interface of the two models converges to zero at a pole in infinity.

(ii) The analysis of the problem domain is performed in two or more computation steps. Different models are used for each step. The outcome of one step is used as input for the subsequent step.

Example: Finite Element Method combined with Discrete Element Method. Figure 3-3 depicts the two computation steps, each analyzed with a different numerical approach. In the first step, the Finite Element Method assumes a continuous ground mass around the tunnel opening. In the second step, joints are introduced forming discrete or rigid block elements along the tunnel boundary. The stresses initially calculated from the Finite

Element analysis are used as input to the rigid block analysis. These stresses simulate the interlocking of the blocks which, in combination with the block weight, simulates realistic conditions.

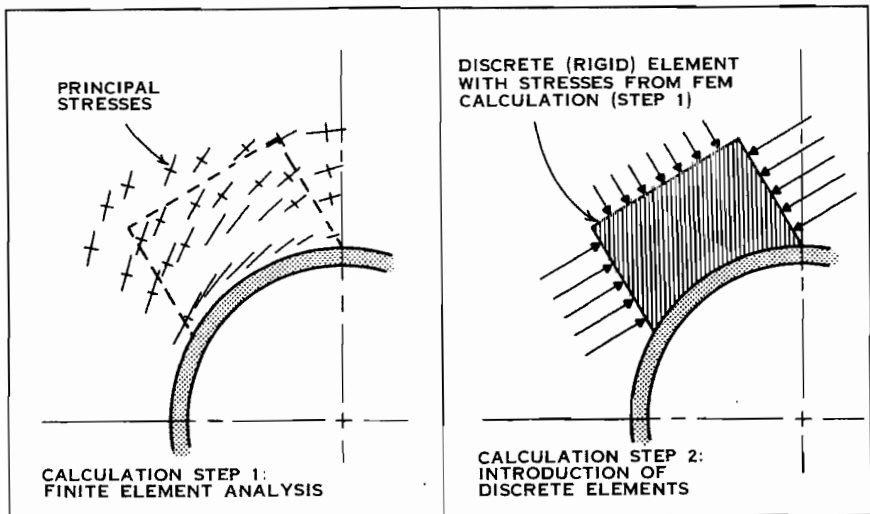


Fig. 3-3. Hybrid Method - finite element method combined with discrete element method.

Example: Finite Element Method combined with Beam Element Method with Elastic Support. The two computation steps are illustrated in Figure 3-4. The combination of methods described here is conducive to a dual lining concept. The Finite Element Method employed for the first computation step analyzes the stress, strain and deformation of the ground mass including the initial or primary lining. It is assumed that the final, or secondary lining is installed at a later point of time. This lining is analyzed by means of the Beam Element Method. Spring elements model the embedment provided by the initial lining and the ground mass. The loads on the secondary lining are determined from the stresses initially calculated from the Finite Element computations,

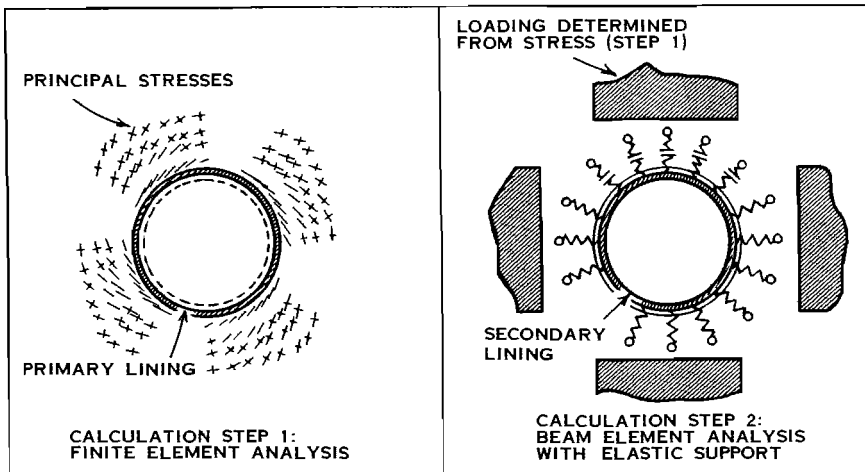


Fig. 3-4. Hybrid Method - finite element method combined with beam element method with elastic support.

(iii) The model is first used that is best suited to validate computation parameters. Subsequently, the validated parameters are used with a different model that best generates the necessary data for design.

Example: Finite Difference Method combined with Finite Element Method. The Finite Difference Method may allow a better validation of parameters that are used as input to the subsequently performed Finite Element computation. An example is, for instance, the analysis of fluid flow in a porous medium (Bolly et al., 1988). The true transmissivity parameter of the medium can be better determined from the Finite Difference Analysis. Subsequently, the fluid flow is analyzed with the Finite Element computation.

3-4.7 Comparison of Numerical Methods

The strengths and weaknesses of the methods discussed in this chapter were summarized above. Conclusions as to the suitability and applicability of a numerical method must be drawn for each individual case.

Typically, a different level of conservatism is associated with the various methods. For instance, Figure 3-5 illustrates

the roof displacements of a tunnel, analyzed with different methods by Laabmayr and Swoboda (1978). The curved line represents the displacements analyzed with the Beam Element Method with Elastic Support, as a function of the tunnel overburden. According to Terzaghi's theory, the load on a tunnel roof remains constant for depths greater than a defined value (see Section 2-4). The straight lines mark the limits of the displacement values calculated from Finite Element computations. The range relates to different rates of load transfer assumed in the various Finite Element computations. The point corresponding to the actually measured roof displacement is also shown in Figure 3-5.

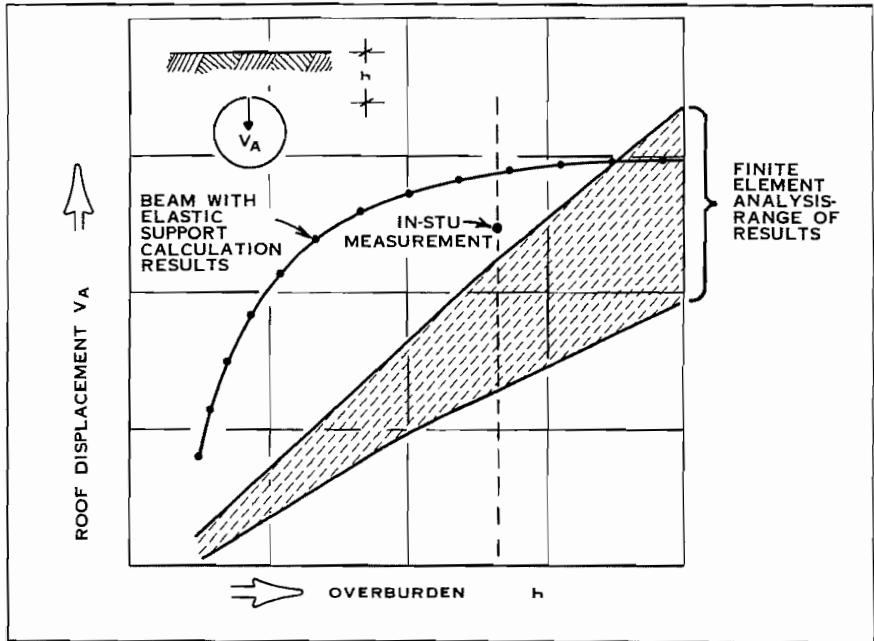


Fig. 3-5. Impact of the numerical method on calculation results.

Similar qualitative results would be obtained at tunnel roof if bending moments were plotted along the vertical axis of Figure 3-5. It appears that the Beam Element Method with Elastic Support yields too conservative values for lower overburdens if Terzaghi's load assumption is used. On the other hand, the analyses according to the Finite Element Method do not include phenomena that would have increased the calculated values. For either method, large experience is required to evaluate the quantitative computation results. This is increasingly true with the more complex numerical methods.

Finally, numerical methods should be compared in terms of calculation efficiency. Simple methods should be employed for simple problems. One evaluation criterion should be the utilization of the computational resources (Schiffman, 1972).

3-5. MATHEMATICAL TREATMENT OF NUMERICAL "ELEMENT METHODS"

Of the numerical methods discussed in Section 3-4, the Finite Element Method, Boundary Element Method, and Discrete Element Method, are summarized under "Element Methods". Common to all these methods is that the main body of the structure is discretized into an assemblage of elements of smaller dimensions. The smaller elements are assumed connected to the neighboring elements only at the common node points. The displacements under a system of loading, determined only at the node points, are then used to find the displacements at any other point in the interior of the elements. Basic element units such as rods, trusses, beams, plates, bricks and shells, are used either alone or in combination to form the final physical shape of the structure.

Basically, all the "Element Methods" use matrix algebra to operate on the mathematical expressions (algebraic, differential or integral) that describe the physical behavior, e.g. stress, strain or displacement, of a problem domain.

3-5.1 Elements of Matrix Algebra

A matrix is a rectangular or square array of parameters arranged in rows and columns and represents a mathematical relationship. The parameters may be numerical, differential or

integrable quantities. A set of linear expressions is shown in Equation 3-1:

$$\begin{array}{rcl}
 a_{11} x_1 + a_{12} x_2 + a_{13} x_3 + \dots + a_{1n} x_n & = & R_1 \\
 a_{21} x_1 + a_{22} x_2 + a_{23} x_3 + \dots + a_{2n} x_n & = & R_2 \\
 \cdot & & \cdot \\
 \cdot & & \cdot \\
 \cdot & & \cdot \\
 a_{n1} x_1 + a_{n2} x_2 + a_{n3} x_3 + \dots + a_{nn} x_n & = & R_n
 \end{array} \tag{3-1}$$

Equation 3-1 can be represented in a convenient matrix notation described by Equation 3-2,

$$\begin{bmatrix} a_{11} & a_{12} & a_{13} & \dots & a_{1n} \\ a_{21} & a_{22} & a_{23} & \dots & a_{2n} \\ \cdot & \cdot & \cdot & & \cdot \\ \cdot & \cdot & \cdot & & \cdot \\ \cdot & \cdot & \cdot & & \cdot \\ a_{n1} & a_{n2} & a_{n3} & \dots & a_{nn} \end{bmatrix} \begin{Bmatrix} x_1 \\ x_2 \\ \cdot \\ \cdot \\ \cdot \\ x_n \end{Bmatrix} = \begin{Bmatrix} R_1 \\ R_2 \\ \cdot \\ \cdot \\ \cdot \\ R_n \end{Bmatrix} \tag{3-2}$$

or,

$$[A] [X] = [R] \tag{3-3}$$

where [A] represents the matrix, and

[X] and [R] represent the vectors

All the arithmetical operations, such as addition, subtraction, multiplication and division or inversion can be performed on a matrix.

A matrix of form [A] can be manipulated to give a lower triangular matrix [L], or upper triangular matrix [U], or a diagonal matrix [D]. For example:

$$[A] [X] = [R] \tag{a}$$

can be manipulated to render

$$[L] [X] = [P] \tag{b}$$

or

$$[U] [X] = [Q] \tag{c}$$

or

$$[D] [X] = [M] \tag{d}$$

such that solving for [X] by (b), (c) or (d) gives the same values of [X] if solved by (a).

The concept entails reducing [A] into an upper triangulation matrix such that $[U] [X] = [Q]$, and finding the values of the elements of [U] and [Q] and then using substitution to solve for the elements of [X]. This Gauss elimination method of finding [Q] and [U] is very conducive to a digital computer. Other methods such as the Gauss-Jordon method, Cholesky method (Ural, 1973) and other methods can be also used to solve the equation 3-3.

3-5.2 Mathematical Formulation in the Finite Element Method

The Finite Element Method (Section 3-4.2) entails that the underground structure is approximated by an assemblage of properly selected finite elements. The finite elements are considered interconnected at a finite number of nodal points or joints. These finite elements are discrete elements.

With the given joint loading, known geometric configuration, and assumed material properties of the finite elements, the joint node displacements and the internal stresses of each finite element are determined by the application of finite element method. This requires the determination of the stiffness matrix for selected finite elements that model the problem domain.

By following the general form of equation 3-3, the internal displacements of the elements in the directions of degrees of freedom are described through the nodal displacement:

$$\{D_i\} = [\phi] \{U_n\} \quad (3-4)$$

where:

$\{D_i\}$ is the internal displacement of a finite element vector,

$[\phi]$ is the shape matrix, and

$\{U_n\}$ is the nodal displacement vector.

By taking proper derivatives, the internal displacements can be converted into internal strains such that,

$$\{\epsilon_i\} = [B] \{U_n\} \quad (3-5)$$

where:

$\{\epsilon_i\}$ is the strain vector, and

[B] is the displacement strain matrix

Once the strain vector is obtained, the internal stresses can be found by using constitutive relationships of the material:

$$\{\sigma_i\} = [D] \{\epsilon_i\} \quad (3-6)$$

where:

$\{\sigma_i\}$ is the stress vector, and

$[D]$ is the strain-stress (or stiffness) matrix.

By invoking the energy equation, that external virtual work equals internal virtual work, one can find the value of the stiffness matrix.

Assuming that $\{R\}$ is the generalized nodal force and $\{\bar{U}_n\}$ is the virtual nodal displacement, the external virtual work can be written as:

$$W_e = 1/2 \{\bar{U}_n\}^T \{R\}$$

and the internal virtual work:

$$W_i = 1/2 \int \{\bar{\epsilon}_i\}^T \{\sigma_i\} dv$$

Equating $W_e = W_i$ follows that

$$\begin{aligned} \{R\} &= [K] \{U_n\}, \text{ with} \\ [K] &= \int [B]^T [D] [B] dv \end{aligned} \quad (3-7)$$

where $[K]$ is the individual element matrix and may be given in local coordinate system.

If a generalized global coordinate system is used, each individual stiffness matrix needs to be transformed into global coordinates and the individual elements assembled to render the global stiffness matrix for the structure as a whole. That is,

$$[K_g] = [T] [K] \quad (3-8)$$

where:

$[T]$ = is the transformation matrix, and

$[K_g]$ = is the local stiffness matrix transformed into a global system of coordinates

$$[\bar{K}] = \sum_{i=1}^n K_i \quad (3-9)$$

where:

$[\bar{K}]$ is the assembled global stiffness of a structure as a whole.

Application of equation

$$[\bar{K}] \{U_n\} = \{R\} \quad (3-10)$$

renders the values of $\{U_n\}$ from which $\{\epsilon_i\}$ and $\{\sigma_i\}$ can be determined by use of the equations 3-5 and 3-6.

3-5.3 Library of Stiffness Matrix

Zienkiewicz (1971) has developed an element stiffness matrix for plane stress, plane strain, 2D, axisymmetric stress, 3D four noded elements, and 3D eight noded elements. Laursen (1978) has developed 3D stiffness matrix for 12 degrees of freedom, 3 displacements and 3 rotations at each two nodes of the beam.

3-6. MODELING FOR NUMERICAL COMPUTATIONS

Numerical computations, as a tool of tunnel engineering, aim to analyze, i.e., reproduce, explain and predict the behavior and response of structures and media subjected to impacts from tunneling. Establishing a model of the "real world" conditions is necessary if physical and mathematical concepts are to be employed for the analysis. The modeling of "real world" conditions is difficult because of the unknowns of the subsurface, the complexity of the subsurface and tunnel behavior, and the problems associated with formulating proper constitutive laws of the ground. Since it is neither possible nor useful to simulate all conditions and parameters in detail, a simplified model must be described. Without simplification, an accuracy might be pretended that could easily prove false. Also, cost considerations typically call for a simplified computation model. Even so, the results gained from the numerical computation must still be of benefit to the engineer and to the engineered product. The experience of the engineer with the numerical tool used is vital for proper interpretation of the results. This includes the understanding of the impacts that specific program character-

istics have on the calculation outcome. Each computation method has its strengths and weaknesses.

Model simplification can be achieved by employing one or several of the following approaches:

- o Three-dimensional conditions modeled in two-dimensions.
- o Utilization of section symmetries.
- o Simplified modeling of the ground and the tunneling process.

3-6.1 Three Dimensions Simulated by Two-Dimensional Model

Three dimensionality of the subground and tunneling conditions has been analyzed by Wittke (1977) and other authors, and can be found in various forms:

(i) Anisotropy of the rock mass (schistose rock, etc.) and discontinuities extending in three dimensions.

(ii) Three-dimensional spatial geometry of the analyzed problem area, for instance in the proximity of portals, pillars, end walls, and the advancing excavation face. Figure 3-6 depicts a tunnel in face proximity where three-dimensionality is encountered in two ways: First, load transfer due to tunneling induced stress redistribution in the subground occurs in directions both transverse and longitudinal to the tunnel axis. Second, displacements occur along the tunnel circumference, in the ground ahead of the tunnel, and at the tunnel face. The latter can represent a stability case for which a three-dimensional analysis may be critical.

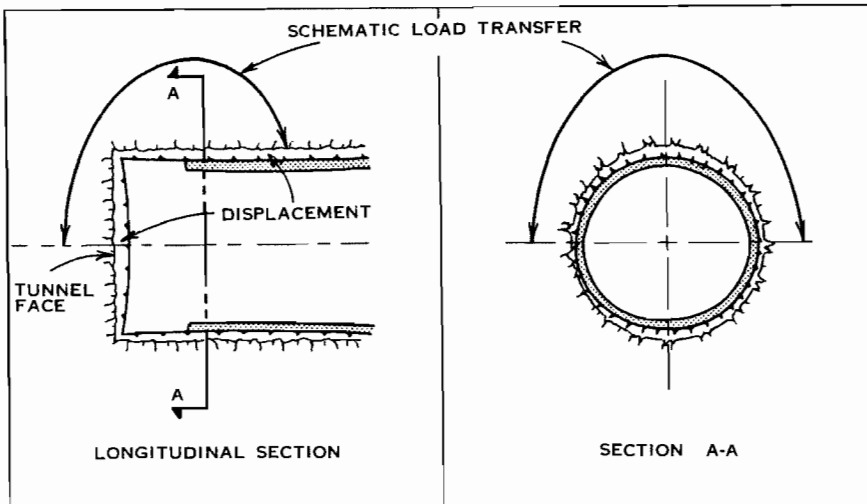


Fig. 3-6. Three dimensionality of load and displacement at the tunnel face.

Despite these three-dimensionalities frequently encountered in nature, a three-dimensional analysis is often not necessary. Instead, a two-dimensional model can be substituted. The decision on whether a two-dimensional or three-dimensional model offers the best solution, should include the following considerations:

(i) The size and complexity associated with a three-dimensional model, compounded by imperfections inherent to any computer program, may adversely affect the calculation results. Also, the description and processing of complicated models promotes inaccuracies and errors with the calculation input development. Similarly, the large number of calculation output parameters can be difficult to process and interpret. In addition to the engineering computer program, a pre-processing and post-processing program is needed to alleviate these problems.

(ii) Calculation Costs: The cost to perform a three-dimensional analysis obviously exceeds that of the two-dimensional analysis. Costs are incurred due to labor and computer use. For the three-dimensional analysis, additional labor relates to the engineer's efforts to prepare the calculation input data and to evaluate and interpret the calculation results. Costs

associated with computer use relate to the type of computer required and the computer time necessary. Figure 3-7 shows the computer cost as a function of the number of unknowns for a problem described by Wittke and Pierau (1976). For the case of a typical Finite Element Analysis, the number of unknowns equals approximately three times the number of nodal points. According to the figure, the calculative cost increases exponentially. In Figure 3-7, approximately 500 unknowns correspond to the base 100% cost.

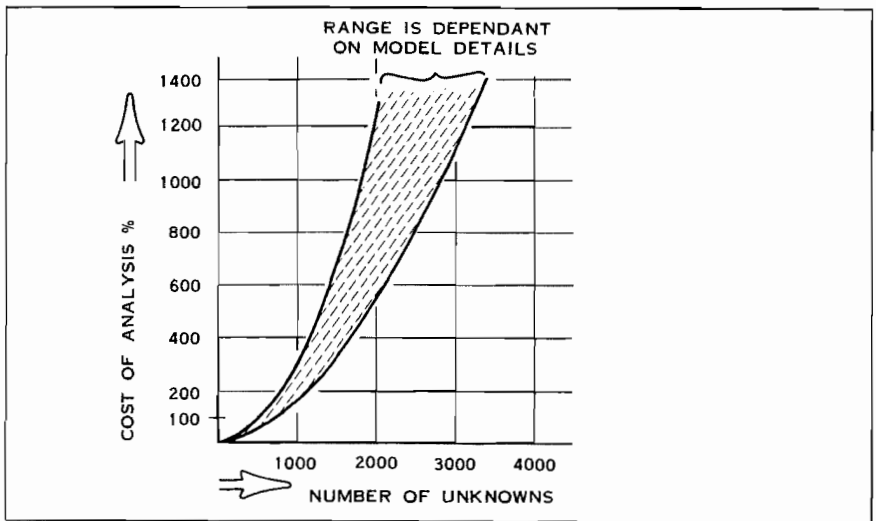


Fig. 3-7. Computer cost as function of the number of unknowns - finite element calculation example.

The simulation of three-dimensional conditions by a two-dimensional model requires experience and the understanding of the relationship between these two models. The proper two-dimensional simulation of the three-dimensional load transfer in face proximity (see Figure 3-6) has proven particularly critical to obtaining valid calculation results. This aspect is discussed in more detail in Section 3-7.2.

3-6.2 Utilization of Symmetry

If the geometry, the ground mass properties, and in situ stresses are symmetrical to the vertical tunnel axis, only half of the continuum must be analyzed. Figure 3-8 illustrates an example of such simplification.

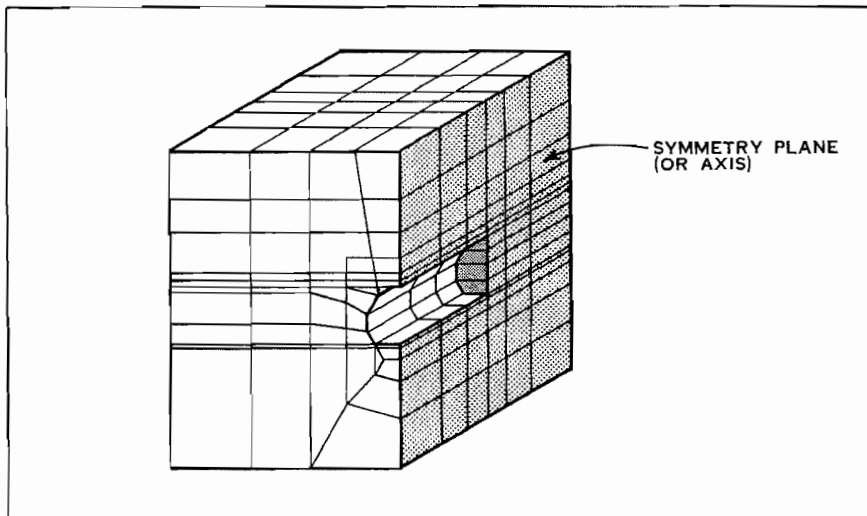


Fig. 3-8. Symmetry of the computation model.

3-6.3 Simplified Modeling of the Subground and the Tunneling Process

The unknowns and complexity of the subground and the tunneling conditions require that simplifications be made for the calculation model. In the following section this is described for the Finite Element Method.

3-7. MODELING WITH THE FINITE ELEMENT METHOD

The Finite Element Method is by large the most popular numerical calculation tool in tunnel engineering. Therefore, the following discussion focuses on this method.

In the United States several general purpose finite element programs are available such as Stardyne, Ansys, Abaqus, MSC Nastran, Cosmic Nastran, Adina, Ease 2, and Marc (Sinha, et al, 1987). Each software program has its own element library. Brebbia (1982) has compiled the advantages and disadvantages of several software programs. His analysis may assist in the selection of a particular software program for the most efficient means of analyzing a particular underground structure. Alternatively, one can conduct its own research to determine the suitability of a particular software program for its own application.

3-7.1 Modeling of the Subsurface

This section follows up on the discussion of the section 3-6: Modeling for Numerical Computations. As was stated before, some simplification of the computation model is necessary. This applies also to modeling of the subsurface.

Care is required in the selection of the finite element mesh that models the medium. It is important that the size and type of finite elements be properly selected to ensure accuracy of analysis, convergence of solution and minimization of rounding errors during numerical calculation.

A large number of elements will usually render high accuracy of analysis but will require larger computer capacity and longer computer runs. This may increase the cost of analysis. The finite elements selected should not create spurious energy modes or cause shear locking or membrane locking. Generally, a patch test (Bathe, 1982) will identify ill-conditioned elements which should be eliminated and modified to obtain a realistic finite element analysis. The aspect ratio (longest/smallest dimension of element) should not exceed three, otherwise considerable calculation errors will be generated.

To some extent, common or similar approaches are used to model the subsurface regardless of actual subsurface conditions. One common approach is that a constitutive law is established that governs the stress/strain relationship of the ground.

Constitutive laws used for geotechnical engineering computations are linear elastic, non linear elastic, linear visco elastic, elasto-plastic, elasto-visco-plastic, isotropic, anisotropic, thermal-dependent or stochastic. In addition, specific modeling and simulation requirements vary with the ground characteristics in question. Specifics of subsurface modeling relate to the different unknown subsurface parameters and the different parameters that affect ground mass behavior. For the purpose of discussing subsurface-specific modeling, a distinction is made in the following between rock subsurface and soil subsurface.

(i) Modeling of a Rock Subsurface. Rock masses modeled for numerical computation can be seen composed of intact rock and discontinuities. The properties and characteristics of both components govern the properties of the rock mass. Accordingly, rock masses are isotropic or anisotropic, discontinuous or continuous, homogeneous or heterogeneous.

For tunnel engineering purposes, rock masses are sometimes modeled as ideal homogeneous and isotropic continuum. At other times models account for heterogeneity or anisotropy, or for discontinuities. Only rarely do models account for all non-ideal rock mass characteristics. The type and necessary sophistication of a rock mass model, or the permissible simplicity to obtain valuable results is discussed below.

Continuum vs. Discontinuum Model: Rock masses typically show some form of discontinuities. Discontinuities occur as through going faults, master joints, and as the result of sedimentation, schistosity, and tectonic jointing. To anticipate and model these features in their details is a formidable task that can usually not be accomplished.

An alternative to modeling individual discontinuities is to approximate their effect on a simplified quasi-continuum rock mass model (Wittke, 1977). The validity of a quasi-continuum model is predominantly determined by two parameters: the joint pattern, and the discontinuity spacing.

A random pattern, that includes multiple joint sets, justifies more a continuum rock mass model than if for instance a parallel texture was prevalent. As for the spacing of discontinuities, their magnitude relative to the size of the tunnel is of relevance. Among other factors, this relationship determines the freedom of the rock blocks to move. If the spacing is small in

comparison with the tunnel dimension, a continuum model is more likely to simulate real conditions.

Modeling of Discontinuities: Both continuum and discontinuum models can include discontinuous features of the rock mass. Hence, discontinuities can be modeled with any of the numerical methods described in this chapter. Parameters of relevance for tunnel engineering are: discontinuity type, scale and orientation relative to the analyzed tunnel, spacing, frequency, openness, roughness, fill material, strength and deformability, hydraulic properties, and time dependent response. These parameters are more or less difficult to anticipate. Therefore, the modeling of specific discontinuities is only rarely attempted, except for major structural features, and is not discussed here in detail.

Modeling of a Quasi-Continuum Rock Mass: The parameters of a quasi-continuum rock mass are governed by the characteristics of the intact rock and the discontinuities. Figure 3-9a displays a qualitative relationship between the degree of rock mass fracturing, expressed in RQD, and the rock mass deformability. Depending on the RQD value, the rock mass elasticity modulus, E_r , amounts to approximately 20% to 60% of the modulus of a respective intact rock core (Cording, 1975) (Heuze, 1980). Similar relationships are the case for strength parameters. If Mohr/Coulomb or a similar failure criterion is used, rock mass strength is expressed in terms of cohesion and friction. Figure 3-9b qualitatively illustrates how strength parameters of a rock mass may be reduced from those for the intact rock (Wittke, 1977). The figure illustrates the effect of a random discontinuity pattern versus that of a parallel discontinuity texture.

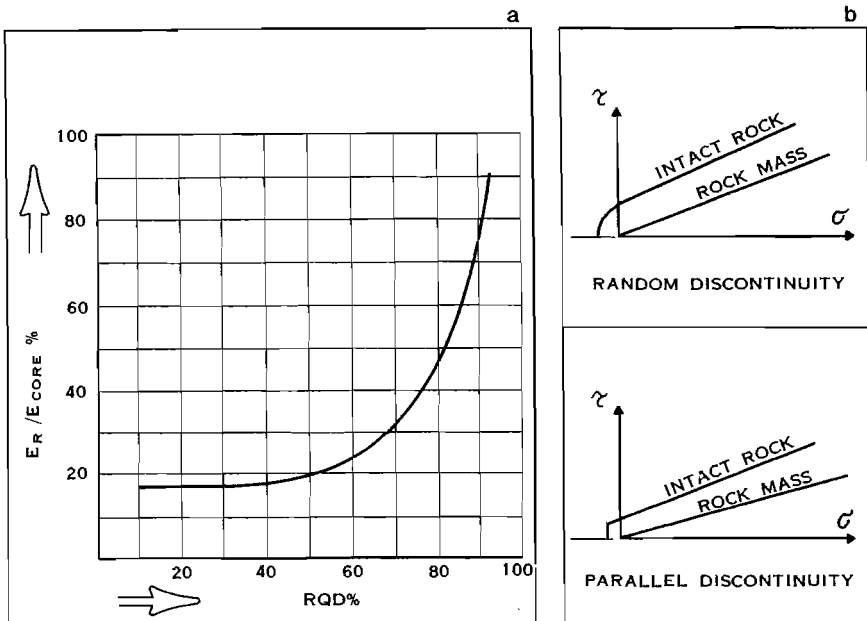


Fig. 3-9. Rock mass properties. (a) deformability modulus. (b) strength.

The constitutive law to be used to simulate rock mass behavior is largely determined by discontinuity characteristics. The selection shall also be tied to the stress level anticipated in the ground. For instance, for a random texture, when stresses well below the corresponding strength are induced, a linear elastic isotropic stress/strain law can be justified. In contrast, a non-linear ground behavior must be allowed if stresses on the order of magnitude of the rock mass strength are anticipated.

The quantitative determination of quasi-continuum rock mass parameters for numerical computations requires large experience of the engineer. An example analysis by Mussger (1984) is illustrated below that reflects a real tunnel situation.

Calculation Example: Jointing characteristics were anticipated to vary significantly along the subject tunnel alignment. Grouping of rock mass parameters was found necessary in order to allow for greater modeling accuracy. Three groups were defined to reflect various rock mass jointing: highly jointed rock (HJ),

medium jointed rock (MJ), and relatively sound rock (R). In order to reduce the calculation effort and to facilitate the allocation of results to actual tunnel locations, the three groups were summarized to two: highly jointed to medium jointed rock (HJ to MJ), and jointed to relatively sound rock (J to R). The respective shear envelopes are qualitatively illustrated in Figure 3-10. Also, the figure shows several unlabelled curves of shear envelopes for comparable rock masses, determined through in-situ measurements or derived from empirical formulations. The shear envelopes required for the subject tunnel analysis were graphically found as the mean values of the applicable unlabelled curves.

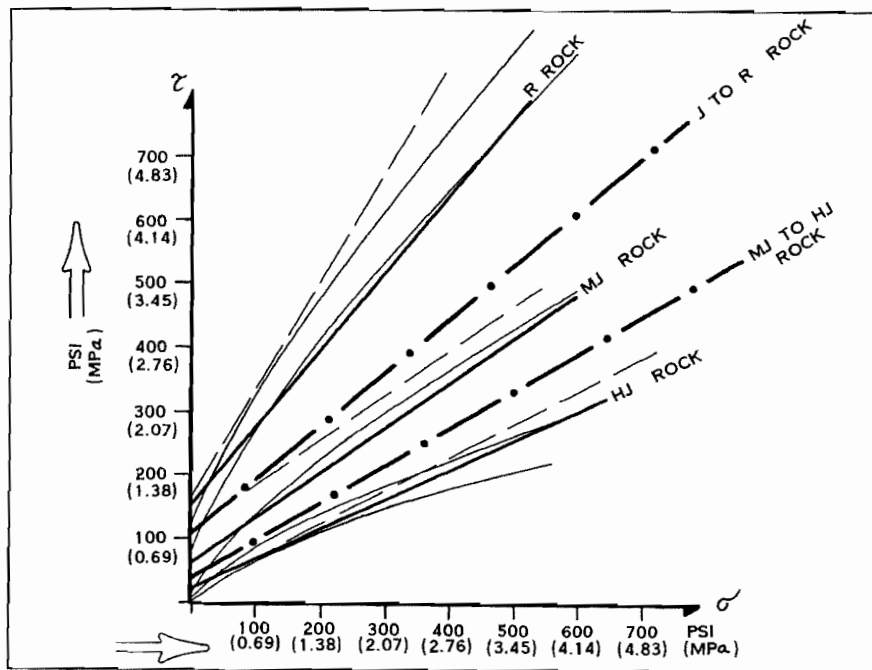


Fig. 3-10. Calculation example - rock mass strength.

The respective calculation parameters of the quasi-homogeneous rock mass are summarized in Table 3-2. Friction and cohesion values are directly taken from Figure 3-10. Deformation moduli were determined from Figure 3-9a. In this case, the RQD values for HJ, MJ and R rock mass types led to rock mass to intact rock elasticity ratios of 0.2, 0.35 and 0.6, respectively.

TABLE 3-2
Calculation example - Rock mass parameters.

| | HJ | MJ | R | MJ TO HJ | J TO R |
|---|--|--|--|--|--|
| DENSITY γ PCF (KN/M ³) | 170 (27.03) | 170 (27.03) | 170 (27.03) | 170 (27.03) | 170 (27.03) |
| POISSON'S RATIO ν | 0.23 | 0.23 | 0.23 | 0.23 | 0.23 |
| MODULUS OF DEFORMATION PSI (MPa) | 0.8×10^6 (5.5×10^3) | 1.4×10^6 (9.6×10^3) | 2.5×10^6 (1.7×10^4) | 1.1×10^6 (7.6×10^3) | 1.7×10^6 (1.2×10^4) |
| SIDE PRESSURE COEFFICIENT K_0 | 0.3 | 0.3 | 0.3 | 0.3 | 0.3 |
| FRICTION ANGLE ϕ | 25 | 35 | 50 | 30 | 40 |
| COHESION C PSI (MPa) | 30 (0.21) | 70 (0.48) | 170 (1.17) | 40 (0.28) | 110 (0.76) |

(ii) Modeling of a Soil Subsurface. Modeling considerations for a soil subsurface may differ somewhat from those for a rock mass. For a rock mass, discontinuities pose the prime problem to the model. By comparison, no distinct discontinuities are typically encountered in soil. Instead, the "intact" soil is often difficult to describe. Problems commonly associated with modeling of the soil subsurface are:

- o The variability of soil parameters obtained from testing is often too high to determine true values. Substantial efforts have been made in recent years to develop constitutive models for soil. By comparison, the reliability of

material constants determined from experimental data has not been addressed adequately (Zaman et al., 1988).

o Soil parameters may vary with time due to changing subsurface conditions. Changing conditions may relate to creep effects or to the impact of ground water. For instance, groundwater lowering during tunnel construction affects the water content and the related soil characteristics. Figure 3-11 schematically depicts a tunnel profile at face proximity. For the example described in the figure (Gnilsen, 1987), groundwater lowering is performed prior to excavation resulting in "partially drained" soil condition. Additional drainage of the soil surrounding the tunnel occurs due to the excavation process. In the figure, "partially drained" conditions are assigned to phase one of the calculation process, relating to the area at and ahead of the tunnel face. With progressing tunnel advance, the tunnel face moves beyond this area, i.e. the area is subject to additional groundwater drainage into the tunnel opening. The resulting "drained condition" is analyzed in phase two. For the described calculation example, phases one and two are also tied to different loading and material parameters. In particular, the increasing strength of the shotcrete lining as a function of time, and changing loading conditions as a function of distance from the tunnel face, are taken into account.

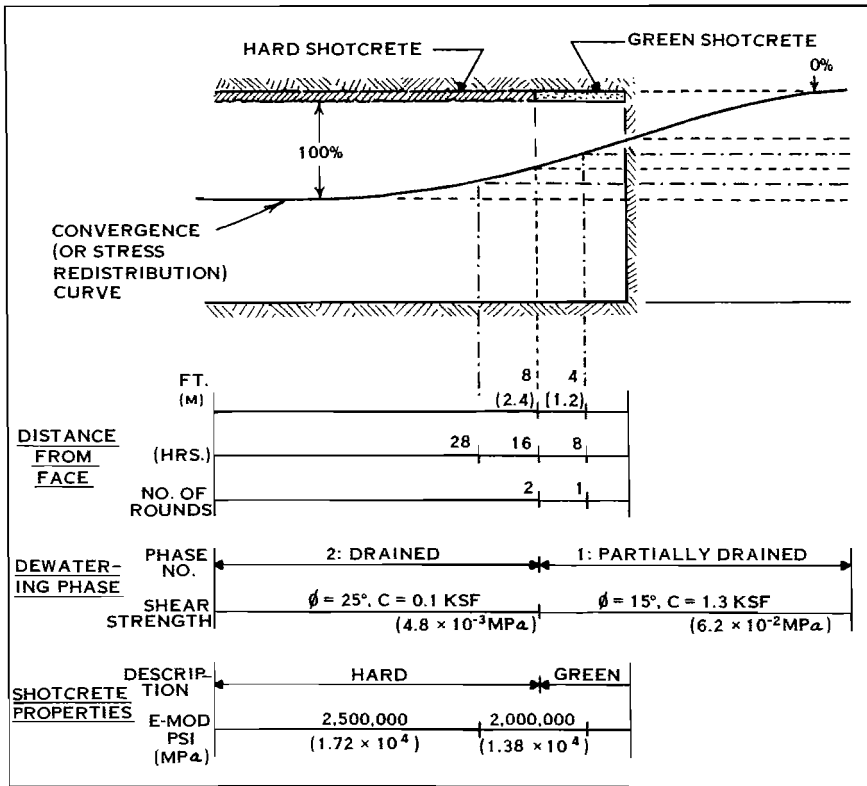


Fig. 3-11. Calculation example - tunnel in soil.

o Changing loading and stress conditions in the soil also relate to the rheologic behavior where encountered. Changing and complex soil response under complex loading conditions represents another difficulty of modeling the soil subsurface.

(iii) Modeling of Subsurface Irregularities. Modeling of discontinuities in a rock mass is described above. Other types of irregularities are common to rock and soil subsurface. For instance, an analyzed section may include ground areas with parameters that differ from those of the remaining section. Such area is referred to as non-homogeneity. Since their location and extent are mostly unknown, parameter studies may be performed.

Figure 3-12 depicts a parameter study performed by Mussger (1984) which analyzed ground stresses and lining deformation.

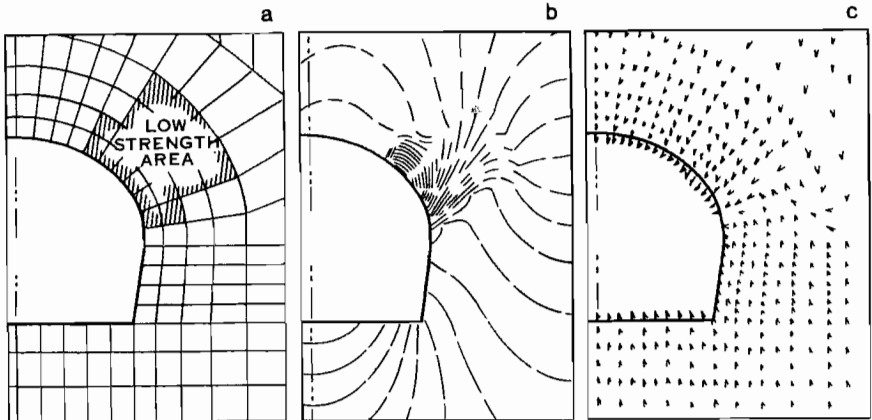


Fig. 3-12. Computation with non-homogeneous host ground. (a) Ground model. (b) Computed Stress Contours. (c) Computed displacement vectors.

3-7.2 Modeling Tunnel Advance and Construction

If realistic results are to be obtained from the numerical analysis, actual construction measures and their sequence must be taken into account. In particular, the excavation and installation of support and lining are important tunnel construction parameters. On the other hand, as discussed before, a simplified model will be necessary.

Different construction approaches and methodologies entail different excavation, support, and lining characteristics. For instance, mechanized full face excavation by means of tunnel boring machine or shield is typically associated with tunnel linings different from those used with sequential tunnel excavation and support. The simulation of tunnel advance and construction must differ accordingly (Kasali and Clough, 1983). Yet, no differentiation is made for the following discussion since generic assumptions are made.

Common to all tunneling is that a three-dimensional structure is created at the tunnel face. Accordingly, stresses and deformations occur in three dimensions. The three-

dimensionality at the tunnel face is schematically shown in Figures 3-6 and 3-13. Regardless of the distance away from the face, arching of the ground develops perpendicular to the tunnel axis. In addition, arching parallel to the tunnel axis is shown at the face. Arching in two or more planes creates three-dimensional load transfer. The simplification necessary with the numerical analysis most often includes a reduction to a two-dimensional model. This is discussed in the following.

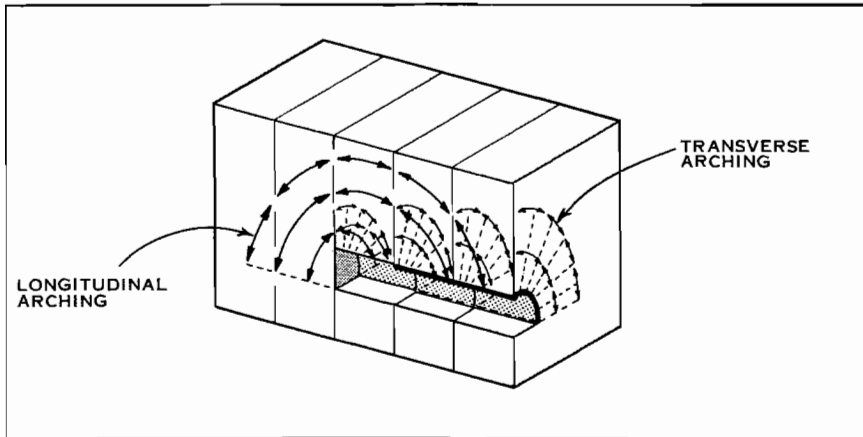


Fig. 3-13. Three-dimensionality at the tunnel face.

Three-dimensional stress and loading conditions at face proximity relate to the stiffness of the ground ahead and around the excavated tunnel. Stiffness is reduced close to the excavated area, and zero where soil has been removed. Understanding the impact of stiffness on ground stress and loading conditions allows for the formulation of a simplified model. Stiffness and stresses are the parameters used to simulate three-dimensionality in a two-dimensional model (Schwartz and Einstein, 1980), (Laabmayr and Swoboda, 1978). Figure 3-14 summarizes the two methods and is used for reference in the following discussion.

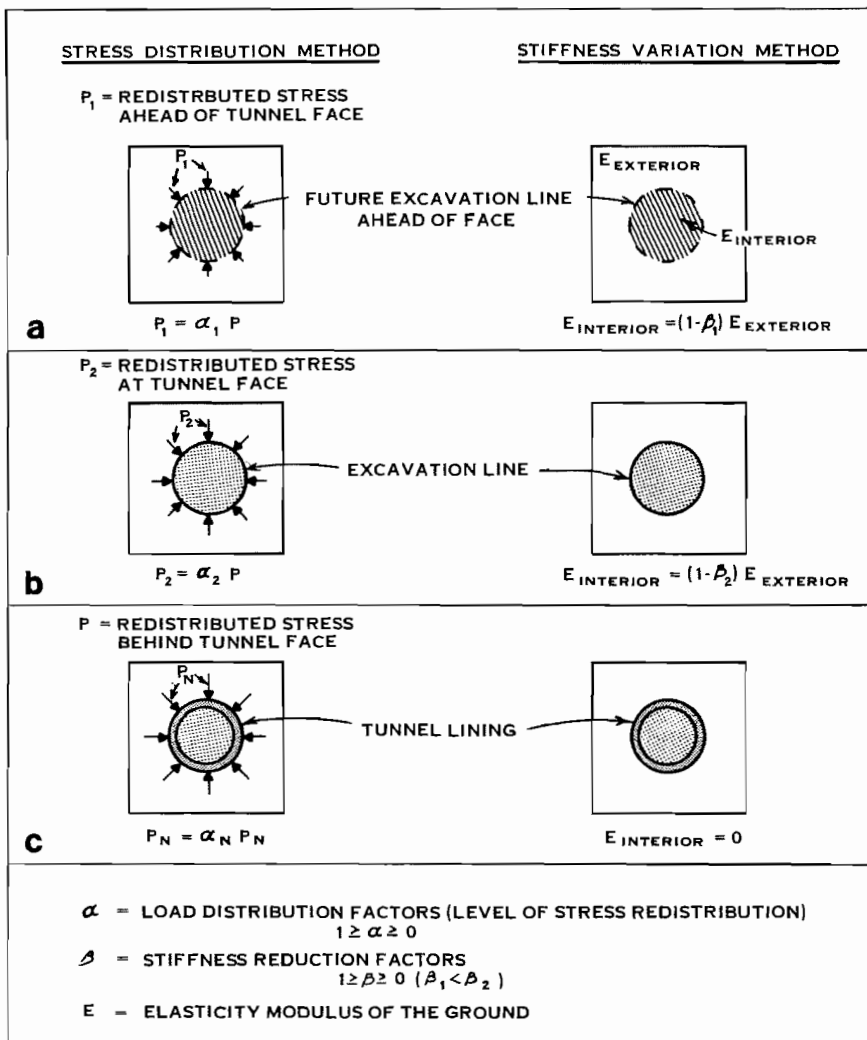


Fig. 3-14. Stress distribution and stiffness variation method - summary.

(i) Stress Distribution Method. Stresses vary within and beyond the area of three-dimensional arching at face proximity. The location of a two-dimensional section used for simplified tunnel analysis lies initially ahead (Figure 3-14a), then at (Figure 3-14b), and finally behind (Figure 3-14c) the advancing tunnel face (see Figure 3-15a).

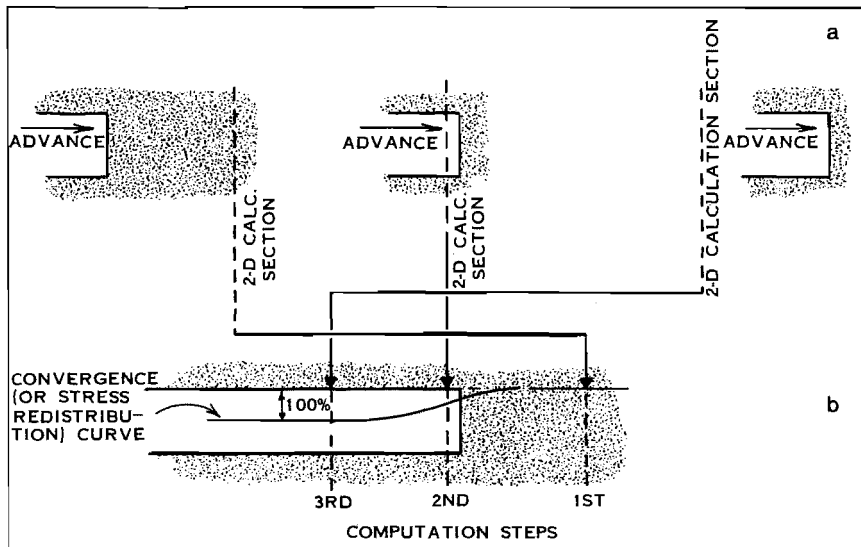


Fig. 3-15. Two-dimensional analysis of three-dimensional tunnel advance - calculation example. (a) True condition with advancing tunnel face. (b) Calculation assumption with increasing stress redistribution at subsequent computation steps.

Figure 3-15b shows the corresponding calculation assumption. In the figure, the curve approximates the variation of stress occurring ahead, at and behind the tunnel face. As shown, stress distribution caused by the tunneling process starts at some distance ahead of the face and is fully accomplished at some distance behind it. The curve shown in the figure is governed by the assumption, that the level of stress mobilization corresponds to the observed displacement in the tunnel. That is to say, that zero displacement relates to zero stress distribution from the tunneling process, and final displacement relates to final stress

distribution. The degree of stress distribution is described by the multiplication factor α . In Figure 3-14, the factors α_1 through α_3 are increasing numbers, smaller than one. In this figure, "p" denotes the finally distributed stress level, with α_3 indicated to equal one.

(ii) Stiffness Variation Method. Alternatively to directly using stress parameters, stress distribution can be determined from stiffness variation. Stiffness variation implies stiffness reduction or stiffness increase of the ground or the lining, respectively. Stiffness reduction of the ground relates to the excavation process. Stiffness increase may be considered as lining elements are introduced. As lining (e.g. shotcrete or grouted segments) increases in stiffness with time, the lining will attract stresses.

The initial stiffness or elasticity modulus, E, of the ground prior to excavation is denoted in Figure 3-14. The reduction of the ground stiffness with nearing tunnel excavation is described by the stiffness reduction factor β . Similar to the correspondence assumed between stress and displacement, the stiffness reduction factor is brought into agreement with Figure 3-15b (Laabmayr and Swoboda, 1978), (Schikora, 1982).

3-8. MODEL VERSUS MEASUREMENT

Data obtained from the numerical computation and verified during and after tunnel construction include: displacement of the surface (settlement), subsurface and tunnel lining; and stresses in the ground, in the lining, and between ground and lining.

The comparison of calculated and measured values is discussed by Bauman (1988), Schikora and Ostermeier (1988), Golser and Hackl (1985), and other authors. One or several of the following purposes may be served:

- o Verification of the design
- o Interpretation of measured values. The corresponding calculated values provide a reference for assessing the significance of in-situ measurements.
- o Validation and calibration of the numerical model.

Model validation and calibration are often necessary to evaluate the effect of model simplification, and to verify the

design assumptions including the material and ground parameters used. These factors have been discussed in Section 3-7. In particular, reference is made to the discussion on the simplified simulation of three-dimensional conditions at face proximity by means of the stiffness reduction or load distribution method (Laabmayr and Swoboda, 1978). Without the experience and understanding gained from tunnel measurements, the proper selection of stress or stiffness parameters is virtually impossible.

The achievement of satisfactory agreement between measured and calculated results may also require model adjustments to account for specifics of the method or model, or shortcomings of the computer program. For instance, unloading of the ground due to tunnel excavation may locally exaggerate the "softness" of the computer model, calling for local increase of ground stiffness beyond actual values. Such "manipulation" may be a legitimate means to the experienced user.

Unless the numerical model is validated or model-specific adjustments are made where necessary, the accuracy of calculation results is difficult to determine. While certain results may deviate considerably from actual measurements, other values may at the same time concur with reality. Validation of the numerical model has been described before in this chapter. For example, the discussion on Back Analysis deals with this subject.

Besides model validation and model adjustment, calculation results may still require interpretation to eliminate impacts from the numerical process. For instance, Figure 3-16 depicts model characteristics that may cause the numerical process to exaggerate true conditions: Corners in the tunnel circumference, or inter faces of ground layers with different stiffness. While either feature can attract stresses in nature, a much larger, unrealistic stress concentration may result from the numerical computation.

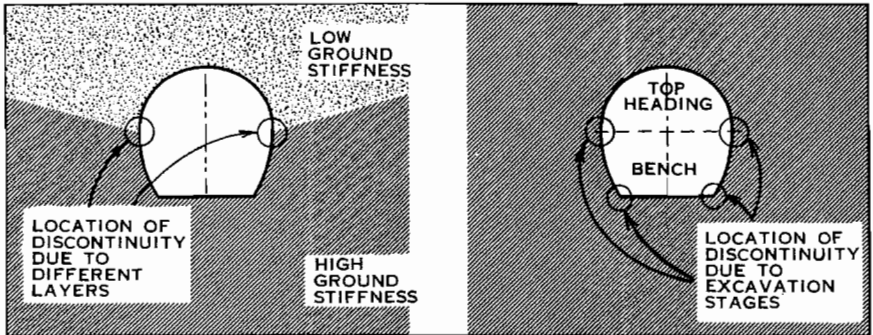


Fig. 3-16. Model characteristics that require result interpretation.

Result interpretation will be facilitated by comparative studies (see Section 3-3), experience of the user, and the understanding of the engineering problem as well as the computer program.

A discussion of qualitative and quantitative calculation results was included in Section 3-3. A comparison of model versus measurement can fit either category. Figure 3-17 illustrates an example analyzed by Gnisen (1986) where both qualitative and quantitative computation results are obtained from a Finite Element analysis, and are compared with observations in the tunnel. The qualitative result, that is difficult to measure or validate in absolute terms, is the distribution of stress concentrations in the ground. In Figure 3-17a, dots indicate the areas where stresses exceeded the elastic limit. The distribution of stress concentrations along the tunnel circumference can be qualitatively verified by observing stress concentrations along and within the tunnel lining. This leads to the quantitative result that is calculated and compared with measurements. Figure 3-17b shows the axial forces and moments in the lining that are induced from the ground stresses in Figure 3-17a. Axial forces and bending moments are converted to lining stresses that can be measured in the tunnel.

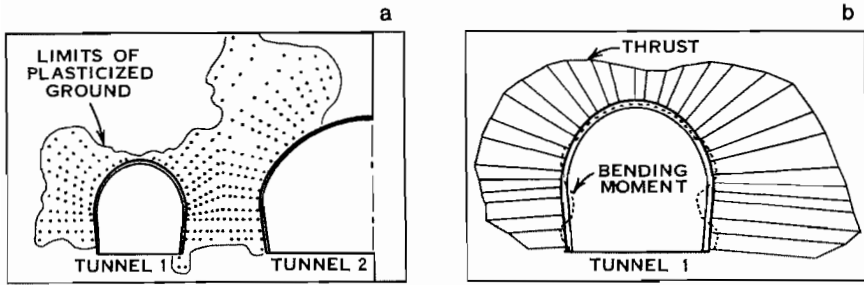


Fig. 3-17. Calculation results - example. (a) Qualitative result: plasticized ground area. (b) Quantitative result: bending moment and thrust in the lining.

Another example of qualitative and quantitative computation results compared with in-situ observations is illustrated in Figure 3-18. The figure shows calculation results and measurements of a tunnel subjected to an earthquake effect. Figure 3-18a compares the original tunnel shape with that observed after an earthquake on the Inatori Tunnel in Japan (Yoshikawa and Fukuchi, 1984). By comparison, Figure 3-18b illustrates the calculation results from the quake simulation. Good correlation between calculated and instrumented results could be found.

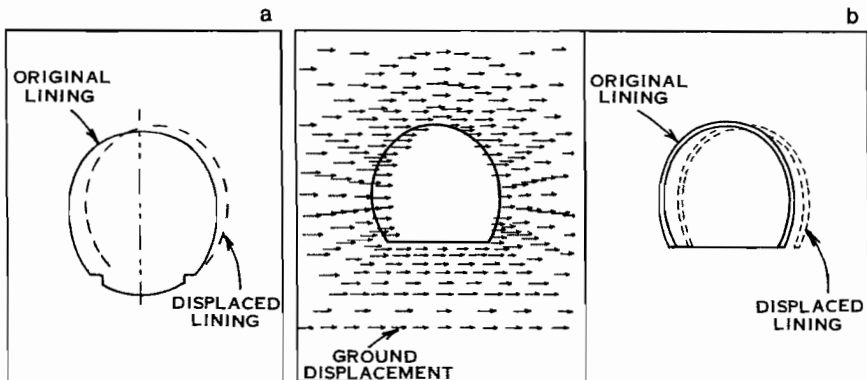


Fig. 3-18. Calculation versus measurement - example 1. (a) Observed tunnel before and after earthquake. (b) Computation results.

An example where interpretation is necessary to explain differences between numerically calculated and measured data is depicted in Figure 3-19. In this case, the settlements computed from the Finite Element analysis are well below the actual values. The interpretation must take into account that no consolidation effect was considered by the numerical computation.

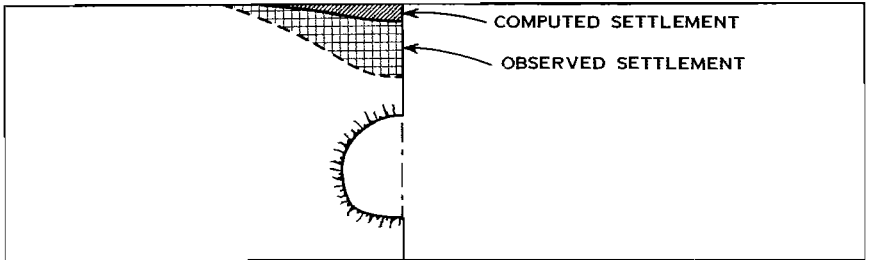


Fig. 3-19. Calculation versus measurement - example 2. Computed versus observed settlements.

By comparison, close concurrence of calculated and measured surface deformations and lining forces is described by Schikora (1982) for the example illustrated in Figure 3-20.

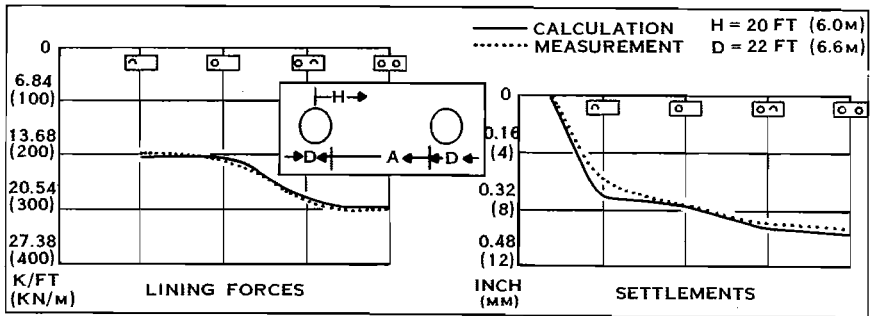


Fig. 3-20. Calculation versus measurement - example 3.

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Chapter 4

ROCK REINFORCEMENT

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4-1 GENERAL

Essentially, rock reinforcement is analogous to concrete reinforcement (Sinha and Schoeman, 1983). Both concrete and rock are strong in compression but weak in tension. While concrete reinforcement supplements the lacking tensile strength of plane concrete, the rock reinforcement enhances the performance of rock mass as a construction material. The rock reinforcement somewhat controls the deformation of the rock mass toward the excavation opening, counteracts the loosening of the strata and, in forms of rock bolts, introduces prestress into the rock mass. This introduction of prestress increases the surficial frictional forces on the discontinuities of rock mass between the individual rock mass units. The increased interjoint friction increases the shear strength, stiffens the roof of excavation thereby augmenting the carrying capacity of the roof, and preserves the keying action of the joint blocks.

Though an Appalachian miner developed the rock bolt in 1870, only since 1922 have rock bolts been used in the USA to ensure the stability of excavation. Some 90 million rock bolts alone were used in the USA in coal mining in the year 1978. In 1986, Atlas Copco estimated (Lock, 1988) that 50 percent of all rock bolting was performed by hand held drilling equipment, 40 percent by mechanized drilling and manual bolting, and the remaining 10 percent was fully mechanized bolting. In civil engineering works for tunneling, shaft driving, and cavern excavating, the use of mechanized bolting is increasing very fast.

The various forms of rock reinforcements are rock studs, rock anchors, rock bolts, split sets, Swellex bolts, and cable bolts. One end of the rock reinforcement has a device which allows the reinforcement to be anchored in the hole. The other end, designed to stay near the excavated face, is fitted with a surface plate which bears against the rock face. Rock bolts are always tensioned; whereas, rock anchors, rock dowels, and studs remain untensioned until loaded by the rock.

Rock studs are basically steel rods that are threaded at both the ends. Rock anchors are similar to rock bolts and are not prestressed or tensioned after

installation. Both the rock studs and anchors require rock movements to become active, otherwise they remain passive and do not take any rock load.

Rock bolts are prestressed or tensioned immediately after installation. Rock bolts, therefore, compress the rock strata and actively share the rock load. Because rock creeps under loading, under the rock loads, the rock bolts may also creep and may loosen the initial prestress with time. This gradual loss of tension in the rock bolts is detrimental to excavation stability and may require retensioning of the rock bolts at a later date or may require a secondary system of rock bolting.

The materials used for rock reinforcement vary widely: timber, fiberglass, steel, and polyester resins. Polyester resins constitute pumped rock anchors that have a tensile strength of nearly 8,000 lb/in² (55 MPa) and a bond strength of nearly 2,000 lb/in² (14 MPa). They are very convenient where, due to space restrictions, rigid long anchors or bolts cannot be used. Reinforcing cables can be used instead of long anchors but pumpable rock anchors sometimes serve the purpose better.

4-2 ROCK REINFORCEMENT

The different types of rock reinforcement can be divided into three classes: mechanically anchored, resin or cement bonded, and frictional. Slot and wedge (not used any more in the USA), or expansion-shell type bolts have mechanical anchors which are installed in the interior of the rock mass and are activated by a pulling, pushing, rotating, or exploding mechanism from the surface. Resin or cement bonded rock bolts or anchors rely on the bonding strength of resin or cement to transfer the rock loads. These bonding agents also provide protection against corrosion of steel, which is considered very desirable. Franklin and Woodfield (1971) found that polyester resin bonded rock bolts were 1.7 to 3.0 times stronger than mechanical rock bolts and required very little displacement of rock strata to transfer the load. Frictional effects between rock reinforcement and rock mass are relied upon for transferring loads in split sets or split tubes (Scott, 1980) and Swellex rock bolts (Atlas Copco, 1983).

The most commonly used rock reinforcement is either tensioned (active) or untensioned (passive). Untensioned rock reinforcement usually transfers the load by suspension and is suitable in somewhat seamy rocks and where small displacements are not a major concern. Where it is necessary to minimize rock loosening, one resorts to tensioned rock reinforcement or installs the untensioned rock anchors as soon as practical.

4-2.1 Split Sets

Split sets are hollow slit cylindrical tubes of adequate thickness. The

longitudinal slit is about 5/8-inch (15 mm) wide. To facilitate entering into the drilled hole, one of the ends of the split sets is tapered and swagged. On the opposite end of the swagged end, a formed ring is welded to the tube to support the surface plate. The drilled hole length for the split sets is about 2 inches (50 mm) longer than the length of split sets. The drilled hole diameter is slightly smaller than the diameter of the split set. When the tapered end of the split set is inserted into the drilled hole and the split set driven into, the hole acts as a die compressing the tube to the size of the hole and partially closing the slot in the process. To be effective, the slot must remain open at least 1/8 inch (3 mm). The compression of the split set tube creates radial forces on the rock thus increasing the frictional forces at the interface of the tube and rock. The increased frictional forces provide stability of the rock mass and prevent rock layers from separating. The split tube rock reinforcement is shown on figure 4-1. Split sets are to be considered only as temporary reinforcing measures in corrosive environments.

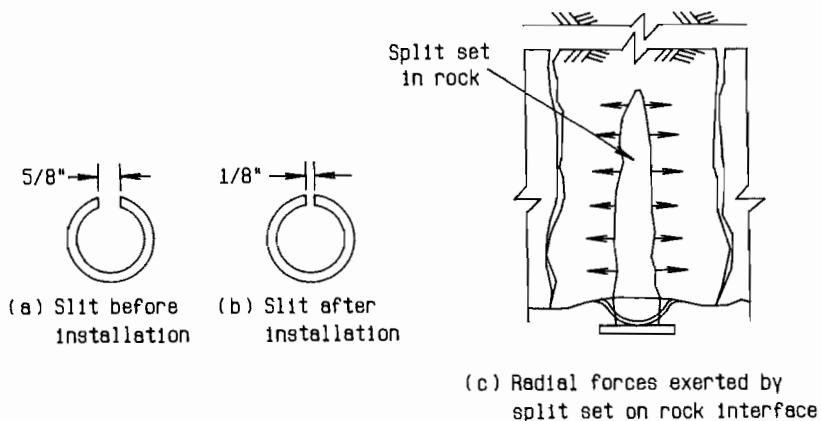


Fig. 4-1. Split Sets.

4-2.2 Swellex Bolts

The Swellex bolts are made out of folded steel tubes (Atlas Copco, 1983) which have an original diameter of 1-39/64 inches (41 mm) and are shown in figure 4-2. The tubes have closed ends. One of the ends carries an orifice through which water under pressure can be introduced which expands back the folded steel tubes to their original shapes. The expanded tubes then tightly fit against the drilled holes. The Swellex bolts adapt to relatively large variations in the drill hole diameters. Use of higher pressure may overexpand the Swellex bolts and may induce additional new fractures in the host rock

which may be undesirable. Swellex bolts are to be considered only as a temporary reinforcing measure in a corrosive environment.

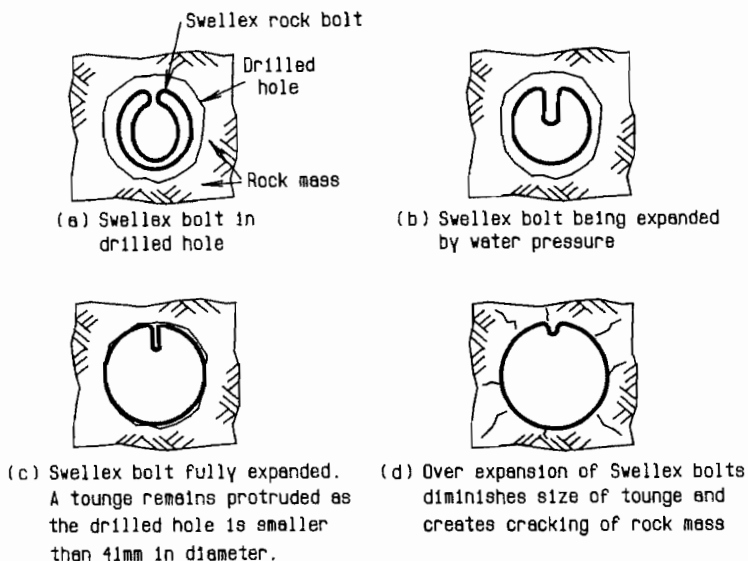


Fig. 4-2. Swellex Bolt.

4-2.3 Cable Bolts

Cable bolts are high strength stranded cables inserted and anchored in very long drilled holes. They are used where very long rock bolts are required.

4-2.4 Pumpable Rock Bolts

Pumpable rock bolts are made by pumping up materials like polyester resins, reinforced thermoplastic resins, glass-filled polyesters, or epoxies that have high tensile modulus. The rock bolt strength of pumpable rock bolts is limited to a maximum value of 8,000 lb/in² (55 MPa).

The pumpable rock bolts are used in situations requiring very long bolts and where coupler connections of rock bolts are not feasible.

4-2.5 Yieldable and Flexible Rock Bolts

In some situations, yieldable and flexible rock bolts may be required. Yieldable or flexible rock bolts carry a yieldable or flexible portion which is threaded and connected with nuts and couplers to their rigid portions.

4-3 TYPES OF ROCK BOLTS

There are over 50 or more types of rock bolts that are available today. Some of the most commonly used rock reinforcements in forms of rock bolts and anchors are shown on figures 4-3 through 4-12. They are slot and wedge rock bolt (fig. 4-3); regular expansion anchorage--headed bolt (fig. 4-4); bail expansion anchorage--solid bolt (fig. 4-5); grouted smooth bar rock bolt with integral grout tube (fig. 4-6); hollow groutable deformed bar rock bolt (fig. 4-7); grouted end anchorage, pumpable type (fig. 4-8); perforated sleeve and mortar type (fig. 4-9); cement groutable rock bolt with resin anchor stop and integrable grout tube (fig. 4-10); grouted end anchorage, polyester resin (fig. 4-11); and threaded bar rock bolt with polyester resin grouted anchorage (fig. 4-12).

Slot and wedge expansion shell type anchors do not provide firm anchorage in several types of soft, decomposed, or spongy rocks. Rock bolts have been used in Gneiss, limestone, clay, and also in concrete. Cut threaded bolts are weaker than rolled threaded bolts. Mechanical anchors require about 2 inches (50 mm) of rock slip to develop strength. Resin anchors require only about 1/2 inch (12 mm) of rock slip to develop strength. Bolts with notched shanks give less rock deformation than those with smooth shanks.

4-4 ROCK REINFORCEMENT INSTALLATION

Installation basically consists of drilling holes, then placing and anchoring the rock reinforcement. Rotary drills usually provide better holes than percussion drills. Although percussion drilling provides good rough surfaces for increased anchorage, it is very difficult to keep the drill holes straight, especially for long bolts. For short bolts, 6 feet (2 m) or less in length, any type of drill can be used. Anchoring may be achieved by using mechanical, chemical, or increased frictional systems. In the USA, of the different bonding materials, the "resins" are being favored over the "cement."

The installation time, a function of standup time of a rock opening, should generally be within 3 hours of excavation and the bolts could be installed within 6 feet (2 m) of the advancing tunnel face. Doing so prevents the rock mass from loosening and makes rock reinforcement very effective.

In the case of tensioned reinforcement, an additional process of stressing the bolts is required.

The measuring of penetration speed of drilling and examining the drilling dust during installation can render valuable information on hardness and condition of the rock. This information may be used to verify design of rock bolts.

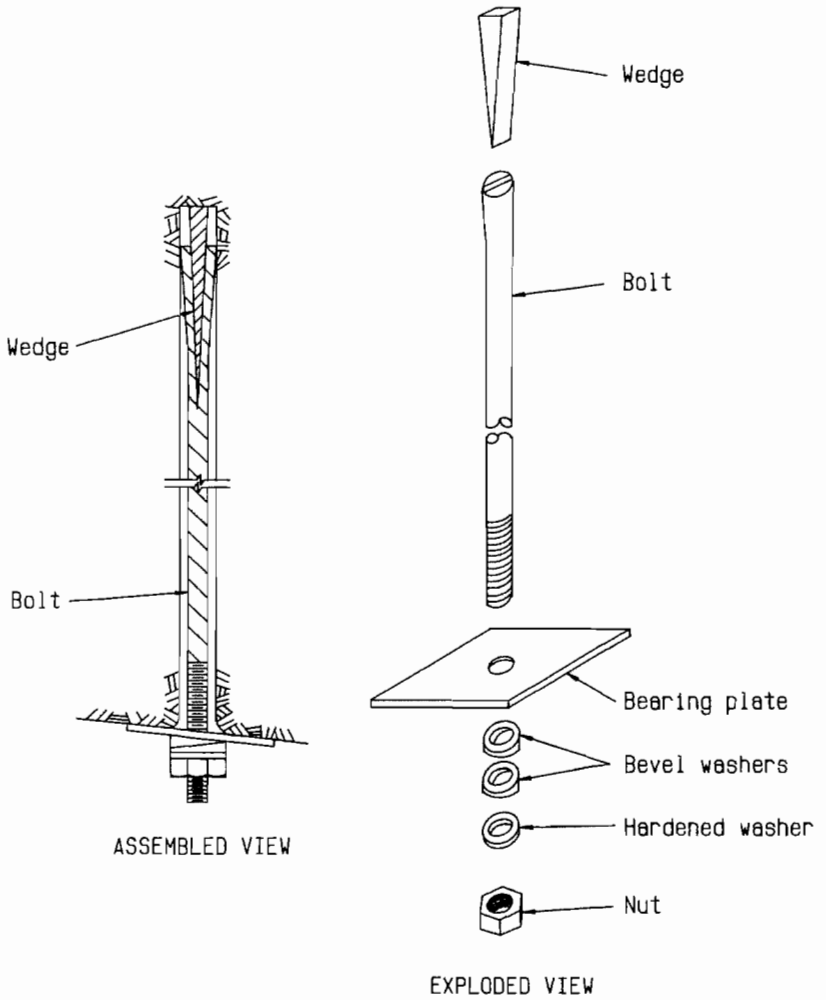


Fig. 4-3. Slot and Wedge Rock Bolt (U.S. Army, 1980).

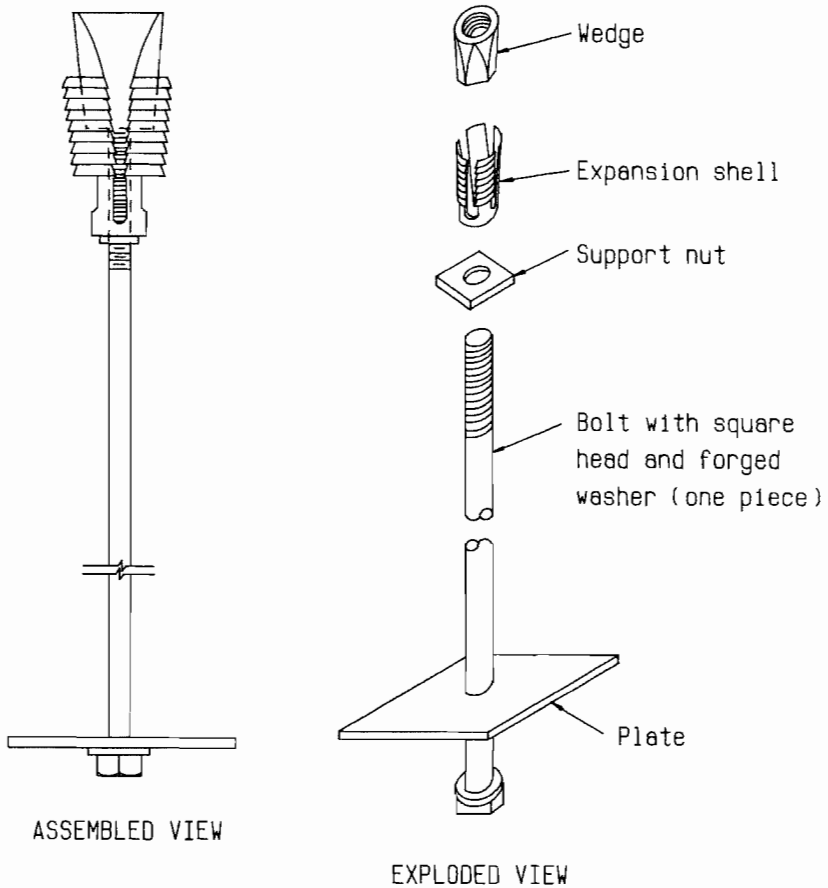


Fig. 4-4. Regular Expansion Anchorage--Headed Bolt (U.S. Army, 1980).

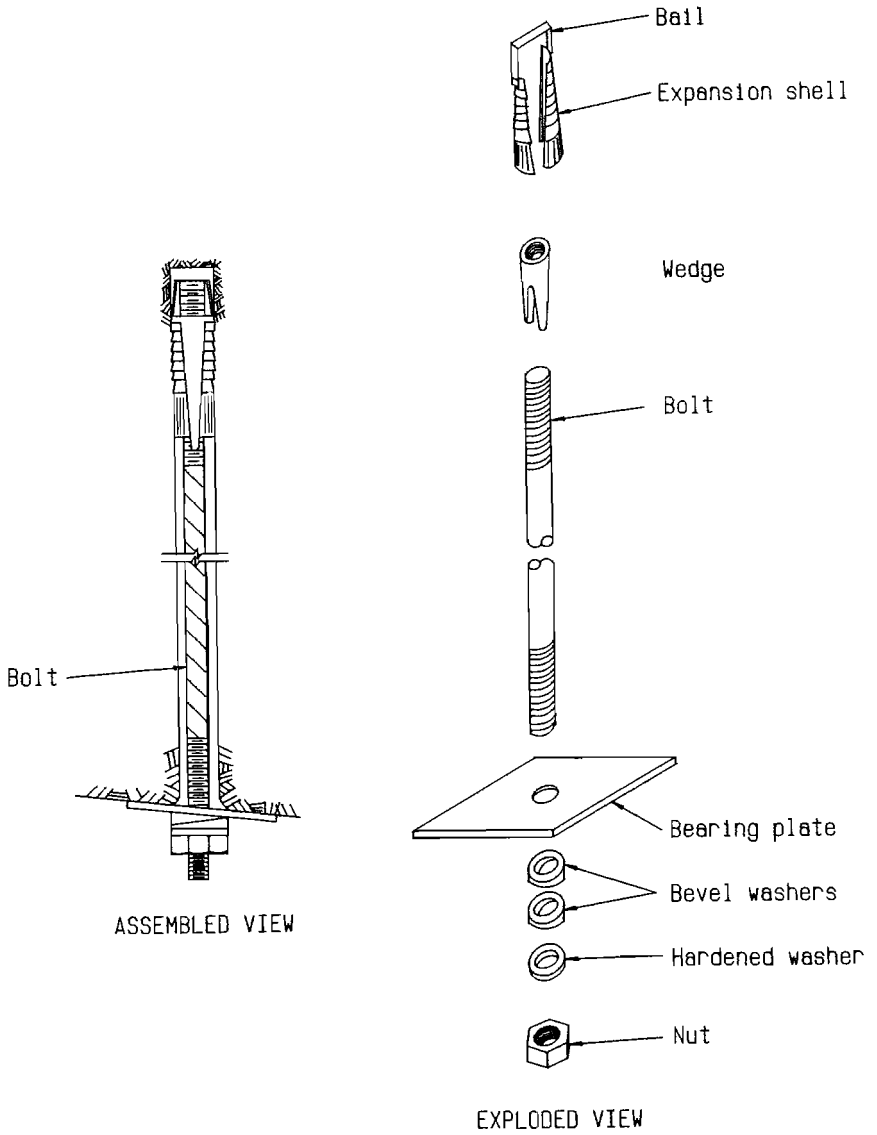


Fig. 4-5. Bail Expansion Anchorage--Solid Bolt (U.S. Army, 1980).

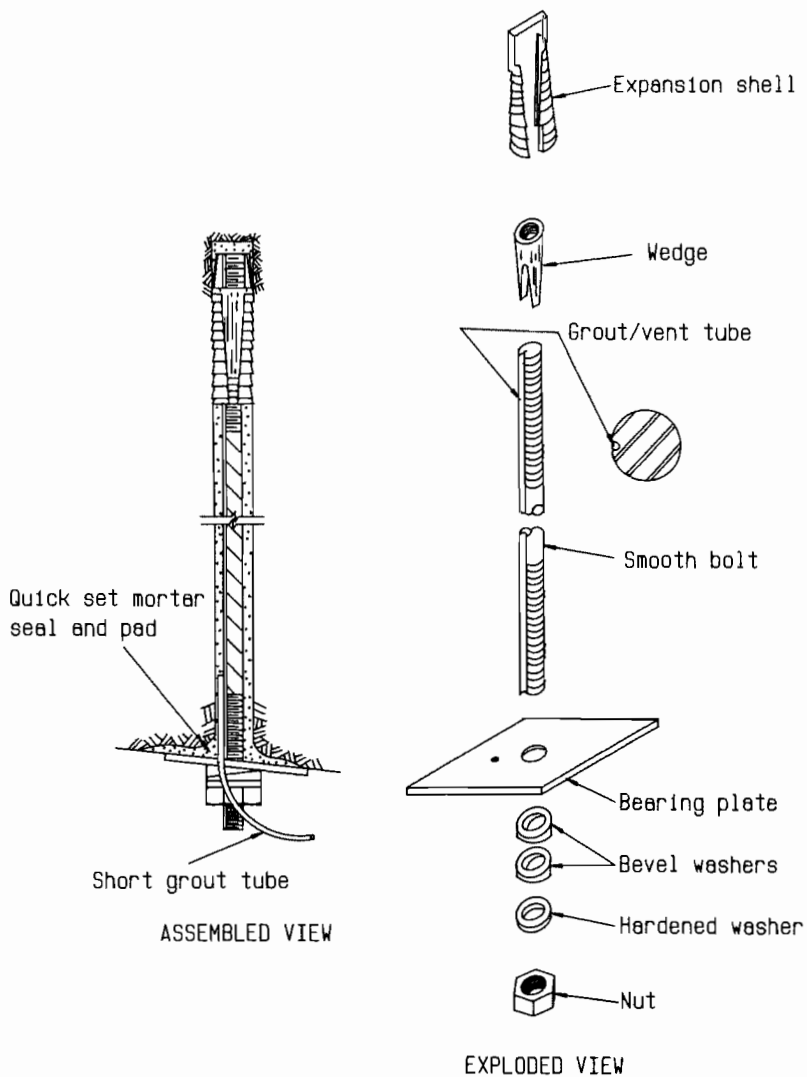


Fig. 4-6. Groutable Smooth Bar Rock Bolt With Integral Grout Tube (U.S. Army, 1980).

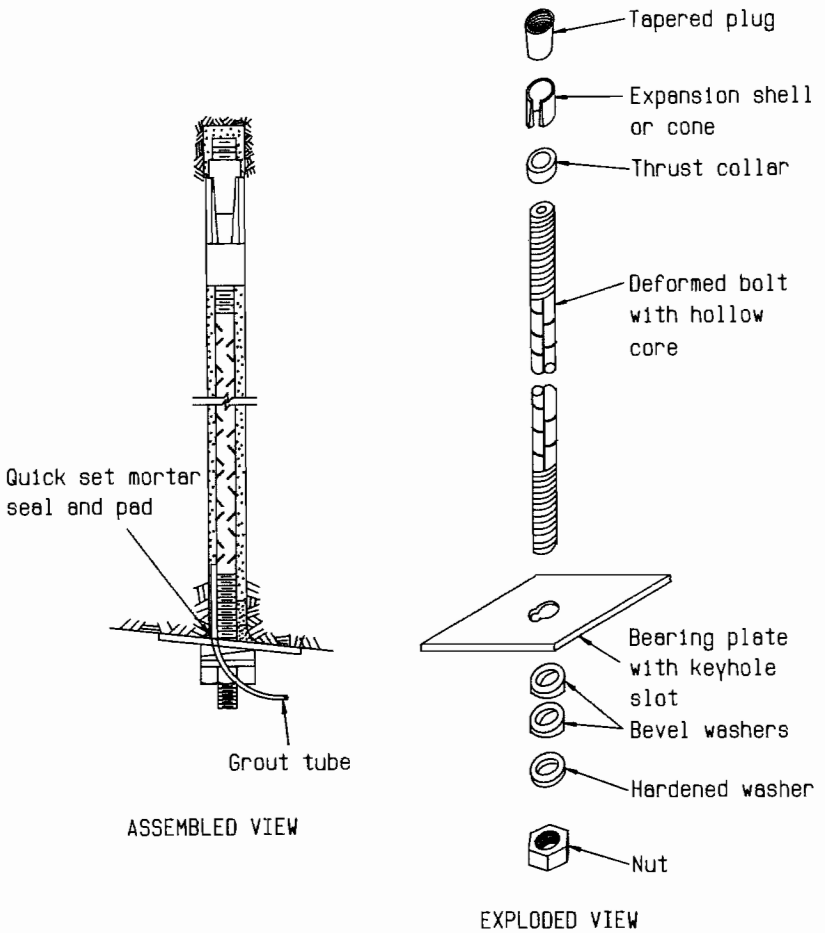


Fig. 4-7. Hollow Groutable Deformed Bar Rock Bolt (U.S. Army, 1980).

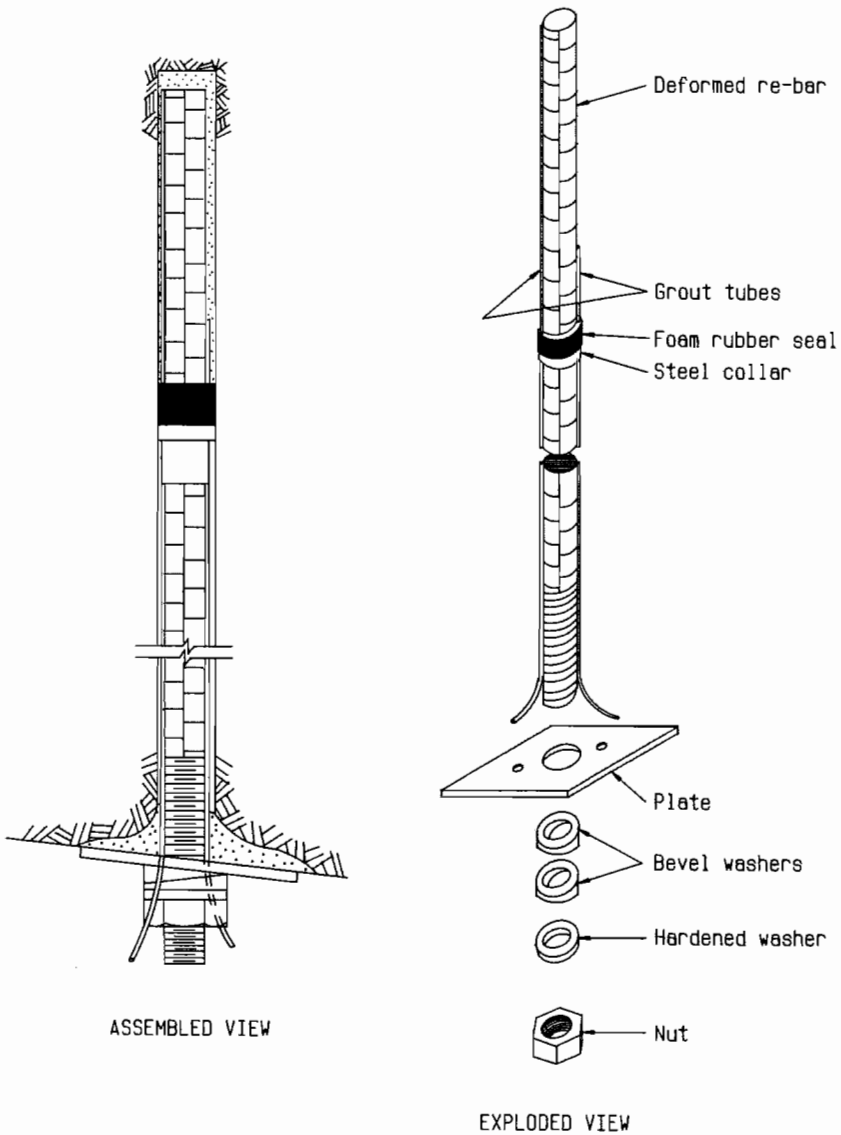


Fig. 4-8. Grouted End Anchorage, Pumpable Type (U.S. Army, 1980).

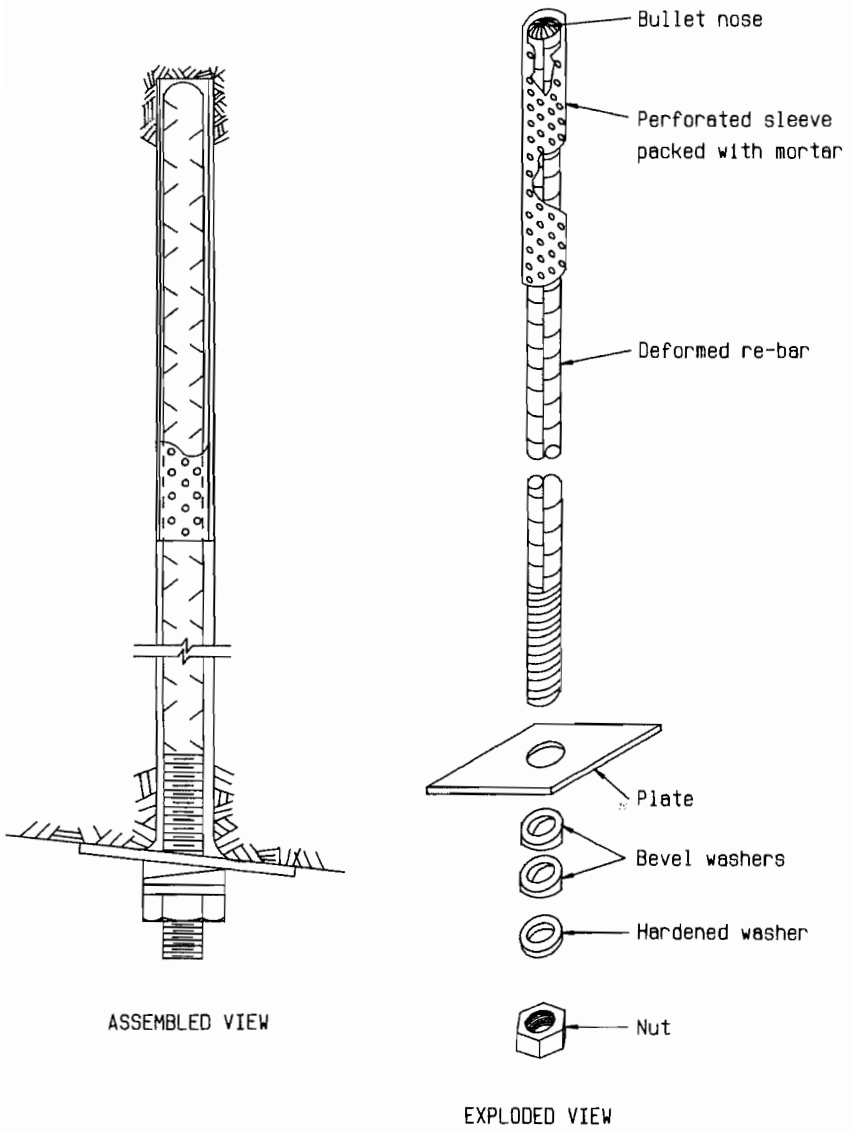


Fig. 4-9. Perforated Sleeve and Mortar Type (U.S. Army, 1980).

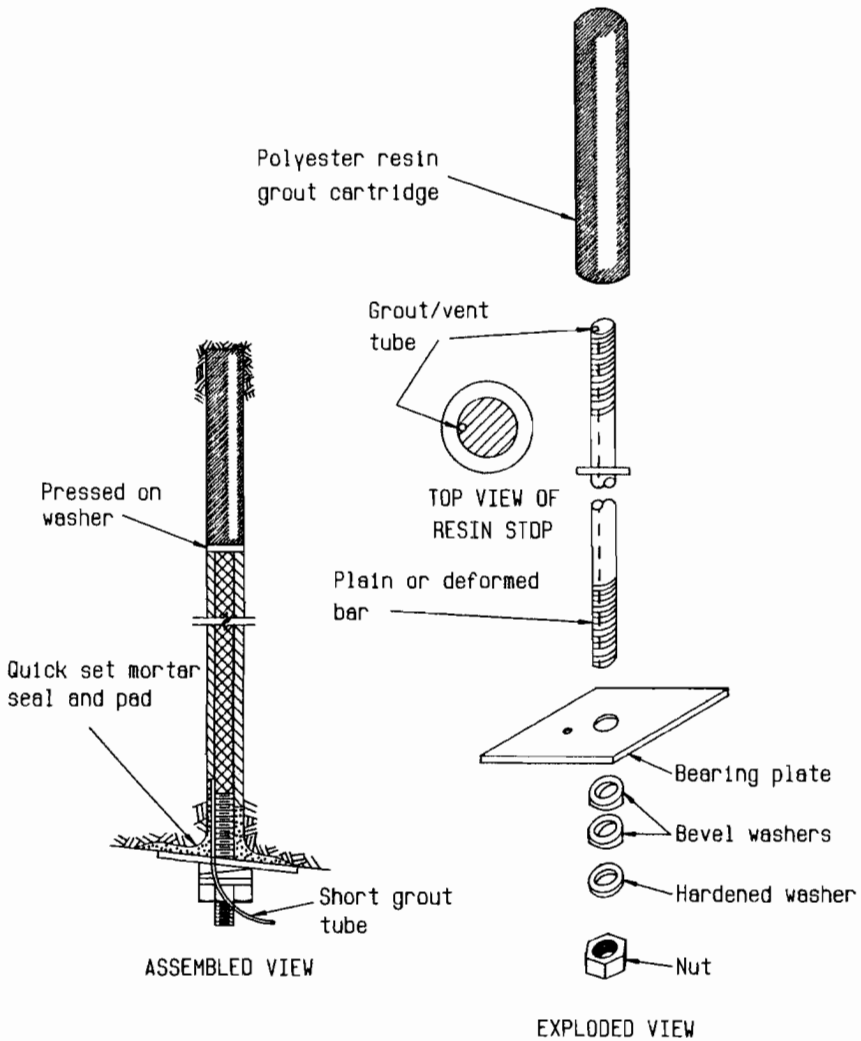


Fig. 4-10. Cement Groutable Rock Bolt With Resin Anchor Stop and Integral Grout Tube (U.S. Army, 1980).

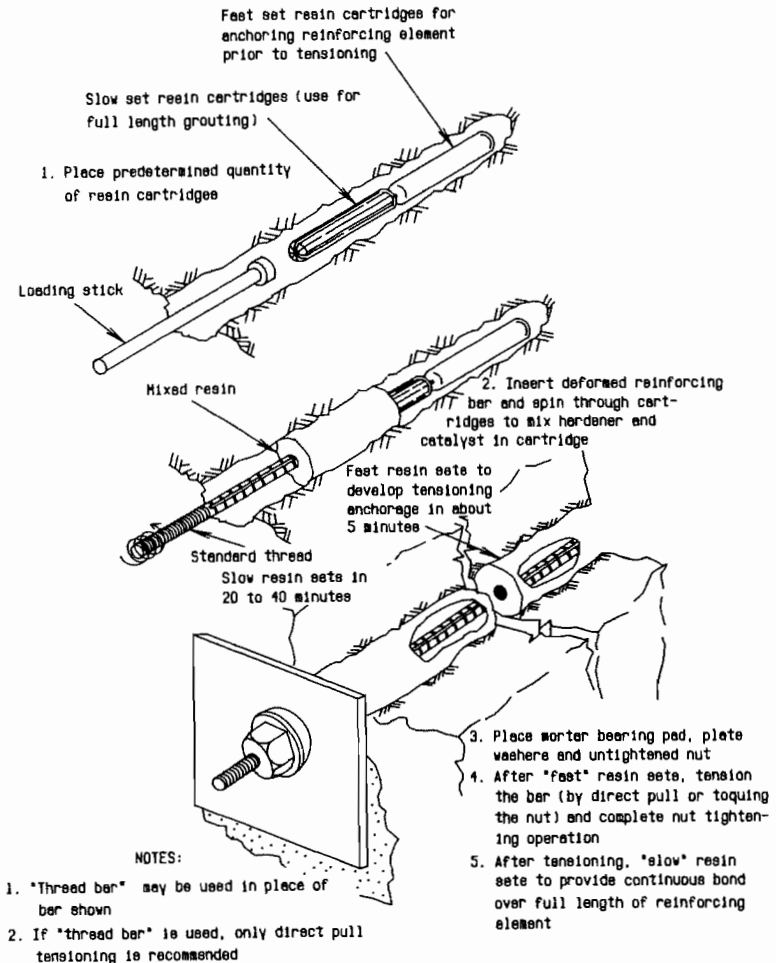
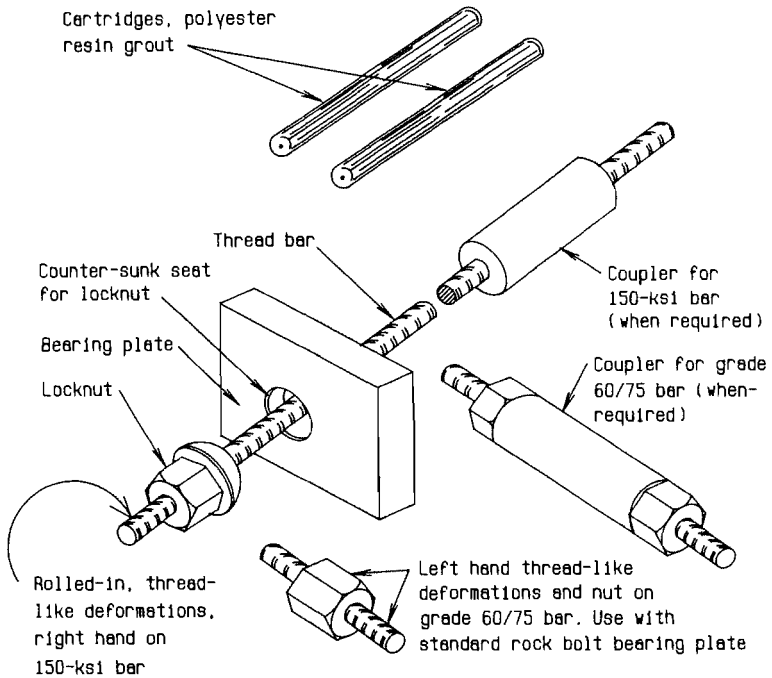
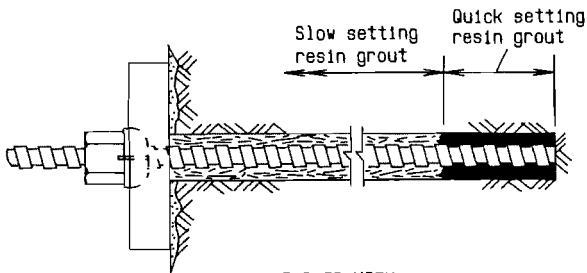


Fig. 4-11. Grouted End Anchorage, Polyester Resin (U.S. Army, 1980).



EXPLODED VIEW



ASSEMBLED VIEW

(Hardware for 150-ksi bar shown)

Fig. 4-12. "Thread Bar" Rock Bolt With Polyester Resin Grouted Anchorage (U.S. Army, 1980).

4-5 PARAMETERS OF DESIGN

Rock reinforcement is usually considered a feasible support system when the anticipated support pressures do not exceed 50 lb/in² (0.35 MPa) and the rock is not very blocky or fissured. The important parameters for designing rock bolts are length, diameter, spacing, pattern of rock bolts (spot, regular, or strapped), and the physical properties both of intact rock and joints. Properties such as tensile, compressive, and shear resistance of intact rock and the cohesion, angle of internal friction, and strike and dip of joints are important parameters in the design of rock reinforcement. The material properties of rock reinforcement such as Young's modulus of elasticity, ultimate stress, and creep properties influence the design of rock reinforcement. Other important parameters are: (a) ratio of horizontal to vertical rock pressures, (b) rock block shape, (c) mean size of rock blocks, (d) size of rock anchors or bolt bearing plates, (e) time dependent deformations and dilatation of the rock mass, and (f) degree of fracturing (Lang et al., 1979).

4-6 DESIGN OF ROCK REINFORCEMENT

When support systems, such as rock bolts and rock anchors, are placed in the interior of the rock mass, the constraints on design become stringent. The analysis then requires consideration of the original in situ stresses, the stresses induced by excavation and prestressing of rock bolts. Rock bolts are considered, in essence, to create point loads; both on the surface and within the mass of the semi-infinite space containing the rock mass. Again, some simplifying assumptions of rock mass properties are required to analyze the stresses induced by rock bolts. Boussinesq's (1885) equations for a point load on surface and Mindlin's (1953) solution for a point load in the interior of the semi-infinite space have been used to solve the loading created by the two ends of the rock bolts.

Because these solutions become very complicated and really do not use the actual varying but grossly simplified and assumed properties of rock masses, the results do not match the actual observations. As such, some simplifying analyses have come into use which are discussed in the following paragraphs. In the case of rock anchors that are fully grouted, the concentrated loading at the ends are eliminated.

4-6.1 Rock Bolt Suspension Theory

The simplest analysis considers the rock bolt as a suspension device transferring the weight of the weaker rock strata, near the opening, to the stronger strata, away from the opening. The total weight of the weaker strata

is carried through the rock bolts to rock strata that are strong and can carry the load. Naturally, the length of the rock bolt should be adequate to traverse through weaker rock zones and provide the necessary anchoring length into the stronger strata for the transfer of load. The cross sectional area of the rock bolt should be sufficient to transfer the load at 50 to 67 percent of the yield strength of the material of the rock bolt (O'Neill, 1966).

Figure 4-13(a) illustrates the stronger strata carrying the load of the weaker strata by nailing or suspension action.

Length of bolt, $L = L_1 + L_2$

L_1 = weak zone dimension

L_2 = anchor length

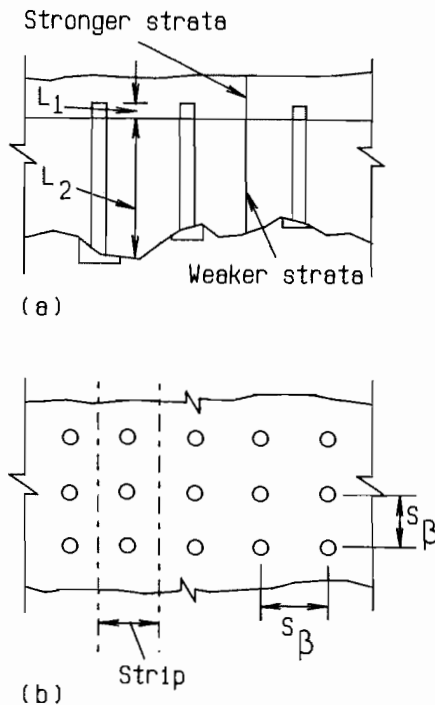


Fig. 4-13. Suspension of Weaker Strata.

Cross sectional area required of bolt A_B is given by equation 4-1:

$$A_B = \frac{(S_B)^2 (\omega_r)(L_1)(F_s)}{0.6 f_{yB}} \quad 4-1$$

where f_{yB} = yield stress of bolt
 S_B = spacing of rock bolts
 ω_r = unit weight of rock
 F_s = factor of safety

If a very strong bolt is used, S_B theoretically tends to become very large. This is not feasible in practice because the low tensile strength of rock will limit S_B from becoming large; the rock in between the bolts will otherwise fail in tension due to bending. Therefore, we must consider the bending effects on the rock between any two rock bolts or rock anchors.

4-6.2 Rock Bolt Bending Theory

If one considers a strip of width S_B supported between two rock bolts shown on figure 4-13(b) and then applies pure bending theory,

$$\frac{\sigma}{Y} = \frac{M}{I} \quad \text{or} \quad \sigma = \frac{M}{Z} \quad 4-2$$

where σ = tensile stress at bottom of the intact rock strata
 Y = distance from neutral axis
 M = moment due to self-weight
 I = moment of inertia
 Z = section modulus = I/Y

One can find the limiting value of S_B that is consistent with the allowable tensile stress in rock in rupture. In this case, the rock between bolts is considered intact which is extremely rare and, therefore, the results derived are extremely hypothetical.

In determining moment values, it is necessary to assign support conditions at the ends of the hypothetical beam of length = S_B between rock bolts. Biron and Arioglu (1983) have considered simple support conditions whereas Hobst and Zajic (1977) have considered the supports to be fixed. Following American Concrete Institute recommendations for continuous beams, the support condition should be between simply supported and fully fixed. Based on this premise, the limiting value of S_B can be found as follows:

$$S_B = \sqrt{\left[\frac{10}{6}\right] \left[\frac{\sigma_t \cdot L \cdot 1}{w_r}\right]} \quad 4-3$$

where σ_t allowable tensile stress of rock in rupture.

This S_B should be reduced by a certain factor of safety to provide some reserve strength to account for the variability in the tensile strength of the rock mass. Application of this concept requires that certain rock properties such as unit weight, tensile rupture strength, and thickness of weak strata must be known. These are sometimes difficult to determine.

This analysis remains simple as long as a single layer of rock is involved; however, such a situation is rarely met in practice. When two or more layers of rocks are to be considered, it becomes proper to assess the upper and lower bound estimates of section modulus by assuming no slip or full slip along the bedding planes of the different rock layers. No slip occurs when the rock bolts are fully effective and the frictional shear resistance of the bedding planes is sufficient to fully transfer stresses and strains to the adjacent rock strata. Full slip occurs when the joint materials are incapable of transferring stresses. In an idealized laboratory situation, Sinha (1972) found that even the most effective stud systems transfer only 50 percent of the stresses and strains to the adjoining layers, thus some slip will always occur along the joint interfaces.

4-6.3 Hidden Arch Theory

Lang (1972), using photoelastic observations, hypothesized a zone of uniform compression after rock bolting. This compression zone can act as a hidden flat arch in the roof of the opening. The compressed zone estimation is based on a 45° angle of dispersion of the applied rock bolt forces. The hidden beam acts similar to a Voussoir arch and this arch is considered to take no tension. The stress diagram at any section, therefore, remains wholly compressive. The ultimate compressive stress diagram, therefore, will always remain triangular. Following the Voussoir arch principle, the thrust at any section of the hidden arch must remain within the middle third of the hidden arch. The depth of the hidden arch must be at least 1/12th of the effective span of the hidden arch.

On the premises that the stress diagram remains compressive, the thrust passes through the middle third, the rock bolt load dispersion angle is 45°, and the internal resisting moment equals that of moment created by external force, one can derive the following equations for the hidden beam shown in figure 4-14.

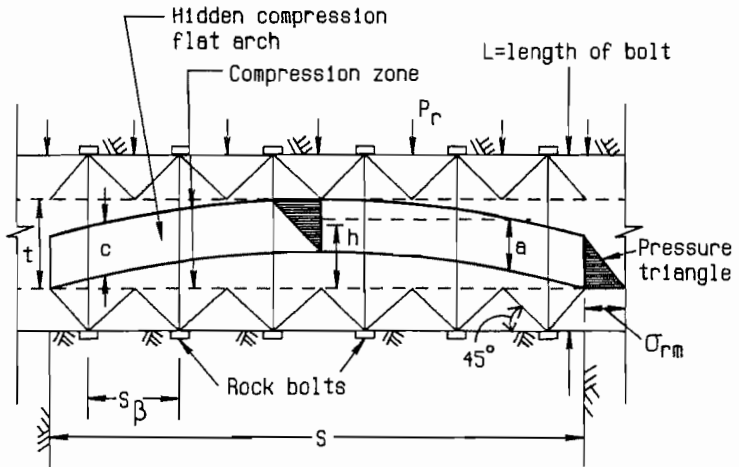


Fig. 4-14. Hidden Flat Arch Formed by Rock Bolts.

$$\text{External moment} = \frac{\rho_r S^2}{8} \quad 4-4(a)$$

where S = effective span of the flat arch
 ρ_r = vertical rock load per unit area

$$\text{Internal forces at crown and abutment sections} = \frac{\sigma_{rm} C}{2} \quad 4-4(b)$$

where σ_{rm} = unconfined compressive strength of rock mass
 C = thickness of the flat hidden arch

$$t = L - 2 \times \frac{S_B}{2} = L - S_B$$

where S_B = spacing of rock bolt
 L = length of bolt

$$\text{Moment arm of internal force} = t - \frac{2C}{3} \quad 4-4(c)$$

$$\text{Internal moment} = \left[\frac{\sigma_{rm} C}{2} \right] \left(t - \frac{2C}{3} \right) \quad 4-4(d)$$

Now external moment = internal moment

$$\text{hence } \frac{\rho_r S^2}{8} = \frac{\sigma_{rm} C}{2} \left[t - \frac{2C}{3} \right] \quad \text{or}$$

$$\rho_r = \frac{4\sigma_{rm} C}{S^2} \left[t - \frac{2C}{3} \right] \quad 4-4(e)$$

To obtain minimum value of ρ_r , one equates

$$\frac{d\rho_r}{dC} = 0 \quad \text{such that}$$

$$t - \frac{4}{3}C = 0 \quad \text{or}$$

$$C = 0.75t \quad 4-4(f)$$

Substituting the value of C from equation 4-4(f) to 4-4(e), one finds

$$\rho_r = \frac{3}{2} \cdot \sigma_{rm} \left[\frac{t}{3} \right]^2 \quad 4-4(g)$$

To increase the value of ρ_r , the incident load, one will have to increase the value of $t = L - S_B$, i.e., length (L) and S_B (spacing) of rock bolt may be adjusted to increase or decrease the value of ρ_r in equation 4-4(f).

The individual load ρ_B in a rock bolt, based on a square tributary area

$$\rho_B = \rho_r S_B^2 F_S$$

For a rectangular bolt pattern $\rho_B = \rho_r \cdot S_1 \cdot S_2 \cdot F_S$. Once ρ_B is found

$$A_B = \frac{\rho_B}{0.67 f_{yB}} \quad 4-4(h)$$

where A_B = area of bolt

S_B = spacing of rock bolt, see figure 4-13(b)

f_{yB} = yield stress of bolt, F_S = factor of safety

S_1 and S_2 are spacings of the rock bolts in the rectangular pattern along the adjacent sides of the rectangle.

4-6.4 Rock Bolt As Equivalent Support

Bischoff and Smart (1975) introduced a concept in which the use of rock bolt reinforcement creates a uniform additional pressure on rock that is equivalent to that taken by steel ribs. This pressure is:

$$\Delta\sigma_3 = \frac{(\sigma_B)(A_B)}{(S_B)^2} \quad 4-5(a)$$

where σ_B = permissible stress in rock bolt

A_B = cross sectional area of bolt

S_B = spacing of bolt

Now $\Delta\sigma_3$ introduces additional strength in the rock mass given by equation:

$$\Delta\sigma_1 = \tan^2 (45^\circ + \phi/2) \Delta\sigma_3 \quad 4-5(b)$$

where ϕ = angle of internal friction of rock joints

and then

$$\Delta T_A = (\Delta\sigma_1)(t) \quad 4-5(c)$$

where ΔT_A = strength of steel ribs

t = effective thickness which can be determined by applying Lang's (1972) approach

also

$$\Delta T_A = \frac{(\sigma_s)(A_s)}{S_s} \quad 4-5(d)$$

where σ_s , A_s , and S_s are stress, area, and spacing of steel rib supports

4-6.5 Empirical Methods

The U.S. Army (1980) suggests empirical methods of assigning length and spacing to rock bolts as shown in table 4-1.

TABLE 4-1

Minimum length and maximum spacing for rock reinforcement.

Minimum length

Greatest of:

- a) Two times the bolt spacing
- b) Three times the width of critical and potentially unstable rock blocks
- c) For elements above the springline:
 1. Spans less than 20 ft - 1/2 span
 2. Spans from 60 to 100 ft - 1/4 span
 3. Spans 20 to 60 ft - interpolate between 10- and 15-ft lengths, respectively
- d) For elements below the springline:
 1. For openings less than 60 ft high - use lengths as determined in c) above
 2. For openings greater than 60 ft high - 1/5 the height

Maximum spacing

Least of:

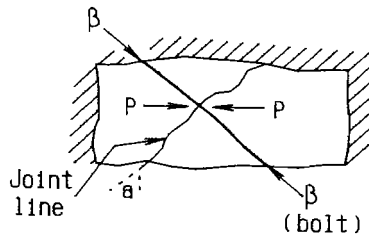
- a) 1/2 the bolt length
- b) 1-1/2 the width of critical and potentially unstable rock bolts
- c) 6 ft

Greater spacing than 6 ft would make attachment of surface treatment such as chain link fabric difficult

Minimum spacing 3 to 4 ft

4-6.6 Joint Friction Approach

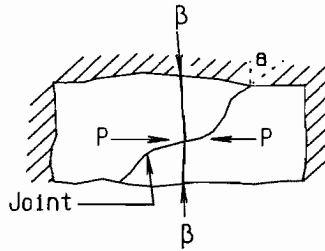
The method discussed by the U.S. Army (1980) for the design of rock bolts from a joint friction approach is shown on figures 4-15. The approach can be extended to a multijoint system.



For stability:

$$\frac{B}{P} > \sin \alpha (\cot \phi - \cot \alpha)$$

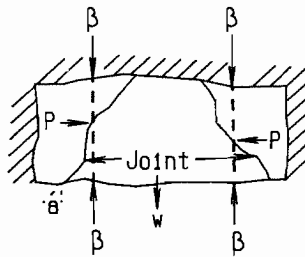
Fig. 4-15(a). Single Joint With Bolt Normal to Joint.



For stability:

$$\tan(\alpha - \phi) < \frac{\beta}{P} < \tan(\alpha + \phi)$$

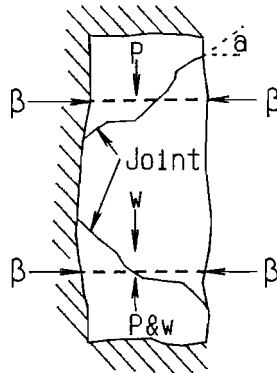
Fig. 4-15(b). Single Joint With Bolt Normal to In Situ Joint Force.



For stability:

$$\tan(\alpha - \phi) < \frac{\beta - 1/2(w)}{P} < \tan(\alpha + \phi)$$

Fig. 4-15(c). Block in Horizontal Surface.



For stability:

$$\tan(\alpha - \phi) < \frac{\beta}{P + 1/2(W)} < \tan(\alpha + \phi)$$

Fig. 4-15(d). Block in Vertical Surface.

4-6.7 Internal Pressure Approach

Cording and Deere (1972) introduced the concept of "internal pressure" for the design of rock bolts to support a sliding critical wedge. Their equation for internal pressure " P_i " is:

$$P_i = P_n \left[1 - \frac{\tan \phi}{\tan \theta} \right] + \frac{Wv}{4 \tan \theta} \quad 4-7$$

where P_n = average normal pressure acting on side of the wedge

ϕ = angle of internal friction

θ = one-half of the included angle of the wedge

v = unit weight of rock

W = width of opening

The rock bolt design has to provide the equivalent pressure of P_i with acceptable margin of safety and the rock bolt anchorage length has to extend a minimum of 4 feet (1.2m) beyond the critical wedge. The derivation of equation 4-7 accounts for the frictional resistance developed at the rock hoints such that the maximum load on the rock bolt equals the weight of the critical wedge less the frictional resistance acting on the sliding wedge planes.

The determination of critical wedge requires the input from an experienced designer and is not determined accurately by any mathematical method.

4-6.8 Reinforced Rock Unit

In a report to the Bureau of Mines, Leeds, Hill, and Jewett, Inc., 1979, introduced a concept of reinforced rock unit. In essence, the concept of reinforced rock unit is a modification of suspension theory discussed at section 4-6.1 in which the shear stresses at the sliding interfaces are included along with the suspension action to resist the rock load. Based on this concept, the tension in a rock bolt is given by equation 4-8.

$$\rho_B = \frac{\alpha v A R}{K \mu} \left[1 - \frac{C}{v R} - \frac{\mu \sigma_h}{v R} \right] \frac{1 - e^{-4K\mu D/R}}{1 - e^{-4K\mu l_B/R}} \quad 4-8$$

where ρ_B = tension in rock bolt

α = varies from 0.5 to 1.0 depending upon the time lapse
between excavation and installation of rock bolts

v = specific weight of rock

A = tributary area of rock reinforcing unit = $S_B \times S_B$
[see fig. 4-13(b)]

$R = \frac{S_B}{4}$ for square pattern

$= \frac{S_1 \cdot S_2}{2(S_1 + S_2)}$ for rectangular pattern

S_1 = spacing of rock bolt in one direction

S_2 = spacing of rock bolt in perpendicular direction to S_1

S_B = spacing of rock bolt

K = ratio of existing horizontal stress to vertical stress

μ = coefficient of internal friction of rock

C = apparent cohesion of rock mass

σ_h = in situ horizontal stress

$D = l_B + Z$

l_B = length of rock bolt

Z = length of rock zone participating in the formation of reinforced rock unit (judgment is needed in assessing the value of Z)
measured parallel to rock bolt

Area of rock bolt can be determined by application of equation 4-4(h).

4-7 PULLOUT TEST

The initial tension in a rock bolt may be reduced during continued excavation due to excavation induced vibrations or due to creeping of rock mass under load. Pullout tests are performed to measure the existing tension in a rock bolt.

Pullout test is simple to perform. About 5 percent of the installed and tensioned bolts are pulled to 80 percent of their yield strength. The rock bolt deformation under pullout test should not exceed the deformation indicated by the elastic theory shown in equation 4-9.

$$\Delta_B = \frac{T_B}{A_B} \cdot \frac{L_B}{E_B} \quad 4-9$$

where Δ_B = deformation of rock bolt during pullout test

T_B = pulling out force

A_B = cross sectional area of rock bolt

L_B = effective length of rock bolt (total length less anchoring length)

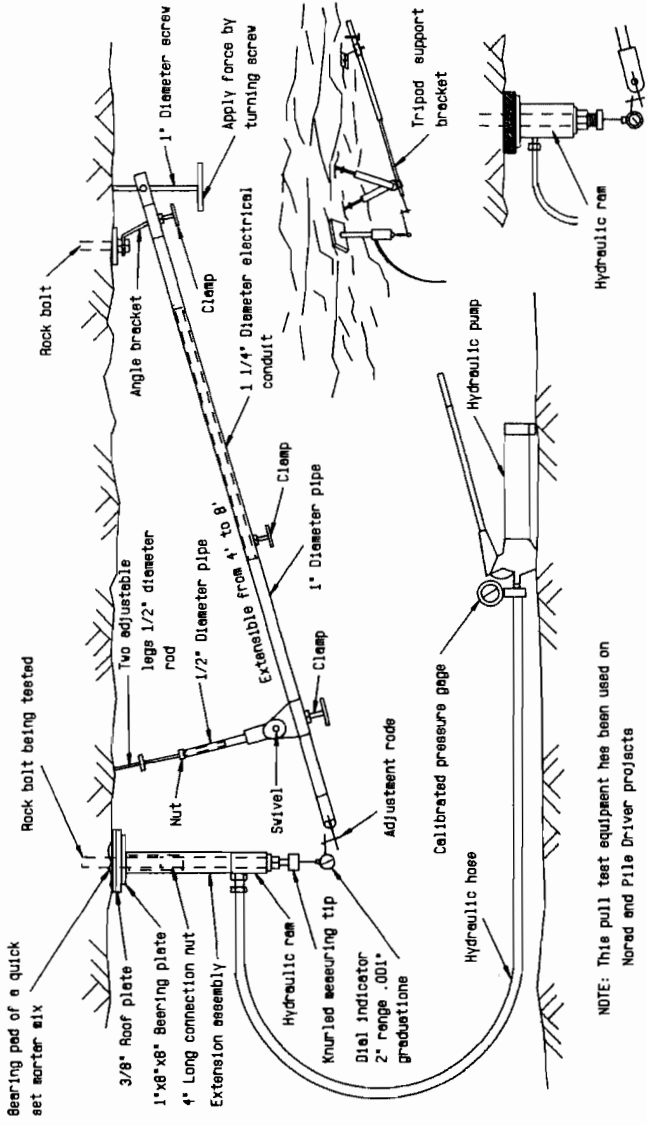
E_B = Modulus of elasticity of rock bolt material

The simplest type of pullout test is shown on figure 4-16 in which a hydraulic ram is used to perform a pullout test. If the rock bolts during pullout testing indicate loss of tension, they need to be retensioned using a torque wrench or similar other devices. The tension induced in a rock bolt by a torque wrench is usually 50 to 60 times the exerted torque from the wrench. For example, a 250 ft-lb of torque will induce 12,500 to 15,000 pounds of tension in a rock bolt.

4-8 ROCK BOLT INSTRUMENTATION

The pullout test described in section 4-7 is a destructive test performed on about 5 percent of the total installed rock bolts. Some of the nondestructive tests employed to test the performance of rock bolts include use of acoustic emission techniques, installation of stress meters, deformation gauges, or use of photoelastic dynamometers.

Details on the use of nondestructive testing mechanisms for testing rock bolts may be obtained from the manufacturer's representative.



NOTE: This pull test equipment has been used on
 Nored and Pile Driver projects

ALTERNATE SYSTEM

Fig. 4-16. Pull Test Equipment (U.S. Army, 1980).

4-9 FIELD OBSERVATIONS

The actual observations performed on several rock bolts indicate the beneficial effects of using longer rock bolts over shorter ones. Continued excavation, dynamic loading and unloading, blasting vibrations, and rock creep, alone or in combination, tend to relax the original tension in a rock bolt. In tight locations, use of rock bolts of different or mixed bolt lengths seem to be very beneficial. Rock temperatures and rock chemicals seem to affect the performance of rock bolts. When moisture or water is present, pullout tests in rock bolts give low values.

Proper bond strength or shear strength is required for resins and cements which may be used to anchor the rock reinforcement to the rock. The bond or shear failure may occur (1) within the bonding material, (2) at the interface of bonding material and rock reinforcement, and (3) at the interface of rock and bonding material. Rock reinforcement material itself may fail during tensioning or the host rock itself may creep under loading; though these failures are rare.

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Chapter 5

UNDERGROUND STRUCTURES IN ROCK

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5-1 INTRODUCTION

Cost of supporting an underground excavation ranges from 12 to 36 percent of the total cost of the final structure. Design of support system, therefore, becomes a significant part of the total process. The initial support system performs the first or preliminary part of supporting the excavation temporarily until the final or secondary or permanent liner is placed.

In designing the final or permanent support system, the contribution of the initial lining, customarily, is completely ignored. This is done because of the consideration that with time, the initial lining support system will deteriorate away and will become structurally useless. Exception to this customary belief is the role of "one pass liners," such as precast concrete or cast iron liners, which serves both as initial and final supports and whose strength is considered to be substantially preserved through their design life.

Both the initial and final support systems are highly indeterminate structures and they are to be designed as such.

5-2 DESIGN OF INDETERMINATE STRUCTURES

If a plane structure has more than three support reactions, it is structurally indeterminate externally. A space structure which has more than six support reactions is considered structurally indeterminate externally. Figure 5-1 shows the common types of supports and the reactions they generate.

The plane truss shown on figure 5-1(a) has six reactions. At "A," one reaction, vertical; at "B," two reactions, one vertical and one horizontal; and at "C," three reactions, one vertical, one horizontal, and one rotational. Therefore, the plane truss of figure 5-1(a) has three degrees (6-3) of external redundancy and must be solved as a structurally indeterminate structure.

The basic design difference between structurally determinate and indeterminate structures is the requirement that in analyzing the structural adequacy of an indeterminate structure, the sectional properties, the material properties, and the precise geometry of the structural member must be known. This requirement alludes that the design of indeterminate structures must

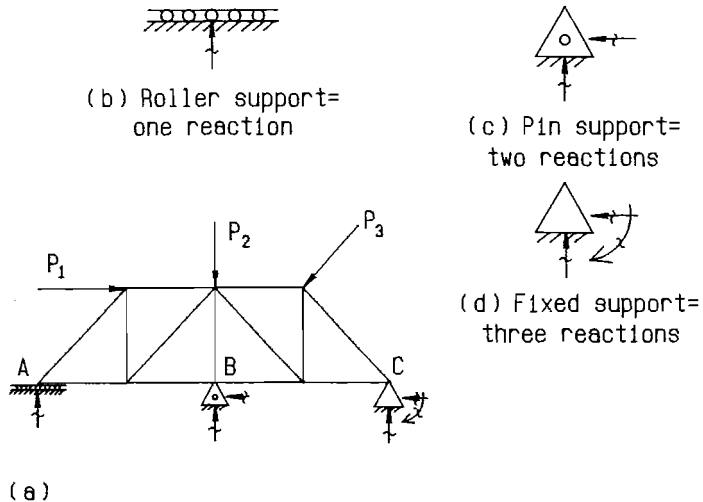


Fig. 5-1. Common Supports and Reactions.

necessarily be an iterative process consisting of (1) an initial assumption of member sizes, geometry, and material properties; and (2) performing the structural analysis of the member with those initially assumed properties with its expected external loads. If the analysis yields stresses and deformations that are within and close to the safe limits of stresses for materials and deformation limits of the structure, then accept that design. If not, then increase or decrease member sizes and/or properties, then reanalyze and redesign until close conformance is obtained with the safe allowable stresses and deformations. Thus, design of indeterminate structures is laborious, time consuming, costly, and sometimes frustrating.

There are several prevalent methods of analysis (Timoshenko and Young, 1965) for indeterminate structures. These methods of analysis are (1) consistent deformation, (2) three moment equations, (3) slope deflection, (4) moment distribution, (5) theorem of least work, (6) virtual work, (7) elastic center, and (8) column analogy.

In the following paragraphs, it is intended to discuss only elastic center and column analogy methods because these two methods are more suitable for the analysis of arches, rings, and bents (see figure 5-2) that are most commonly used for underground structures.

5-2.1 Elastic Center Method

Elastic center is the "center of gravity" of elastic weights. The term

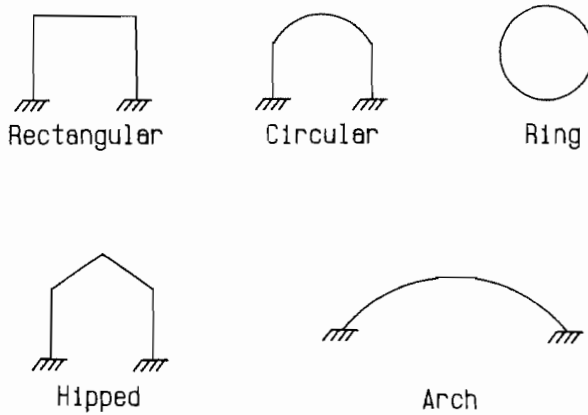


Fig. 5-2. The Common Underground Structural Elements - Bent, Ring, and Arch.

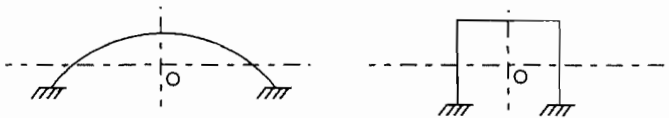


Fig. 5-3. Elastic Centers "O"

$\frac{ds}{EI}$ is called the elastic weight.

Where "ds" is the infinitesimal length at which "E," the modulus of elasticity, and "I," the moment of inertia or second moment of area, can be considered to be constant. Of course, "E" and "I" both may vary along the length of the member. According to established definitions, at center of gravity, the products

$$\int x \frac{ds}{EI} = 0 \quad 5-2(a)$$

$$\int y \frac{ds}{EI} = 0 \quad 5-2(b)$$

and if one of the axis "x" or "y" are principal axis, then the product

$$\int xy \frac{ds}{EI} = 0.$$

5-2(c)

Now let us examine the arch of figure 5-4(a). If the principle of superposition be applicable, then under generalized loading, the arch of figure 5-4(a) is structurally equivalent to those of arches shown on figures 5-4(b) and (c).

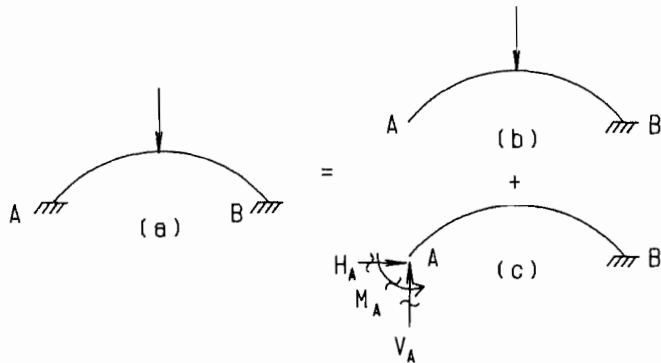


Fig. 5-4. Equivalent Arches Shown on Right.

Figure 5-4(b) is the statically determinate part under only generalized structural loading and figure 5-4(c) is the statically indeterminate part with unknown reactions only H_A , V_A , and M_A acting. The aim is to find H_A , V_A , and M_A .

The generalized strain energy "U" of a structure is given by equation 5-2(d).

$$U = \frac{1}{2} \int \frac{m^2 dx}{EI} + \frac{1}{2} \int \frac{s^2 dx}{AE} + \frac{1}{2} \int \frac{T^2 dx}{GJ}$$

5-2(d)

Where the first term is due to moment, the second term is due to axial thrust and the third term is due to torsion and

m = moment

s = thrust

T = torsion

A = cross sectional area

I = moment of inertia

J = polar moment of inertia

E = modulus of elasticity

G = modulus of rigidity

In arches, the contribution of the first term in equation 5-2(d), namely

$$\frac{1}{2} \int \frac{m^2 dx}{EI}$$

is usually much larger than the second and third terms. The third term is usually ignored because, by design, one does not introduce any kind of torsional forces in the arch. Again, the contribution of the second term due to thrust, namely

$$\frac{1}{2} \int \frac{s^2 dx}{EI}$$

is usually ignored, unless the arch is extremely flat.

Thus, the strain energy

$$U = \frac{1}{2} \int \frac{m^2 dx}{EI}$$

can be construed to consist of contributions of generalized external forces and unknown reactions H_A , V_A , and M_A .

According to Castigliano's second theorem (Parcel and Moorman, 1955), because support "A" does not move or rotate, the partial derivatives of strain energy with respect to unknown reactions give the movement of the unknown reaction.

$$\text{Hence, } \delta h_a = \frac{\partial u}{\partial H_a} = \int \frac{m}{EI} \frac{\partial m}{\partial H_a} dx = 0 \quad 5-2(e)$$

$$\delta v_a = \frac{\partial u}{\partial v_a} = \int \frac{m}{EI} \frac{\partial m}{\partial v_a} dx = 0 \quad 5-2(f)$$

$$\delta m_a = \frac{\partial u}{\partial M_a} = \int \frac{m}{EI} \frac{\partial m}{\partial M_a} dx = 0 \quad 5-2(g)$$

where m is the combined moment due to external loads and the unknown reactions. The integration becomes quite involved and the accuracy is not lost if the " \int " is substituted by summation sign " Σ " when the structural member is considered to consist of 10 or more segments.

Now let us find the elastic center of the area. In order to do that, let us

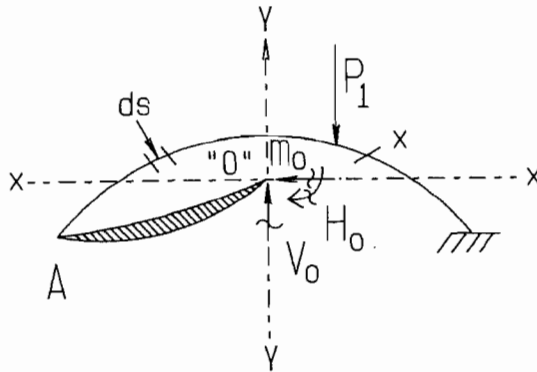


Fig. 5-5. "O" the Elastic Center Shown Connected to Support "A".

move the support reactions to the center of the elastic arch by connecting the free end "A" of the arch by an imaginary member of infinite rigidity EI . The imaginary member is shown hatched on figure 5-5. By doing so, we have not changed the original elastic weight of the arch because

$$\sum \frac{ds}{EI} \text{ for "AO"} = \frac{1}{\infty} = 0.$$

The unknown reactions H_A , V_A , and M_A have been replaced by H_0 , V_0 , and M_0 acting at "O." Let us now assign the following symbols:

$$P = \sum \frac{M_s ds}{EI} \qquad M_x = \sum \frac{M_s y ds}{EI} \qquad M_y = \sum \frac{M_s x ds}{EI}$$

$$A = \sum \frac{ds}{EI} \qquad I_x = \sum \frac{y^2 ds}{EI} \qquad I_y = \sum \frac{x^2 ds}{EI}$$

$$I_{xy} = \sum \frac{xy ds}{EI} \qquad 5-2(h)$$

where M_s = Statistically determinate moments of the external load when indeterminate reactions are considered removed temporarily (solely for the purpose of analysis).

x and y = Distances measured along "x" and "y" axis for the centroidal point of segments "ds" from the elastic center.

The moment at any point "x" on the structure (see figure 5-5) is

$$M_x = M_s - M_0 + H_0 y - V_0 x \quad 5-2(j)$$

Performing partial derivative of strain energy with respect to "H₀" we get, see equation 5-2(e),

$$\int \frac{M_x}{EI} \frac{\partial M_x}{\partial H_0} ds = 0 = \Sigma (M_s - M_0 + H_0 y - V_0 x) y \frac{ds}{EI} = 0 \quad 5-2(k)$$

because $\int \frac{y ds}{EI} = 0$ at elastic center and

$$\int xy \frac{ds}{EI} = 0 \text{ because "y" is axis of symmetry}$$

equation 5-2(k) reduces to

$$\Sigma \frac{M_s y}{EI} ds + \Sigma H_0 \frac{y^2 ds}{EI} = 0$$

Using symbols of equations 5-2(h), the expression reduces to

$$M_x + H_0 I_x = 0 \text{ or}$$

$$H_0 = \frac{-M_x}{I_x} \quad 5-2(l)$$

Similarly, equations 5-2(f) and 5-2(j) will yield expression

$$\Sigma \frac{M_s x}{EI} ds - \Sigma V_0 \frac{x^2 ds}{EI} = 0 \text{ or}$$

$$M_y - V_0 I_y = 0 \text{ or } V_0 = \frac{M_y}{I_y} \quad 5-2(m)$$

and equations 5-2(g) and 5-2(j) will yield

$$\Sigma M_s \frac{ds}{EI} - \Sigma M_0 \frac{ds}{EI} = 0 \text{ or}$$

$$P - M_0 A = 0 \text{ or } M_0 = \frac{P}{A} \quad 5-2(n)$$

To summarize, $M_0 = \frac{P}{A}$, $V_0 = \frac{M_y}{I_y}$, and $H_0 = \frac{M_x}{I_x}$

Once M_0 , V_0 , and H_0 are found, because of the infinite rigidity of the arm "AO" (see figure 5-5).

$$H_A = H_0, V_A = V_0, \text{ and } M_A = M_0 + H_0 y - V_0 x \quad 5-2(o)$$

Now let us solve some numerical examples.

5-2.2 Illustrative Example

Example 1. Find the distance of the elastic center from the crown of an arch. The arch has a radius of "r," the central half angle = α , and the E and I are constant throughout. The arch is shown in figure 5-6.

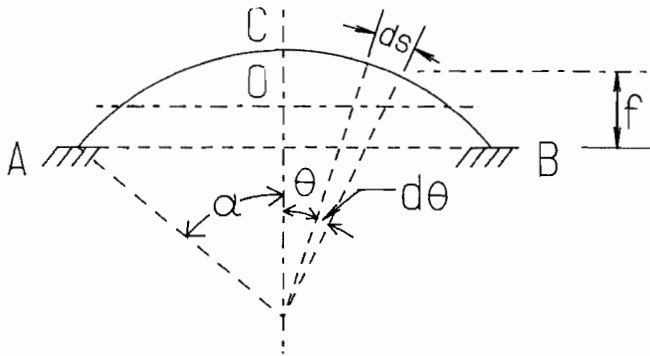


Fig. 5-6. Calculations for an Elastic Center.

Let us take $EI = 1$ and consider the moment about line AB of elemental length ds of arch located at a distance of θ from the center. The element length ds subtends an angle of $d\theta$ at center. Now $ds = r d\theta$, distance $f = r (\cos \theta - \cos \alpha)$, moment of element ds about line AB = $r (\cos \theta - \cos \alpha) r d\theta$. Hence, centre of gravity distance of arch from line AB

$$= \frac{2 \int_0^\alpha (r \cos \theta - r \cos \alpha) r d\theta}{2 \int_0^\alpha r d\theta}$$

$$= \frac{r^2 [\sin \theta - \theta \cos \alpha]_0^\alpha}{r [\theta]_0^\alpha}$$

$$= \frac{r}{a} [\sin \alpha - \alpha \cos \alpha]$$

$$\text{Distance } CD \text{ (see figure 5-6)} = r - r \cos \alpha - \frac{r}{a} (\sin \alpha - \alpha \cos \alpha)$$

$$= r - \frac{r}{a} \sin \alpha$$

5-2(p)

$$= \frac{r}{a} (\alpha - \sin \alpha)$$

Example 2. Let us find reactions H_A , V_A , and M_A for a bent for a cavern. The bent is shown on figure 5-7.

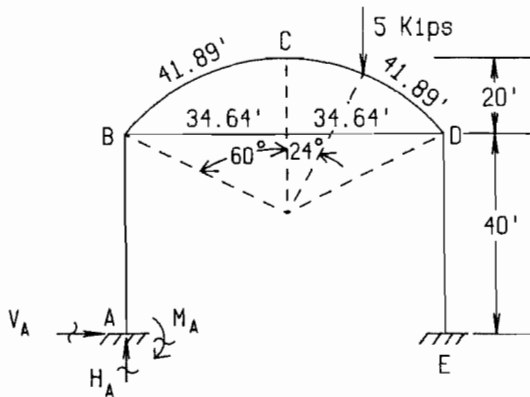


Fig. 5-7. Bent for a Cavern.

The I of column and arch are different but

$$\frac{ds}{EI} \text{ of columns and arch} = 1.$$

Using equation 5-2(p), (1), find the elastic center for arch assuming

$$\frac{ds}{EI} = 1$$

$$d = \frac{40}{1.047} (1.047 - .866)$$

= 6.915 feet from crown "C"
 = 13.085 from line BD, and then,

(2) find the elastic center of the bent ABCDE shown in figure 5-8.

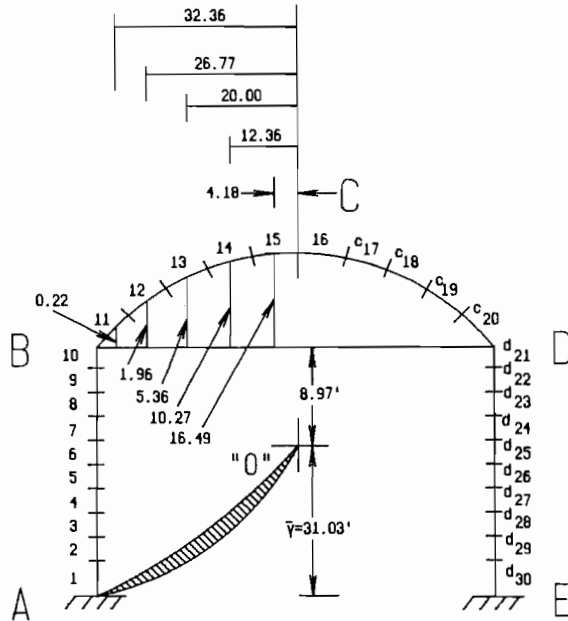


Fig. 5-8. Bent Analysis: "O" = Elastic Center.

Taking moments of areas at line AE and assuming

$\frac{ds}{EI} = 1$ for each element 1 through 15, C_{16} through C_{20} and D_{21} through D_{30}

(see figure 5-8)

for each segment of arch and columns, we get

$$\bar{y} = \frac{10 \times 20 + 10 \times (60 - 6.915) + 10 \times 20}{30} = 31.03$$

$$\text{Now } A = \Sigma \frac{ds}{EI} \text{ for bent} = 10 + 10 + 10 = 30$$

$$\begin{aligned} I_x = \Sigma y^2 \frac{ds}{EI} &= 10(31.03 - 20)^2 + 2[(8.97 + 16.49)^2 + (8.97 + 10.27)^2 \\ &\quad + (8.97 + 5.36)^2 + (8.97 + 1.96)^2 + (8.97 + 0.22)^2] \\ &\quad + 10(31.03 - 20)^2 = 5288.52 \end{aligned}$$

$$\begin{aligned} I_y = \Sigma x^2 \frac{ds}{EI} &= 10(34.64)^2 + 2(4.18^2 + 12.36^2 + 20^2 + 26.77^2 + 32.36^2) \\ &\quad + 10(34.64)^2 = 26,666.68 \end{aligned}$$

As shown in figure 5-9, the

load position = $r \cos 24^\circ = 40 = 16.27$ from center

= $34.64 - 16.27 = 18.37$ from the right column.

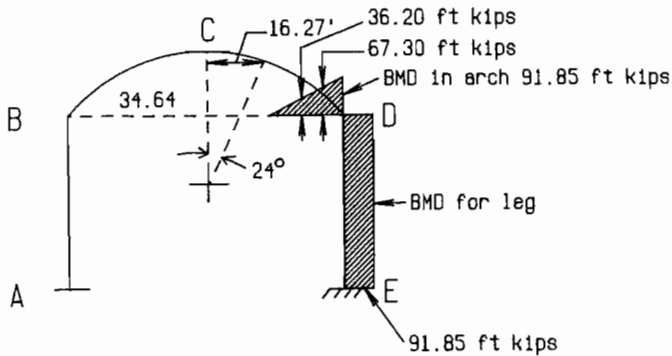


Fig. 5-9. Bending Moment Diagram (B. M. D.) End "A" Assumed Free.

$$\text{calculation of } P = \Sigma \frac{M_s ds}{EI}$$

$$\text{Element } C_{17}C_{18} \text{ (see figure 5-8)} = \frac{1}{2} \times 36.20 \times 1 = 18.10$$

$$C_{18}C_{19} = \frac{1}{2} (67.30 + 36.20) \times 1 = 51.75$$

$$C_{19}C_{20} = \frac{1}{2} (91.85 + 67.30) \times 1 = 79.58$$

$$C_{20} = 91.85 \times 1 \times 10 \text{ through } d_{29}d_{30} = \underline{918.50}$$

$$\text{Hence } P = 1,067.93$$

$$\text{Calculation of } M_y = \Sigma \frac{M_s \times ds}{EI}$$

$$\text{Element } C_{17}C_{18} = 36.20 \times 20 = 724.00$$

$$C_{18}C_{19} = 51.75 \times 26.77 = 1,385.35$$

$$C_{19}C_{20} = 79.58 \times 32.36 = 2,575.21$$

$$D_{21} \text{ through } D_{30} = 91.85 \times 34.64 \times 10 = \underline{31,816.84}$$

$$M_y = 36,501.40$$

$$\text{Calculation of } M_x = \Sigma \frac{M_s \times y \times ds}{EI}$$

$$\text{Element } C_{17}C_{18} = 36.20 \times 14.33 = 523.05$$

$$C_{18}C_{19} = 51.75 \times 10.83 = 560.45$$

$$C_{19}C_{20} = 79.58 \times 9.19 = 731.34$$

$$D_{21} \text{ through } D_{30} = 91.85 \times 1 \times 10 \times (-)11.03 = \underline{(-)10,131.06}$$

$$M_x = -8,316.22$$

$$\text{Hence } M_0 = \frac{P}{A} = \frac{1,067.93}{30} = 35.60 \text{ kips}$$

$$H_0 = \frac{-M_x}{I_x} = \frac{-(-8316.22)}{5288.52} = 1.57 \text{ kips}$$

$$V_0 = \frac{M_y}{I_y} = \frac{36,501.40}{28,666.68} = 1.27 \text{ kips}$$

Hence $H_A = H_0 = 1.57$ kips

$V_A = V_0 = 1.27$ kips

$M_A = M_0 + H_0\bar{y} + V_0\bar{x} = 35.60 + 1.57 \times 31.03 - 1.27 \times 34.64 = 40.32$

Once M_A , H_A , and V_A are found, moment at any point "x," taking counterclockwise moments as positive, can be found to be

$$M_x = M_s - M_A - V_A x + H_A y \quad \text{or}$$

$$M_x = M_s - M_0 + H_0 y - V_0 x$$

where x and y are coordinates of point x from support A.

5-2.3 Column Analogy Method

Column analogy method is analogous to elastic center method.

In elastic center method, moment at any point

$$M_x = M_s - M_0 + H_0 y - V_0 x$$

whereas, in column analogy method

$$M_x = M_s + M_i$$

where M_s = statically determinate and

M_i = statically indeterminate moment

$$\text{where } M_i = \frac{P}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y} \quad 5-2(r)$$

Proper regard to sign (+ or -) has to be provided when summing expressions in equation 5-2(r). Variables P, A, M_x , I_x , M_y , and I_y were defined in formula 5-2(h). One can easily detect that equation 5-2(r) is the equation for moment for a biaxially eccentrically loaded column.

The solution of an indeterminate structure by column analogy method follows the same sequence of analysis as shown in section 5-2.1, considering that the structure is equivalent to a column loaded with elastic load of

$$\frac{M_s ds}{EI}$$

5-2.4. Commentary

The discussions in sections 5-2.1, 5-2.2, and 5-2.3 indicate the complexities of the analysis in solving indeterminate structures. In an actual design office, resource allocation is not justified for calculations from first principles. Usually, design charts, tables, and graphs are available which provide values for moments and reactions at support points for different kinds of bents and frames for different kinds of uniform, varying, and partial loadings. Using those tables, charts, and graphs, factors are determined for calculating the values of unknown moments and reactions. Based on the premise that method of superposition is valid, the values of unknown moments and reactions for different sets of loading are calculated separately and individually and then combined together.

Alternatively, a software package can be used to perform the analysis. Several software packages are available to solve indeterminate structures on personal computers. It is essential that before using a software package, one should thoroughly test the software for accuracy against previously designed structures. The software packages usually carry undesirable disclaimers from the points of view of engineering practitioners. Confidence in the use of the software will not be available unless it is tested against previously designed structures.

5-2.5 Design of a Tunnel by Empirical Methods

Example 3. Design the initial and final support system for a finished 12-foot (3.66 m) diameter tunnel which will be 3 miles (4.8 km) long. The cover on the tunnel ranges from 20 feet (6.10 m) to 200 feet (60.96 m). The boreholes reveal that the tunnel will pass through andesite, tuff, and conglomerate. Forty-five to 95 percent of joints and fractures are filled with calcite, dolomite, and clay. Some joints are quite open. It contains vugs 1 inch (25 mm) to several inches wide. The joint spacing is assumed 1 foot (0.30 m) apart. Joint surfaces are hard and are slightly rough. The average values of RQD, weight, compressive strength, and Poisson's ratio are shown in table 5-1.

TABLE 5-1
Rock properties.

| | RQD | Weight lb/ft ³ | Unconfined compressive strength lb/in ² (MPa) | Poisson's ratio | Modulus of elasticity 10 ⁶ /in ² (GPa) |
|--------------|----------|------------------------------|---|--------------------|--|
| Andesite | 80 - 100 | 160 | 14,500 (100) | 0.21 | 6 (41.38) |
| Tuff | 75 - 90 | 123 | 4,300 (30) | 0.15 | 4 (27.58) |
| Conglomerate | 60 - 80 | 160 | 8,700 (60) | 0.20 | 2 (13.78) |

There are four sets of prominent joints as shown in table 5-2.

TABLE 5-2
Joint description.

| Joints | Strike | Dip |
|--------|-----------|---------|
| A | N. 14° W. | 85° E. |
| B | N. 43° E. | 86° NW. |
| C | N. 82° W. | 58° S. |
| D | N. 25° W. | 13° SW. |

The strike and dip are considered unfavorable for the tunnel. The permeability test by Packer method gave the following permeabilities:

| | |
|--------------|--|
| Andesite | 0 to 720 feet per year (0 to 220 m/yr) |
| Tuff | 0 to 15 feet per year (0 to 4.57 m/yr) |
| Conglomerate | 0 to 10 feet per year (0 to 3.05 m/yr) |

The expected flows during construction will be 50 to 150 gal/min (13.2 to 13.9 l/minute) per 30 feet (10 m) of tunnel.

The petrographic examination indicated that some tunnel zones during excavation would encounter montmorillonite clays which will have an expansive value of up to 55 percent. The crown of the tunnel at a few locations will be 15 feet (4.6 m) below ground water table.

Solution. Because the tunnel is somewhat long, it will be desirable to have several designs for different geological zones through which the tunnel will pass. It would be improper to design the tunnel for the weakest geological zone and have that design applicable for the full length of the tunnel. Contrary to this, for a short length tunnel, it may be advantageous to design

the supports for the weakest geological zone and let that design prevail for the full length of the short tunnel.

In this example case, one should have three different designs: one each for andesite, tuff, and conglomerate zones. For brevity of discussion and because the intent is to demonstrate a design methodology, only one design will be presented for the weakest zone, tuff, and the design for andesite and conglomerate zones will be left out.

(i) Bieniawski's method. The rock mass rating for the tuff zone; using table 2-13 of chapter 2 is shown in table 5-3.

TABLE 5-3.
Calculation of Rock Mass Rating.

| Parameters | Range | Rating |
|-------------------------------|-------------------------------|---------------------------------------|
| Uniaxial compressive strength | 100 MPa | 3.5 [1/2 of (2 + 5), see table 2-13] |
| RQD | 75 - 90 | 17 |
| Spacing of joints | 0.3 | 15 [1/2 of (10 + 20), see table 2-13] |
| Strike and dip | Unfavorable | 6 |
| Condition of joint | Open joints with gouge < 5 mm | 5 |
| Ground water inflow | 13 - 39 L/m | 6.5 [1/2 of (5 + 8)] |
| Total | | 53.0 |

The calculated rock mass rating is 53 and, therefore, it is classified as a fair rock, class No. III (see table 2-13), with an average standup time of 1 week, and it can sustain an unsupported span of 3 meters. Because the tunnel has a clear span of 3.66 meters, initial supports will be required. According to table 2-15, the initial support system could be (a) mainly rock bolts spaced 3 to 6 feet (1.0 to 1.5 m) plus wire mesh and 1 inch (25 mm) of shotcrete in crown where required; or (b) mainly shotcrete 4 inches (100 mm) in crown and 2 inches (50 mm) in sides plus occasional wire mesh and rock bolts where required; or (c) light sets 4 w 13 spaced 5 to 6 feet (1.5 to 2 m) center to center.

The above example shows the ease with which the initial support system can be designed using tables 2-13 through 2-15, of chapter 2.

(ii) "Q" system. In order to design the initial supports for the same

tunnel, using the "Q" system, described in section 2-5, one selects the parameters J_n , J_r , J_a , J_w , and SRF with the help of tables 2-4, 2-5, and 2-6, as shown in table 5-4.

TABLE 5-4.
Selection of Parameters for "Q" System.

| Description | Parameter | Value |
|--------------------------|-----------|-------|
| 4 joint sets | RQD | 82.5* |
| Rough joints | J_n | 15 |
| Montmorillonite fillings | J_r | 3 |
| Medium inflow | J_a | 8 |
| Loose open joints | J_w | 0.66 |
| | SRW | 5.0 |

*Mean of 70 to 90

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF} \quad 2-5$$

$$= \frac{82.5}{15} \cdot \frac{3}{8} \cdot \frac{0.66}{5.0} = 0.272$$

The ESR (excavation support ratio) for permanent excavation, see table 2-7, equals 1.6.

$$\text{Hence equivalent dimension} = \frac{3.66}{1.6} = 2.288$$

From figure 2-2 for equivalent dimension of 2.288 and Q of 0.272, the needed support is type 29 for initial support which, from table 2-8, for $RQD/J_n > 5.0$, $J_r/J_n > 0.25$, and span/ESR of 1.0 to 3.1 is a systematical bolting of untensioned but grouted bolt, spaced on a pattern of 3 feet (1 m) apart plus shotcrete 1 inch (25 mm) thick.

Based on the discussions presented at 5-2.5(i) and 5-2.5(ii), it is recommended that for initial supports, one use 10-foot-(3-m-) long untensioned but grouted 1-inch-(25-mm) diameter rock bolt at a pattern bolting spaced 3 feet (1 m) apart and use 1-inch-(25-mm) thick shotcrete.

(iii) Design of permanent support. Calculation of roof load by (1) using equation 2-4 of chapter 2

$$P_{\text{roof}} = \frac{2.0 J_n \left(\frac{1}{2} - \frac{1}{3} \right) Q}{3 J_r}$$

$$= \frac{2.0 \times 15^{\frac{1}{2}} \times 0.272}{3 \times 3} - \frac{1}{3} = 1.329 \text{ kg/cm}^2 = 2.722 \text{ kips/ft}^2$$

(2) from table 2-2, based on the host geology that the rock is very blocky and seamy, the rock load will come from a loosened zone of rock having a height of

$$\begin{aligned} H_R &= (0.35 \text{ to } 1.10)(B + H_t) \\ &= 0.35 \times 24 \text{ to } 1.10 \times 24 \\ &= 7.2 \text{ ft to } 26.4 \text{ ft} \end{aligned}$$

Because tuff weighs 123 lb/ft³,

$$\begin{aligned} \text{roof load} &= 123 \times 7.2 \text{ to } 26.4 \times 123 \\ &= 0.8856 \text{ to } \underline{3.247} \text{ kips/ft}^2 \end{aligned}$$

It would be desirable to design the permanent support system for the average vertical load. Thus, the average vertical load is $1/2(2.722 + 3.247) \text{ kips/ft}^2 = 3 \text{ kips/ft}^2$.

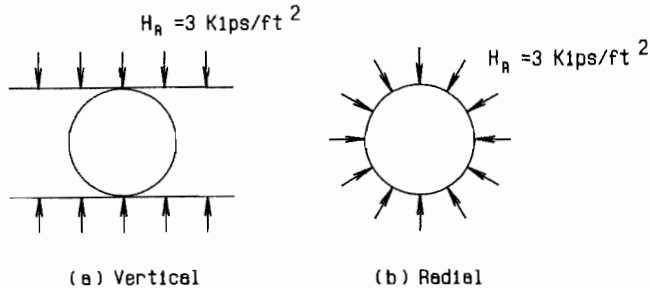


Fig. 5-10. Tunnel Liners Under Load (a) Vertical and (b) Radial.

Figure 5-10 is schematics of a tunnel liner, internal diameter of 12 feet (3.66 m), liner thickness of 6 inches (150 mm), and two loading conditions of 3 kips/ft² (0.14 MPa) (1) vertical, figure 2-10(a), and (2) radial, figure 2-10(b). The in situ modulus of deformation of rock = $1.34 \times 10^6 \text{ lb/in}^2$.

This problem can be solved by the use of elastic center or column analogy methods (see sections 5-2.1 and 5-2.3). Alternately, moment or reactions tables or a computer software package can be used. A software package analysis gave the following results shown in table 5-5.

TABLE 5-5
Stresses in the lining.

| Loading condition | Maximum stress (lb/in ²) | Minimum stress (lb/in ²) |
|---|---|---|
| I Only vertical loads with self weight of liner | 714 (compressive) | 283 (tensile) |
| II Only radial loads and liner's self weight | 404 (compressive) | 163 (tensile) |
| III Only radial loads, self weight neglected | 272 (compressive) | 272 (compressive) |

The beneficial effect of radial load is self evident in reduced stresses and no tensile stresses at any other section. Vertical loads only acting is a severe loading condition. It introduces moments and generates tensile stresses near the crown sections. A thorough backpacking behind liner reduces the possibility of inducing bending moments. This backpacking can be done by using suitable backfill grouting methods.

Concrete can withstand a tensile stress of 10 percent of its compressive strength. In this case, it can withstand a tensile stress of 300 lb/in², which is higher than the 283 lb/in² of the developed tension. The assumed section, therefore, is quite satisfactory. In case of a higher calculated stress, the section would have to be increased in thickness or higher strength of concrete or reinforced concrete will have to be used.

The stress calculations were performed by the use of equation 5-2(s) for combined thrust and moment; namely

$$\sigma = \frac{P}{A} \pm \frac{M}{Z} \quad 5-2(s)$$

where P is direct thrust, A is cross sectional area, M is bending moment, and Z is the relevant section modulus of the cross sectional area.

For radial load with self weight of structure ignored, the stress can be directly found with the use of equation 5-2(t).

$$\sigma_t = \frac{PD}{2t} \quad 5-2(t)$$

where P is the radial pressure, D is the mean diameter, and t is the lining thickness. This formula does not include the effect of rock support and the self weight of liner is not included.

Based on the use of equation 5-2(t), the stress in the liner comes to be 260 lb/in² which is less than 272 lb/in² of table 5-5, loading condition III. This is not a significant difference. The difference is due to disregard of the rock support in equation 5-2(t). Therefore, as a preliminary and quick check, use of equation 5-2(t) is recommended.

It is necessary to test the lining against buckling failure by the equation 5-2(u).

$$P_{CR} = \frac{3EI}{r^3} \quad 5-2(u)$$

where P_{CR} is the critical buckling load, E is the modulus of elasticity of concrete, I is the moment of inertia, and r is the radius of lining.

For the given liner, $E = 57,000 \sqrt{f'_{c28}}$; $P_{CR} = 599.42 \text{ lb/in}^2 = 86 \text{ kips/ft}^2$, which is greater than 3 kips/ft^2 and hence the assumed section can be recommended to be used. (The f'_{c28} is the 28 days unconfined compressive strength of 28-day-old concrete.)

5-3 SHAFT

Elliptical shaped shafts are no longer used. Either rectangular or circular shape is common for a modern shaft. Circular shafts are easier to machine drill or raise bore. Twenty feet (6 m) is the limit of raise bored shafts. Conventional shafts could be as large as 34 feet (10.36 m) in internal diameter. Shafts 116 feet (35 m) have been successfully excavated (Tunnels and Tunneling, 1986). The deepest shaft in South Africa is 11,500 feet (3505 m) deep. Table 5-6 shows the hole straightness in drilling shafts at Crown Point, New Mexico, USA.

TABLE 5-6
Hole straightness.

| | Depth of shaft (feet) | Diameter of shaft (feet) | Hole deviation (feet) |
|-------------|--------------------------|-----------------------------|--------------------------|
| Shaft No. 1 | 2,243 | 10 | 1.0362 |
| Shaft No. 2 | 2,188 | 6 | 1.33 |
| Shaft No. 3 | 2,188 | 6 | 0.84 |

Note: 1 ft = 0.3048 m.

The conventional method of drilling a shaft is by mechanical shovels (grabs) or by drill and blast methods. For deeper shaft blind drilling, raise boring or down slushing can be used.

In blind drilling, the shaft excavation is done through a mud or slurry filling. The slurry or mud provides passive support to the excavated faces of the opening. More mud or slurry is introduced as the shaft is excavated to deeper depths.

Raise boring and down slushing require a small sized pilot shaft to the full depth of the proposed shaft. Also, at the lower end of the pilot shaft there must be an access tunnel or adit through which the mucking can be removed. The cutter head of the raise boring machine is placed in the access tunnel or adit just below the center of the pilot hole. The stem of the raise boring machine is then lowered from the surface into the pilot shaft and is connected to the cutter head. The head frame of the raise boring machine then pulls the cutter head up and starts the rotation of the cutter head. The rock so excavated drops on the floor of the access tunnel and is removed. The process is continued until the full depth of shaft is excavated.

In down slushing method, the excavated rock is pushed down from the excavation face through the pilot shaft and the muck is removed through the access tunnel or drift. Mechanical excavation or drill and blast can be used for down slushing excavation.

In a host competent rock, the excavated shaft may remain unlined. It may need to be supported otherwise. Initial linings of steel ribs, timber cribs, rock bolts, and shotcrete may be used. The final lining may be of in situ concrete, precast; reinforced or plain concrete; cast iron tubings; liner plates; steel liners; and steel liners sandwiched between concrete. The thickness of steel lining may range from 9/16 to 3-1/8 inch (14 to 80 mm). Steel liners thicker than 1-5/16 inch (35 mm) will require welding from both sides. All welds need to be radiographed for proper jointing. The 28-day strength for concrete for shaft lining may range from 4,000 to 12,000 lb/in² (27.6 to 82.8 MPa), the common being 6,000 lb/in² (41.4 MPa) in the USA. Typical linings for shafts are shown on figure 5-11.

The external loads for the design of liners may be due to gravity, water, and the superimposed loads. In very deep shafts, release of tectonic stresses due to shaft excavation may induce rock bursts which can be controlled by methods discussed in chapter 7.

For shafts through weak zones, pregrouting with cement or chemicals may become necessary to maintain stability of excavation during construction. Because of the difficulties in future accesses for repair and maintenance, it is customary to use a larger factor of safety in the design and construction of shafts. The recommended factor of safety is 3.

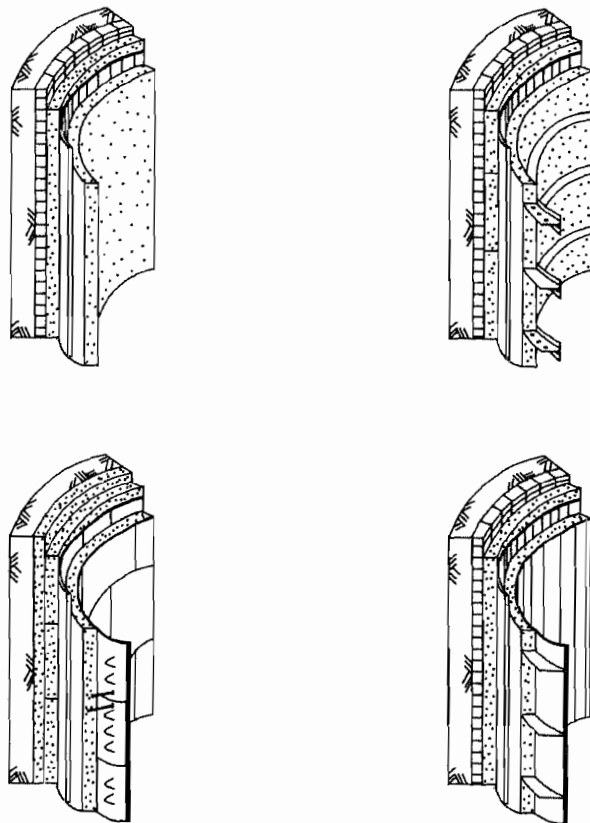


Fig. 5-11. Typical Shaft Linings.

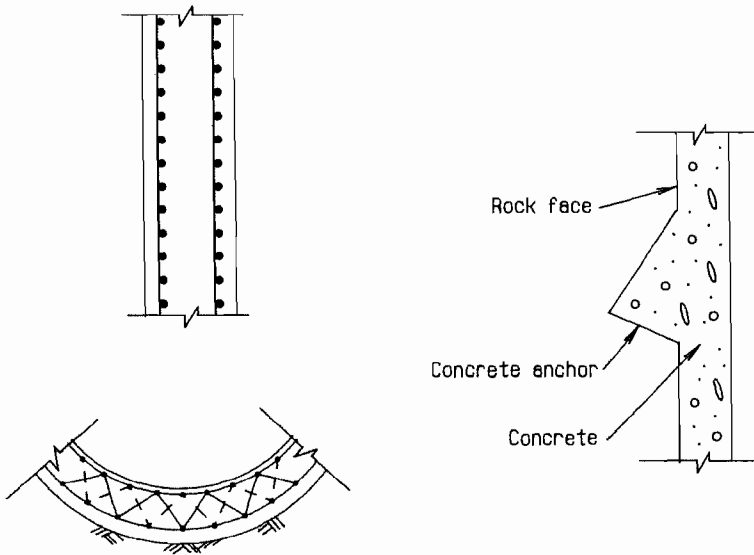


Fig. 5-11. Typical Shaft Linings (continued).

5-3.1 Design of a Shaft

A shaft is usually less severely loaded than a tunnel at the same depth of consideration but requires checks for both vertical and horizontal stabilities of opening. It is desirable to find in situ stresses by conducting tests. The hydrofracturing test is more advantageous for determining in situ stresses at great depths. In absence of in situ tests, the following equations may be used to estimate in situ pressures:

$$\sigma_v = \sigma_{zz} \quad 2-10$$

$$\sigma_H = \sigma_{xx} = \sigma_{yy} = \frac{\nu}{1 - \nu} \sigma_{zz} \quad 2-11$$

The factor $\frac{\nu}{1 - \nu}$ may be substituted by "K."

In engineering material " ν ," Poisson's ratio, can only attain a maximum value of 0.5, therefore K should not exceed the value of unity. But in many engineering situations, K has been found greater than 1. Hoek and Brown (1980) found the value K to range from

$$(0.3 + \frac{100}{z}) \text{ to } (0.5 + \frac{1,500}{z})$$

where "z" is the depth in meters.

Amadei et al. (1988) recommended the value K for ith layer in a stratified orthotropic rock mass to be

$$K_i = \nu_i' = \frac{E_i}{E_i'} \left[\frac{1}{1 - \nu_i} \right] \quad 2-11(a)$$

where ν_i' = Poisson's value in vertical direction for ith layer

ν_i = Poisson's ratio in horizontal direction for ith layer

E_i = modulus of elasticity in horizontal direction for ith layer

E_i' = modulus of elasticity in vertical direction for ith layer.

5-3.2 Model Selection

If the onset of plasticity is generated in a horizontal plane, a two-dimensional analysis for the design will be quite satisfactory. If a vertical instability occurs, it would be required to perform a three-dimensional analysis. In order to ascertain when a two- or three-dimensional analysis is required, it is necessary to define a parameter,

$$K_{CR} = 0.5 + \frac{1}{2K_p} - \frac{\sigma_{UM}}{2K_p\sigma_{ZZ}} \quad 5-3(a)$$

$$\text{where } K_p = \frac{1 + \sin \phi}{1 - \sin \phi}$$

σ_{UM} = unconfined rock mass compressive strength

$$K > K_{CR} \text{ or } K > 1$$

where k = in situ horizontal pressure/in situ vertical pressure

then horizontal plasticity will occur (McCreath, 1980). This occurs in most of the situations in rock shafts and, hence, two-dimensional analysis will seem justified in most of the cases.

If $K < K_{CR}$ or $K < 1/2$, then vertical plasticity will occur that will require a three-dimensional analysis. A three-dimensional analysis could be performed with a finite element software package. Care must be taken to use an aspect ratio not to exceed 3, for selecting the dimensions of elements in finite element analysis.

5-3.3 Estimation of Pressure for Shaft Lining Design

Equation 5-3(b) provides a means of estimating the pressure on a shaft lining. The equation is based on Talbore's (1957) recommendations for elastic rock.

$$P_{SL} = \left[\left[\frac{C}{K_p} + \sigma_H (1 - \sin \phi) \right] \left[\frac{r}{R_p} \right]^{K_p - 1} \right] - \left[\frac{C}{K_p} \right] \quad 5-3(b)$$

P_{SL} = pressure on shaft lining

$$\text{where } C = \frac{\sigma_{UM}}{2(K_p)^{\frac{1}{2}}}$$

σ_H = in situ horizontal rock pressure and, if not available, can be estimated by the use of equation 2-11.

ϕ = angle of friction

$K_p = (1 + \sin \phi) / (1 - \sin \phi)$

r = radius of shaft

R_p = relaxed zone radius

Equation 5-3(c) shows Terzaghi's (1943) recommendation for calculating P_{SL} .

$$P_{SL} = \left[\frac{2}{K_p + 1} \left[\sigma_H + \frac{\sigma_{UM}}{K_p - 1} \right] \left[\frac{r}{R_p} \right]^{K_p - 1} \right] - \left[\frac{\sigma_{UM}}{K_p - 1} \right] \quad 5-3(c)$$

with the symbols carrying the same meaning as of equation 5-3(b).

The relaxed zone radius " R_p " can be found by the use of equation 2-17

$$R_p = r \left[\left[\frac{2}{K_p + 1} \right] \left[\frac{\sigma_{UC} + P_0 (K_p - 1)}{\sigma_{UC}} \right] \right]^{\frac{1}{K_p - 1}} \quad 2-17$$

or by using equation 5-3(d) proposed by Greenslade and Richards (1981).

$$l = R_p - r = \frac{r}{\left[\left[\frac{\sigma_{UM}}{P_0} + (K_p - 1) \right] / (K_p + 1) \right]^{\frac{1}{2}}} \quad 5-3(d)$$

P_0 may be equal to $\sigma_H = \sigma_{xx} = \sigma_{yy}$

For shafts through cohesive or cohesionless soil, the pressures can be estimated by using any soil mechanics principles (Terzaghi, 1943).

5-3.4 Evaluation of Stresses

For shafts with $K > 1$ or $K > K_{CR}$ (see section 5-3.2), the radial stress σ_r and tangential stress σ_θ can be evaluated by use of equations 2-14(a) and 2-14(b).

$$\sigma_r = \sigma_H \left[1 - \left[\frac{r}{R} \right]^2 \right] + P_i \left[\frac{r}{R} \right]^2 \quad 2-14(a)$$

$$\sigma_\theta = \sigma_H \left[1 + \left[\frac{r}{R} \right]^2 \right] - P_i \left[\frac{r}{R} \right]^2 \quad 2-14(b)$$

For determining stresses at a distance R in plastic zone, substitute r with R_p and P_i with σ_p where σ_p is the stress at the boundary of elastic and plastic zone, $\sigma_p = P_0$.

5-3.5 Vertical Instability

When $K < K_{CR}$ or $K < 1/2$, then vertical instability can occur. A three-dimensional analysis will be required. If the instability is caused on a single plane of weakness, the analysis becomes very simple.

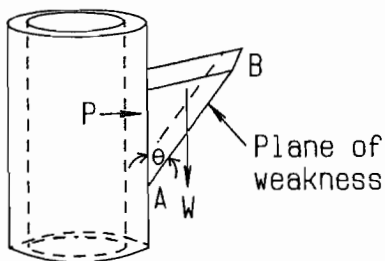


Fig. 5-12. Vertical Wedge.

If AB on figure 5-12 is the plane of weakness, and W is the total load of the block of rock which makes an angle of " θ " with the vertical face of shaft, then the force causing sliding down the face AB

$$= W (\cos \theta - \sin \theta \cdot \tan \phi)$$

where ϕ is the angle of friction at the sliding plane.

The horizontal force "P" required to provide resistance to falling along the plane AB

$$= P_{SL} (\sin \theta + \cos \theta \cdot \tan \phi_1)$$

where ϕ_1 is the angle of friction at the interface of lining and the rock. Thus, the factor of safety is given by equation 5-3(e).

$$FS = \frac{P_{SL} (\sin \theta + \cos \theta \cdot \tan \phi_1)}{W (\cos \theta - \sin \theta \cdot \tan \phi)} \quad 5-3(e)$$

FS should not be less than 3. "P_{SL}" can be calculated from equation 5-3(b or c).

When the vertical sliding is due to sliding on more than one plane, the analysis becomes more involved but follows the same principles. In that case, one has to find the total weight of block that is trying to fall down and also find the resistive forces inclusive of skin friction resistance at the shaft and rock interfaces. Once the forces causing sliding and resisting are calculated, the factor of safety can be determined.

If the weight of the shaft lining cannot be resisted by the developed skin friction at the rock and shaft interface, then the lining must be supported against vertical sliding. A foundation block, rock bolts, or concrete anchors (see figure 5-11) can be used to provide vertical supports to the shaft wall. The concrete anchors, if used, must not be spaced more than 150 feet (45 m) apart. A concrete anchor supporting a shaft wall is shown in figure 5-11.

5-3.6 Breakdown of Cost

Table 5-7 provides a cost breakdown of a shaft at Crown Point, New Mexico.

TABLE 5-7
Cost breakdown.

| | Cost in percent |
|------------------------|-----------------|
| Site preparation | 6.9 |
| Rig | 20.2 |
| Bits and stabilization | 10.9 |
| Logging and surveying | 1.4 |
| Casing | 21.0 |
| Welding of casing | 12.0 |
| Cement for grouting | 6.9 |
| Fuel cost | 2.6 |
| Supervision | 2.9 |
| Excavation | 15.2 |
| | 100.0 |

A vertical shaft is less expensive than a declining shaft when communicating the same vertical depth and having same internal cross sectional area. Shaft with -5° (8.7 percent grade), -10° (17.6 percent grade), and -15° (26.8 percent grade) have been drilled but all have shown to be more expensive than the vertical shaft: -5° being the most expensive, -10° moderately expensive, and -15° being more expensive than vertical shaft. The unit cost for shaft construction decreases with the depth of shaft. Deeper shafts cost less per unit vertical depth than shallow shafts.

5-4 CAVERN

An underground opening having a cross sectional area of $1,000 \text{ ft}^2$ (120 m^2) or more (Einstein, 1987) and an axial dimension of not exceeding 15 times the lateral dimension is classified as a "cavern." It is a large opening and always requires a three-dimensional analysis by empirical, analytical or numerical methods. Caverns are used for underground pumped storage, powerhouses and powerplants, subway stations, storage facilities, parking garages, swimming pools, shelters, testing facilities, and recreational uses.

The roof of the cavern, traditionally, is a circular arch but it could be multiradial or elliptical to suit the host geology. Trapezoidal roofs with side haunches limiting the length of flat spans of the roof were used for the caverns at Drakensburg, South Africa, and Poatina, Tasmania. The sides of the cavern could remain straight or curved. The invert of the cavern could be straight or remain curved. Some of the shapes of caverns are shown on figure 5-13.

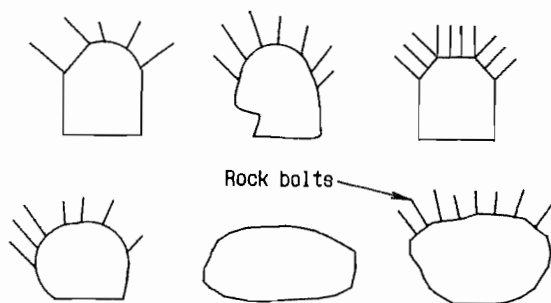


Fig. 5-13. Different Geometrical Shapes of Caverns.

The support systems for roof, sides, and invert are usually a designed system of rock bolts, but, very often, bents and frames have been used to support the excavation. The load which the support system has to sustain

depends on the prevailing geological characteristics, used sequence and method of construction, and the existing hydrogeological conditions. Heading and bench method, and multidrift method are known to reduce the loads on the support system.

5-4.1 Analysis and Design of Cavern

In order to analyze the support system for a cavern, the loads on roof, sides, and inverts have to be estimated. The analysis should be performed for the combination of loading which creates the maximum stress and deformation. Figure 5-14 shows the schematic loading for a cavern.

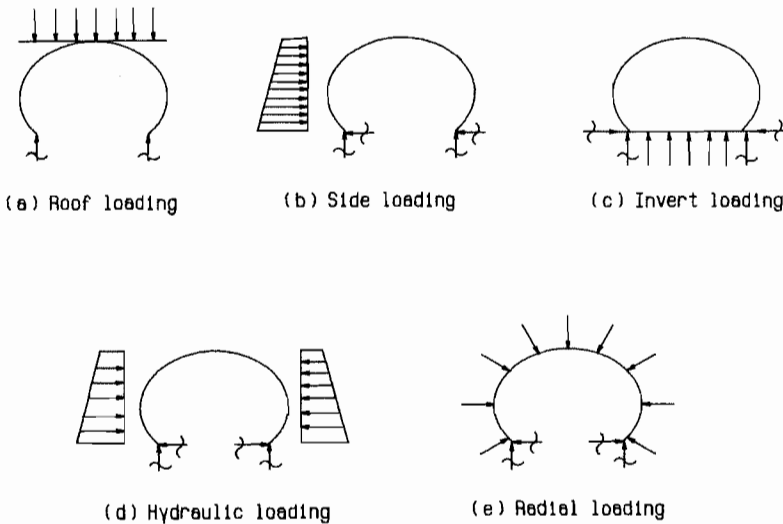


Fig. 5-14. Schematic Loading for Analysis.

In analyzing the support system for a cavern, the ground structure interaction may be ignored. This will be true in the case when a plastic liner is placed between the rock face and the structural support system. If the invert is cast at a later time, the structural analytic model for a crown loaded cavern in a two-dimensional model will appear to be that shown on figure 5-14(a). The ground contribution can be represented by a series of radial or transverse springs or a combination thereof as shown on figure 5-15. Care must be taken to deactivate the springs when tensile forces developed in the springs exceed 50 percent of the tensile strength of the geological media.

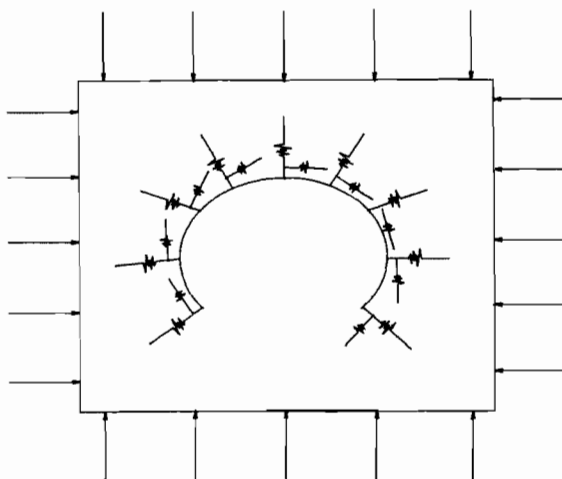


Fig. 5-15. Analytical Model for Supports (Radial and Tangential Springs).

The loads can be estimated by using Terzaghi's method (section 2-4) or "Q" system (section 2-5) or by conducting a hydrofracturing test or performing in situ stress measurements. Once the load has been estimated and a model for analysis selected, the analysis can be performed by using analytical (sections 5-2.1 and 5-2.2) or numerical methods.

Three-dimensional finite element programs such as ADINA (Bathe, 1975), which has options of element birth and death, have been used successfully. Boundary element, distinct element, and hybrid element programs can also be used. A list of such programs and their capabilities were listed by Brebbia (1982).

It is important when using finite element programs developed by others that the user has access to the pre- and post-processing softwares and that the finite element program is first tested on an already existing design. The pretesting will provide confidence in the numerical analysis.

5-4.2 Closed Form Solution

Closed form solutions for cavern require that the boundary of the three-dimensional opening must be expressible as a mathematical function of the coordinate system. This sometimes seriously limits the application of the closed form solution for the analysis of caverns because the boundaries cannot be expressed as a mathematical function of the coordinate system. Because three-dimensional analysis solutions always involve the use of representative elastic constants in the three coordinate systems, the solutions sometimes become quite complicated, cumbersome, and unmanageable. Using three-

dimensional elasticity, stress distribution about spherical, ellipsoidal, and spheroidal cavities have been obtained (Timoshenko and Goodier, 1951). It must be stated that a two-dimensional analysis for an opening in an infinite medium is independent but the three-dimensional analysis is fully dependent on the elastic constants.

5-4.3 Block Analysis

In order to design a rock bolt system, the size of blocks must be estimated. The factor of safety for rock bolts used for stability of cavern opening must not be less than 3 because of the difficulty in accessibility for future maintenance. The rock block size determination will require the study of discontinuities, estimation of coefficient of friction, and cohesion of joint material and fillings. Once these are determined, the rock bolts can be designed by methods discussed in chapter 4.

5-4.4 Other Models

In complicated geological environment and complicated geometrical shape of opening, it might be advantageous to analyze the structure on a three-dimensional physical scaled model. The failure mode of the roof, sides, and inverts can also be studied on "base friction tables" and the structure modified accordingly.

5-5 PRESSURE TUNNELS AND SHAFTS

Sometimes tunnels and shafts have to convey fluids, oil, or water or gas under pressure. The tunnels and shafts have to be strong enough so that they adequately confine and transport the fluid under pressure. When fluids escape into the host medium, they may adversely impact the environment and create undesirable socio-economic effects. If the rock is competent and contains very little discontinuities, it can convey water under pressure without a lining (Broch, 1984). However, before recommending an unlined pressure tunnel, in addition to geology, one must evaluate the increased size requirement of tunnel and increased cost of maintenance of an unlined tunnel or shaft. The permissible velocity for an unlined tunnel or shaft is 5 feet per second (1.5 m/s) against a lined tunnel of 20 feet per second (6 m/s). Thus, a larger tunnel or shaft will be required for an unlined tunnel or shaft than a lined one to pass the same discharge.

Depending on geology, an unlined pressure shaft has been designed to sustain 1,493 feet (455 m) of pressure head (Bergh-Christensen, 1982), and another one was being investigated to sustain 3,215 feet (980 m) of pressure head without requiring a lining (Bergh-Christensen and Kjolberg, 1982). Because air leaks at a much faster rate, 1,000 times that of water, it may be essential to

provide an impermeable lining for gas or air flow shafts and tunnels. In the following sections we will discuss only design of pressure tunnels which carry water. Pressure shafts that carry water are designed similar to pressure tunnels and are checked for vertical and horizontal stability, discussed in section 5-3.

5-5.1 Pressure Tunnels

The leakage of high-pressure water can create hydraulic jacking of the strata; loss of fluid and pressure; raise ground water tables; flood basements of nearby structures; create new surface springs in colluvium, glacial moraines, weathered rock, and lacustrine deposits; induce rock instability; and create landslides or mudslides by increasing the pore pressures and reducing the shear strength of the slope. They can also induce deterioration of the rock mass if the rock is erodible or contains dissolvable gauges. Limestone, gypsum, and salt rocks may suffer considerable rock mass deteriorations due to leakage of water. Figure 5-16 shows the horizontally and vertically formed separations (cracks) due to vertical and horizontal jacking of the strata by leaking waters.

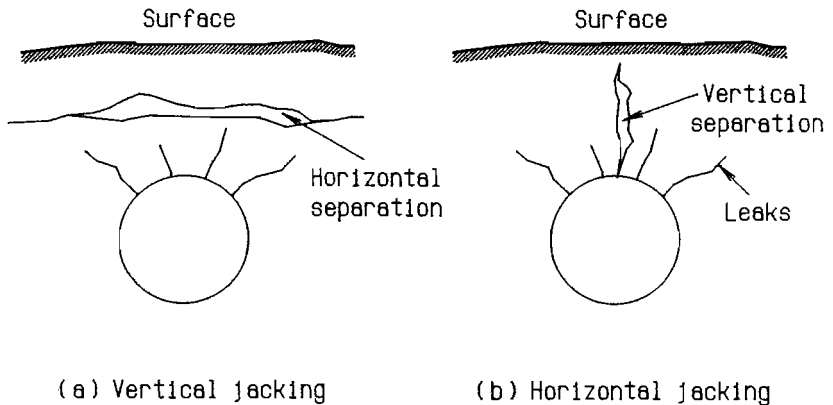


Fig. 5-16. Hydraulic Jacking of Ground.

In a failure study of pressure tunnels (Brekke and Ripley, 1986), it was found that 59.25 percent of tunnel failures were attributed to water leakage, 22.33 percent to rock mass deterioration, 12.96 percent to buckling of steel lining, and 5.56 percent to rupture of lining due to internal pressure. Thus, while designing pressure tunnels and shafts, emphasis on leakage control becomes necessary. To reduce leakage, the allowable stress in reinforcement of

a concrete lining of pressure tunnels is limited to 12,000 lb/in² (82.75 MPa) (Pinkerton et al., 1964).

5-5.2 Lining for Pressure Tunnels

Plain concrete, reinforced concrete, mild steel continuous membrane, and a combination of reinforced concrete with steel liner can be used to line a pressure tunnel. Table 5-8 provides an empirical guide for types of liners to be used for different internal heads of water pressures.

TABLE 5-8
Internal heads of water and types of liners.

| Internal heads | | Pressures | | Type of liners |
|----------------|-----------|-----------------------|-------------|---|
| (feet) | (meters) | (lb/in ²) | (MPa) | |
| 115 | 35 | 50 | 0.34 | Plain concrete |
| 115 - 230 | 35 - 70 | 50 - 100 | 0.34 - 0.69 | Reinforced concrete |
| 230 - 506 | 70 - 154 | 100 - 220 | 0.69 - 1.52 | Mild steel, continuous membrane |
| Above 506 | Above 154 | > 220 [†] | > 1.52 | Steel liners and reinforced concrete lining |

Internal head must include the consideration of operating conditions. The operating conditions must include normal surge pressures and surges due to emergency closure of the pressure tunnel. The internal design pressure head may be 100 to 200 percent of the normal static head.

The minimum thickness of steel liners is also controlled by handling stresses. The minimum thickness for handling should be the maximum of the following T_s shown in equations 5-5(a).

$$1) T_s = \frac{2R_s + 20}{400} \quad 5-5(a)$$

$$2) T_s = \frac{R_s}{144}$$

$$3) T_s < \frac{1}{4} \text{ inch (6 mm)}$$

$$4) T_s = \frac{1}{250} \text{ to } \frac{1}{300} \text{ of internal diameter}$$

Additional thickness must be provided for corrosion. Steel corrodes 0.005 inch per year (0.127 mm/yr) (Uhlig, 1971). For concrete liners, the thickness

$$5) T_C = \frac{1}{50} \text{ to } \frac{1}{55} \text{ of the internal diameter}$$

where T_S = thickness of steel liner in inches

T_C = thickness of concrete liner in inches

R_S = internal radius of steel liner in inches

5-5.3 Cover

Steel liners are required when the natural rock vertical cover is less than the required cover shown in equation 5-5(b) (which is a modified equation) (Bergh-Christensen and Dannevig, 1971),

$$V_C = \frac{H_W v_w FS}{v_r \cdot \cos \beta} \quad 5-5(b)$$

where V_C is vertical cover measured normal to the pressure tunnel alignment

H_W is the static head of water

v_w is the unit weight of water

v_r is the unit weight of rock

FS is acceptable factor of safety (usually 2.0)

β is the angle which the tunnel makes with the horizon

The U.S. Bureau of Reclamation and U.S. Army Corps of Engineers prefer V_C not less than 0.45 to 0.5 of H_W .

Steel liners with covers less than or equal to V_C are to withstand the full hydrostatic head as a freestanding structure and any contributing effects of host rock in resisting the internal pressure is completely ignored.

Steel liners are also required for pressure tunnels if the minimum horizontal or side cover of rock on the pressure tunnels or shafts is less than twice V_C .

5-5.4 Internal and External Pressures

The host media may take up to 0 to 70 percent of the internal pressure, 0 percent being for incompetent and 70 percent being for the most competent rock. Media participation in sharing internal pressure can be induced by increasing the competence of host rock by pressure grouting. The required lining thickness is reduced when the host media shares the internal pressure. Internal pressure for design must include surge pressures during normal operations and emergency shutdowns (see section 5-5.2).

With time, external water pressures build around the pressure tunnels and pressure shafts. The external pressure build up ranges from 30 to 100 percent of the design internal pressure. The external water pressures can be reduced by using drainage galleries, French drains, relief drains, weep holes, and by grouting around the pressure tunnel or pressure shaft.

A combined system using drainage and grouting can be effectively used to reduce and control the external water pressures. Pressure tunnel linings must be designed to withstand a minimum of 75 lb/in² (0.52 MPa) of external pressure (Amstutz, 1953).

5-5.5 Pressure Tunnel Design

Pressure tunnel lining has to be designed for (1) external geological loading as a nonpressure tunnel (see section 5-2), (2) for only internal hydraulic pressure, and (3) for only external hydraulic pressures. The stresses in these three loading conditions are to be superimposed for design.

(i) Design for internal pressures. The design for internal pressure requires the determination if the internal pressure is fully resisted by the lining alone or in combination with the host rock. If steel takes the full internal pressure, the allowable stress in steel is considered to be the lower of 70 to 100 percent of the yield stress and 50 to 67 percent of the ultimate tensile stress. When host rock takes partial internal pressure, the allowable stress in steel lining is considered to be the lower of one half to two thirds of the yield stress and one third the ultimate tensile stress. The thickness for a free-standing liner is given by equation 5-5(c).

$$t_{SL} = \frac{P_i R_s}{\sigma} \quad 5-5(c)$$

where t_{SL} = thickness of steel liner

P_i = internal pressure

R_s = radius of steel liner

σ = allowable stress

If the steel lining is encased in concrete, due to shrinkage it is possible that an annular gap forms near the interface of the steel and concrete linings. Customarily, the annular gap thickness is considered to vary from 2×10^{-4} to 4×10^{-4} times the radius of the steel lining. The formation of annular gap is dependent on the temperature variation and is given in equation 5-5(d).

$$\Delta_G = \alpha \Delta_T R_S$$

5-5(d)

but not less than $(2 \times 10^{-4} \text{ to } 4 \times 10^{-4})R_S$

where Δ_G = gap thickness

α = coefficient of expansion of steel = 6.5×10^{-6} in/in/oF

Δ_T = range of temperature variation

R_S = internal radius of steel liner

The sharing of internal pressure by host rock can be determined by the application of elastic theory. The steel liner is considered as a thin cylinder and the concrete lining as a thick tube. The steel liner must bridge over the annular gap and come in contact with the concrete encasement. The concrete is considered to remain in contact with the cracked rock. In figure 5-17 is shown the schematics of the liners; only one half section has been shown for clarity.

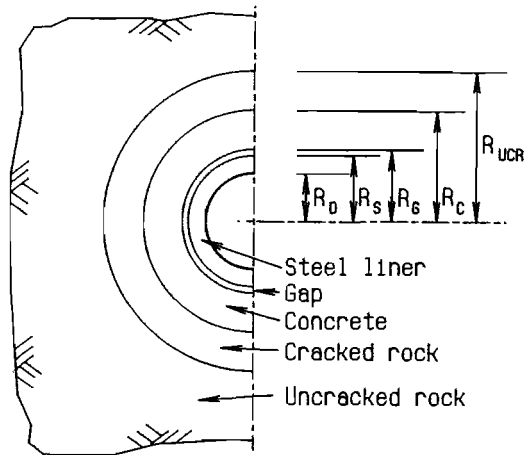


Fig. 5-17. Steel Liner Encased in Concrete

In order to maintain contact, at the interfaces, equation 5-5(e) must hold good

$$\Delta_S = \Delta_G + \Delta_C + \Delta_{CR} + \Delta_{UCR}$$

5-5(e)

where Δ_S = deformation of steel lining
 Δ_G is given in equation 5-5(d)
 Δ_C = deformation of concrete lining
 Δ_{CR} = deformation in the cracked rock zone
 Δ_{UCR} = deformation in the uncracked rock zone

In order to find the various terms in equation 5-5(e), we will assume that the plane strain conditions hold, P_i is the internally applied known pressure, and P_s is the unknown pressure taken by the steel liner only.

Because steel lining is considered as a thin cylinder, stress in steel lining is given by equation 5-5(f).

$$\sigma_s = \frac{P_s R_s}{t_{SL}} \quad 5-5(f)$$

And because plane strain conditions exist, strain

$$\epsilon_s = \frac{\sigma_s}{E_s} (1 - \nu_s^2)$$

Where E_s is the modulus of elasticity of steel lining
 ν_s is the Poisson's ratio of steel lining

and hence $\Delta_S = \epsilon_s R_s = \frac{\sigma_s}{E_s} R_s (1 - \nu_s^2)$ or

$$\Delta_S = \frac{P_s R_s^2}{t_{SL} E_s} (1 - \nu_s^2) \quad 5-5(g)$$

Because no other forces act, it may be shown that (see figure 5-17)

$$2 \pi R_G (P_i - P_s) = 2 \pi R_C (P_C)$$

where P_C is the pressure at the concrete and cracked rock interface or

$$P_C = \frac{R_G}{R_C} (P_i - P_s)$$

$$\begin{aligned} \text{Average pressure in concrete} &= \frac{1}{2} \left[(P_i - P_s) + \frac{R_G}{R_C} (P_i - P_s) \right] \\ &= \frac{1}{2} (P_i - P_s) \left[\frac{R_C + R_G}{R_C} \right] \end{aligned}$$

$$\begin{aligned}
 \text{Hence } \Delta_C &= \frac{1}{2} (P_i - P_s) \frac{R_C + R_G}{R_C} \frac{(1 - \nu_C^2)}{E_C} (R_C - R_G) \\
 &= \frac{1}{2} \frac{(P_i - P_s)}{E_C} \frac{(1 - \nu_C^2)}{R_C} (R_C^2 - R_G^2)
 \end{aligned} \tag{5-5(h)}$$

Pressure " P_d " at the cracked and uncracked zones interface is similarly given by the expression

$$P_d = \frac{R_G}{R_{UCR}} (P_i - P_s)$$

$$\text{Average pressure in cracked rock zone} = \frac{P_C + P_d}{2}$$

$$= \frac{1}{2} \left\{ \frac{R_G}{R_C} (P_i - P_s) + \frac{R_G}{R_{UCR}} (P_i - P_s) \right\}$$

$$= \frac{1}{2} (P_i - P_s) R_G \frac{R_{UCR} + R_C}{R_{UCR} \cdot R_C}$$

$$\Delta_{cr} = \frac{1}{2} (P_i - P_s) \frac{R_G}{R_{UCR} R_C} \frac{(1 - \nu_{cr}^2)}{E_{cr}} \frac{(R_{UCR}^2 - R_C^2)}{2} \tag{5-5(i)}$$

In order to find Δ_{UCR} , we apply the elastic theory for deformation of an infinite mass for a circular excavation.

$$U = \frac{1 + \nu}{E} RP \quad \text{and get}$$

$$\Delta_{UCR} = \frac{1 + \nu_{UCR}}{E_{UCR}} \cdot R_{UCR} \frac{R_G}{R_{UCR}} (P_i - P_s)$$

$$= (P_i - P_s) \frac{1 + \nu_{UCR}}{E_{UCR}} R_G \tag{5-5(j)}$$

Now all the terms of the equation 5-5(e) being known,

$$\begin{aligned}
 P_s = & \frac{[d\Delta_T R_s + 0.5 P_i \{ (\frac{1 - \nu_c^2}{E_c}) \frac{1}{R_c} (R_c^2 - R_G^2) + \\
 & \frac{R_G}{R_{ucr} R_c} \frac{(1 - \nu_{uc}^2)}{E_{cr}} (R_{ucr}^2 - R_c^2) + \\
 & 2 (\frac{1 + \nu_{ucr}}{E_{ucr}}) R_G \}]}{[\frac{R_s^2 (1 - \nu_s^2)}{t_{SL} E_s} + 0.5 \{ (\frac{1 - \nu_c^2}{E_c}) \frac{1}{R_c} (R_c^2 - R_G^2) + \\
 & (\frac{R_G}{R_{ucr} R_c}) (\frac{1 - \nu_{uc}^2}{E_{cr}}) (R_{ucr}^2 - R_c^2) + \\
 & 2 (\frac{1 + \nu_{ucr}}{E_{ucr}}) R_G \}]} \quad 5-5(k)
 \end{aligned}$$

Once P_s is found, the design of steel and concrete lining can be performed.

Though equation 5-5(k) seems formidable, it is easy to perform calculations in steps as shown in equations 5-5(g) through equation 5-5(j) in terms of P_s and $(P_i - P_s)$ and then apply equation 5-5(e) to solve for P_s .

Similar discussions on sharing of internal pressure by host rock were provided by Patterson et al., 1957, assuming plane stress conditions, and by Vaughan, 1956, assuming linear pressure variation with the lining.

As can be realized that the preceding discussion of pressure sharing by rock is based on the creation of a cracked zone of rock having an external radius of R_{ucr} which can be approximately estimated to be $3 \times R_D$ or alternatively R_{ucr} can be estimated by use of equation 5-3(d).

For a more realistic analysis, it is recommended to perform a two-dimensional finite element analysis using shell elements and representative rock, concrete, and steel material properties. A three-dimensional finite element analysis is more expensive to perform but may be desirable for large projects.

(ii) Design for external pressures. The pressure tunnel lining as designed in section 5-5.5(i) must be checked against external pressure. The magnitude of external pressure is difficult to predict. Historically, the external head for design has varied from 15 percent to 100 percent of the static head of the pressure tunnel but not less than the head exerted due to existing ground water table. Because external head usually exerts compressive stresses in circular pressure tunnel or shaft linings, the lining has to be checked against buckling

stresses. The buckling resistance could be increased by increasing the thickness of liners. Use of stiffener rings to increase buckling resistance is not popular any more because such uses require larger excavations and create concrete placement difficulties. Drainage galleries, pipes, or drains can be used to reduce the external water pressures, thereby decreasing the buckling forces and requiring thinner lining.

Some of the simpler equations to determine critical buckling loads are given in equations 5-5(1) and 5-5(m).

$$P_{cr} = \frac{2E}{1 - \nu^2} \left(\frac{t}{D}\right)^3 \quad 5-5(1)$$

$$P_{cr} = \frac{3EI}{R^3L} \quad 5-5(m)$$

where P_{cr} = critical buckling pressure
 t = thickness of lining
 D = diameter of lining
 E = modulus of lining
 R = radius of lining
 L = length between stiffener rings

For a comprehensive treatment of buckling of steel liner, the reader may reference Windenburg and Trilling, (1960); Steel Plate Engineering Data, vol. 4, (1984); and Amstutz, (1953).

5-6 INTERSECTIONS

Intersections are essential units of an underground structure and connect shafts with tunnels or caverns. Intersecting tunnels, shafts, and caverns form configurations which may be either a cross, tee, yee, or L-shaped, as shown on figure 5-18.

An intersection should be examined from the points of view of (1) stress concentration, (2) roof or strata separation, and (3) opening instability.

The structural analysis of an intersection requires a three-dimensional analysis by finite element, boundary element, or boundary integral methods. Alternatively, three-dimensional analysis using physical modeling or photo-elasticity techniques can be used. Many available software programs assist in performing the numerical analysis. Previous analysis of intersections have indicated that the three-dimensional analysis zone may extend to distances lying within two to six times the maximum dimension of intersecting

openings. The distances were measured from the intersection of the center lines of intersection units. The sizes of main and auxiliary intersecting units, the ratios of existing ground stresses, and the angle of intersections influence the distance of the three-dimensional analysis zone and also the magnitude of stress concentration.

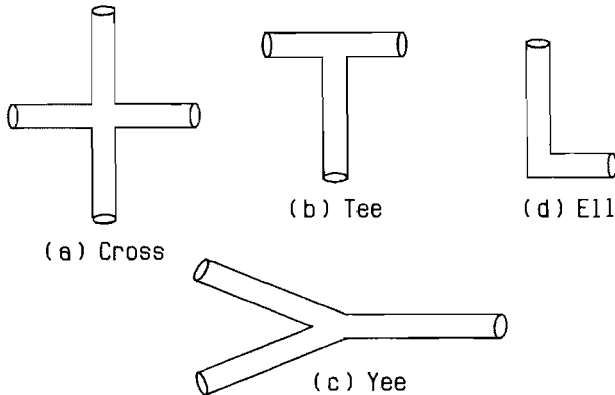


Fig. 5-18. Configurations of an Intersection.

For a "T" intersection, the stress concentration factors generally range from 1.0 to 6.0, under action of uniaxial ground condition ($\sigma_1 \neq 0$, $\sigma_2 = \sigma_3 = 0$). When the principal stress factors are greater than zero, the stress concentration factor seems to be lower than when the principal stress factors equal zero. Brown and Hocking, (1976), found the stress concentration to be as high as 35.7 in pillars of a "Y" intersection when principal stress factor (k) was zero.

If facilities for three-dimensional analysis do not exist, then one can use some approximation methods. One approximation method consists of superimposing the stresses obtained from equations 2-12 in one arm of the intersection with Poisson's ratio times the stresses obtained in other arms by using the same equations 2-12. While superimposing, the directions of stress vectors must be given due consideration.

Some designers reduce the three-dimensional problem to two-dimensional analysis for the intersection by providing a breakout section at the intersection. The breakout has to be reinforced to form either a system of beams, columns, frames, or arches, that will support the load coming at the intersection. The nearby zone at the breakout section hosts hidden beams, columns, frames, or arches to which the auxiliary units of the intersection are connected. Lew, (1976), found that intersections forming "Y" configurations

create higher stress concentration factors compared to intersections forming 90° configurations. He found that the the crown at the intersection deflected 40 percent larger than the crown of sections lying at distances six times the diameter of the tunnel.

5-7 MULTIPLE TUNNELS

As stated earlier, creation of an underground opening forces the native stresses and displacements to readjust. Based on simple elastic theory, this zone of readjustment can include points that are located up to five times the diametral distance from the center of the opening. Thus, if another tunnel is located such that the pillar thickness equals $9(R_1 + R_2)$, where R_1 and R_2 are diameters of two adjacent tunnels, then the readjustment of stresses and displacements will be independent of the second tunnel. In actuality, rock is not that homogenous and as such, an elastic theory may not be applicable. In practice, if the pillar thickness equals the diameter of the largest adjacent tunnel, the multiple openings behave as a single opening (Barla and Ottoviani, 1974). In case the intervening pillar is unable to resist the roof load, then the two adjacent tunnels must be treated as a larger tunnel, as if the opening spans to cover both the tunnels.

Complicated configurations for multiopenings will require a three-dimensional numerical analysis which in itself is costly and time consuming. A two-dimensional numerical analysis is much easier to run and provides 5 to 10 percent more conservative design compared with a three-dimensional analysis.

For five circular holes spaced one diameter apart, the stress concentration factor is 3.28 where for an infinite row of holes, it is 3.24 (Obert et al., 1960). Based on experimental data, Obert recommended the value of stress concentration factor in pillars for multiple openings to be

$$K = C + 0.9 \left[\left[\frac{W_0}{W_p} + L \right]^2 - 1 \right] \quad 5-7(a)$$

where C = stress concentration factor for a single opening under unidirectional stress field

W_0 = width of opening

W_p = width of pillar

The equation 5-7(a) is valid for circles and ovaloids that have height to width ratio of 0.5 to 2.0.

5-8 VERY LARGE STRUCTURES

Very large underground structures may be required for gymnasiums, storage, recreation centers, convention halls, and civil defense. They are designed as a very large cavern. The construction is usually heading and bench and the method of excavation is drill and blast. Rock bolts and shotcrete are used as temporary support. The final support is a structural support as necessitated by the functional requirement of the underground structure. Sometimes through pilot holes, special arch ribs are excavated and filled with reinforced or plane concrete. These reinforced or plane concrete arches are then used as structural members to support the future very large underground openings.

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Chapter 6

DESIGN AND ANALYSIS OF UNDERGROUND STRUCTURES IN SWELLING AND SQUEEZING ROCKS

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6-1 THE PHENOMENA

Both swelling and squeezing cause an inward movement of the tunnel* periphery over time. The intensity of the movement rate and the magnitude of the displacements often vary over the tunnel surface depending on the geology, on the original stress state and on the shape of the tunnel. Swelling is due to volume increase caused by water uptake and often occurs without yielding, while squeezing is essentially associated with creep caused by exceeding a limiting shear stress. Nevertheless, in dilatant material, squeezing can also be associated with volume increase, while on the other hand, swelling induced stresses and material modifications may cause time dependent yielding. Swelling and squeezing can occur in both rock and soil. Note that in brittle rock, excessive stresses will lead to rock bursts (see Chapter 7); while this phenomenon is very different from squeezing, the underlying cause, i.e. stresses exceeding a certain limit, is similar.

Most occurrences of swelling ground are associated with argillaceous soil or rock; swelling in anhydrite or mixed anhydrite-argillaceous rock is less frequent but may actually cause the most severe problems. Squeezing, as the preceding description of the phenomenon implies, can occur in any soil or rock as long as the particular combination of induced stresses and material properties pushes some zones around the tunnel beyond the limiting shear stresses at which creep starts.

So far the swelling and squeezing phenomena have been described as time dependent movements. Clearly if these movements are completely or partially inhibited by the tunnel support, they will cause substantial stresses in these supports. Swelling and squeezing can thus lead to significant load increases in, and eventual failure, of tunnel supports which may in turn be accompanied by significant movements. Depending on type and dimension of the

* The term tunnel is used in this chapter to describe any artificially created underground opening.

support on the one hand, and tunnel geometry and geologic conditions on the other hand, and tunnel geometry and geologic conditions on the other hand, any combination of deformation and support load may result. The cases discussed in the next section illustrate these phenomena.

6-2 Some Cases

Swelling and squeezing rock have caused problems in tunneling since underground openings have been created by man. Detailed documentation on swelling and squeezing problems in tunnels exists since the start of railroad tunnel construction in the last century. Rziha (1867) discusses a number of cases, in particular the Czernitz tunnel, which was built in the 1850's. Fig. 6-1 shows the substantial invert heave and crown deformation of the final opening while Fig. 6-2 illustrates deformation in the crown drift. From the description of the geology (gypsum, gypsum with clay, sand) and of the observed phenomena, it is not quite clear if swelling or squeezing predominates. This may thus be a good example for the combination of these modes of deformation.

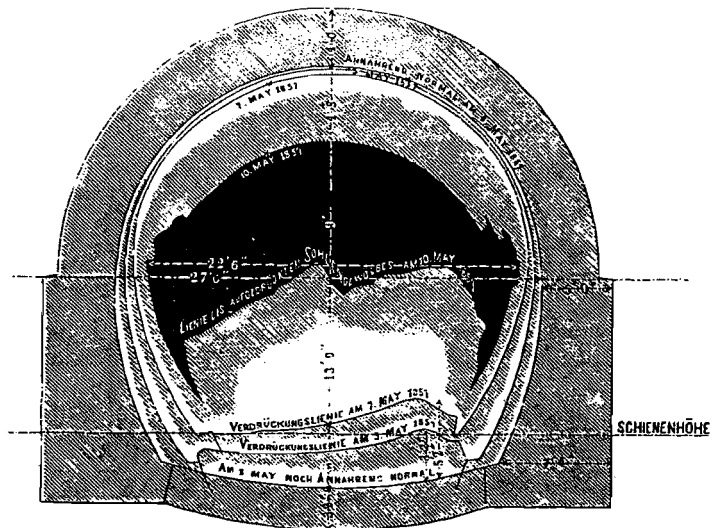


Fig. 6-1. Invert and crown displacements in Czernitz Tunnel (From Rziha, 1867).

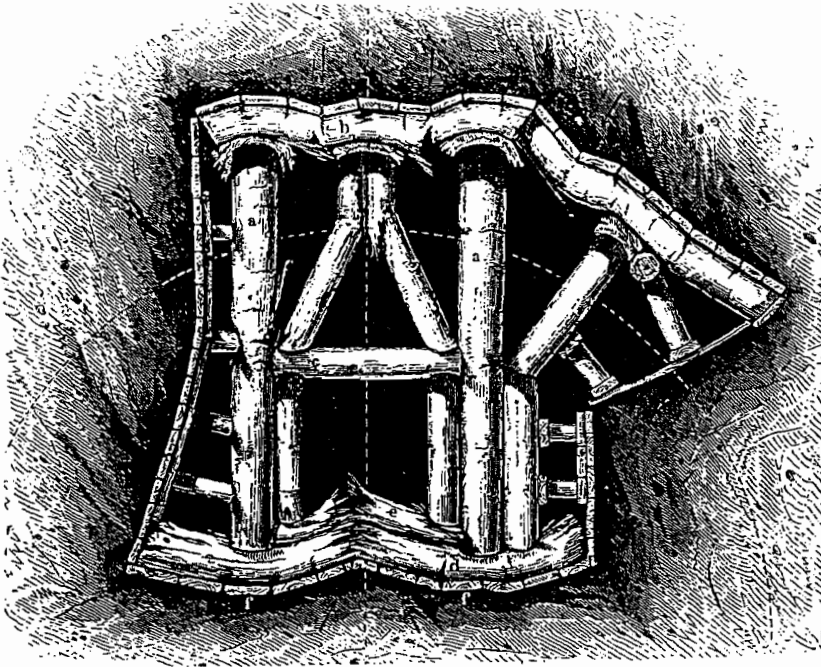


Fig. 6-2. Deformation in crown drift of the Czernitz Tunnel (from Rziha, 1867).

As a consequence of the mechanism underlying squeezing, i.e. stresses exceeding a particular material related limit, it is not surprising to find extreme squeezing in cases where high to medium overburden combines with altered rock or fault zone materials. For instance: both the Gotthard railroad tunnel in the Alps (built 1872-82) as well as the parallel road tunnel built 100 years later suffered substantial delays from squeezing conditions. In the Moffat tunnel in the Rocky Mountains squeezing conditions in altered and faulted rock brought the work to a stop until the Lewis forpoling method was applied (Beaver, 1973, Lovering, 1928; note that Lovering uses the term "swelling" but describes phenomena which are "squeezing"). Similar problems occurred in other Rocky Mountain tunnels such as the Roberts tunnel (Wahlstrom, 1973) and the Straight Creek (Eisenhower) tunnel where a shield was lost and contractors had to be replaced.

A number of cases involving swelling problems, and on which detailed documentation exists, are listed in Table 6-1. Not only the magnitude of displacements and stresses is remarkable, but the total duration of the movements (over 100 years in some cases), and the extreme rates during the early phases after excavation. A few of the cases listed in Table 6-1 will now be discussed in more detail.

Kappelberg Tunnel

This is the most extreme case known to the author. It is a railroad tunnel in southern Germany completed in 1880 and originally built for two tracks but with only one centrally placed track left at present. A total invert heave of approximately 15 feet (4.7 m) occurred in the most critical zone. During the initial period after construction until 1887 the tracks had to be lowered each year by 10 inches (25 cm) at this location; up to 1926 swelling rock had to be removed and the track lowered 26 times! As will be discussed later, one has to realize that the total heave would probably not have reached this magnitude if the swelling rock had not been removed. Also remarkable is the fact that an upward crown displacement of 2 feet (0.64 m) took place in these critical sections (see Schaechterle, 1926 and Einstein, 1979 for further comments). Obviously the tunnel had to be continuously repaired. In 1982/83 a major reconstruction was undertaken in which some sections received a reinforced inner shotcrete skin while the sections with major swelling problems were rebuilt by constructing on articulated 20-inch (50 cm) thick liner of circular shape inside of and partially overlapping the original support (Kurz et al. 1984).

Bozberg Tunnel

This two-track railroad tunnel of 8200 feet (2500 m) length in the Swiss Jura mountains is located in marl, clayshale and anhydrite and was built between 1871-75 (Fig. 6-3). Even during construction, invert heave and abutment convergence occurred. The abutments in some sections were reconstructed several times and invert arches were built in 1903-05 in the most strongly affected sections; these, however, were rapidly destroyed by the highly sulfatic water. The drainage channel was frequently destroyed; the subsequent flooding of the invert increased swelling and, combined with the traffic induced vibrations, led to pumping of fines into the balast. Since 1923 regular measurements were made; the results are plotted in Figs. 6-4 and 6-5. Both figures show the dependence of swelling magnitude on rock type. In these zones new invert arches were constructed between 1963 and 1967 (Fig. 6-3). Fig. 6-3 also shows the inward displacement of the original open arch support.

TABLE 6-1
Performance of tunnels in swelling rock

| Tunnel construction period dimensions | Rock type | Invert heave | | Period | Overburden in m | Remarks | | |
|--|---|--|---------------|-----------------------------|---|--|--|--|
| | | Per year mm | Total mm | | | | | |
| Häzberg 1871-75 w = 9m h = 8.5m 2-track railroad | Anhydrite | 7.1-1.4 | 270 (450 est) | 1923-54 | | | | |
| | Lias marl | 4.5 | 150 | | | | | |
| | Opalinus clayshale | 4.5-5.8 | 180 | | | | | |
| | Molasse marl | 5.2-10.6 | 330 | | | | | |
| Ricken 1903-08 w = 7.4m h = 7.0m 1-track railroad | Molasse marl | 14.5-24.0 | 250-400 | 1904/08-64 | | Estimate from abutment convergence Invert plate Track level | | |
| | | 21.4-25.7 | | 1916/17 (14 months) | | | | |
| | | 5.6-9.0 | | 1910-17 | | | | |
| | | 10.8-11.8 | | 1940/41 and 42/47 | | | | |
| | | 5.0-6.9 | | 1965-66 120 days | | | | |
| | | During construction estimated heave: 500-750 | | 1966-67 520 days | | | | |
| Storage tunnel | Molasse marl | 15.7 | | 144 days | 50 | | | |
| Kappelsberg 1878-80 w = 9m h = 7m 2-track railroad (1 track operated only) | Gypsum marl Gypsum anhydrite Marl Gypsum - Keuper | 1. 40.8 | 2. 52.5 | 3. 50.3 | 4700 | 1903-07 | 80 | 1. Station 48400 2. Station 48500 3. Station 48525 |
| | | 29.0 | 25.0 | 28.0 | | | | |
| | | 24.0 | 24.0 | 25.0 | | | | |
| | | 23.0 | 23.0 | 23.0 | | | | |
| | | during first years, 250mm/year | | | | | | |
| | | of: 640 | | | | | | |
| Grenchenberg 1-track railroad | Molasse marl Opalinus clay shale | 1. 5.1-6.2 | 2. 4.6 | 3. 5.5 | | | 1. No invert cover 2. Invert arch r = 3.65m 3. Invert plate | |
| | | 9.3 | 5.0 | | | | | |
| | | 60 mm in 10 days | | | | | | |
| | | During construction | | | | | | |
| Genevreville 1855-58 2-track railroad | Anhydrite Gypsum | 60 mm in 10 days | | | | | During construction | |
| | | During construction: "1m in a few weeks" | | | | | | |
| Upper Hauenstein 1853-56 2-track railroad | Anhydrite Keuper marl Dogger marl | av. 6. max 10 | | | 2 years 1972-74 | | | |
| | | av. 4.5 max 8 | | | | | | |
| | | av. 6. max 11 | | | | | | |
| Lower Hauenstein 1912-15 w = 10.8m h = 9.5m 2-track railroad | Anhydrite and Marl | 1. 4.5 | 2. 1.4 | 3. 4.5 | 1937-51 (14 years) 1968-73 (5 years) | | 1. Anhydrite 2. Keuper marl 3. Opalinus - Clay shale 4. Dogger marl 5. Elfinger marl | |
| | | During construction: "1m in a few weeks" | | | | | | |
| | | During construction: "1m in a few weeks" | | | | | | |
| Bruggwald 1907-10 1-track railroad | Molasse marl | 50 | | 2 weeks during construction | 70/max | | | |
| Wagenburg (North tube) Invert (pilot) drift 1943 w = 3m h = 2.7m | Gypsum-Keuper | 100 | 1021 | 1st year | 50-60 | Crown lift: 300 mm Contact stresses of 4 MN/m ² on invert plate in a test section in pilot drift | | |
| | | (7-8) | | 1974 in South tube | | | | |
| Belchen 1963-70 w = 12m h = 10m (Double tube highway tunnel) | | Invert: 650 | | During construction 7 years | 100-150 | Concrete fiber | | |
| | | Drainage channel: 900 | | | | | | |
| | | 750 | | | | Contact stresses: 3.5 MN/m ² 0.3 MN/m ² | | |
| | | | | | | 27 MN/m ² 3 MN/m ² Opalinus | | |
| Tunnel construction dimensions | Rock type | Per year mm | Total mm | Period | Overburden in m | Remarks | | |

Such inward displacements and the related voids created behind the liner have been observed in a number of other cases.

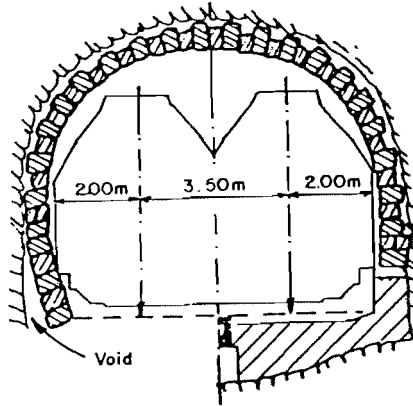


Fig. 6-3. Bözberg Tunnel. Left side of figure: abutment convergence. Right side of figure: reconstructed liner and new invert arch (from Beck and Golta, 1972).

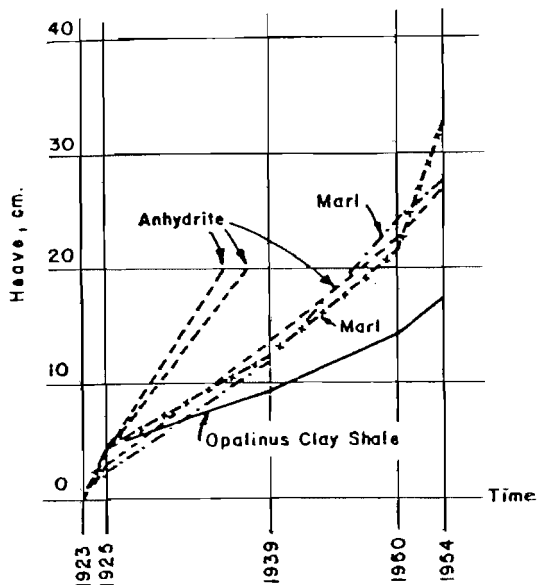


Fig. 6-4. Bözberg Tunnel. Invert heave (from Grob, 1976).

BÖZBERGTUNNEL
DEFORMATION MEASUREMENTS 1923-1954

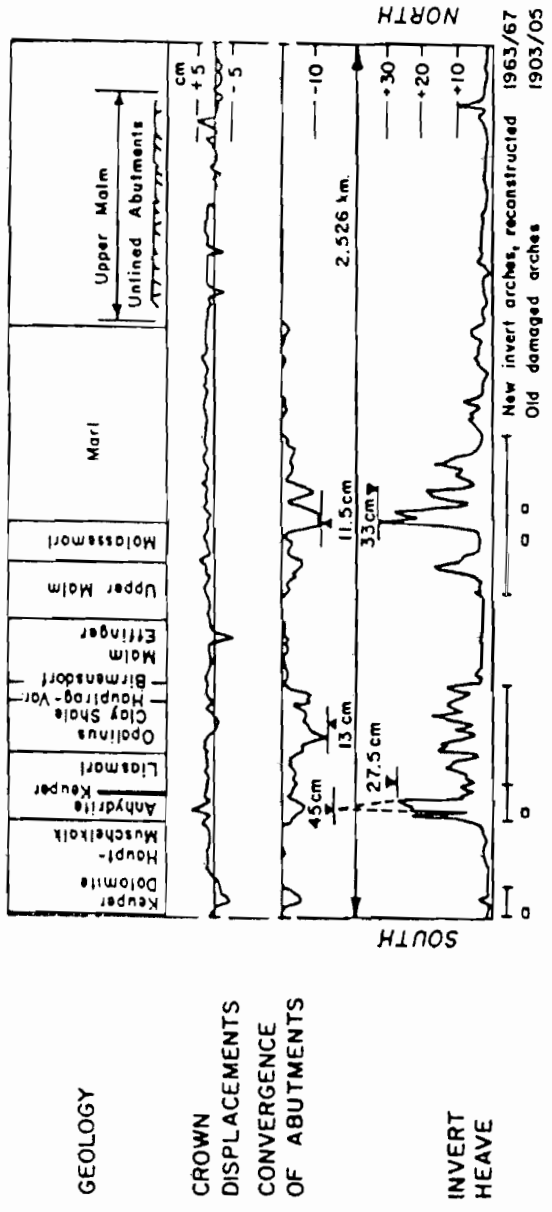


Fig. 6-5. Bözberg Tunnel. Longitudinal cross section and displacements 1923-1954 (from Grob, 1976).

Belchen Tunnel

This is a 2-mile (3.2 km) long highway tunnel also in the Swiss Jura mountains which was built 1964-1970. Its two parallel two-lane tubes are located in marl, clayshale and anhydrite (Fig. 6-6). The case has been discussed in a number of publications, (Hunder et al., 1970; Grob, 1972; Einstein et al., 1976) and thus only the most important aspects are mentioned here. The tunnel was constructed by excavating (in each tube) two invert drifts and then enlarging them to the full cross-section. After the enlargement to the full cross-section in the Keuper formation (which contains anhydrite), the invert level rose 3 feet (0.9 m) within a few months. An invert arch was built with a radius of 34 feet (10.4 m) and a thickness of 18 inches (0.45 m) (Fig. 6-7); however this was sheared off shortly after construction and an additional 2 feet (0.6 m) of invert heave occurred. A new invert arch with a smaller radius of approximately 27 feet (8.12 m) and greater thickness 34 inches (0.85 m) was therefore built (Fig. 6-7). Stress cells were placed at the rock-concrete interface of the invert arch to measure the contact stresses, while the fiber-stresses in the concrete were backfigured from deformation observations and a few overcoring tests. In 1974 the maximum contact stresses were 500 psi (3.5 MN/m^2) and the maximum fiber stresses were 3800 psi (27 MN/m^2). Similar observations but with significantly smaller stress magnitudes, (0.3 MN/m^2 i.e. 40 psi contact stresses, 3.0 MN/m^2 i.e. 400 psi fiber stresses) were made in the clayshale.

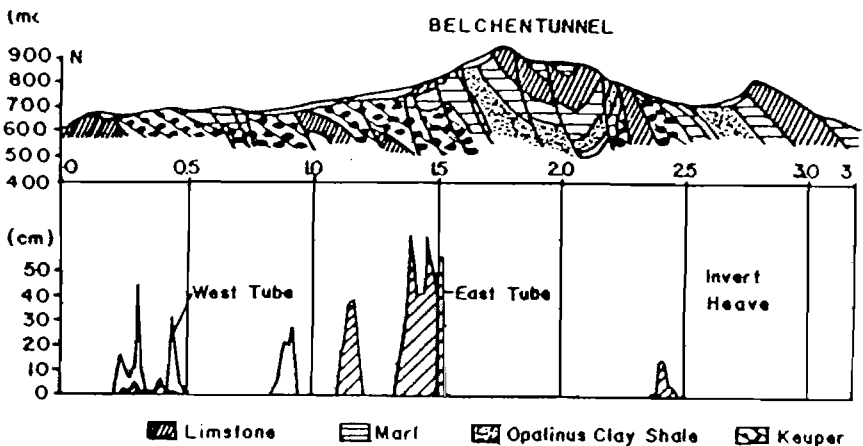


Fig. 6-6. Belchen Tunnel: Longitudinal cross section and invert heave (from Grob, 1972).

Indications of continuing swelling led to a new observation program starting in 1980. At that time the stress cells at the clay shale-concrete interface were still working, and the following contact stresses were observed.*

TABLE 6-2
Contact Stresses Rock-Invert Arch, in Clay Shale Section of Belchen Tunnel

| | Minimum | Maximum | Mean |
|------|------------------------|------------------------|------------------------|
| 1980 | 0.08 MN/m ² | 0.60 MN/m ² | 0.30 MN/m ² |
| 1986 | 0.11 MN/m ² | 0.64 MN/m ² | 0.30 MN/m ² |

while the concrete fiber stresses were

TABLE 6-3
Fiber Stresses in Support, in Clay Shale Section of Belchen Tunnel

| | Minimum | Maximum | Mean |
|------|------------------------|------------------------|------------------------|
| 1980 | 0.42 MN/m ² | 4.65 MN/m ² | 2.29 MN/m ² |
| 1986 | 1.32 MN/m ² | 8.25 MN/m ² | 3.44 MN/m ² |

The stress cells in the Keuper (containing the anhydrite) did not function anymore and were substituted in 1976 by 16 stress cells at the concrete rock interface in one of the three ventilation shafts which produced the following observations:

* Values in these tables are given in SI units; 1 MN/m² is approximately 140 psi.

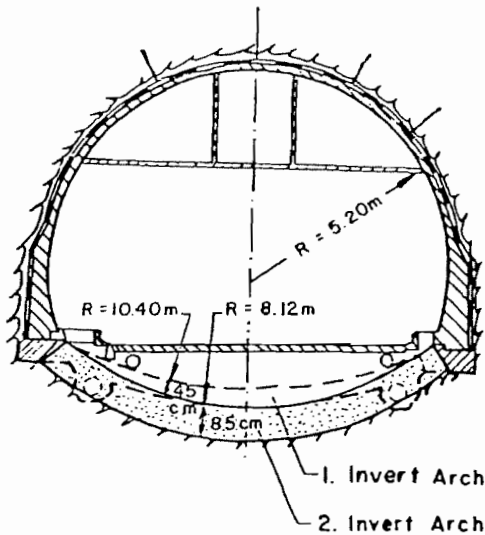


Fig. 6-7. Belchen Tunnel. Cross section with original (1.) and reconstructed (2.) invert arch (from Grob, 1972).

TABLE 6-4

Contact Stresses Rock-Shaft Liner, in Keuper Section of Belchen Tunnel

| | Maximum | Mean |
|------|------------------------|------------------------|
| 1980 | 4.01 MN/m ² | 2.29 MN/m ² |
| 1986 | 4.36 MN/m ² | 2.27 MN/m ² |

(These values exclude one stress cell in which stresses greater than 10 MN/m² were observed). These observations are particularly significant in that they not only show high and still increasing stresses, but that this occurs in a vertical circular shaft.

In the same Keuper zone, concrete fiber stresses were measured with load cells in horizontal slots in the tunnel liner; the derived stresses ranged from 850 to 1000 psi (6 to 7.5 MN/m²). No direct comparison with the previously determined fiber stresses is, however, possible, due to different locations and different measurement - and evaluation methods.

Also interesting in this case is the extensive investigation of various countermeasures and the strong indications about the underlying mechanisms. It was, for instance, quite conclusively shown that suction of water through pores and small cracks had the greatest effect on swelling while joints seemed to play a much smaller role. Grout injection into the joints stopped the flow of water but not the swelling.

Another important result is the fact that an invert arch per se does not necessarily stop invert heave, but that the proper combination of curvature (approaching that of a circular shaped opening) and thickness is necessary to provide the required resistance.

These are only a few cases. They illustrate very clearly the consequences of swelling rock on tunneling. The great number of tunnels in swelling rock built in Central Europe over the past decades and the many tunnel projects in this geographic area not only produced many additional case histories, but also summaries and reviews (e.g. Steiner, 1987; Kovari, 1987) and most importantly, a number of test tunnels and test sections (Kuhnenn et al., 1979; Kirschke, 1987).

To conclude these comments on tunnel case histories in swelling ground, it should be emphasized that, although they represent a wide variety of rocks ranging from shales to marls to anhydrite and mixed rocks, there are other materials notably fault gouge and argillaceous soils which are also associated with swelling phenomena.

6-3 Definition of Swelling and Squeezing Mechanisms

6-3.1 Swelling*

The swelling mechanism is a combination of physico-chemical reaction involving water and stress relief. The physico-chemical reaction with water is usually the major contributor to swelling, i.e. the volume increase with time, but it can only take place simultaneously with or following stress relief.

- The major physico-chemical mechanisms involving water are:
With Clay Minerals:

* This section corresponds to the discussion of swelling mechanisms in "Characterization of Swelling Rock" (ISRM, 1983), with some comments added.

Water is absorbed at the exterior surface of clay minerals and it is taken up at internal surfaces of clay minerals having expandable layers. Swell pressure depends on the interparticle distance of clay particles and intraparticle distance between expandable layers. Swell heave depends on the amount of clay particles and on the number of expandable layers.

Rock with expandable layer type particles swell more than those with particles that absorb water only externally.

With Anhydrite-Gypsum:

Hydration of anhydrite ($\text{CaSO}_4 + 2\text{H}_2\text{O} \rightarrow \text{CaSO}_4 \cdot 2\text{H}_2\text{O}$) produces gypsum with a maximum volume increase of about 60%.

Anhydrite does usually not transform itself directly into gypsum by association of water; anhydrite is usually dissolved by water followed by gypsum precipitation from this solution.

60% volume increase only occurs if completely dry anhydrite without voids is exposed to water in an open system. Naturally occurring anhydrite with voids will show a smaller volume increase; the same holds for a closed system with limited water supply.

Hydration of anhydrite takes place on its surface. Therefore, massive anhydrite with few fissures does practically not swell, while finely divided anhydrite does swell substantially.

An important combined swelling mechanism occurs if the same rock contains anhydrite and clay minerals, e.g. finely divided anhydrite in shale or marl. The latter provides the required water and ensures continuous supply through its pores.

With Pyrite and Marcasite:

Oxidation produces sulfates which in turn can react with calcite, and then precipitate to produce gypsum.

Oxidation of pyrite, which does not require water, causes swelling. The calcite reaction and precipitation of gypsum cause secondary swelling, which can however only take place if the water evaporates.

Other physico-chemical mechanisms involving water and leading to volume increase are freezing and weakening of diagenetic bonds, the latter however only indirectly:

Although freezing of water leads to substantial time dependent volume increase, it is not considered a swelling mechanism. Also,

particularly in rock, a substantial part of this mechanism involves frost fracturing or extension of existing cracks in freeze-thaw cycles. It is thus a mechanism also involving breaking of diagenetic bonds.

Weakening or breakdown of diagenetic bonds reduces restraints on clay particle swelling. An example is the dissolution of calcium carbonate cementation.

Swelling caused by freezing and breaking of diagenetic bonds will not be considered in this chapter.

- Exsolution of gases in weakly cemented materials can also cause volume increase. Swelling caused by this mechanisms will not be considered in this chapter.
- The physico-chemical reactions can only take place in combination with stress relief, in particular:
 - Stress relief can cause negative pore pressures which in turn can lead to flow of water. If such water inflow occurs, drained volume increase and volume increase caused by one or more of the physico-chemical mechanisms will result.
 - Stress relief can cause fissures which in turn facilitates flow of water and lead to volume increase through one or more of the physico-chemical mechanisms.
 - Stress relief can cause fissuring of diagenetic bonds which in turn reduces restraint on swelling by one or more of the physico-chemical mechanisms.
- Indirect effects that cause swelling in combination with physico-chemical reactions are:
 - Shearing (e.g. of faults) and tension cracking will usually break diagenetic bonds and increase the surface area that is exposed to water. Swelling through one or more of the physico-chemical mechanisms will thus be facilitated.
 - Weathering and alteration of non-swelling minerals can produce swell susceptible minerals.
 - Swelling of fault gouge and joint fillers are prime examples.

Swelling expresses itself by volume increase and thus isotropic or anisotropic (depending on composition and texture) strains in all directions if uninhibited. Constraint in any direction causes a buildup of stresses in that direction; complete inhibition of swelling produces the maximum possible stresses.

As shown in Fig. 2-9, excavating a tunnel leads to a decrease in radial stresses and a decrease or increase of tangential stresses near the eventual/future tunnel periphery. If the stress changes cause an overall stress reduction, the conditions for stress relief are fulfilled and swelling will occur in the manner described above.

A good indicator of the overall stress conditions relevant to swelling is the first stress invariant which expresses the volumetric state of stress:

$$I_1 = \sigma_{11} + \sigma_{22} + \sigma_{33} \quad (6-1)$$

as will be seen in Section 6-5 some analysis methods use the change in the first stress or strain invariants to derive swelling stresses or strains.

The usual in situ stress conditions before excavation will produce a stress relief in invert and crown after excavation, in particular below a flat invert. Given these stress changes (see also Fig. 6-8), it is thus not surprising that swelling usually occurs in tunnel inverts, particularly if they are flat. Naturally, the gravity driven flow of water in the ground and construction water on the invert aggravate the conditions. As mentioned before, swelling under constraint will produce stress increases, and, if these stresses are high enough, they will stop the swelling. Curved tunnel shapes, particularly if the radius is small, provide conditions where swelling will quickly lead to natural constraint. A liner will provide additional constraint; particularly an arch shaped liner will not only stand up to considerable loads but will also lead to a rapid buildup of counterstresses. This is the major reason why swelling in the usually curved and lined tunnel crowns is relatively small. In the sidewalls of a tunnel the stress conditions are usually such that no stress relief occurs. The occasionally observed movement of sidewall liners is in most cases the indirect effect of invert movements (see e.g. Fig. 6-3).

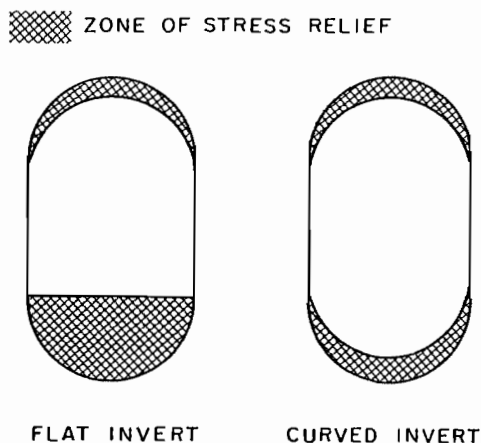


Fig. 6-8. Different extent of stress relieved zones depending on invert shape.

6-3.2 Squeezing

Squeezing is the time dependent shear displacement of the ground which causes the tunnel periphery to move inward. The squeezing mechanism can consist of any one or a combination of submechanisms, namely

- creep (or otherwise expressed viscous behavior) in the particles of the intact material such as the grains in rock and soil. Creep of individual particles may be due to viscous behavior of the crystal structure or unstable crack propagation.
- creep along the interfaces between particles of the intact material, and
- creep along larger scale discontinuities such as bedding and foliation surfaces, joints and faults. These creep mechanisms involve the three well known components (primary, secondary and tertiary) and typical combinations thereof (Fig. 6-9).

Usually, the creep mechanism(s) underlying squeezing is of visco-plastic nature but, particularly at low stresses, some of the strains may be recoverable i.e. visco-elastic behavior occurs.

Creep usually occurs at stress levels below the short term shear strength of a material. The results of short duration strength tests are thus not very useful for determining creep susceptibility and the type of creep mechanism.

Creep and thus squeezing can occur without volume change. In cases of dilatant behavior, squeezing will be associated with volume increase. It is also possible that pore or cleft water dissipation occurs with squeezing, in such circumstances consolidation and volume decrease will be associated with squeezing.

It should be noted that these comments on volume change apply to idealized conditions in which a free body of the material is thought to deform over time under constant stresses. In a tunnel, inward movement of the ground surrounding will encounter increasing constraint, particularly if the periphery is curved (Fig. 6-10), which may change the stress conditions and thus the squeezing process. Considering again the comments about stress states around a tunnel, one sees that at many locations at the tunnel periphery, a simultaneous stress increase in one direction and a stress decrease in another direction takes place. Particularly, simultaneous loading/unloading (Fig. 6-11a), but in lower resistance materials also anisotropic unloading, (Fig. 6-11b) will push the ground toward the yield or failure state.

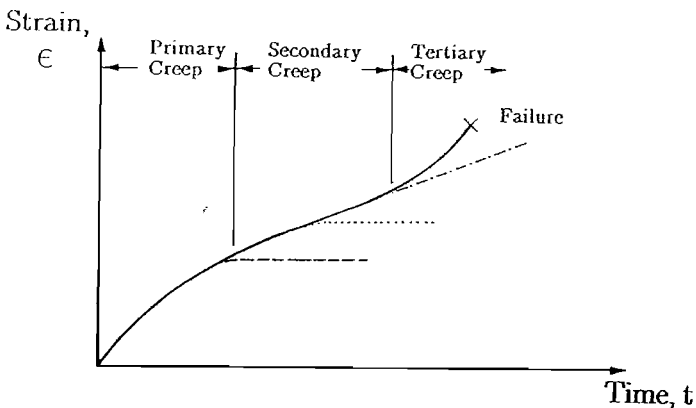


Fig. 6-9. Typical components of creep behavior and combinations.

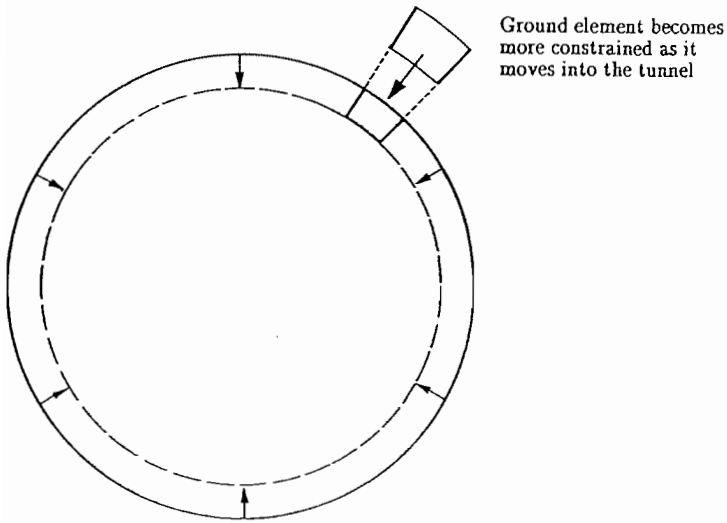


Fig. 6-10. Inward movement of tunnel periphery causes increased constraint.

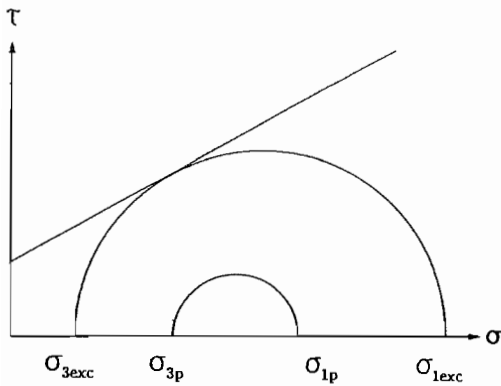


Fig. 6-11a. Mohr stress circle for simultaneous loading/unloading in different directions (σ_{1p} = primary stress state, σ_{1exc} = stress state after excavation).

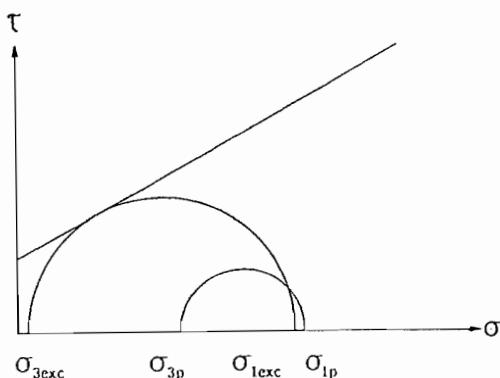


Fig. 6-11b. Mohr stress circle for anisotropic unloading (σ_{ip} = primary stress state, σ_{iexc} = stress state after excavation).

As stated before, creep and thus squeezing usually starts at stress levels below the failure stress level. Also, in viewing Figs. 6-11a and b, it should be kept in mind that they represent the behavior of intact materials; if joints exist and creep takes place mostly along them, the stresses at which creep occurs will depend on the orientation of the joints.

From what has been stated above, sidewall squeezing will predominate under conditions where the primary vertical stresses (i.e. the stresses before excavation) are greater than the primary horizontal stresses. Conversely, if the horizontal primary stresses are greater than the vertical ones, invert and crown will be most severely affected. Similar to swelling, straight (flat) walls (inverts, crowns) will lead to less favorable stress conditions and thus to more pronounced squeezing.

Also, the previously mentioned increasing constraint as material squeezes into the tunnel (Fig. 6-10) may, in contrast to swelling, not necessarily reduce or inhibit squeezing but make it more severe depending on the original stress state (Fig. 6-12).

Liners or tunnel supports, in general, will produce counterstresses, particularly as a reaction to the squeezing displacements. Since the liner induced stresses usually act in the direction of the minor principal stress, this will have a favorable effect. However, if these counterstresses together with an impermeable liner produce a pore (cleft water) pressure increase the effect may be counterproductive.

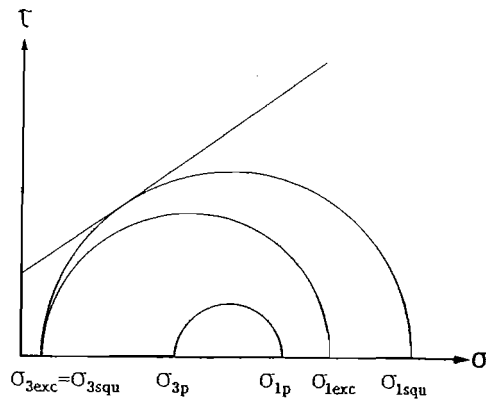


Fig. 6-12. Mohr stress circle showing the constraint effect caused by squeezing (σ_{1p} = primary state of stress; σ_{1exc} = state of stress after excavation, no squeezing; σ_{1squ} = state of stress caused by squeezing).

6-3.3 Combined Swelling and Squeezing

This aspect, which so far has received mostly intuitive consideration, has recently been treated more rigorously (Bellwald et al., 1987) and is subject to ongoing investigations. Based on the behavior of overconsolidated clays one can assume that rapid unloading, as it occurs around a tunnel during excavation, will lead to an increasing deviator stress simultaneously with the development of negative pore pressures (the latter aspect has been discussed before). This in turn causes the stress change shown with circle 2 in Fig. 6-13. In these undrained conditions, the ground will be in a safe stress range as far as squeezing is concerned. Swelling will start, however, caused by negative pore pressure induced suction. As the negative pore pressures dissipate the stress state will move closer to failure and creep (i.e. squeezing) will start. If the swelling zone is constrained, as for instance the zone under the tunnel invert which is laterally constrained, inhibited swelling will lead to a stress buildup. The major principal stress in Fig. 6-13 will increase and squeezing is more likely. It is even possible that failure (yielding) will occur, a fact which has been experimentally proven by Sun Jun et al., (1984); (see also Bellwald, et al., 1987).

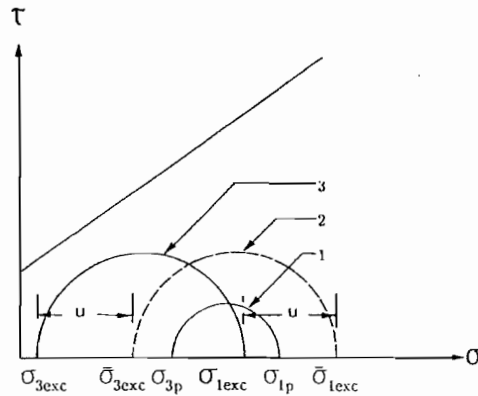


Fig. 6-13. Mohr stress circles for combined swelling and yielding (squeezing). (Simplified illustration; assuming no positive pore pressures exist in situ).

1. Primary stress state.
2. Stress state after excavation in terms of effective stresses, undrained (negative pore pressures).
3. Stress state after excavation in terms of total stresses, undrained. This is also stress state in terms of effective stresses, which is approached as negative pore pressures dissipate.

6-4 Laboratory Testing for Swelling and Squeezing

6-4.1 Swelling Tests

Laboratory tests can be relatively simple identification tests or quantitative swelling tests. With the former one can estimate if swelling may occur and possibly get a rough idea on the swelling potential, i.e. the possible magnitude of swelling displacements and stresses. Quantitative tests are intended to be used in conjunction with rigorous analytical and some empirical predictions.

IDENTIFICATION TESTS. The best known simple identification test is the Casagrande test for Atterberg Limits. A number of correlations between Atterberg limits and swelling potential exist. Most of these relations apply to expansive soils, but Brekke and Howard (1973) provide one for fault gouge, and Katzir and David (1968) one for swelling marl, that include quantitative relations between the liquid limit and swelling pressures (from laboratory swelling tests). The fact that a number of different relations exist between Atterberg limits and swell pressures is an indication that Atterberg limits may provide an estimate but can probably not be the basis for generally valid relations. Even more important are the effects of specimen preparation;

remolding or oven-drying of the natural specimen which may be necessary for Atterberg limit testing can modify the swelling behavior. Another relatively simple identification procedure is water imbibing of a specimen exposed to 100% humidity. Brekke and Howard (1973) were able to correlate the amount of imbibed water and swelling pressure (obtained from laboratory swelling tests). The results of the Franklin slaking test can also be related to the swelling potential (Franklin, et al., (1972).

In specific, well defined lithologic zones or subzones it may also be possible to relate the unconfined compressive strength (or point load index) to swelling potential.

The determination of specimen mineralogy is probably the best identification test; since these methods also provide substantial quantitative information, they will be discussed below.

QUANTITATIVE TESTS FOR SWELLING POTENTIAL. Mineralogic-textural testing can be used to identify potentially swelling minerals and in many circumstances to provide a quantitative estimate of the expected swelling pressures (stresses due to swelling under a particular set of constraint conditions; this will be further discussed below). In the case of the DLVO-theory (Madsen, 1976) mineralogic-textural testing is the basis for quantitative derivation of swelling pressures. X-ray diffraction is ideally suited for mineralogic testing. Not only is it possible to identify the minerals contribution to or inhibiting swelling (clay, anhydrite, pyrite, calcitic diagenetic bonds) but also to determine their quantitative proportion. In addition, a quantitative estimate of swelling potential is possible (e.g. with tests run at different water contents, or in the case of clays, with tests run fully saturated with glycol, completely dried at 300°C and rehydrated with glycol).

It is often necessary to not only know about the existence and relative quantities of swelling minerals but to also know their textural distribution (e.g., the interlayering of anhydrite and clay minerals (shale) can significantly increase swelling (Grob, 1972; Lippmann, 1976). Also, the parallelity of clay particles can have a significant effect. Investigations with the scanning electron microscope are well suited for these purposes.

Other tests in the category of quantitative testing may be needed for particular swelling prediction methods, e.g. the DLVO theory requires the determination of the clay mineral surface (Madsen, 1976).

Probably best known in the area of quantitative testing for swelling potential are "swelling tests" in the narrow sense. In these tests intact specimens obtained from the field (cores, block samples) or pulverized material are subjected to a particular combination of water and applied stresses and the associated volume change of the specimen is observed:

In a free swelling tests, intact specimens are placed in a container (Fig. 6-14) and submerged in either distilled water, or water from the site or water with a special chemical composition. Usually only vertical displacements are measured; it is desirable to also observe radial (lateral) displacements e.g. with a calibrated band placed around the specimen. The strains versus time are plotted as shown in Fig. 6-15. Usually, one is mostly interested in the maximum (axial/radial) swelling strain. (Note that this test is completely unconfined!). If the material is pulverized, free swelling tests range from placing the pulverized material in a test tube and adding water to analogous procedures in beakers or other open containers. (Note that such tests may not represent unconfined conditions.)

At the other extreme are the tests to determine the maximum axial stress. The specimen is placed in an oedometer type ring inside an open cell, (Fig. 6-16) subject to a small seating load (for details see ISRM, 1988), and the cell is then filled with water. Any axial heave (displacements) is immediately compensated by tightening the screws which will increase as shown in Fig. 6-17 and reach the maximum axial swelling stress (often called maximum swelling pressure or simply swelling pressure). This stress is a conservative approximation of the swelling stresses that are to be expected.

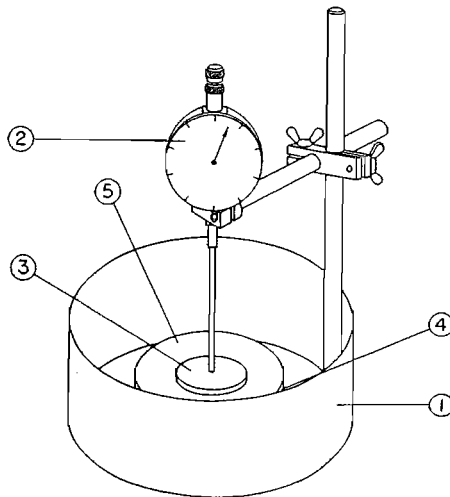


Fig. 6-14. Free swell test apparatus (1: Container, 2: Gauge, 3: Platen, 4: Band to measure circumferential stress, 5: Specimen).

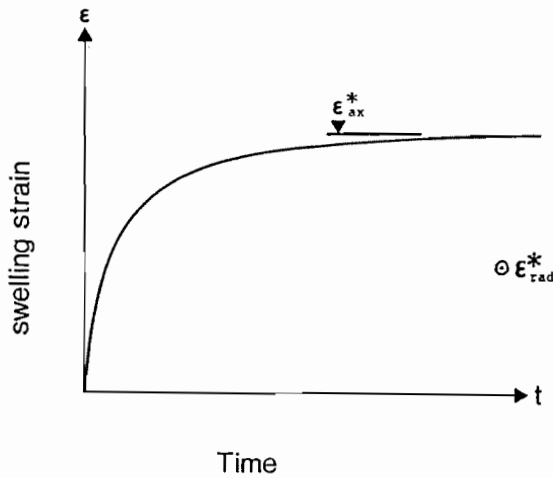


Fig. 6-15. Free swell test, swelling stress - time curve (ϵ_{ax}^* = max. axial swell strain, ϵ_{rad}^* = max. radial swell strain).

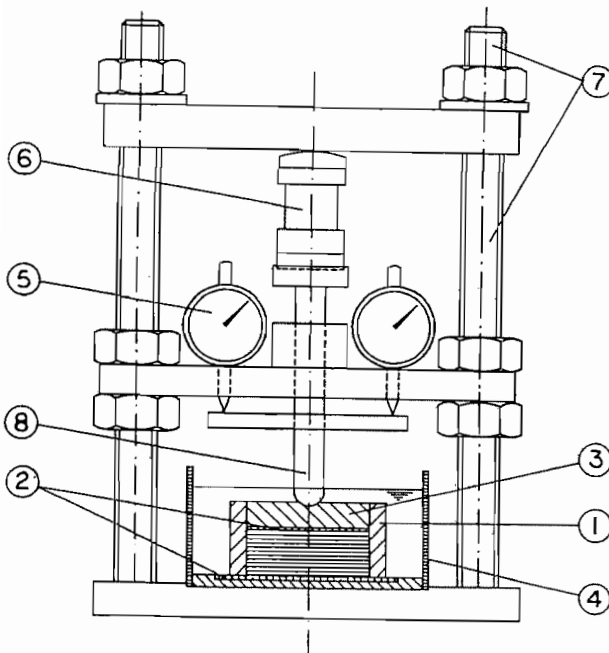


Fig. 6-16. Maximum axial swelling stress apparatus. (1. Oedometer type ring, 2. Porous plates, 3. Top platen, 4. Container, 5. Gauges, 6. Load cell, 7. Screws and frame).

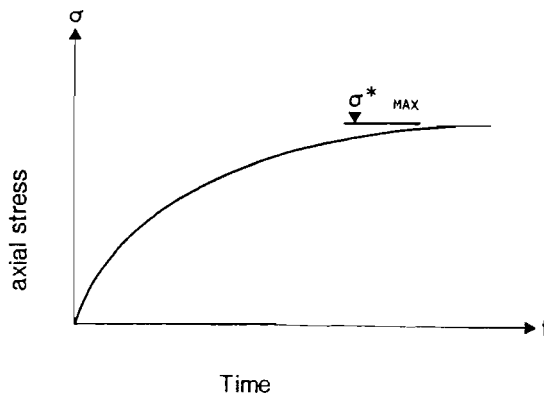


Fig. 6-17. Maximum axial swelling stress test, swelling stress - time curve (σ_{\max}^* = maximum axial swelling stress = maximum swelling pressures).

The free swelling test and the maximum swelling stress tests are meant to provide a rapid assessment of the swelling potential both regarding deformation and stresses. They allow one to check if swelling is a problem and to get an idea on the time it takes to develop the stresses/displacements. If these tests indicate that swelling is significant, so called "axial swelling stress versus axial swelling strain tests" have to be conducted. Since, in reality, the conditions are neither completely constrained nor unconstrained, the stress/strain test has to reflect this. The apparatus is very similar to the maximum swelling stress equipment with an oedometer type ring in an open cell (Fig. 6-18). The loading arrangement, however, allows one to control the applied loads. The test is conducted by first applying a load (corresponding to stress σ_A in Fig. 6-19) under dry conditions. Water is applied at this point by filling the cell, and swelling occurs for the stress σ_A . Afterwards the specimen is unloaded in a stepwise manner and swelling is allowed to take place in each step before further unloading. This test is a modification of the well known Huder-Amberg (1970) oedometer test. In the Huder-Amberg test (Fig. 6-20) the specimen is subjected to several load-unload cycles under dry conditions before applying water at D' with swelling to D and subsequent unloading/swelling in steps entirely analogous to the above mentioned stress/strain test (the unloading steps are smoothed in Fig. 6-20).

So far missing as a standard test is one in which not only axial but also lateral stresses are measured and controlled. Research is under way to remedy this (Sun Jun, 1984; Bellwald, et al., 1987).

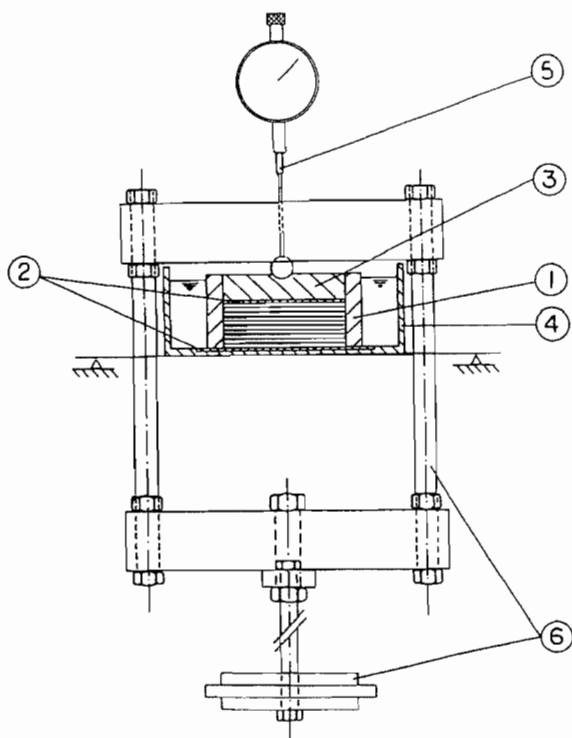
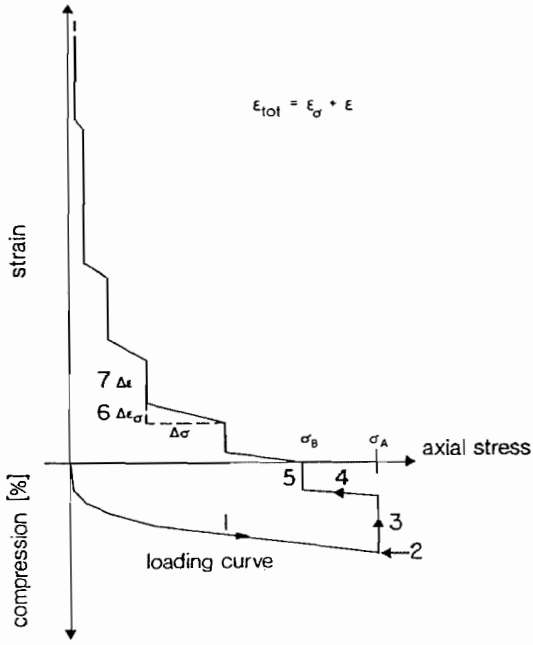
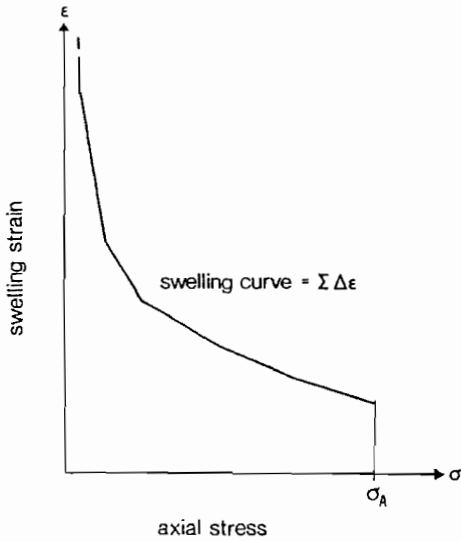


Fig. 6-18. "Axial swelling stress versus axial swelling strain" apparatus. 1. Oedometer type ring, 2. Porous plates, 3. Top platen, 4. Container, 5. Gauge, 6. Load frame).



- a. Stress strain curve
 1. Loading in dry condition, 2. Water added, 3. Swell heave, 4. Loading decrement to σ_B , 5. Swelling at σ_B , 6. Elastic strain, 7. Swell strain.



- b. Swell stress strain curve obtained from 6-19a, considering only swelling strain $\Delta\epsilon$.

Fig. 6.19. Axial swelling stress versus axial swelling strain test.

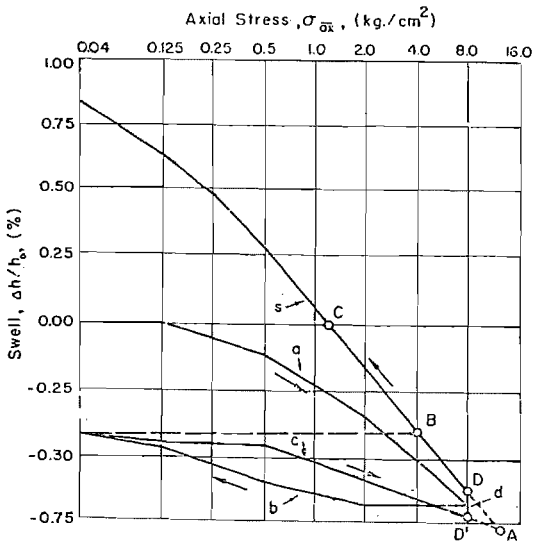


Fig. 6-20. Swelling test by Huder and Amberg, (1970), a, b, c, dry loading and unloading; D' water added; d swelling to D; s swell curve obtained by stepwise unloading.

6-4.2 Tests for-Squeezing

Appropriate tests for this purpose are triaxial tests in which the deviator stresses are held constant and the deformation over time is observed. Consolidation, pore pressure and external stress conditions have to simulate reality in the same way that triaxial tests for other purposes have to do. For a good review of testing methods, of results and of their use in predicting time dependent behavior around tunnels see Semple et al., (1973).

6-5 Empirical and Analytical Methods for Tunnels in Swelling and Squeezing Rock

6-5.1 Introduction

As shown in Chapter 2, empirical tunnel design methods are based on ground classification systems. One or more ground parameters are related to a particular support dimension or a range of possible support dimensions. Empirical methods are derived from observations of ground characteristics and of installed support dimensions (possibly support performance) in a particular

location of a tunnel. One has to be aware of the fact that empirical methods are only applicable to the ground conditions and supports which have been covered in the underlying cases. (See also Einstein et al., 1979 and Steiner and Einstein, 1980).

Analytical methods, (in the context of this report, "analytical" includes both closed form and numerical approaches) rely on first principles or approximations thereof. In other words, constitutive relations of the material are formulated and used, with the appropriate geometric and stress field boundary conditions, to make predictions. Limitations of the analytical methods are mostly caused by the usual simplifications in the constitutive relations and boundary conditions.

6-5.2 Empirical Methods for Swelling and Squeezing Ground

Many of the empirical methods for rock and soil tunneling include swelling and squeezing ground as one or several of the ground classes. In the lines below, three of these methods (the Terzaghi-, RQD-, and O-methods) are specifically described; some comments are also made on other methods. In addition a specifically swelling oriented empirical method, the one by Brekke and Howard (1973), is also discussed. In order to avoid repetition of other chapters of this book and of the open literature, only the minimum necessary to understand and apply these methods is given below.

Terzaghi - Method

Terzaghi makes recommendations for squeezing and swelling rock and soil. The Terzaghi classification consists of a definition and the rock load formula amplified by explanations in the text (see Proctor and White, 1946, and Table 2-2): Classes 7 and 8 in Table 2-2 relate to squeezing and class 9 to swelling rock.

In the accompanying comments in Proctor and White (1946) Terzaghi provides the original formulation for squeezing rock with

$$H_p = H_p 10 \frac{B+H_t}{20} \quad (6-2)$$

where $H_p 10$ is the rock load on a 10-foot by 10-foot (~ 3 m x 3 m) tunnel. He states that values of $H_p 10$ for shallow tunnels (less than several 100 feet overburden) have been observed to start with 23 feet (~ 7 m) and to reach final values of 42 feet (~ 13 m); at depths of more than 1000 feet (~ 300 m) the corresponding values are 30 and 70 feet (9 and 21 m). (The numerical constants

for classes 7 and 8 in Table 2-2 are evidently derived from these $H_p/10$ values.) From our research (Steiner and Einstein, 1980) it seems that Terzaghi extracted these rock loads from Bierbaumer's (1913) work, who observed and interpreted tunnel support performance. The explanations in the text for classes 7 and 8 are otherwise limited. Squeezing rock is simply characterized as rock containing clay, but squeezing itself is only phenomenologically described, namely as time dependent inward movement of the tunnel periphery.

For swelling rock, i.e. class 9 in Table 2-2, Terzaghi mentions a case of a shallow tunnel in which observed swelling pressures were three times the overburden. For deep tunnels he mentions observed swell pressures of 10 to 20 tons per square feet (1 to 2 MN/m²). The latter corresponds roughly to the 250 feet (~ 75 m) overburden given for class 9 in Table 2-2. It is important to mention that Terzaghi states explicitly that swell pressure primarily depends on the swelling capacity of the rock and that general rules depending on height and width of the tunnel as given for the other classes cannot be established. He states that it is not even known if swell pressures depend on the tunnel size. Nevertheless, he also points out that swelling does not prevent the ground around the tunnel, and at some distance from it, to support itself as in any other rock. Again, as for squeezing rock, no detailed explanation on how to recognize swelling rock in the field is given; however, Terzaghi recommends to determine the swelling behavior through an oedometer type swell test but he does not provide any relations between oedometer type test behavior and support loads.

Interestingly, Terzaghi also makes suggestions on detailed design of supports in squeezing and swelling rock. In essence, he recommends to overexcavate between the steel ribs and to place narrow flange ribs such that as much free movement as possible take place. (see also Section 6-6).

Soil: (see "Earth Tunneling with Steel Supports," Proctor and White, 1977). Using unconfined compressive strength q_u and the pressure p_r exerted on the "block of clay located above of the working face" (see Fig. 6-21) a load ratio L_r is defined as

$$L_r = 100 \frac{p_r}{q_u} = 100 \frac{H}{q_u} \left(w - \frac{s_u}{R} \frac{3.14 + \frac{R}{H_t}}{1 + 1.57 \frac{H_t}{B}} \right) \quad (6-3)$$

where $s_u = \frac{q_u}{2}$ and $w =$ unit weight

Depending on the magnitude of L_r no squeeze, squeeze requiring some and squeeze requiring major restraint (such as compressed air) is predicted

(Fig. 6-22). Load recommendations for squeezing clay are as follows (refer to Fig. 6-23):

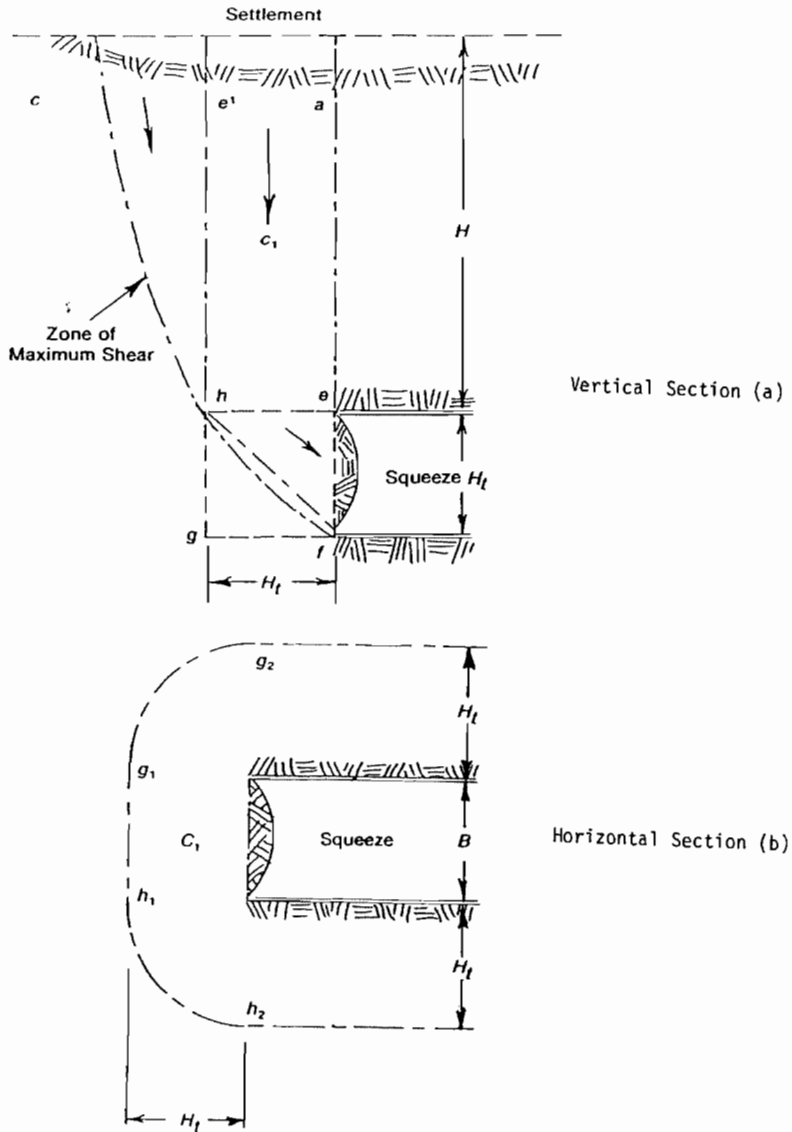


Fig. 6-21. Squeezing in soil tunnels. Geometric assumptions (from Proctor and White, 1977).

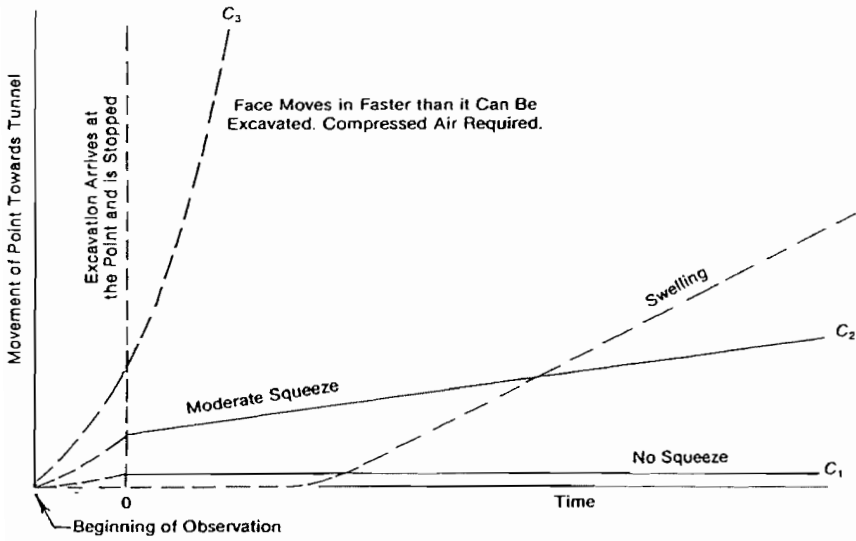


Fig. 6-22. Squeeze curves in clay (from Proctor and White, 1977).

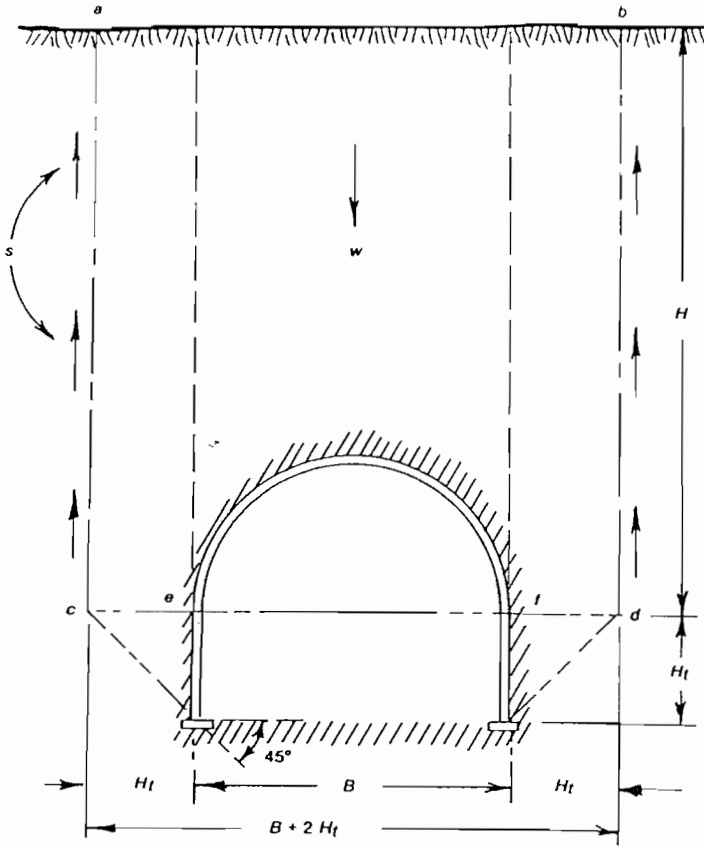


Fig. 6-23. Squeezing in soil tunnels. Load geometry (from Proctor and White, 1977).

Homogeneous squeezing clay:

$$\text{unit load on roof: } p = wH \frac{2 s_u H}{B + 2H_t} \quad (6-4)$$

$$\text{unit load on sidewall: } P_h = p + \frac{1}{2} w H_t - q_u \quad (6-5)$$

If there is stiffer clay behind the sidewalls (i.e. clays with a q_u that is substantially greater than wH):

$$\text{unit load on roof: } p = wH \frac{s_u H}{B} \quad (6-6)$$

unit load on sidewall: no load

If the opposite occurs, i.e. softer clay behind the tunnel sidewalls, a much wider zone than $B + 2H_t$ is affected and

$$\text{unit load on roof: } p = wH \frac{2s_u H}{B + 6H_t} \quad (6-7)$$

$$\text{unit load on sidewall: } Ph = p - q_u \quad (6-8)$$

These relations are valid for permanent supports. For temporary supports only 1/2 of the shearing resistance s_u is assumed to be mobilized.

RQD - Method (Deere et al., 1968)

This RQD based classification and the associated supports are shown in Table 6-5. Squeezing and swelling ground is expressed in a single class and not related to RQD. The rock load values correspond to those of Terzaghi.

Q-System (Barton et al., 1974)

Recall Equation 2-5, Chapter 2.

$$Q = \frac{RQD}{J_n} \frac{J_r}{J_a} \frac{J_w}{SRF} \quad (2-5)$$

Swelling and squeezing is directly considered in J_a (Table 2-5) and in SRF (see Table 2-6). Conditions which may produce squeezing (and possibly swelling) enter also indirectly through low RQD, high J_n , low J_r and possibly low J_w . As described in Chapter 2 "Q" is related to a range of suitable supports obtained from a study of over 200 cases. Several of these cases include swelling/squeezing rock; nevertheless, it seems advisable to check the underlying cases against any new applications which can be done by consulting the original publication (Barton, et al., 1974).

Other empirical methods do not specifically mention swelling or squeezing conditions but can include these via considerations of standup time (Laufer (1958), RMR (Bieniawski, 1979) or via consideration of strength and acting stress conditions (Franklin, 1976); Louis 1974). On the other hand, Wickham et al. (1974) in their RSR method specifically exclude squeezing conditions.

TABLE 6-5
 Deere's RQD-support relations (after Deere et al., 1968).
 GUIDELINES FOR SELECTION OF PRIMARY SUPPORT FOR 20-FT TO 40-FT TUNNELS IN ROCK

| Rock Quality | Construction Method | Steel Sets | Alternativ Support Systems | | | | | | |
|---|---|--------------------------|----------------------------|-------------------------|----------------------------|---|---------------------------------|-----------------|--|
| | | | Rock bolts ¹ | | Shotcrete ² | | Additional Support ^b | | |
| | | | Weight of Sets | Spacing ^c | Spacing of Pattern Bolts | Requirements and Anchorage Limitations ² | | Total Thickness | |
| Excellent RQd > 90 | Boring machine Drilling and blasting | Light | None to occasional | None to occasional | None to occasional | None to occasional | None | None | (Conditional use in poor and very poor rock) |
| Good RQd=75 to 90 | Boring machine Drilling and blasting | Light | None to occasional | None to occasional | None to occasional | None to occasional | None | None | |
| Fair RQd=50 to 75 | Boring machine Drilling and blasting | Light to medium | Occasional to 5 to 6 ft | Occasional to 5 to 6 ft | Occasional mesh and straps | Local application | None | None | |
| Poor RQd=25 to 50 | Boring machine Drilling and blasting | Medium to heavy circular | 3 to 4 ft | 3 to 5 ft | Mesh and straps required | Local application | None | None | Provide for rock bolts |
| Very poor RQd=25 (Excluding squeezing and swelling ground) | Boring machine Drilling and blasting | Medium to heavy circular | 2 to 4 ft | 2 to 4 ft | Mesh and straps required | Local application | None | None | Provide for rock bolts |
| Very poor, squeezing or swelling ground | Both methods | Very heavy circular | 2 ft | 2 to 3 ft | Mesh and straps required | Local application | None | None | Provide for rock bolts |

Note: Table reflects 1969 technology in the United States. Groundwater conditions and the details of jointing and weathering should be considered in conjunction with these guidelines, particularly in the poorer quality rock. Shotcrete anchors may be difficult or impossible to obtain anchorage with mechanically anchored rock bolts in poor and very poor rock. Grouted anchors may also be unsatisfactory in very wet tunnels because shotcrete exposure is limited, only general guidelines are given for support in the poorer quality rock. Changing requirements for steel sets will usually be minimal in excellent rock and will range from up to 25 percent in good rock to 100 percent in very poor rock. In good and excellent quality rock, the support requirement will in general be minimal but will be dependent on joint geometry, tunnel diameter, and relative orientations of joints and tunnel.

TABLE 6-6

Classification of gouge material (from Brekke and Howard, 1973).

| Dominant Material in Gouge | Potential Behavior of Gouge Material | | Initial Identification of Gouge Material | |
|---|--|--|--|-----------------------------|
| | At Face | Later | Gradation | Atterberg Limits |
| Swelling clay | Free swell, sloughing. | Swelling pressure and squeeze against support | | Liquid Limit LL > 30% |
| | Swelling pressure and squeeze on shield. | or lining, free swell with down-fall or wash-in if lining inadequate | >15% -No. 200 | or Imbibed Water > 5% |
| Inactive clay | Slaking and sloughing caused by squeeze. Heavy squeeze under extreme conditions. | Squeeze on supports or lining where unprotected, slaking and sloughing due to environmental changes. | Sieve | Liquid Limit LL < 30% |
| | | | | or Imbibed Water < 6% |
| Chlorite, talc, graphite, serpentine | Ravelling | Heavy loads may develop due to low strength, in particular when wet. | - | Plastic Index PI < 5% |
| Crushed rock fragments or sand-like gouge | Ravelling or running; standup time may be extremely short. | Loosening loads on lining. Running and ravelling if unconfined. | <15% -No. 200 Sieve | - |
| | Favorable condition | May dissolve, leading to instability of rock mass. | - | - |

To conclude the comments on empirical approaches, the method by Brekke and Howard (1973) shall be mentioned, which is specifically aimed at considering the behavior of fault gouge and includes characterization of swelling conditions and their consequences on support requirements (Table 6-6).

6-5.3 Analytical Methods for Tunnels in Swelling and Squeezing Ground

6-5.3.1 Introductory Comments

Given the wide variety of methods and the limited space in this book, a compromise between practically useable presentation and encompassing treatment had to be found. What will be done in the following, is to present a few closed form solutions (analytical methods in the narrow sense), in detail, followed by a brief review of other closed form methods and by a review of numerical solutions.

6-5.3.2 Analytical and numerical methods for tunnels in swelling ground

Inverse settlement method (Huder and Amberg, 1970; Grob, 1972; also Kovari, 1987): This method was originally based on the Huder-Amberg swell test (mentioned in Section 6.4.1) but can be also based on the standard swell test mentioned in the same Section. The concept and procedure are as follows:

- In most cases swelling in the zone under the invert and the associated invert heave is the most important problem.
- The oedometer swell test represents a suitable model test for swelling of an unsupported invert (assuming that the oedometer specimen represents the ground under the invert).
- The swell stress-strain behavior is obtained from the oedometer or similar swell test by deducting the elastic unloading strain from the total strain in the Huder-Amberg test (Fig. 6-20) or by using the procedure of the standardized test shown in Fig. 6-19. (Note that the standardized swell test (ISRM, 1988) is only meant to be applied to argillaceous rocks. The Huder-Amberg test has been applied to argillaceous rocks, anhydrite and mixed rocks.)
- The swell stress-strain curve is assumed to be a straight line in a "linear strain-log stress" plot. The assumption of a straight line behavior is a good approximation of the behavior of many argillaceous rocks and some anhydrites. Its general applicability has to be viewed with caution. (The major difference between argillaceous and anhydrite swelling is the irreversibility of the latter. This does not play a role in the original inverse settlement analysis, which essentially only determines the heave of an unsupported invert, but will be an important point to consider in other applications.)

- The stress change (decrease) caused by tunnel excavation is expressed by the ratio σ/σ_0 where σ = stress after excavating and σ_0 stress before excavation. These stresses are radial stresses with regard to the tunnel. They are axial stresses in the oedometer. σ/σ_0 is plotted in Fig. 6-24.

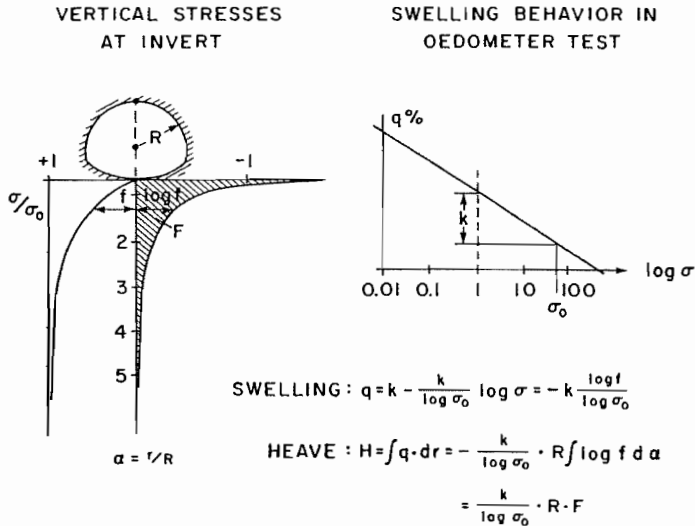


Fig. 6-24. Inverse settlement method (after Grob, 1972).

- Swelling strain and swell heave (displacement) can then be computed as shown in Fig. 6-24, the latter through integration of swell strain over the depth of the zone which is unloaded (destressed) due to excavation.
- To predict support behavior one can use the swelling stress-swelling strain curve and the above mentioned strain-heave relationship to derive a stress-heave curve. This is a so called characteristic curve* relating the stress applied at a particular point of the tunnel circumference to its displacement (Fig. 6-25). Fig. 6-25 is taken from Kovari et al. (1987); they essentially follow the original procedure by Grob except for making the following simplifying assumptions: 1. Swelling is limited to a zone one tunnel diameter (D) deep from the tunnel periphery. 2. The swell strain ϵ can be distributed uniformly or linearly decreasing or logarithmically

* The characteristic curve concept is discussed in Chapter 2.

decreasing over the depth D (Fig. 6-26). Swell heave can be expressed as $H = k D$, where k is a factor expressing the strain distribution.

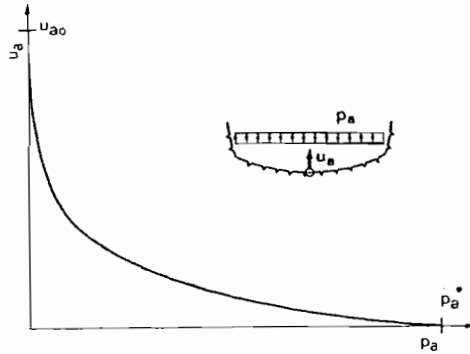


Fig. 6-25. Characteristic curve for invert heave due to swelling (from Kovari, et al., 1987). u_a = invert heave, ρ_a = stress

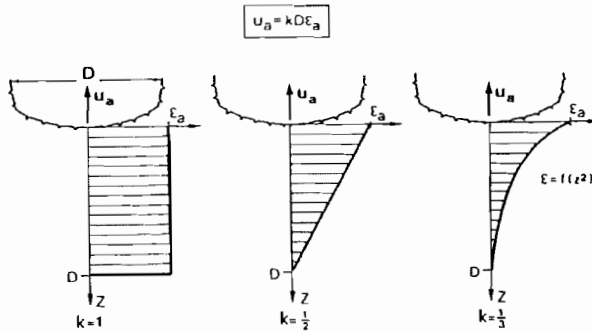


Fig. 6-26. Swell heave u_a depending on swell strain (ϵ_a) distribution. Type of distribution expressed by factor k (from Kovari et al., 1987).

The swell heave prediction with the inverse settlement method is thus entirely analogous to a simplified settlement analysis. Consequently it is also possible to subdivide the swelling zone, if different materials exist, and if their behavior has been determined with suitable (oedometer type) tests.

Application of this method is only entirely correct as long as the test conditions correspond to those in reality. This means that the lateral stresses which initially exist, and which are produced during the test in the oedometer should be like the real ones; this is rarely the case. The swell stress predictions are similarly only correct if the particular stress state is reached by stepwise unloading and by letting the strains dissipate in each step. The use of the characteristic curve (as shown in Fig. 6-25 above) to estimate support loads can thus only be a first approximation (in reality the invert e.g. is completely unloaded followed by a stress increase as the ground deforms against the invert structure).

Einstein-Bischoff Method (Einstein et al., 1972; Einstein and Bischoff, 1976; Einstein, 1981): This method makes also use of the Huder-Amberg - or of the standard swelling test. Its most important characteristic is the consideration of the three dimensional stress state, which was shown to be very significant through observations in tunnels by Golta (1967), and by the authors of the method. Expressed in a few words, swelling is influenced by the three dimensional state of stress in that swelling in one direction is strongly affected by the stress states in the other directions.

The concept and procedure is as follows:

As shown before (Section 6-3.1) the original or primary stress state will be modified by excavation (sometimes this stress state after excavation is called secondary stress state) and further modified by subsequent swelling or squeezing. As also shown in Section 6-3.1 it is possible to consider the three-dimensional nature of the problem through the first stress invariant,

$$I_1 = \sigma_{11} + \sigma_{22} + \sigma_{33} \quad (6-1)$$

Excavation produces a reduction of the first stress invariant in some zones around the tunnel. Swelling will take place in these zones until the stresses produced by swelling cause the resulting stress invariant (first invariant of the stresses after excavation plus first invariant of the swelling stress) to equal the original or primary first stress invariant:

$$I_{1exc} + I_{1swel} = I_{1prim} \quad (6-9)$$

Swelling stresses are produced by natural confinement and by artificial supports.

- The swelling stress-strain behavior is obtained from the Huder-Amberg or from the standard swell test. Again the elastic strains are not considered as was the case in the inverse settlement method. Assuming that the test conditions represent zero lateral strain, and that elastic conditions prevail, the corresponding first stress invariant can be computed with the knowledge of Poisson Ratio:

$$I_1 = \sigma_{ax} \frac{1+\nu}{1-\nu} \quad (6-10)$$

where σ_{ax} = axial swelling stress as measured in the laboratory test (see Section 6-4.1).

Since I_1 is proportional to σ_{ax} , the swell stress-strain curve as obtained from oedometer type tests can be used to produce the first stress invariant versus axial strain (first strain invariant) curve (Fig. 6-27) by simply modifying the stress scale. (The first strain invariant and axial strain are equal for conditions of zero lateral strain).

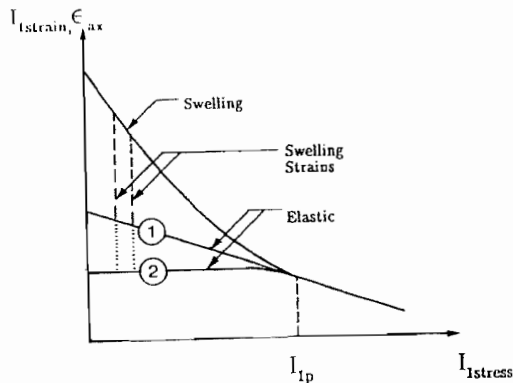


Fig. 6-27. Axial swell stress-strain curve for oedometer type tests expressed in form of stress/strain invariants. I_{1p} = first stress invariant in primary state of stress. (Note: this curve can be derived from the results of tests according to Huder-Amberg (see Fig. 6-20) or ISRM (see Fig. 6-19b). In the former case the "elastic behavior" is expressed by elastic curve 1, in the latter by elastic curve 2 which is horizontal since the ISRM results already exclude elastic strains.)

- The primary stress state and the stress state after excavating the tunnel i.e., the corresponding first stress invariants, are determined assuming elastic conditions and, usually also, plane strain (the latter is however not necessary). In the zones around the tunnel where the stress state after excavation produces a first stress invariant smaller than the primary one, swelling occur; these are the so called swelling zones.
- Swell heave for cases without artificial supports is computed using the first stress invariant versus swelling strain curve (Fig. 6-27). The swell strain is obtained as the difference between the elastic strain after excavation and the point of interest on the swell curve (usually complete unloading for the 'no artificial support' case). How to obtain this difference depends on the derivation of the swell curve as indicated in Fig. 6-27. The heave is then obtained through integration over the extent of the swelling zone. The procedure is thus the same as for the inverse settlement method with the exception that it is performed on the basis of the three dimensional stress state.
- Swell stress on a support is completed in three steps:
 1. The stress-deformation behavior of the support is derived for any desired point at the tunnel periphery.
 2. The swell heave for the same point is computed as described above for the no-support case. From this one deducts the heave that takes place between excavation and support placement. This can be either obtained from field observations or from laboratory test observations and using the empirical relation developed by Bischoff (1975):

$$\frac{\epsilon_{lab}^{1.5}}{\epsilon_{field}^{1.5}} = \frac{t_{lab}}{t_{field}} \quad (6-11)$$

where ϵ = swell strains, t = time over which strains develop.

3. Support stress and displacement (from step 1) have to be equal to the swell stress and heave (from step 2). This is done analogously to Kovari's (1987) extension of the inverse settlement procedure. Here, however, one considers the three dimensional stress state. This is done by combining the first stress invariant after excavation with the stresses imposed on the ground by the support (Fig. 6-28). Swell strain and heave are calculated for this "revised" first stress invariant and steps 2 and 3 are repeated. An iterative procedure is thus required to

arrive at the equilibrium ground and support stresses and displacements. The method is however sufficiently simple to accomplish this by hand in a few steps.

- An important aspect of this method is the inclusion of in situ monitoring. Given the uncertainty in transferring swell behavior as observed in a oedometer type test to reality, it is considered essential to monitor the actual behavior. Monitoring together with the capability of adapting design and construction, make it easier to deal with changes in swelling behavior.

Although this method is somewhat more realistic than the inverse settlement method by including the third dimension, it is still simplified in that elastic conditions are assumed and in that, in essence, only a particular point of the support is considered. The fact that the deformation of a point influences the remainder of the support and the ground around it is not taken into account.

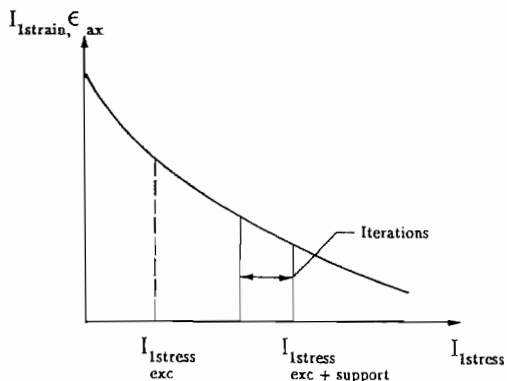


Fig. 6-28. Einstein-Bischoff procedure for determining stresses and strains in tunnels with support using the $I_{1\text{stress}} - \epsilon$ curve.

Method by Gysel (1977) with extension by Gysel (and Bellwald)(1987): This method remedies the limitations of the Einstein-Bischoff method by considering ground structure interaction of the entire support (Note that for the closed form solution this support has to be circularly shaped; the principle of the method could be extended to other shapes employing numerical approaches). The concept and procedure of the method are as follows:

- The swell displacement (heave) - stress relation is essentially analogous to that of Einstein-Bischoff in that changes in the first stress invariant are related to swell strains which are in turn integrated over the relevant depth. A linear relation between the first swell strain invariant and the logarithm of the first stress invariant is used, analogous to the linear relation used in the inverse settlement method. The invariants are computed assuming elastic conditions. To compute swell displacements in a particular direction, notably in the radial direction which is of greatest interest, an assumption has to be made regarding the distribution of swell strains. Gysel (1987) discusses a number of possible assumptions (see also Wittke-Rissler (1976) method below).
- Elastic displacement caused by the excavation are computed and subtracted from the swell displacements (it is also possible to consider presupport swell displacements).
- The support-stress displacement relation is computed according to Morgan (1961) and Muir Wood (1975) analogous to the flexibility and stiffness approach described in Section 2-10, Chapter 2). This relation depends on a factor describing the support stress distribution.
- Analogous to the precedingly discussed methods, the support stress-displacement and the ground stress-heave (displacement) at a particular point on the tunnel circumference are set equal. This allows one to circulate the above mentioned support stress distribution factor - which in turn is used to compute the support forces and moments.

While this method includes ground structure interaction of the entire support, it is still limited by the underlying assumption of elasticity, tunnel shape and simplified ground and support stress states.

Wittke-Rissler method (Wittke and Rissler, 1976): This is the numerical analog of the Einstein-Bischoff and Gysel methods. The underlying concept is the three dimensional approach of Einstein and Bischoff. This is implemented in a finite element procedure which allows one to eliminate some of the simplifications of the Einstein-Bischoff and Gysel methods. It is possible to consider any shape opening (in contrast to Gysel) and to consider the complete ground structure interaction rather than only the behavior of a single point (in contrast to Einstein-Bischoff). As in the Gysel method it is however necessary to make an assumption regarding the distribution of the strain components which make up the first strain invariant (the principal strains are assumed to be proportional to the principal stress differences between the primary state of stress and the state of stress after excavation).

The Wittke-Rissler approach is incorporated in a three dimensional finite element analysis and is thus an attractive design tool. Nevertheless, one has to be aware of the fact that it is based on oedometer type tests, which are essentially one-dimensional, with assumptions on the behavior in the other directions, and it is also based on elastic behavior. The Wittke-Rissler method, by the way, considers only an increase in swell strains (and of stresses if constrained) but not the possibility of a reversal (i.e. consolidation is not included); in essence swelling is considered irreversible which is correct for anhydrite but not for argillaceous ground. This minor limitation could be easily corrected, however. Both the Wittke-Rissler and Gysel approaches assume that swelling will only take place if the first stress invariant becomes smaller than the stress invariant at which no axial expansion occurs in the swell test (stress state at D in Fig. 6-20). This is in contrast to the Einstein-Bischoff assumption where swelling takes place in all stress states below the primary state of stress but where swelling produces counterstresses to eventually stop itself at stress state above B (Fig. 6-20).

Another finite element based method is Rheostaub, (see Kovari, et al., 1983); its underlying assumptions are similar to Wittke's. It goes beyond Wittke's in that a non-linear 'swell stress (first stress invariant)-strain (first strain invariant)' relationship can be used; in other words the actually observed relationship can be modelled. Nevertheless, also this method is based on the oedometer test and the elastically derived invariants. Rheostaub has, however, elasto-plastic capabilities and it could handle (time dependent) yielding in combination with swelling, if the appropriate test results did exist.

Frohlich (1968, 1987) introduces the capability to treat anisotropic swelling. Argillaceous rocks, in particular, are bedded and swelling is much more pronounced in the direction perpendicular to the bedding than parallel to it. Based on swell tests Frohlich developed constitutive relations for anisotropic swell behavior and included them in a finite element procedure.

To conclude this discussion on methods for designing swelling in tunnels, two approaches based on the characteristic curve method shall be briefly outlined:

Peck (1969) illustrates ground structure interaction with the schematic diagram of Fig. 6-29a. (Note that in contrast to other applications of the characteristic curve method an equivalent ring (support) load rather than the support pressure is plotted. (The former is simply proportional to the latter.) In Fig. 6-29b, Peck then shows that the ring load can still increase

after support and ground have attained the equilibrium point, particularly for swelling clays. Peck (1969) also mentions that, in general, the ring load should be calculated with the full overburden stress, which should also be sufficient for swelling clays; he goes on to say that for overconsolidated clays the ring load should be calculated with $1/2 (1+K_0)p_z$. i.e. a value greater than the overburden stress.

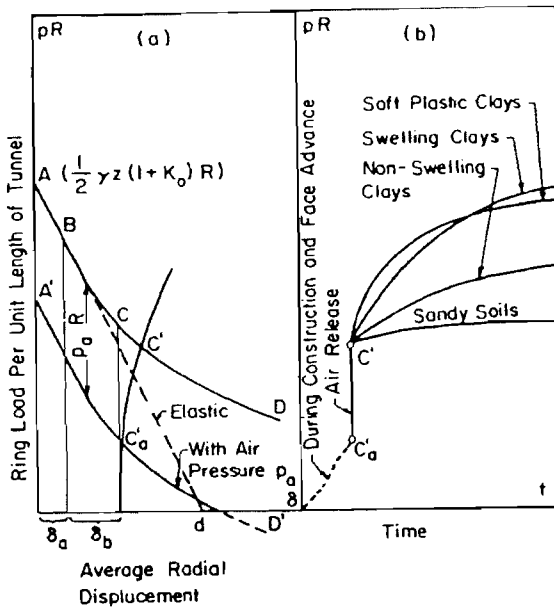


Fig. 6-29a. Characteristic curve for ground and support. b. Time-support load curve (from Peck, 1969).

Lombardi (1973) extended his elasto-plastic characteristic curve method to include swelling. His method allows one to include a volume increase in the plastic zone. While this was originally intended to consider dilatant behavior, it can also be used for swelling. This is done by relating the plastic volume increment to time. Volume increments are based on experience which may make the use of the method difficult.

As is the case in most characteristic curve based methods the two above mentioned ones essentially represent one specific point of the tunnel periphery.

6-5.3.3 Analytical and Numerical Methods for Squeezing Ground

Any number of methods for analyzing creep exist and several are specifically aimed at tunnel applications. In the lines below the method by Semple et al., (1973) using Aiyer's tunnel analysis (Aiyer, 1969) will be described as a representative example:

The creep relation can be formulated as follows:

$$\epsilon = B \exp(\beta \Delta) (t/t_1)^\lambda$$

where t_1 = time at which R and β are defined

Δ = stress level = ratio of applied deviator stress to strength, determined in a conventional constant rate of strain triaxial test

B , β , λ are constants, defined as shown in Fig. 6-30a and 6-30b.

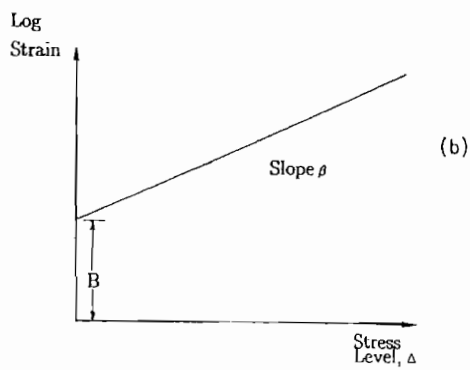
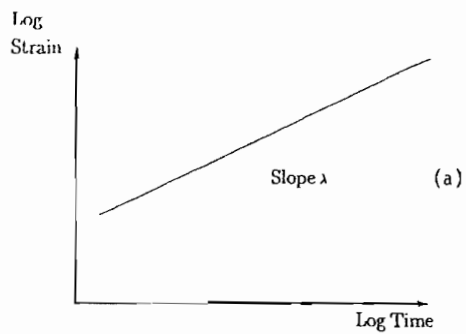


Fig. 6-30a, b. Definition of creep factors (after Semple et al., 1973).

For the fault gouges tested by Semple et al., (1973), the representative value are $B = 0.1$ to 0.4% $\beta = 2.5$ to 5.0 $\lambda = 3\%$ to 9% .

The creep relation can be incorporated in Aiyer's iterative method (Aiyer, 1969) to produce strains at the periphery of a circular unsupported tunnel or stresses and strains of the lining (with known elastic properties) of a tunnel (assuming uniform far field stress).

In materials investigated by Semple et al., (1973) varying B and β magnitudes had no effect on the ground movement of unsupported tunnels, these movements were, however, directly proportional to λ and so were the radial stresses acting on a support. Also, time dependent ground movements are directly proportional to overburden pressure.

Similar creep relations have been used either in closed form approaches or finite element methods. An example of the latter is Pheostaub (Kovari, 1983) mentioned previously.

In squeezing conditions the problem is not so much in finding suitable analysis methods, but in making certain that the material properties are properly determined.

6-5.4 Analysis of Combined Swelling and Squeezing

It has been shown in Section 6-3.3 that swelling, through the pore pressure changes (dissipation of negative pore pressures) and through the production of very high stresses in the constrained direction, may actually cause yielding. On the other hand, yielding may break diagenetic bonds and provide thus the basis for increased swelling.

None of the analytical or numerical procedures developed so far allows one to treat such a combined behavior. (Rheostaub comes closest but requires some work to include the pore pressure effects.) Nevertheless, it seems relatively straightforward to incorporate such capabilities in most of the closed form approaches and certainly in the finite element methods. For the time being, it seems worthwhile to wait for a better characterization of the combined swell- γ yield process from several studies which are under way.

6-5.5 Concluding Comments on the Analysis of Swelling and Squeezing for Tunnel Design

A number of empirical, closed form and numerical methods exist that allow one to predict the consequences of swelling and squeezing in tunneling. However, one should be aware of the fact that these methods are either based on a limited number of cases (empirical approaches) or make limiting assumptions (closed form and numerical approaches). This is particularly so regarding swelling where the underlying laboratory test is usually an oedometer type

test, and the observed behavior in the laboratory is assumed to correspond to that in the field. Elastic behavior is also a common assumption. Finally, most of the swelling analyses are based on the behavior of argillaceous rock, and one assumes this behavior to be applicable to anhydrite. Such an assumption is problematic if counterstresses cause a reversal of swelling which is only possible in argillaceous rock but not so in anhydrite. Also, the linear swell stress-strain curve in the semilogarithmic presentation may be a reasonable approximation for argillaceous material but not for anhydrite.

6-6 Design and Construction of Tunnels in Swelling and Squeezing Ground

6-6.1 Basic Concepts

Swelling and squeezing in tunneling can be handled by either preventing them or by designing the tunnel such that the consequences of time independent deformation can be accommodated. The former approach will be termed "active," the latter "passive." In many instances passive and active measures will be combined. As will be seen below, active and passive measures encompass both design and construction. While integration of design and construction should be considered in all types of construction, it is essential in underground construction and indispensable in underground construction involving problematic ground such as swelling and squeezing materials. As a matter of fact, in such conditions not only design and construction but also operation and maintenance have to be considered as a whole.

6-6.2 Design and Construction of Tunnels in Swelling Ground

Passive Approach

Shaving off of ground is the simplest passive design-construction approach. It has been used in many tunnels during construction and, as was shown in the case histories of Section 6-2, also during operation. This underscores what has been said above, that not only design and construction but also operation/maintenance need to be considered. On first sight, the shaving off approach seems to contradict solid engineering, which is to provide the client with a structure requiring a minimum of maintenance. However, it may in many cases be the most cost effective approach. This is particularly so, if the internal structure is built in a way to make the ground removal easily possible. Nevertheless, one has to be aware of the fact that removing swelling ground prevents this ground from building up counterstresses; the total heave will thus, in most instances, be greater than if the ground were left in place.

Frangible backpacking, compressible joint elements and many similar features make it possible for some swelling displacements to take place in a largely

uninhibited manner or with minor restraints before exerting stresses on the support. These support systems are usually conceived in such a manner that the supports will eventually carry some load and produce swell reducing counterstresses; they are thus a mixture of the passive and active approach. Nevertheless, the underlying concept is to let most of the swell displacements take place before significant forces are produced. Fig. 6-31 shows a number of possibilities involving several which have been practically applied or are planned, ranging from frangible backpacing to compressible joint elements to compression slots. The reader is also referred to Section 6-6.3 where so called yielding supports in squeezing conditions are shown; they serve basically the same purpose as the designs shown in Fig. 6-31.

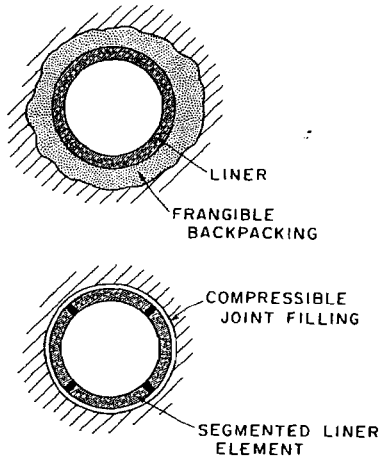
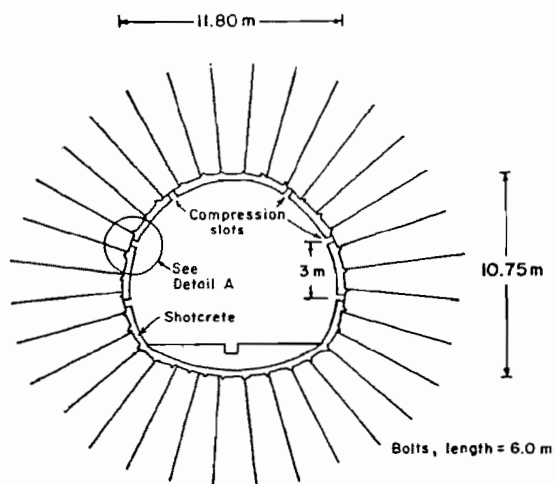


Fig. 6-31. Design of tunnels in swelling rock; passive approach.
a. Frangible backpacing and compressible joint fillers.



CROSS-SECTION OF TUNNEL WITH COMPRESSION SLOTS

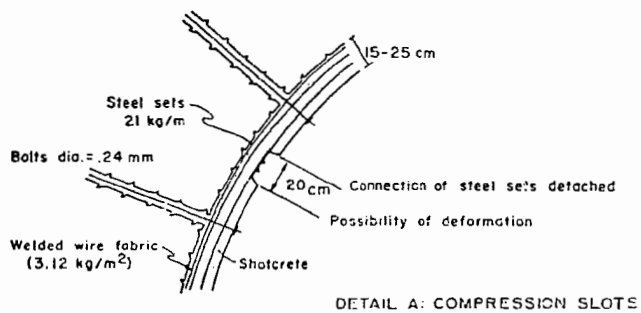


Fig. 6-31. Design of tunnels in swelling rock; passive approach.
b. Compression slots (after Pöchhacker, 1976).

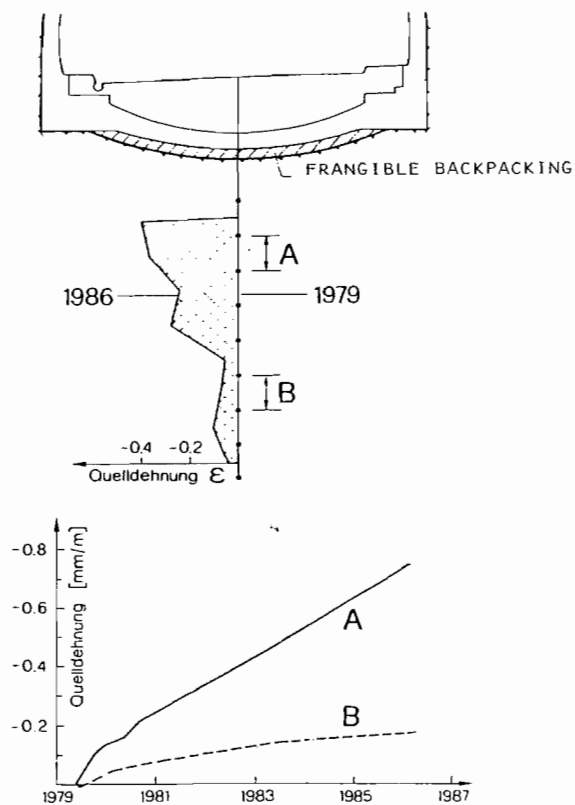


Fig. 6-31. Design of tunnels in swelling rock; passive approach.
c. Frangible backpacking under invert (after Kovari, et al., 1987).

Active Approach

From what has been described in Section 6-3.1, it is evident that preventing or limiting swelling can be attained by preventing water from entering the distressed zones, by reducing its effect if it enters, by preventing distressing or by a combination of these measures.

Chemical inhibitors have found application in the oil industry to prevent swelling in boreholes in argillaceous rocks. They are based on modification of the electrolyte content of the pore water. In order to be effective they have to penetrate the low permeability rock which limits the size of the zones that can be effectively treated. Although these procedures were successful in boreholes, this is unlikely in tunnels, at least with the present application methods. Hull et al., (1980) describe a variety of crystallization inhibiting agents that prevent gypsum growth and which, if periodically applied, may be sufficient to prevent invert heave in anhydrite containing rock.

The stress in the ground can be affected by shaping the tunnel cross-section, by applying counterstresses or a combination of both:

The shape of the tunnel cross-section can be chosen such that the first stress invariant remains close to the original (primary) state of stress. This will reduce the swelling potential. It is important to remember however, that this simple statement is based on assumptions of material isotropy and to some extent on assuming elastic behavior. The result of this design criterion will usually be a curved tunnel shape. Curved shapes have the additional advantage that larger counterstresses will develop than for flat surfaces. Swelling of ground below curved surfaces will produce a substantial increase in lateral stresses due to the development of an arch effect (Fig. 6-32b). This is less so under flat surfaces where lateral stresses may lead to static instabilities in form of buckling and are thus limited in magnitude (Fig. 6-32a).

Artificial arches are the logical extension of the curved surface concept. Swelling displacements will be restrained by such structures, and they will mobilize counterstresses (in addition to the naturally mobilized ones). In many cases the major stress relief occurs in the crown and invert. While constructing a crown arch is a common and easily performable procedure, curved inverts and construction of invert arches is problematic unless full face tunnel boring machines are used. Nevertheless, excavating an invert arch by drilling and blasting may be the only successful approach to limit swelling. A point in case is the Belchen tunnel which has been described in Section 6-2. Recall that the original design prescribed an invert arch with large radius; this failed and had to be replaced by an arch with smaller radius and greater thickness (Fig. 6-7).

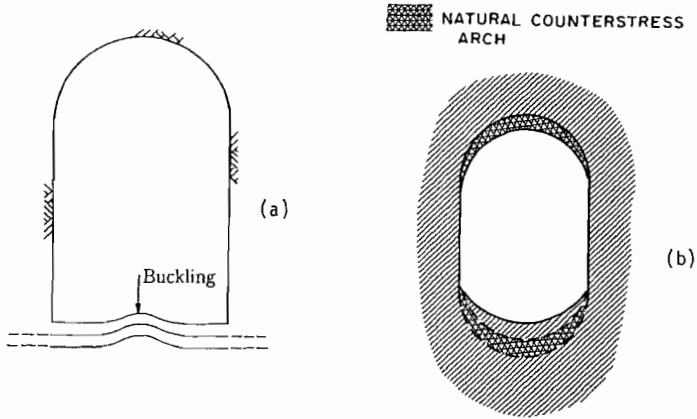


Fig. 6-32. Design of tunnels in swelling rock; active approach.
 a. Buckling of flat invert caused by excessive swell heave.
 b. Buildup of natural counterstresses under curved invert.

Bolting or anchoring the final support (liner) (see Fig. 6-33) is another method to apply counterstresses. This approach can be used with tunnel surfaces of any shape. It is important that the bolts are anchored beyond the destressed zone (see Einstein et al., 1972). If the swelling stresses are large it may be technically difficult and economically infeasible to place a sufficient number of bolts.

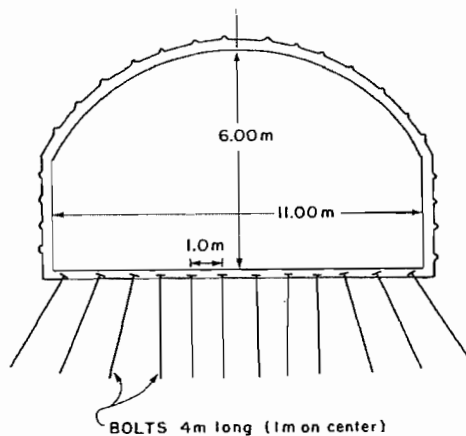


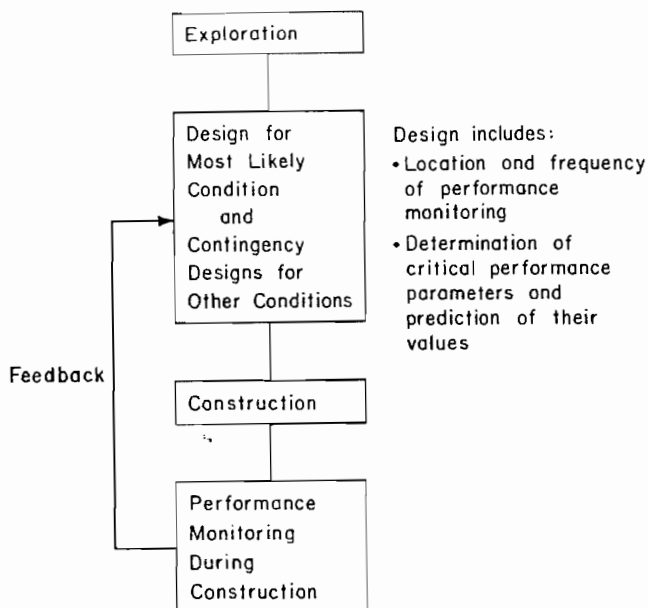
Fig. 6-33. Design of tunnels in swelling rock; active approach. Bolted invert plate.

The comments above concentrated on describing design features. It has to be reemphasized that design and construction have to be integrated. One obvious and already discussed aspect of this is the elimination or reduction of water penetrating into the ground. Drainage and possibly sealing after each round (cycle) is thus a must when tunneling through swelling rock. The drainage facilities (ditches, pipes) should be built such that no leakage into the ground can occur. This usually means that the ditches have to be flexibly lined and that flexible pipes have to be used to accommodate swell displacements. In any case, the construction process has to be planned such that adjustments to the drainage facilities (and possibly to the surface seals) can be made.

The uncertainty in predicting swell displacements and particularly swelling time requires that construction be adaptable. Observational approaches in which different contingency designs are developed and selected depending on the observed conditions should thus be used. This, by definition, involves monitoring of displacements and possibly also of loads. Amongst the designs discussed above several are particularly well suited to be adapted; compressible joint elements (see Brunar et al., (1985), and comments in Section 6-6.3) and particularly bolts can be adjusted both in number and in load capacity. If shotcrete is used, some adaptation of support flexibility through compression slots and thickness is also possible. If adaptable procedures are used, they have to be thoroughly planned as outlined in the diagram below (Fig. 6-34).

As mentioned earlier, operation and maintenance has to be considered in the design-construction process. The uncertainty in swelling behavior may cause swell displacements and stresses to increase during operation. This can be handled by conservative design involving passive, active or combined approaches. In the case of passive approaches it may lead to ongoing maintenance work and some interruption of the tunnel operation. Examples are repositioning of railroad tracks or repaving of roadways, possibly involving some shaving off of the ground. Another possibility is to increase the tunnel cross section such that swell heave can be accommodated with minor adjustments which can be handled as a part of regular maintenance procedures. Many other possibilities exist. The decision on what design to use has to consider all the consequences on construction, operation and maintenance. An optimum combination can be chosen on the basis of economic criteria or a combination of economic and other (e.g. aesthetic, comfort) criteria. Such well structured decision making processes should, as a matter of fact, be applied to any type of tunnel; they are of particular benefit in swelling rock tunnels.

OBSERVATIONAL METHOD



Knowledge of design parameters is updated through monitoring and feedback into design.

Fig. 6-34. Principle of adaptable/observational approach to tunnel design and construction.

6-6.3 Design and Construction of Tunnels in Squeezing Ground

The basic approach and a number of the specific design-construction procedures used in swelling ground apply here also. The review below will thus refer as much as possible to Section 6-6.2.

Passive approaches in squeezing ground will be aimed at accommodating the displacements. The passive design will usually be in form of compressible joint material and compression slots rather than in form of frangible backpacking. As a matter of fact the compression slots (Fig. 6-31b) were developed for the NATM in the Tauern tunnel (see e.g. Pochhacker, 1976; Steiner, et al., 1980) to accommodate strongly squeezing ground with

displacements of 1-1/2 feet (40 cm) and more. The compressible joint elements shown in Fig. 6-35 were developed for application in ground which is strongly deformed (due to mining). Other types of compressible joint elements exist ranging from wooden pieces inserted between concrete liner elements to flat jacks. Most of these elements represent a combination active-passive procedure in that they follow the characteristic curve of Fig. 6-35, i.e., they react in a stiff manner before they yield. This is advantageous because, as will be discussed below, some counterstresses are beneficial.

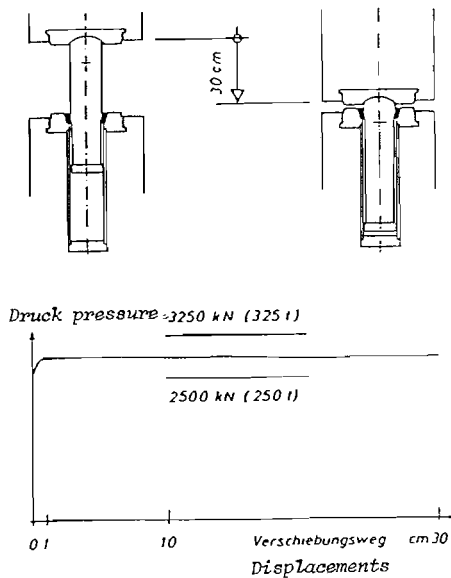


Fig. 6-35. Compressible joint elements type Meypo (from Brunar et al., 1985).

Active approaches in squeezing ground conditions are aimed at producing stress conditions preventing or eliminating further creep. This can be achieved by structures of sufficient stiffness against which the ground deforms and builds up counterstresses such as the artificial arches, or by structures (structural elements) which directly apply counterstresses (such as bolts).

The same comments regarding adaptable (observational) methods that were made in Section 6-6.2 apply here. As a matter of fact one of the "classic" adaptable methods, the NATM, was developed to efficiently and effectively handle squeezing ground deformation.

6-7 CONCLUSIONS

The consequences of swelling and squeezing ground on tunneling do usually not endanger life, but they can be extremely costly. Full or partial destruction of a tunnel can occur both during construction and during the entire lifetime of a tunnel. The problem is made worse by the fact that the available characterization methods and thus the prediction methods for swelling and squeezing ground are not entirely adequate. Nevertheless, it is possible to analyze and design tunnels in such ground conditions such that they perform satisfactorily. This requires a good understanding of the underlying mechanism, of the limitations of the applied analyses and, particularly, of the suitable construction procedures. Most important is the integration of design and construction; this usually involves monitoring during construction and the possibility to adapt the design, if necessary.

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Chapter 7

UNDERGROUND STRUCTURES IN ROCK BURST ZONES

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7-1 INTRODUCTION

Rock burst is a term used to describe rock failures ranging in magnitude from the explosion of small fragments of rock from underground excavation faces or side walls to sudden collapse of a large section of a tunnel or an excavation. A burst is, therefore, defined as a sudden and violent explosion of rock in or around an excavation. Failure is normally associated with high stress and brittle or brittle-elastic materials; bursting may also be associated with desorbed gas, and this type of failure is termed an outburst. These events have the potential to inflict severe injury to mining personnel. Invariably production is disrupted, and entry closure will result. Usually smaller bursts occur in openings of limited size, such as tunnels or development drifts, whereas larger bursts are more likely to happen in extensively mined areas. Rock bursts may have damaging effects on rock surrounding other openings, as well as on rock within the vicinity of tunnelling.

The severity and frequency of rock bursts usually increase with depth. The cause of this increase is attributed to the increased weight of the overburden strata, and correspondingly, the increasing stress in the rock with depth. However, depth is not the only factor that can contribute to rock bursts. Bursts have been reported in excavations under only 1,000 ft (304.8 m) of cover. Generally, bursts in shallow excavations occur infrequently and are not as severe. In most deep, burst prone, underground excavations the depth at which bursts are first experienced is usually below 2,000 ft (609.68 m), in most instances, they become a serious problem after 3,000 ft (914.41 m). However, some underground operations have excavated at depths greater than 5,000 ft (1,524 m) without bursts. This indicates that site-specific conditions other than depth are also important factors.

This chapter discusses factors affecting rock bursts and methods to predict their occurrences. Methods to avoid burst conditions using destressing techniques are also discussed.

7-2 BURST MECHANISMS

The exact causes of rock bursts are very difficult to determine, and reliable prediction is nearly impossible. While localized high-stressed zones are common to all burst occurrences, other factors may act independently or together to cause a burst. The contributing factors shown in figure 7-1 are combined under four major categories: strain energy, geology, physical properties, and opening design (Haramy et al., 1985).

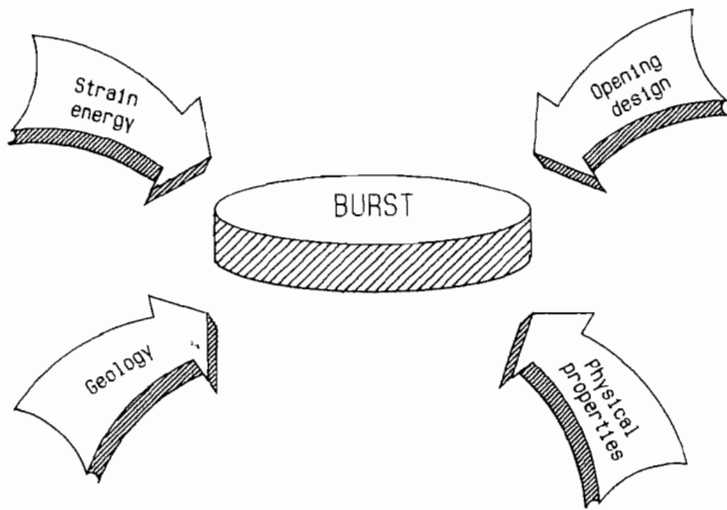


Fig. 7-1. Burst contributing factors.

7-2.1 Strain Energy

Rock has the capacity to store large amounts of strain energy before failing. The higher the maximum strain energy that can be stored in a given type of rock, the more likely the rock will have the tendency to burst. The total energy stored depends on rock mechanical properties, lateral confinement, and the magnitude of the applied stress. The maximum strain energy stored per unit volume in a uniaxially loaded rock is $\frac{\sigma_{vc}^2}{2E}$ (Obert and Duvall, 1967), where σ_{vc} is the uniaxial compressive strength and E is the modulus of elasticity. This relationship indicates that the type of rock generally associated with bursts is hard, strayed (large compressive strength), and brittle (large E). However, in recent studies Babcock (1984) indicated that any rock may burst in a laboratory environment, and presumably in a mine situation, given the right confinement conditions. Magnitude of the stress is

largely dependent on the number, size, and shape of openings. Before the excavation, the rock mass is in equilibrium. Entry (opening) development redistributes stress in the rock mass and results in permanent deformation around the opening. Stress increases around the opening until a critical level is reached, once the capacity of the rock to store strain energy is reached, any additional stress will cause the rock to burst.

An energy index (W_{ET}) obtained from laboratory-derived stress-strain curves can be used to distinguish between bursting and nonbursting rock. Nonbursting rock tends to plastically or visco-elastically deform under stress, while bursting rock tends to build up stress and fail violently. Figure 7-2 shows

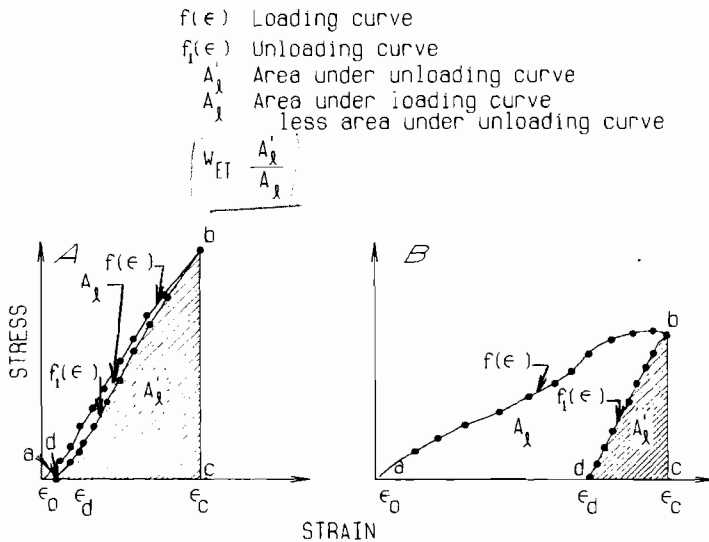


Fig. 7-2. Loading and unloading curves for (a) burst-prone, and (b) non-burst-prone samples (Neyman et al., 1972).

typical loading and unloading curves for laboratory rock samples. The area under the unloading curve is termed A'_λ . The area between the loading and unloading curves is termed A_λ , and the energy index is a comparison of the two areas, A'_λ and A_λ , and can be calculated by the following equation (Neyman et al., 1972):

$$W_{ET} = \frac{A'_\lambda}{A_\lambda} = \frac{\int_{\epsilon_d}^{\epsilon_c} f_1(\epsilon) d\epsilon}{\int_{\epsilon_0}^{\epsilon_c} f(\epsilon) d\epsilon - \int_{\epsilon_d}^{\epsilon_c} f_1(\epsilon) d\epsilon} \quad (7-1)$$

where W_{ET} = energy index

A'_e = energy elastically accumulated in a sample

A_p = energy losses due to permanent strain

ϵ_c = total strain

ϵ_d = permanent nonelastic strain

ϵ_0 = strain at zero load.

The larger the W_{ET} , the more the rock is liable to burst; strong rock stores more strain energy and as a result is more liable to burst violently. Figure 7-3 shows a relationship between uniaxial compressive strength of rock and relative violence of fracture (Hill and Denklaus, 1961).

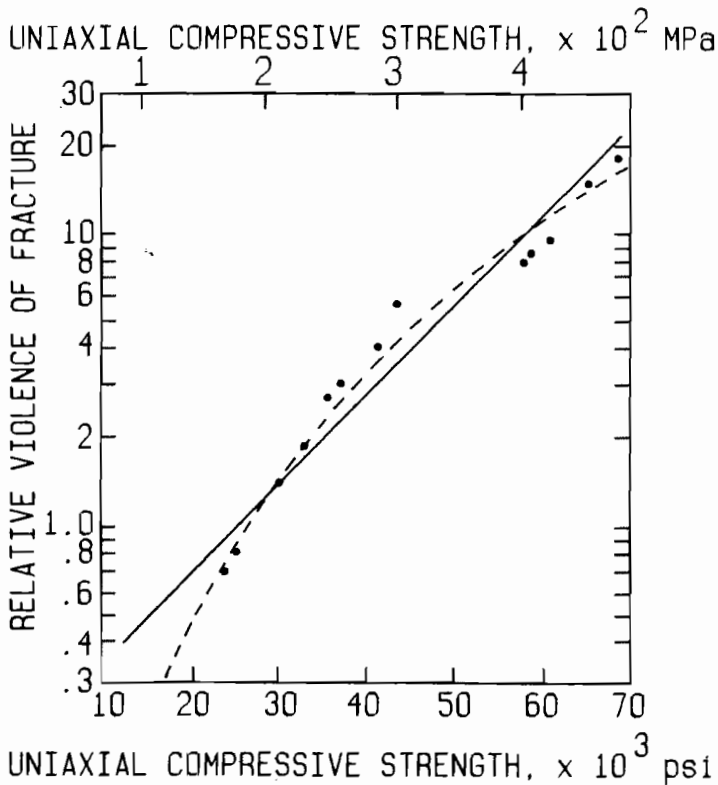


Fig. 7-3. Relationship between rock compressive strength and relative violence of fracture.

Failures caused by impact loading may also occur because failure of strong

roofs in underground excavations can transmit an induced stress wave over a great distance, resulting in simultaneous bursts over a large area. This effect occurs when sudden loading causes high stresses to develop in a progressive manner.

7-2.2 Geology

In underground excavations, geological features and rock physical properties are factors over which the engineer has no control. The location and orientation of geological anomalies, such as faults, folds, dikes, and joints, often contribute to burst conditions. Interactions between anomalies and bursts are difficult to quantify, and opinions on the subject vary significantly. For example, substantial disagreement exists on the effect that faulted areas have on bursts. In studies of Upper Bavarian seams in West Germany, it was noted that bursts usually occur in nonfaulted areas, while faulted areas have few bursts (Lama, 1967). Stress concentrations along fault surfaces can cause many microcracks to form, releasing excess stresses; the faulted area is less likely to burst. Faults can absorb high amounts of strain energy and reduce the number of bursts. Contrary to these observations, substantial evidence indicates that bursts occur more frequently in and around fault zones. At the Spring Hill Mine in Canada, strong bursts occurred near fault areas during development of inclined headings (Campbell, 1958). At the Sunnyside Mines, Utah, the occurrence and magnitude of bursts increased considerably as the openings were developed through 18-ft (5.5-m) displaced fault zones. As a result, alternate means of support were needed to support the entries. In Upper Silesia, Poland, the Andreas No. 3 seam is folded by an overthrust fault along a 1,500-ft (457.2-m) zone. Below the fault, explosive bursts occurred, while the area above the overthrust zone was free from bursts. The 360-ft (109.7-m) depth cannot justify this abnormality. Scientists reasoned that stress was released above, but not below, the fault zone (Lama, 1967). In South African mines, bursts normally increase as the mining face approaches faulted zones (Hackett, 1962). Figures 7-4 and 7-5 show the influence of faults and dikes on burst prediction at underground excavations in the Witwatersand district, which are 6,000 to 9,000 ft (1830 to 2790 m) deep (Hill and Denkhaus, 1961).

Experience indicates that stress levels will occur in dikes present in the rock being excavated. The increased temperature, metamorphism, and recrystallization of rock near dikes may increase the spontaneous energy and uniformity of rock mass (Mohr, 1956). It is believed that the presence of dikes causes weakness in the mine structure and hence causes an increase in burst production in the proximity of a dike, as shown in figure 7-5.

Mining under synclinal folds also increases the frequency of burst.

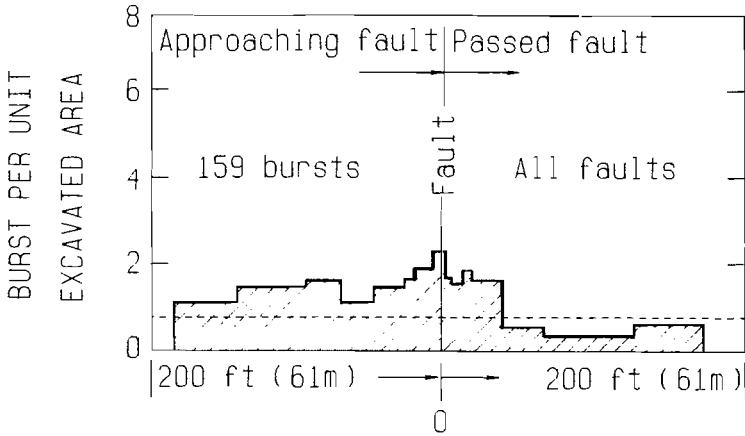


Fig. 7-4. The effect of faults on burst occurrences.

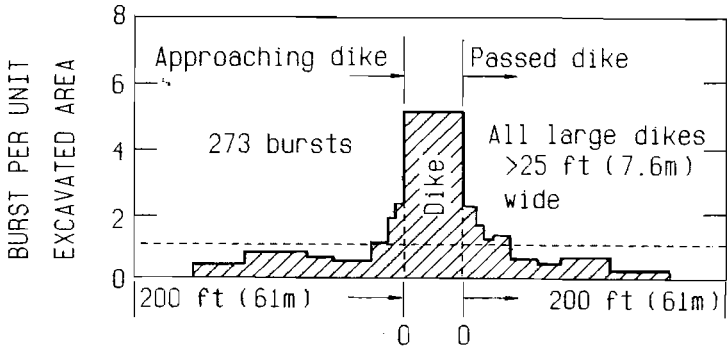


Fig. 7-5. The effect of dikes on burst occurrences.

Synclinal folds are normally jointed as a result of high lateral ground stresses. At the Knawow Colliery in Poland, crosscuts developed beneath small synclinal fold axes were difficult to maintain; however, as excavation advanced through the synclinal region, crosscuts remained stable. At the Miechowicz Colliery in Poland, excavation under synclinal folds resulted in a large number of bursts (Lama, 1967).

Increasing depths, thick overburdens, and steeply dipping excavations are generally synonymous with burst conditions. Steep terrain, with resulting fluctuation in overburden pressure, and strong overlying strata that may resist caving are contributing factors to bursts. In general, the extent and degree

of geologic disturbance in a rock deposit can create burst-prone conditions and should be carefully studied prior to planning underground excavations.

7-2.3 Physical Properties

High stress buildup is closely related to the rock mass physical properties. The properties of the rock in the opening zone and nearby strata have a significant effect on burst conditions. The type of rock generally associated with rock bursts is qualitatively described as hard, strong, and brittle. In terms of mechanical properties, the rock has an unconfined compressive strength, σ_{vc} , from 15,000 to 60,000 psi (100 to 420 MPa) and a modulus of elasticity, E , ranging from 6×10^6 to 14×10^6 psi (4 to 9.7 GPa). Conversely, rock types with similar strength and elastic modulus properties are known to be mineable at depths with no occurrence of bursting (Obert and Duvall, 1967).

7-2.4 Opening Design

Inadequate planning or improper design can contribute to and increase the occurrence of bursts in underground openings. The sequence of excavating and interaction between adjacent openings or workings are important factors to

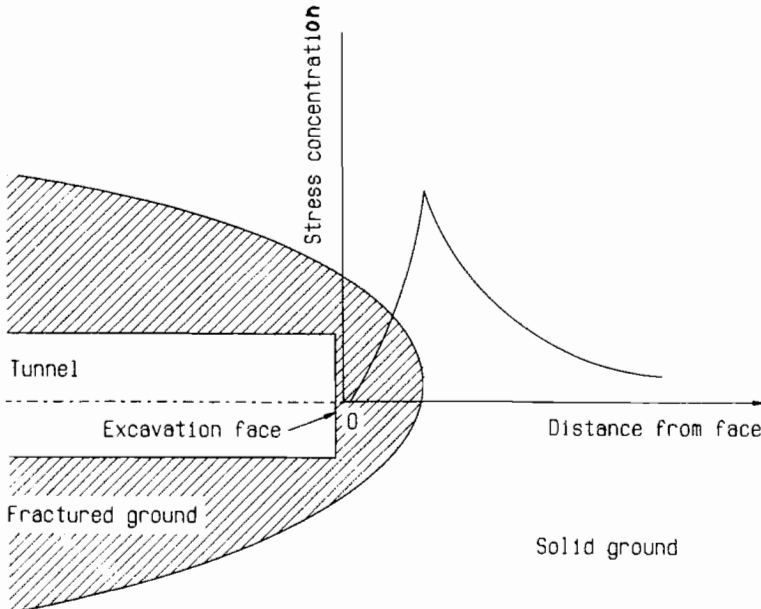


Fig. 7-6. Abutment zones around an excavated tunnel.

consider.

For a simplified case, the opening creates a pressure arch or abutment zone around the opening as shown in figure 7-6. The excavating and blasting during development also create a stress-relieved zone in the proximity of the opening. Developing tunnels or drifts within the abutment zone created by other workings or high-stress areas may result in violent bursts. Sharp corners, remnants, islands, and other underground structures that cause or enhance stress concentrations are locations for many and severe bursts, which can be significantly reduced if proper planning is implemented.

7-3 DETECTION OF ROCK BURST-PRONE AREAS

Researchers agree that rock bursts are caused by stress buildups due to overburden thickness, properties of strata in the near vicinity of openings, geologic conditions, design, and sequencing. Coates (1981) presented a list of ground control principles for underground excavation planning and emphasized the effect of opening geometries and sequencing on burst production.

Determining whether an underground structure is burst prone is simple: If the structure has experienced violent failures or seismic events, it is said to be burst prone, although this may not seem very scientific, many factors affecting bursts join together to cause a burst. Just because one opening is as deep as the next one, which may have a burst problem, does not mean that the first opening is burst prone.

The simplest course of action in determining burst-prone areas is to observe and document events surrounding bursts that have already occurred. While it is nearly impossible to categorize all events prior to a burst, simple physical occurrences could have been the cause. For instance, do bursts occur when excavating in a certain direction? Are the bursts related to the magnitude of the blast and size of shots? Are the bursts occurring in a special or unique geologic zone? Does drilling seem different prior to the burst? There are a number of manmade or man-controlled occurrences that should be recorded to possibly prevent future bursts. Visual confirmation can be sought by observation of the unit support deformation with time, together with localized rock failure characteristics such as violence and fragment shape and size.

From a more scientific standpoint, several methods are currently being researched as possible detection methods of burst-prone conditions. Many research efforts have been directed towards detecting burst-prone areas before excavation. State-of-the-art equipment and technology have been utilized to the fullest extent. Ground deformation monitoring using simple closure meters or survey techniques with stress meters can also indicate stress buildup.

The available detection methods are not completely reliable nor ready for application in an area that is rapidly excavated. The following discussion

includes brief descriptions of several methods such as microgravity, photoelastic, and on-site detection device methods. The most promising and most widely used method, the microseismic method, will be discussed in greater detail.

7-3.1 Microgravity Method

The microgravity method (Fajkiewicz, 1983) uses rock mass deformations, changes in gravity intensity, and changes in density distribution to evaluate burst-prone areas. Stresses in rock mass disturbed by underground mining will redistribute and reach a new equilibrium. The resulting density distribution is a function of a change in rock mass volumetric strain. Strain energy released in the form of a burst is approximately equal to the change in gravity intensity.

Changes in density distribution occur with postmining rock mass deformations and produce measurable changes in gravity microanomalies. This appears in four stages of brittle fracture of rocks in multiaxial compression: crack closure, fracture initiation, critical energy, and maximum deformation (rupture).

Rock bursts may be predicted depending on the gravity anomaly sign recorded; a negative gravity anomaly occurs shortly before a burst, while a positive gravity anomaly indicates a burst is not likely to occur. A negative gravity anomaly is caused by an increase in volume and a decrease in rock density.

7-3.2 Photoelastic Method

When certain plastics and optical glasses are subjected to stress and viewed under polarized light, the patterns of interference become visible and can be related to applied rock stress intensity and direction. Two photoelastic techniques were developed at the University of Sheffield to predict impending rock bursts (Grabis et al., 1976):

1. The disk technique is fairly inexpensive and does not require extensive expertise. Plastic material is bonded to the rock under load and placed on the surface of an existing excavation or at the flattened end of a borehole. The disk technique is best suited for igneous rocks with good elastic properties under high stress.

2. The prestressed meter technique uses a device, containing an appropriate glass plug, placed inside a borehole and prestressed against two opposite points on the hole wall by jacking platens operating through wedges. The stress magnitude is given in terms of an interference fringe pattern, which can be seen under polarized light. The fringe pattern is measured with a special mechanism calibrated in fringe units and gives stress magnitudes. The prestressed meter technique is limited by the depth of the borehole due to viewing limitations.

7-3.3 On-Site Burst Detection Device

The on-site burst detection device can be attached to an underground mine surface and provides a signal of the stress concentrations in the surrounding rock to indicate if a burst may occur. The principle behind this device comes from the fact that rock under stress produces a current varying greatly with rock type. Current is recorded for a 24-hour period, and the detector automatically resets if the stresses remain below a certain level.

7-3.4 Microseismic Method

The microseismic method, also known as the subaudible sounding or acoustic emissions (AE) method, is a geophysical approach that detects subaudible rock noise (microseisms) associated with yielding of the rock. The microseismic method is used to detect high-stress zones and potentially unstable areas and is based on experimental evidence that the rock undergoes small-scale displacements that result in the release of seismic and sometimes acoustic energy (Jackson, 1984).

Rock testing indicates increased noise rate is associated with increased stress, the rate increase becoming pronounced as the ultimate stress is approached. Noise-rate curves prior to failure in different geological material often exhibit similar slopes even though the material and failure mechanisms are different (Blake et al., 1974). A pattern of increased activity followed by a dramatic seismicity decrease period immediately prior to a burst in underground excavations has been noted by several researchers (Brady, 1978; Leighton, 1982). Laboratory studies have also shown that similar seismic events precede specimen failure.

Changing stress conditions that contribute to rock bursts also cause rock noises, which can be detected and analyzed using microseismic techniques. The source location of individual rock noise can be accurately located in three-dimensional space.

Figure 7-7 shows a schematic of a typical microseismic system. The geophones, attached to the walls of the opening, translate the rock noise from acoustic waves, which may be undetectable by the unaided ear, to electrical signals. The electric signals are then analyzed to determine their source location.

7-4 ROCK-BURST PREVENTION

High stress is the common denominator in the burst problem. Causes of high stress can be traced to numerous factors that present a very complicated problem. Burst prevention may be achieved by proper planning and opening design, sometimes including an active stress relief program incorporated during excavation. As discussed earlier, the excavation sequencing, geometry and

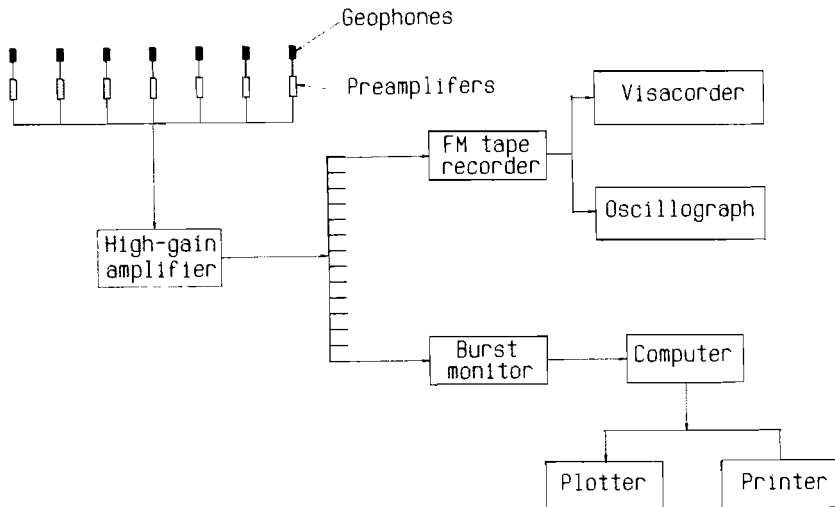


Fig. 7-7. Schematic of a microseismic system.

number of entries, geology, and physical properties should all be included in the design to minimize advancing through stress concentration areas which may be burst prone (Haramy et al., 1987). Additional support may be required, such as extensive bolting, hoist rope lacing, and mechanisms that were effective in South African gold mines to hold the fragments of fractured rock in place (Orthlepp et al., 1983). Orthlepp shows different patterns and examples of the use of mesh and lacing in conjunction with rock bolts and grouted cables and steel rods to stabilize tunnels in deep gold mines. These supports were effective for reinforcing tunnels following seismic events of magnitude exceeding 4.0 on the Richter scale. If the planning and design are not successful in controlling bursts, a destressing method should be used.

7-4.1 Destressing Methods

There are two theories related to destressing:

(i) Reducing stress concentration by intentionally fracturing the rock. As a result, the load is transferred to another part of the mine structure and the destressed area will not burst violently. The theory is simple, but controlling the extent of the fracture and rate of load transfer is not always feasible.

(ii) Inducing rock burst. There is a direct correlation between stope blasting and seismic event frequency. Triggering an increased number of bursts during excavation, by blasting all shot holes in as short an interval as

possible, and where feasible, firing all stopes simultaneously, resulted in reducing bursting activity on shift (Willan et al., 1985). Triggering was related to the cumulative effect of the blast vibration on an already highly stressed rock mass in a state of unstable equilibrium.

Some of the most widely used destressing techniques are-

(i) Volley firing: In this method explosions are used to fracture a previously located, highly stressed zone to a certain depth before excavation. The method is used before opening development to advance the abutment zone away from the active working face. Depending on the location and magnitude of stress, different hole patterns are used.

(ii) Auger drilling: In this method, stress relief is induced by drilling large-diameter holes into highly stressed area. A hole or series of holes in the rock will structurally weaken the rock and cause failure at a reduced stress level; stress buildup cannot then occur. The maximum possible borehole diameter depends on the sensitivity of the rock or area being drilled. Violent occurrences during drilling require the use of smaller diameter holes.

(iii) Hydraulic fracturing: This method uses the injection of fluid under pressure to overcome compressive stresses and cause material failure by creating fractures or fracture systems in a porous medium. This method is time consuming and is not recommended during the excavation process. Also, fracturing a very large and highly stressed area causes loads to be redistributed and may create burst conditions elsewhere.

In general, incorrect use of stress relief methods may sometimes increase the potential for rock bursts. Full understanding of each method, the mine stress concentration areas, and where the stress will be distributed as a result of destressing is essential.

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Chapter 8

UNDERGROUND STRUCTURES THROUGH SEISMIC ZONES

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8-1 INTRODUCTION

The objective of this chapter is to provide a brief summary on the seismic characteristics, effects of ground motion on tunnels and other underground structures, seismic design, and analyses of underground openings.

8-2 SEISMIC CHARACTERISTICS

Typically, earthquakes are classified in accordance with modes of generation as tectonic, volcanic, collapse, or explosion. Tectonic earthquakes are associated with relative displacement along new or existing faults. Volcanic earthquakes are produced by volcanic eruptions. Collapse earthquakes are associated with such events as landslides, collapse of roof or caverns, or rockbursts. Explosion earthquakes are associated with detonation of chemical or nuclear devices.

The seismic criteria discussed in the following sections deal mainly with tectonic activities. Other types of earthquakes can be quantified to obtain ground motion in similar manner. To evaluate seismic characteristics for design of underground structures, the following seismic input criteria are used: (i) the size of earthquake, (ii) the intensity and the frequency content of ground motion, and (iii) the duration of the strong motion.

8-2.1 Size of earthquake

Earthquake magnitudes are used to represent size of earthquakes. Several magnitude scales are used to represent the size. The most common magnitude scales are: the local magnitude (M_L), the surface wave magnitude (M_S), the body wave magnitude (M_B), and the moment magnitude (M_W). The definition and application of each type of magnitude scale are summarized in table 8-1. The relative values of magnitude scales are exhibited in table 8-2. The magnitude can be correlated with the energy released by the earthquake, fault rupture length, area affected by the strong motion, and the maximum fault displacement. The relationship between fault rupture length (L) and magnitude (M) can be

TABLE 8-1

Definition and application of magnitude (Housner and Jennings, 1982).

| Magnitude | Definition | Application |
|------------------------|---|--|
| Local (M_L) | Logarithm of peak amplitude (in microns) measured on Wood-Anderson seismograph at distance of 100 km from source and on firm ground. | Used to represent size of moderate earthquake. More closely related to damaging ground motion than other magnitude scales. |
| Surface Wave (M_S) | Logarithm of maximum amplitude of surface waves with 20-second period. | Used to represent size of large earthquakes. |
| Body Wave (M_B) | Logarithm of maximum amplitude of P-waves with 1-second period. | Used to represent size of large, deep-focus earthquakes which do not generate strong surface waves. |
| Moment (M_W) | Based on total elastic strain-energy released by fault rupture, which is related to seismic moment M_0 , where $M_0 = G \cdot A \cdot D$; G = modulus of rigidity (rock) A = area of fault rupture surface D = average fault displacement | Used to distinguish between two very large events of different fault lengths. |

TABLE 8-2

Relative values of magnitude scales.

| Magnitude | M_1 | M_2 | M_3 | M_4 | M_5 | M_6 |
|-----------|-------|-------|-------|-------|-------|-------|
| M_W | 5.0 | 6.0 | 7.0 | 8.0 | 9.0 | 10.0 |
| M_L | 5.2 | 6.1 | 6.8 | 6.9 | 6.9 | 6.9 |
| M_S | 4.5 | 6.0 | 7.3 | 7.9 | 8.2 | 8.3 |
| M_B | 5.4 | 6.3 | 7.3 | 7.6 | 7.9 | 7.9 |

expressed as suggested by Mark (1977):

$$\text{Log } (L) = 1.915 + 0.389M$$

8-1

where L is the average fault rupture length in meters.

8-2.2 Intensity and frequency content of ground motion

The intensity of ground motion can be expressed by qualitative and quantitative measures. Qualitative measures are based upon observed effects of

the earthquake motion on people, structures and their contents. The various intensity scales are Modified Mercalli, Rossi-Forel and Arias. Quantitative measures correspond to parameters obtained directly from ground motion time histories. Peak acceleration, peak velocity, and spectrum intensity are some of the parameters that have been used in estimating design earthquakes. To measure the frequency content of the ground motion, a frequency spectrum is needed. Two types of frequency spectrum commonly used are response spectrum and Fourier amplitude spectrum. Response spectrum is defined as a relationship between the maximum response of a single-degree-of-freedom oscillator and its frequency at different damping factors. The response is usually plotted in a logarithmic tripartite form as shown in figure 8-1. Fourier amplitude spectrum is associated with the amplitude of the relative velocity for an undamped single-degree-of-freedom oscillator at the end of the record and its frequency.

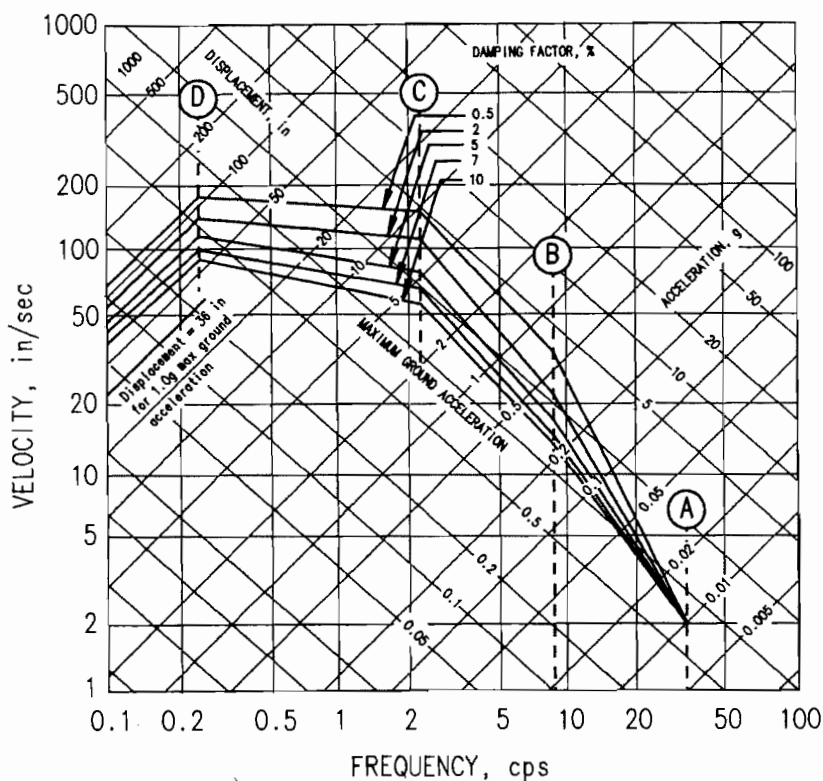


Fig. 8-1. Response spectra - Scaled to 1 g horizontal ground acceleration.

8-2.3 Duration of the strong motion

The duration of the strong motion effects the influence of earthquake motion on the response of the structure. There is no single widely accepted approach for quantifying the duration of a strong motion for a given ground motion record.

8-3 EFFECT OF GROUND MOTION ON UNDERGROUND STRUCTURES

The effect of earthquakes on underground structures may be broadly grouped into three classes -- faulting, ground failure, and shaking.

(i) Faulting includes direct primary shearing displacements of bedrock which is generally limited to relatively narrow seismically active fault zones. Sliding along a geologic fault introduces stresses that may be significantly higher than the magnitude induced by shaking. It is not practical to design an underground structure to restrain major displacement in the order of several centimeters to meters. It is more feasible to avoid sensitive areas or to accept the displacements and to localize the damage and to provide means to accommodate repairs. These features typically consist of either an excavation of an oversize section through the fault zone and use of a flexible support system or incorporation of a flexible coupling if the tunnel is lined.

(ii) Ground failure - Damage caused by ground failure may be associated with rock or soil slides, liquefaction, soil subsidence and other effects of ground motion. This type of damage can be mitigated by careful site investigation in the vicinity of tunnel portals and other shallow excavation areas. The site investigations required to determine potential for liquefaction and simplified methods to quantify factor of safety against liquefaction is summarized in the section 8-4.

(iii) Shaking - Damage due to shaking for lined tunnels may include spalling, cracking or failure of the liner. Shaking may also reduce shear strengths of the soil and rock mass above the tunnel and subsequently tunnel support system may have to withstand additional loads. For unlined tunnels, such vibrating motion may cause block motion, spalling, rock fall, or local opening of joints. The response of an underground structure to shaking will be influenced by the shape, depth of excavation, the properties of soil and rock mass around the opening, and the intensity of ground motion. Based on data compiled by Dowding and Rozen (1978), no damage should be experienced by an underground structure in rock if the particle velocity due to ground motion is below 20 cm/sec. Similar relationship is not available for underground structures in soil.

8-4 LIQUEFACTION OF SOILS

8-4.1 Definition

Liquefaction is a phenomena which includes the loss of shearing resistance or development of excessive strains under monotonic or cyclic loading.

8-4.2 Geological and geotechnical observations

Earthquake-induced instabilities of cohesionless soils occur most frequently in geologically recent deposits. Tsuchida (1970) proposed the grain-size distribution boundary curves as shown on figure 8-2 to identify soils that may liquefy. Seed et al. (1976) showed that it is unlikely that soil with $D_{20} > 0.7$ mm would ever develop a condition of initial liquefaction, provided there are no overlying or intervening layers of low permeability to impede drainage, where D_{20} = grain size in mm, which is 20 percent finer by weight.

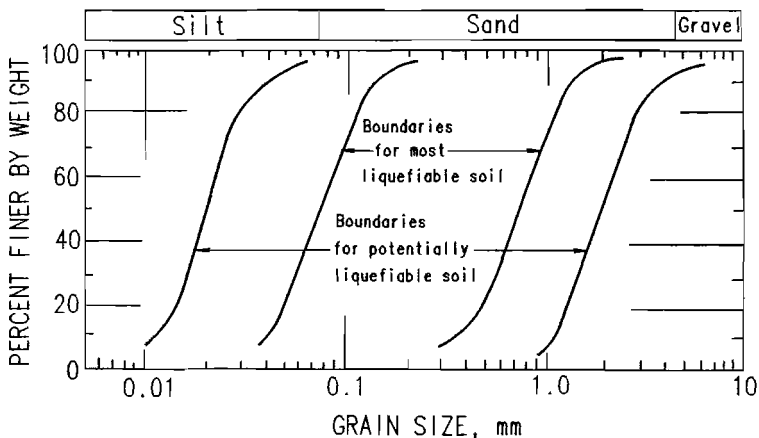


Fig. 8-2. Qualitative evaluation of liquefaction by grain size (Tsuchida, 1970).

8-4.3 Simplified analysis

Figure 8-3 exhibits the correlationship of modified blow count with stress ratio at sites with silty sands with a magnitude (M_L) of 7.5. τ_{av} is the average peak shear stress and σ'_0 is the initial vertical effective stress.

Modified blow count is the corrected blow count which would be measured at an effective overburden stress of 1 ton/ft². Modified blow count is represented as:

$$(N_1)_{60} = C_N ER_m N_m / 60$$

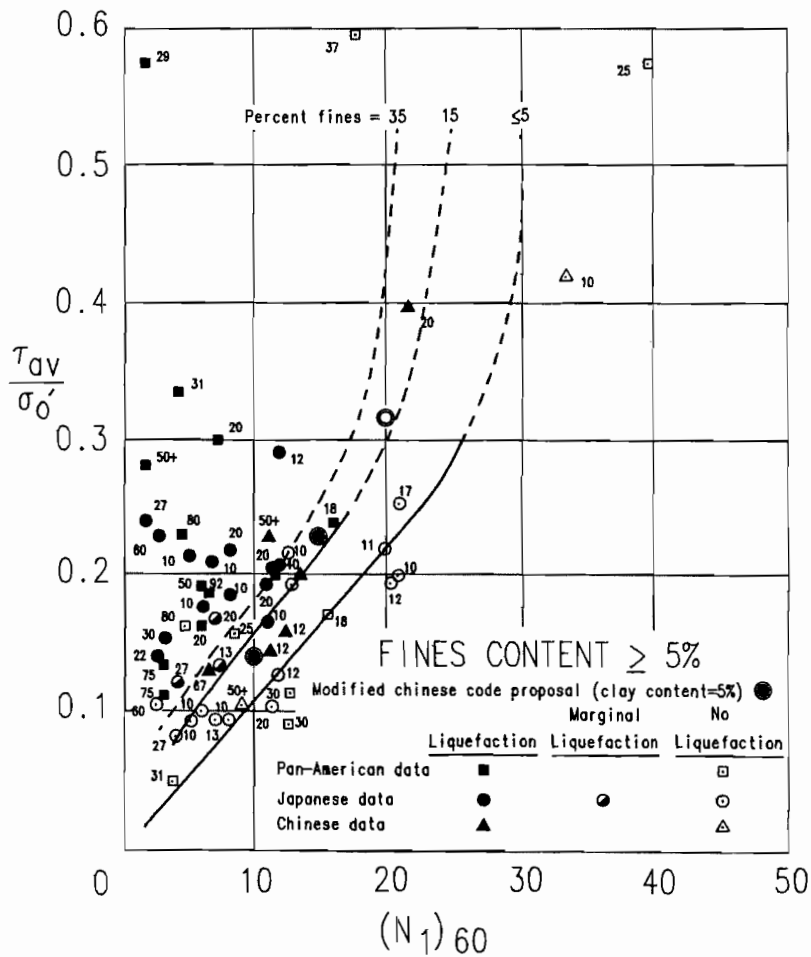


Fig. 8-3. Relationship between modified blow count and stress ratio at sites with silty sands with a magnitude (M_L) of 7.5 (Seed et al., 1984).

where N_m = measured blow count

ER_m = corresponding energy ratio in percent (see table 8-3)

C_N = overburden correction factor (see figure 8-4)

$$= \left(\frac{1}{\sigma'_0}\right)^{1/2}$$

σ'_0 = effective overburden stress in tons/ft²

TABLE 8-3

Summary of energy ratios for SPT Procedures (Seed et al., 1984).

| Country | Hammer type | Hammer release | Estimated rod energy, ER_m (percent) | Correction factor for 60 percent rod energy |
|---------------|-------------|--|--|---|
| Argentina | Donut | Rope & pulley | 45 | 45/60 = 0.75 |
| China | Donut | Free-fall | 60 | 60/60 = 1.00 |
| | Donut | Rope & pulley | 50 | 50/60 = 0.83 |
| Japan | Donut | Free-fall | 78 | 78/60 = 1.30 |
| | Donut | Rope & pulley with special throw release | 67 | 67/60 = 1.12 |
| United States | Safety | Rope & pulley | 60 | 60/60 = 1.00 |
| | Donut | Rope & pulley | 45 | 45/60 = 0.75 |

Stress ratio induced by a ground motion with the maximum acceleration of a_{max} at ground surface can be expressed as follows:

$$\tau_{av}/\sigma'_0 = 0.65 a_{max} \sigma_0 r_d / \sigma'_0 g$$

8-3

σ_0 = total overburden stress

r_d = stress reduction factor with depth

= 1.0 at ground surface

0.9 at a depth of 35 ft

Factor of safety against liquefaction can be computed as a ratio of stress ratio based on modified blow count and the induced stress ratio by ground motion. A factor of safety of greater than 1.35 is considered safe.

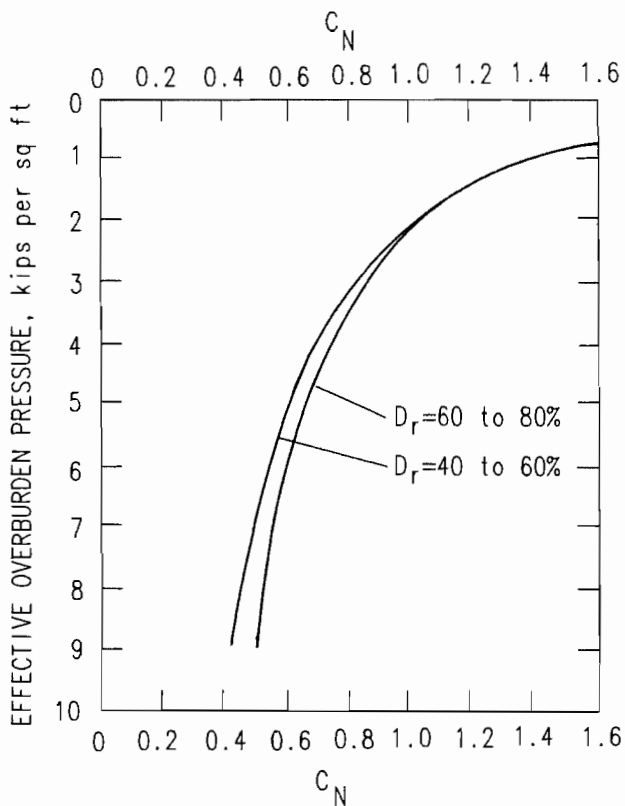


Fig. 8-4. Relationship between overburden correction factor (C_N) and effective overburden stress (Seed et al., 1984).

8-5 SEISMIC DESIGN OF UNDERGROUND STRUCTURES

Earthquake engineering of underground structures mitigates the possible damage from two principal sources, namely ground motion and fault rupture. The design approach is dependent upon the type of structure and the medium in which the structure is built. The discussion of analysis methods and design is divided into sections on underground structures in soil and in rock. Damage is unavoidable if an underground structure crosses a fault that slips; however, certain design features may help reduce the damaging effects of fault movement.

8-5.1 Underground structures in soil

Lateral earth pressures on underground structures increase during seismic events. The Mononobe-Okabe (Seed and Whitman, 1970) theory of dynamic soil pressure can be used to compute the active earth pressure during an earthquake as follows:

$$P_{AE} = 1/2 \gamma_s H_w^2 (1 - K_v) \cdot K_{AE} \quad 8-4$$

$$\text{where } K_{AE} = \frac{\cos^2(\phi - \theta - \psi_w)}{\cos \theta \cos^2 \psi_w \cos(\delta + \psi_w + \theta) E^2}$$

$$\theta = \tan^{-1} \frac{K_h}{1 - K_v}$$

$$E = 1 + \left\{ \frac{\sin(\phi + \delta) \sin(\phi - \theta - \psi_q)}{\cos(\delta + \psi_w + \theta) \cos(\psi_q - \psi_w)} \right\}^{1/2}$$

γ_s = unit weight of soil

H_w = the vertical dimension of the structure

ϕ = angle of friction of soil

δ = angle of friction of structural material with soil

ψ_q = slope of ground surface behind the vertical dimension of the structure

ψ_w = slope of the tunnel wall to vertical

K_h = horizontal design ground acceleration (in g's)

K_v = vertical design ground acceleration (in g's)

Mononobe and Okabe theory described by Seed and Whitman (1970) considered the total pressure P_{AE} computed by equation 8-4 would act on the wall at the height of $H_w/3$ above the base. However, the vertical walls are restrained at the top and bottom in the underground structure. Therefore, it is more appropriate to apply the additional lateral earth pressure at midheight, so as to distribute it uniformly over the depth of the structure.

Seed and Whitman (1970) proposed that the increment of dynamic pressure ΔP_{AE} above the static pressure could be approximated by

$$\Delta P_{AE} = \frac{1}{2} \gamma_s H_w^2 \cdot \frac{3}{4} K_h \quad 8-5$$

Using the equation 8-5, the design based on static loading, excluding seismic effect, can probably withstand ground acceleration up to about 0.25 g if allowable stresses are increased by one-third. A two-dimensional finite element model, subjected to an input seismic motion can be used to calculate stresses in tunnel. The sectional forces can be evaluated using computer models of elastic beams for the structure and lumped masses and springs for the soil medium.

8-5.2 Special considerations in design

(i) Rock intrusions. The underground structures in soil mass should not be cast directly against the rock. Kuesel (1969) suggests at least a 2-ft (0.6-m) over-excavation filled with soil or aggregate backfill.

(ii) Abrupt changes in cross section. Discontinuities in the structure will be subjected to differential rotation and translation due to the difference in the structural stiffnesses at those changes in cross-sections. The joints at the section of changes in cross section should be designed to accommodate the differential deformations.

(iii) Corner reinforcements for rectangular tunnels and culverts. For seismic considerations, the reinforcements on the inside faces of the tunnel should be extended into the top and bottom slabs and hooked at the far faces as shown in figure 8-5.

(iv) Abrupt soil-rock interfaces. If an underground structure is excavated through an abrupt interface between large soil and rock masses, a flexible joint should be provided to accommodate relative motion.

(v) Excess pore pressures. Excess pore pressures may develop a potential for uplift of the structure. Gravel drains with proper filter criteria around the structure will permit excess pore pressures to dissipate, thereby reducing the uplift.

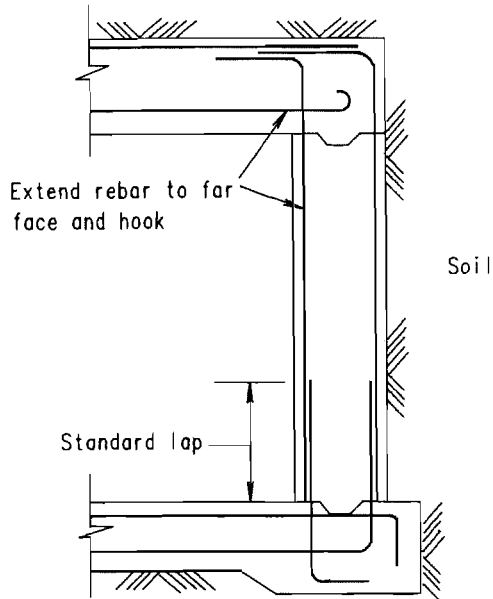


Fig. 8-5. Corner reinforcement for rectangular tunnels in soil.

8-5.3 Underground structures in rock

Underground structures in rock are often quite different from those in soil. In general, competent rock permits larger spans of openings and may need little or no lining for stabilization. Stress computations for dynamic loading should consider seismic wave propagation normal to the tunnel axis and wave propagation parallel to the tunnel axis. The wave propagation normal to the axis results in dynamic stress concentrations in the circumferential stresses around the cavity. The analysis can be modeled as a two-dimensional plane strain problem that can be treated by classical methods for circular cavities and by finite element or finite difference methods for noncircular cavities. Waves propagating parallel to the tunnel axis result in axial and curvature deformation, which may cause opening of joints, possibly leading to rock fall. A three-dimensional model is required to analyze stresses for seismic waves propagating parallel to the axis.

8-6 ANALYSIS OF UNDERGROUND STRUCTURES

The response of underground structures may be described by axial, curvature, and hoop deformations (figure 8-6). Axial deformations are represented by alternating regions of compressive and tensile strains that travel as a wave

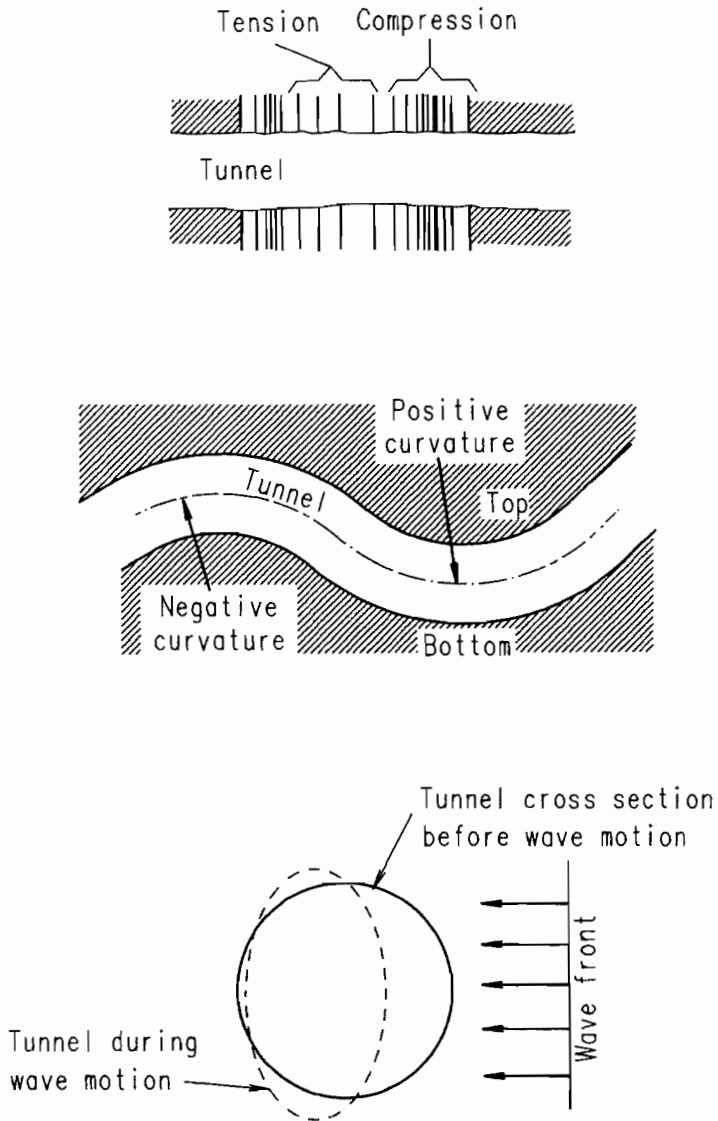


Fig. 8-6. Axial, curvature, and hoop deformations of tunnel.

train along the axis. Curvature deformations produce alternate regions of negative and positive curvatures propagating along the tunnel. For positive curvatures, the rigid liner will be in compression on the top and in tension on the bottom. For the tunnel with flexible lining or with no lining, the tunnel in positive curvatures experiences tensile strains on the top and compressive strains on the bottom. Hoop deformations are caused by the wave fronts propagating normal or nearly normal to the tunnel axis. The effects of deformations include hoop stress concentrations and the entrapment and circulation of seismic wave energy around the tunnel which is possible only when wavelengths are less than the radius of the tunnel.

8-6.1 Axial and curvature deformations

The dynamic stresses due to seismic waves are superimposed upon the existing static state of stress in the rock and in the liner. Compressive stresses due to dynamic loading added to the static stresses may cause spalling along the tunnel circumference due to buckling. Tensile seismic stresses subtracted from the compressive static stresses may result in tensile stresses, which may open joints in rock, loosening of rock bolts and create a potential rock fall from the tunnel roof and walls.

A one-dimensional model can be used to determine axial and curvature deformations for submerged transportation tunnels, subway tunnels in soils, and steel or concrete pipes. Such structures can be considered as a structural beam and common structural analyses can be employed. The revised method of Kuribayashi et al. (1974) utilizes a velocity response spectrum for base rock, evaluated from observed strong motion accelerations.

Three-dimensional models are needed to analyze large tunnels and caverns for axial and curvature deformations. Computer programs such as ADINA (1981), NONSAP (Bathe et al., 1974) currently provide the basic tool for axial and curvature deformation analysis.

Free field stresses estimated by the following equations have been used to make a qualitative evaluation of stability in several studies.

$$\sigma_{\max} = \pm \rho V_p |v_{\text{peak}}| \quad 8-6$$

$$\tau_{\max} = \pm \rho V_s |v_{n,\text{peak}}| \quad 8-7$$

where σ_{\max} = maximum axial stress
 τ_{\max} = maximum shear stress
 ρ = density of the material
 V_p = P-wave velocity

- V_s = S-wave velocity
 v_{peak} = peak particle velocity in the direction of propagation
 $v_{n,\text{peak}}$ = peak particle velocity normal to the direction of propagation

8-6.2 Hoop deformations

The concentration in the hoop stresses due to deformation may be estimated from expressions for free-field stresses.

$$\sigma_{\text{max}} = \pm K_1 \rho v_p |v_{\text{peak}}| \quad 8-8$$

$$\tau_{\text{max}} = \pm K_2 \rho v_s |v_{n,\text{peak}}| \quad 8-9$$

where K_1 = the dynamic stress concentration factor for a P-wave (figure 8-7)

K_2 = the dynamic stress concentration factor for an S-wave (figure 8-8)

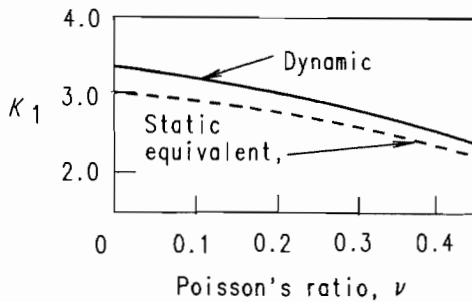


Fig. 8-7. Relationship between dynamic stress concentration factor K_1 for P-wave and Poisson's ratio.

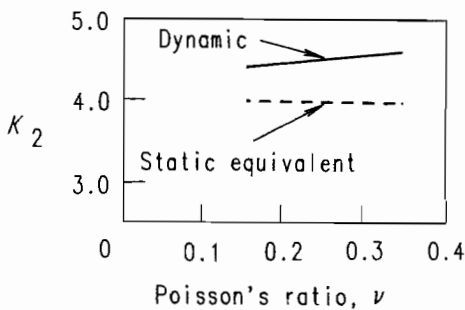


Fig. 8-8. Relationship between dynamic stress concentration factor K_2 for S-wave and Poisson's ratio.

These simple expressions to estimate peak dynamic stresses around an unlined cylindrical cavity can be extended to lined tunnels. Mow and Pao (1971) investigated the case of a P-wave incident upon an elastic liner of arbitrary thickness embedded in an elastic medium. The solution is dependent upon ratios of the P-wave velocities, shear moduli and the Poisson's ratio of the two materials, and the ratio of the outer and inner radii of the liner. The maximum dynamic stress concentration factor for the medium, K_m , and for the liner, K_l , is shown in figure 8-9 and 8-10, respectively. Poisson's ratio for both materials were set to 0.25 and the dimensionless parameters were defined by

$$\bar{\mu} = \frac{G_m}{G_l}$$

$$\bar{\alpha} = \frac{V_{Pm}}{V_{Pl}}$$

$$\bar{r} = b/a$$

where G_m = shear modulus of the medium

G_l = shear modulus of the liner

V_{Pm} = P-wave velocity through the medium

V_{Pl} = P-wave velocity through the liner

b = outer radius of the tunnel

a = inner radius of the tunnel

Mente and French (1964) present similar results for an S-wave incident on an elastic liner. Numerical models such as finite element and finite difference methods can be employed very efficiently for analyses of hoop deformations. These modeling procedures permit consideration of lined and unlined cavities, arbitrary cross-sectional shape, rock joints, nonhomogeneous material properties, and rock bolts, among others.

8-7 AVAILABLE NUMERICAL MODELS

The five approaches reported in the literature are briefly described. The approaches are: (1) the finite difference method, (2) the finite element method, (3) boundary integral method, (4) the method of characteristics, (5) the lumped parameter method. In finite difference method, the continuous derivatives with respect to the spatial variables are replaced by ratios of changes in the unknown variables over a small but finite spatial increment. Finite element models are discretized by dividing the body into an equivalent system of finite elements (see chapter 3). The solutions of equations of

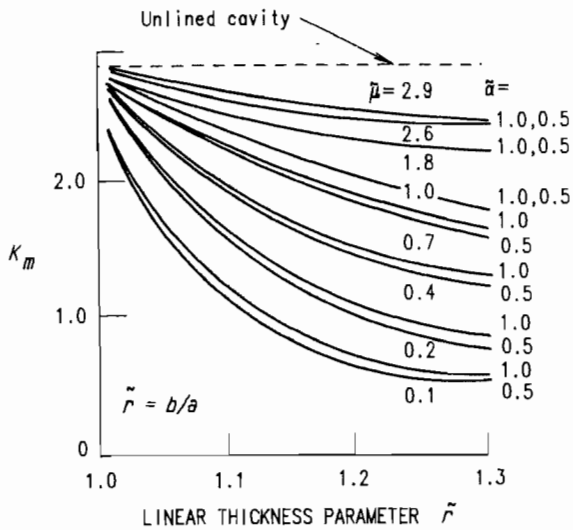


Fig. 8-9. Relationship between medium dynamic stress concentration factor K_m and liner thickness parameter \tilde{r} .

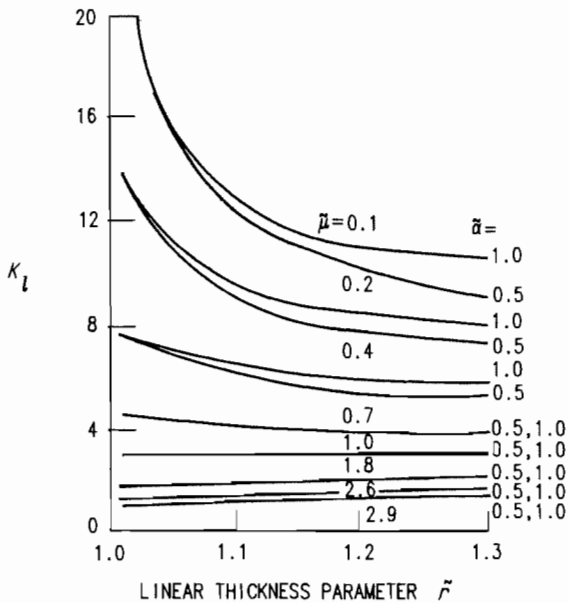


Fig. 8-10. Relationship between medium dynamic stress concentration factor K_l and liner thickness parameter \tilde{r} .

motion can be indirectly obtained in the frequency domain and then transformed into the time domain by employing the inverse Fourier transformation. In boundary integral methods, only the boundaries of the tunnel and underground cavern are represented by a finite number of segments. The method of characteristics (Desai and Christian, 1977) is employed by solving the wave propagation equation. The set of partial differential equations governing the propagation of waves in a medium is converted into a set of ordinary differential equations in time domain only, using paths of propagation. In lumped parameter models, the masses are physically lumped and are connected by springs and dashpots.

8-7.1 Computer programs for dynamic analysis

Many computer programs based on finite element method and wave equations are available that are well suited for analyses of underground structures. The following programs are widely used for the above application:

- (i) SHAKE
- (ii) FLUSH
- (iii) ADINA

(i) SHAKE. (Schnabel et al., 1972) - This computer program can be used to analyze the free-field response. The soil medium is modeled as a system of horizontal visco-elastic layers of infinite horizontal extent, and an equivalent linear model is used to represent the strain-dependent behavior of material properties of each soil layer. The medium can be subjected to input motion from vertically propagated shear waves. A continuum solution to the one-dimensional wave equation is employed. The solution is carried out in the frequency domain and is then transformed back into the time domain through the use of Fast Fourier Transformations.

(ii) FLUSH. (Lysmer et al., 1975) - This computer program can be used to compute the two-dimensional response of a soil-structure system. The soil medium is assumed to be comprised of a system of homogeneous visco-elastic horizontal infinite soil layers. An equivalent linear model is used to represent strain dependent material properties. The medium can be subjected to vertically incident shear waves. The soil-structure system can be modeled using either a conventional plane strain model or a modified two-dimensional model that attempts to simulate three-dimensional effects through the use of in-plane viscous dampers attached to each nodal point of the soil medium. The soil medium is bounded by a rigid base and by transmitting boundaries along the sides.

(iii) ADINA. (Adina Engineering, 1981) - This computer program is a general purpose finite element program for the two-dimensional and three-dimensional analysis. The library of constitutive models permits inclusion of linear and

nonlinear materials. The input can consist of horizontal and vertical motions. Several solution techniques are available, which include explicit and implicit formulations.

8-7.2 Recommended procedures

Although sophisticated analyses tools are available to investigate the dynamic response of underground structures to seismic loading, simple procedures of analysis should be employed as a starting point for any analysis of underground structure. If the results of the preliminary analyses suggest that special precautions will be required to assure satisfactory performance, more rigorous analyses may be justified.

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Chapter 9

SHOTCRETE FOR SUPPORT OF UNDERGROUND OPENINGS

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9-1 GENERAL

Shotcrete is "concrete shot from a fire hose." As used in underground work, experience and empirical rules dictate a total thickness of only two to six inches (usually four) (50 to 150 mm, usually 100 mm) of shotcrete, to support even very large openings. This is in sharp contrast to earlier North American designs of steel ribs for initial "temporary" support, followed by "final" cast-in-place concrete linings, usually at least 12 inches (300 mm) thick. Shotcrete obviously works, but the use of a thin and relatively fragile layer of shotcrete means that caution must be used by designers, and workmanship in the field is all-important.

Shotcrete originated with drill+blast work, but the use of modern Tunnel Boring Machines (TBM's) makes the task of good shotcrete design more complex. Steel-fiber-reinforced shotcrete (SFERS) with microsilica is recommended as sturdy, economical, and conservative.

9-2 INTRODUCTION

Shotcrete is concrete placed by shooting the cement, aggregates, water and various additives (accelerator, retarder, plasticizers, steel fibers, microsilica, etc.) through a hose using compressed air. Shotcrete was first developed in the early 1900's by Carl Akeley of the Smithsonian Institution, to spray on molds of animals in the museum. In 1915, the Allentown Cement Gun company bought Akeley's patent. In the 1930's, Rabcewicz began to use shotcrete for tunnel support in Iran and later in Europe, leading eventually to his concept and theories (with co-workers) of the New Austrian Tunnel Method (NATM). In North America, E.E. Mason used gunite (shotcrete) to seal a tunnel roof in 1957. Use of steel-fiber-reinforced shotcrete (SFERS) in tunnels was pioneered in North America, following research by Parker and others in the 1970's at the University of Illinois. The use of microsilica in shotcrete was pioneered in Scandinavia in the 1970's, and SFERS with microsilica has been used in a number of underground openings in North America in the 1980's (Rose, 1985).

The term "gunite" was used by Allentown. More recently, the term "gunite" has come to refer to mixes with small-sized aggregate (less than 1/4") and the term "shotcrete" refers to larger-sized aggregate mixes. The distinction is unimportant.

All early shotcrete performed used the "Dry Mix" method, where dry materials were BLOWN by compressed air through the delivery hose, with water added to the mix at the last possible moment, at the shotcrete nozzle. In recent decades, reliable "Wet Mix" equipment has become available where a conventional wet concrete mix is PUMPED through the hose to the nozzle, where compressed air and accelerator is added. Many contractors tend to prefer the Wet Mix method because of less dust, less rebound, and higher production capability. However, the Wet Mix method requires a reliable source of Wet Mix concrete to be delivered to the site, whereas Dry Mix shotcrete can be made up in small batches and applied promptly to the area of distress, whenever and wherever needed. Both methods satisfy designers' requirements.

European tunnel shotcrete practice, developed from drill+blast excavation, has strongly influenced all shotcrete designers. European practice typically includes a wire mesh installed after the first 2-in (50-mm) layer of shotcrete is placed. Following installation of the mesh, or Welded Wire Fabric (WWF), the second 2-in (50-mm) layer of shotcrete is placed. North American designers consider the installation of this wire mesh, or WWF, to be awkward, time consuming and expensive. Further, the WWF will vibrate when the shotcrete second layer is applied, and a weak lamination is typically found within the shotcrete at the WWF location (King, 1980).

The use of steel fibers mixed throughout the shotcrete to produce a steel-fiber-reinforced shotcrete (SFERS) was pioneered in the USA. Work by Parker (1975), Henager (1981), Rose et al. (1981), Morgan (1984) and others, led to the use of SFERS in several tunnels in the USA and Canada. At present, the world's longest tunnel supported solely by SFERS is the 9450-linear-foot Stanford Linear Collider (SLC) tunnel in California (Rose, 1986).

9-3 ROCK LOADS

9-3.1 Terzaghi, Barton, and Bieniawski

We know that very thin layers of shotcrete can support even very large underground openings, but we are not always sure how they do it. The design of shotcrete is still largely based on empirical European practice. It is helpful to briefly discuss the several theories of rock loads, in an attempt to understand how such thin shotcrete linings can accomplish effective support where heavy steel ribs and cast-in-place (CIP) concrete linings were previously used.

Karl Terzaghi (1946), Dick Bieniawski (Bieniawski, 1973) and Nick Barton

(Barton et al., 1974) (see chapter 2), among others, created rock classification systems which attempt to predict the rock load which may act on tunnel supports. Each is based on the geologist's description of the condition of the rock to be encountered. Terzaghi's system is much simpler than Barton's or Bieniawski's system. The other systems are more complex, which gives an illusion of increasing sophistication.

Allowing for a reasonable range of differences in the rock and/or the geologist's judgement and description, Terzaghi and Barton and Bieniawski all give roughly comparable rock loads (Einstein et al., 1979; Rose et al., 1981a; Rose, 1982). Only the Terzaghi system will be discussed here, but the conclusions apply to all systems.

The Terzaghi classification has dominated North American design for many years, is familiar to all senior workers in the field, and is still used. The Terzaghi rock loads are based on Terzaghi's field observation of old tunnels. Where no support was provided, the roof rock gradually fell out over a large number of years to certain heights and configuration, depending upon the type of rock. Terzaghi's method was to design steel rib support to hold up a rock load equivalent to all of the rock which could ever loosen or fall out of the roof and wall over a long period of time, thus finally exerting a dead load on the steel rib. Theoretically, over a very long period of time the full rock load might be exerted. Very few measurements exist, however, which indicate that in real life the full rock loads are mobilized against the steel ribs. It is now considered that if the rock is supported promptly, only a small part of the full Terzaghi rock load will ever develop. Therefore, steel ribs are not needed, and shotcrete (placed promptly) will suffice to hold the rock.

9-3.2 Voegele and Goodman Computer Models

Voegele (1978) in 2-D, and Goodman and Shi (1981) in generalized 3-D have greatly clarified our understanding of real rock behavior. Their work does not calculate actual rock loads, but using computer graphics they independently show how the mass of rock attempts to be self-supporting, due to interlocking of blocks and pieces. If certain key pieces are held safely in place (by shotcrete, and/or rock bolts, etc.) the entire mass is held stable. Should minor unravelling occur, however, these key pieces may become undermined and eventually fall out. The result is instability of a large number of interlocked pieces, or a "rock fall" or "tunnel failure" when a key piece falls. Shotcrete prevents such minor unravelling, and inhibits failure of key blocks. Thanks to the computer graphics, it is easier to understand how a thin layer of shotcrete, often with rock bolts, can promote stability of large openings.

9-3.3 Tom Lang's Bucket and Hair Net

In the late 1950's Tom Lang (1957) pioneered work on rock bolts and shotcrete at the Snowy Mountain project in Australia, using photoelasticity, mathematical theory, and large boxes full of gravels in the laboratory. One of his most striking illustrations was to fill an ordinary 12- to 18-in water bucket with crushed rock (1/2- to 2-in sizes). A few 1/8-in rods were attached to the bottom of the bucket, and were surrounded by the gravel. An old-fashioned ladies' hair net was held in place over the gravel by washers and nuts attached to the rods. The nuts and washers on the rods were hand tightened, simulating rock bolts. Then the bucket was turned upside down, and it was seen that nothing fell out. The mass of gravel stayed intact inside the upside-down bucket.

Tom Lang then took a match and burned the hair net. In a moment, a few small pieces of gravel fell out. Then suddenly, as key blocks were rearranged and/or fell out, the entire mass failed and fell out of the bucket. In Lang's demonstration, the rods and washers simulated the rock bolts, and the ladies' hair net simulated the shotcrete.

9-3.4 Time Dependent Rock Loads

Ordinary loads begin to act on a tunnel only after a period of time. At the instant of first opening, of course, the tunnel is (at least temporarily) stable. By definition in such a case, no unsafe load yet exists. However, unless the rock is unusually excellent, if nothing is done and no support at all is provided, then the rock may gradually begin to readjust. Eventually, small pieces of rock may fall out of the roof and wall. Terzaghi-style loading of the tunnel will begin.

In the field, real failures of shotcrete have occurred when rock loads were rearranged over a period of time (often due to the action of clay and ground water) and rock interlocking subsequently became ineffective. The thin layers of shotcrete are rarely so regular, smooth and linear that they are actually acting as a structural member or arch, as this term is used by structural engineers. The lines of thrust within a shotcrete arch are often so irregular and skewed that structural engineers may calculate that such an arch cannot possibly hold up the predicted Terzaghi/Barton/Bieniawski rock load (see chapter 2).

However, the shotcrete can be understood to act as a thin membrane which presents unravelling and ensures that the rock mass itself remains intact. Finite Element Method (FEM) studies show the lines of thrust in such a case are actually passing through the rock mass itself, as much as one diameter behind the walls and crown, and heavy rock loads in very large openings can easily be carried on such thick rock "abutments".

The concept of "Standup Time" has been addressed by Laufer (1958) and Deere et al. (1969). This is an extremely difficult subject, and very little scientific work has been done on it. However, the point is that for a certain period of time, almost any type of ground is self-supporting. Following this certain period of "Standup Time", local unravelling and collapse begins. For our purposes, if shotcrete is applied before the "Standup Time" has elapsed, the process of unravelling and instability is considered halted. The tunnel has been stabilized and is safe, even though the shotcrete layer is typically very thin (only four inches or so).

9-4 CONSTRUCTION OF UNDERGROUND OPENINGS

Construction using shotcrete must be related to the method of excavation. The European NATM concepts often used in shotcrete design were developed from drill+blast tunnels. As discussed below, the North American tendency to strive for very rapid advance using machines to excavate means that shotcrete designers must not blindly rely on the old European concepts.

9-4.1 Construction Using Drill+Blast

European shotcrete practice evolved from observations in drill+blast highway and railroad tunnels. Drill+blast provides a cyclic operation: after the blast, the workmen allow the smoke to clear for perhaps twenty minutes, and then come in to bar down any loose rock. The shotcrete equipment can be brought directly to the face even before the muck pile is removed, and shotcrete workmen can stand on the muck pile if necessary. The rock could be washed down, and shotcrete placed in perhaps as little as an hour after the blast. For a 16-ft-diameter tunnel, for instance, the relatively short advance by drill+blast (say 12 feet per blast) would require only about 4 cubic yards (4 cubic meters) of shotcrete for the first 2-in (50-mm) layer on the walls and crown; this could be placed in another hour. The tunnel would clearly be stabilized promptly, before any significant loosening occurred.

The green shotcrete would have perhaps 4 to 6 hours to bond well to the rock (say 50 to 75 psi) (0.4 to 0.6 MPa) to survive the next blast vibrations, and hold up any potential "key blocks" and prevent ravelling. The second 2-in (50-mm) layer would be placed later, after the welded wire fabric (WWF) "reinforcement" is unrolled and tacked to the surface of the walls and crown.

In this cyclic drill+blast system, little variety is possible and specifications would be simple and easy to follow: shotcrete promptly after each blast.

9-4.2 Construction Using Roadheaders

In the 1940's, a new machine appeared in some kinds of underground work.

The Roadheader, usually track mounted, cuts the rock with a rotating cutter head spiked with replaceable cutters on a boom. Limited to softer rocks of perhaps 7,000 to 10,000 psi (48 to 70 MPa) compressive strength, Roadheaders were first used in mining soft coal beds. "Milling" type machines have the cutters rotating in line with the axis of the boom, throwing the rock cuttings sideways. "Ripping" type machines have the cutters rotating perpendicular to the boom, throwing the rock directly downwards. Thus, Ripping-type Roadheaders do not have a lateral reaction component that tends to "wobble" the Roadheader sideways, and they may be more efficient excavators in tunnels. Both types of Roadheader cut the ground continuously and carry the muck by conveyor belts to the rear of the machine for deposition in a muck pile, for subsequent removal by load/haul/dump units.

With Roadheaders, there is no fixed, predictable and convenient time to stop mining, move the machine out of the way, and bring in the shotcrete crew. On the other hand, even the new, large, 70-ton Roadheaders are only about 25 feet (8 m) long, and they can be moved back away from the face whenever the foreman or the operator decides to do so. The shotcrete can be brought up to the rear of the Roadheader and the men can haul the shotcrete hose around the Roadheader to the face without extreme difficulty. If the foreman is willing to halt production, back up the Roadheader and bring in the shotcrete crew, the shotcrete will be applied promptly, before the rock begins to loosen. Roadheaders excavate the rock more gently than does the drill+blast method, which is, of course, a great advantage. But it can be seen that intangibles (the job foreman's decisions and gambling instincts) have been introduced, which are almost out of the designer's control. Furthermore, as shown later, the contractor has an economic imperative to excavate as far as possible before shotcreting, which implies that the rock stands open longer, and loosening may occur.

9-4.3 Construction Using a TBM

In 1952, the first modern Tunnel Boring Machine (TBM) was built by James Robbins for F.K. Mitty, to excavate a 25-ft-diameter tunnel in the Pierre Shale at Oahe Dam in South Dakota. Since then, more than 130 TBM's have been built by the Robbins Company alone, and a number of other companies worldwide also manufacture TBM's. These TBM's typically fill the entire tunnel and rotate a circular cutter head in the face, thus excavating the rock with numerous cutters mounted concentrically on the cutter head. Very hard rock can now be continuously mined with very little disturbance. TBM's mine the ground so gently that in some instances even very soft zones in badly faulted ground have been successfully excavated by hard rock TBM's (for instance, the Spirit Lake Tunnel at Mount St. Helens, built by Peter Kiewit Construction Co. of

Omaha, Nebraska. This tunnel, designed by the U.S. Corps of Engineers, Portland, Oregon, was one of the first to use steel-fiber-reinforced shotcrete with a TBM).

However, in modern TBM's the "trailing gear" of belts and muck cars makes the entire TBM several hundred feet long, and to date very little success has been achieved in finding a way to shotcrete near the face with a TBM. It is not possible to back the TBM up; shotcreting can be done only after the TBM has continued forward out of the way. In most TBM designs, especially those of the Robbins Company, it is essentially impossible to provide a convenient space or "window" in the TBM for shotcreting. However, for a Melbourne, Australia, project, Tony Peach of Boretch of Solon, Ohio, did successfully modify a Jarva TBM to provide a small shotcrete compartment directly behind the cutter head. This is to date apparently the only truly successful shotcrete job with a TBM. All other projects have had to wait for several days until the TBM and all its trailing gear have passed and exposed the rock to the shotcrete crew. Shotcreting is done several hundred feet behind the face at best, and designers must be aware of this fact.

9-4.4 Construction Problems

In a drill+blast tunnel, the normal shape is a horseshoe tunnel. Shotcrete is normally required on the crown and walls only. A Roadheader tunnel could be any shape; again, normally, shotcrete would be on the walls and crown only. Shotcreting in the invert to "close the ring" would interfere with normal traffic loads and shotcrete is not normally placed across the invert. A TBM tunnel is always circular, and designers sometimes seem to have an absent-minded tendency to show the circular tunnel completely shotcreted all around. Such placement of the shotcrete in the invert is impractical. Any shotcrete in the invert will suffer considerable traffic distress; the designer should think this problem out carefully. If the designer insists on "closing the ring" on a circular TBM tunnel, or even a horseshoe Roadheader or drill+blast tunnel, precast concrete segments in the invert should be used.

In drill+blast work, the designer could reasonably expect that shotcrete would be placed every 4 to 6 hours, after the tunnel had advanced a short distance (12 feet or so). The European theories of shotcrete evolved from such drill+blast methods.

In North America, high labor costs have led to the development and use of highly mechanized methods of tunnel construction, using Roadheaders and TBM's. These machines excavate the rock gently, no doubt greatly increasing the "Standup Time" to significantly more than drill+blast standup time, as well as minimizing the rock load. However, with modern tunnelling machines, the application of shotcrete (or any other support) may not take place until after

some 10 to 20 feet (Roadheader) or even 200 feet of advance (for a TBM) has taken place, i.e., conceivably up to several days later. Designers and contractors may wish to provide extra strength in the shotcrete, for instance by using microsilica, to account for the delays and uncertainties brought about by modern machine tunneling.

9-5 SHOTCRETE DESIGN

9-5.1 Time Dependent Properties of Shotcrete

As discussed earlier, ordinary underground openings have a certain "Standup Time" and will remain stable for some time before rock loads brought about by unravelling and loosening (or squeezing or swelling) occur. The application of shotcrete to prevent unravelling, and to stabilize the "key blocks" should be prompt. Most properties of shotcrete are time-dependent.

At the instant of application, of course, the shotcrete is a liquid. To adhere directly overhead, and hold its own weight against gravity, a 2-in-thick shotcrete layer must have a bond with the rock of about 25 PSF, or 0.17 psi. In the field, it is commonly found that overhead layers more than 2 inches thick are likely to fall off during initial placement (with the exception of Dry Mix microsilica shotcrete, discussed later). Thus, the commonly specified 2-in-thick layer is, in fact, an empirical thickness, which is the practical maximum that can be placed with conventional shotcrete without excessive falloff.

For shotcrete, modern specifications require that the initial set of the cement plus accelerator must occur in about 1 to 2 minutes and the final set in about 12 to 20 minutes, so that in a short time, the shotcrete is quite firm to the touch. In about 8 hours the bond will increase to about 75 psi on clean, rough rock, and in 24 hours the bond will reach its maximum value to several hundred psi.

Simple calculations will show that shotcrete with about 75-psi bond over the crown of an underground opening has sufficient bond to hold a "key block" of any reasonable size in place, with a large safety factor, even if conditions should be so dramatically changed that the key block is suddenly without side friction or any other support (which it must have had a few moments previously in order to stay in place at all). For this reason, shotcrete on clean, rough rock with good bond can be placed in the crown only; it will stay in place in the crown and needs no assistance from wall shotcrete to help carry vertical roof loads. As discussed later, however, shotcrete on wet or soft ground with poor bond should always be extended all the way down the wall and founded firmly on the tunnel invert, to act as a structural arch and thus avoid falloff due to bond failure.

9-5.2 Rebound

The compressed air provided to the shotcrete hose (which is usually about 2-in diameter) may be several hundred CFM, and the exit velocity of the air from the nozzle can be 350 to 500 ft/sec. The lighter particles of cement, steel fibers, (if used) and fine sand are carried at higher velocities than the heavier aggregate particles. The heavy slugs of shotcrete will exit the nozzle at about 115 ft/sec, fly one or two meters and hit the rock wall at about 105 ft/sec (about 75 mph), according to careful measurements by Opsahl (1985).

The initial impact of the new shotcrete on the tunnel wall results in a high percentage (about 30-50%) of the heavier material rebounding off the wall and on to the floor. Parker et al. (1975) made a methodical study of rebound, and calls this stage "Phase I". Parker found that in a short time, however, a thin layer (about 0.4 inches) of cement paste forms on the wall, and subsequent quantities of shotcrete bury itself in this paste and rebound is much reduced (to 5-15%); Parker calls this "Phase II". Placing shotcrete in two layers means that the high-rebound Phase I stage takes place twice. The total rebound lost with two 2-in-thick layers of shotcrete is about twice that which would occur with a single 4-in layer (i.e., only one limited Phase I stage). However, as noted earlier, layers thicker than 2 inches (except for Dry Mix microsilica shotcrete) tend to fall off during placement.

Rebound is increased when Welded Wire Fabric (WWF) is used, as discussed later.

9-5.3 Mix Design

In most respects, shotcrete mix design is essentially the same as concrete mix design. At the Atlanta Research Chamber (Rose et al., 1981) the conventional shotcrete, for each cubic yard, had 660-lb cement (7 sacks); 1,790-lb fine aggregate (sand, to No. 4 screen); and 1,300-lb coarse aggregate (gravel to 1/2 in); i.e. the sand:gravel ratio was 60:40. The same mix was used when steel fibers were added (115 lb/CY). At different projects the proportions have been different, and the sand:gravel ratio may reach 70:30. Generally, the mix design is the contractor's responsibility, as long as certain design parameters are achieved.

Three to six percent (by dry cement weight) accelerator is commonly added to the shotcrete mix. For jobs located at remote sites or with special requirements, an attempt may be made to heat or cool the mix, add retarders, plasticizers and/or other additives, which may affect the behavior of the shotcrete significantly.

When steel fibers are specified, even the best and toughest fibers do no good if less than about 80 lb/CY are used. Most types of deformed steel fiber require 100-125 lb/CY to be effective. Special tests (Rose, 1985; Morgan,

1984) should be specified to ensure satisfactory results using steel fiber reinforcement.

When microsilica is specified, no less than 8 percent of the cement dry weight should be used. Since some microsilica is sold in liquid form with water added, care must be taken that the water is not incorrectly included in the percent; inquire from the manufacturer for the correct dry weight when liquid microsilica is used.

9-5.4 Layer Thickness

Experience has led to a virtually uniform rule that each shotcrete layer should be 2 inches (50 mm) thick, with a total thickness varying from 2 to 6 inches (50 to 150 mm), depending upon the circumstances. Four inches total thickness is common, even for very large openings. Designers who specify shotcrete thickness greater than 8 to 12 inches (200 to 300 mm) will find that cast-in-place concrete probably would have been cheaper.

9-5.5 Welded Wire Fabric (WWF)

European practice includes a 6 by 6- or 8 by 8-in wire mesh, or welded wire fabric (WWF), placed between the shotcrete layers for "reinforcement". This WWF is commonly installed after the first 2-in (50-mm) shotcrete layer, by pinning the WWF to the tunnel roof and walls with short rock bolts at about 4-ft (1.2 m) centers. Many North American designers note that the WWF will sag, that extra shotcrete will be required to fill irregularities behind the WWF, and that the impact of shotcrete on the WWF causes vibrations which create a plane of weakness or even actual voids in the shotcrete (King and Pease, 1980). Inspection shows that WWF "reinforcement" at best is located near the neutral axis, and under load, the WWF "reinforcement" does little work. As discussed later, the use of steel fibers mixed throughout the shotcrete addresses these problems.

9-5.6 Shotcrete in Soft Ground; Squeezing or Swelling Ground

If bond is sufficient, as for example on clean, rough rock, shotcrete can be placed in the crown only. However, in wet or soft ground, the bond can fail, or sometimes the ground immediately behind the shotcrete is so soft and/or wet that the ground will fail in tension, pull out and fall, leading to voids or major shotcrete falloff. In such ground it is imperative to shotcrete from the invert up, so that the shotcrete shell can act as a free-standing arch if the bond fails. The tunnel invert should be protected from excessive traffic which could erode and undermine the footing of the shotcrete arch in such wet and/or soft ground. Shotcrete falloff of tens of feet along the tunnel has been reported in such conditions (King and Pease, 1980; Rose, 1985).

In squeezing or swelling ground, shotcrete may be particularly suitable because it will exhibit distress in pertinent areas, yield slowly, and permit repair with further layers of shotcrete. In some cases it may be considered desirable to "close the ring" with a thick layer of shotcrete in the invert. Such invert shotcrete will, of course, harden rapidly and can tolerate modest traffic loads, but may require some protection in early hours. It may be practical to place additional invert shotcrete (or poured concrete with accelerator) on the weekends. The interference of invert shotcrete with construction activity is real, however, and must be acknowledged (financially) by the engineer and owner.

9-5.7 Shotcrete Incompatibility with Timber Lagging

Shotcrete is compatible with steel ribs, or European-style "lattice girders," where the shotcrete encases these steel supports. This is because with some effort, good workmanlike nozzlemen can completely encase steel ribs or lattice girders in shotcrete without leaving voids.

Timber lagging, however, is totally incompatible with shotcrete. Many designers and specification writers and inspectors overlook this fact, and timber lagging has sometimes been wrongly and/or inadvertently permitted in some tunnels. Timber lagging placed with steel ribs or lattice girders forms a "rat's nest" of wood, located out of sight behind the shotcrete. Shotcrete placed over this "rat's nest" of miscellaneous timber is merely concealing these substantial voids from view. The shotcrete may not be in contact with the ground at all. The action of time and groundwater may loosen the ground in these areas, and new ground loads of uncertain magnitude and direction could develop on the thin shotcrete lining. On some projects, using a low pressure grout (about 5 psi), attempts have been made to fill the voids around such timber lagging after the shotcrete has covered it from view. Clearly, a balance must be achieved between popping loose adjacent good shotcrete by excessive grout pressure, versus failing to fill the numerous voids. In general, it is simplest and best to remove and replace such void-filled unsatisfactory shotcrete.

In ground where, for some reason, lagging (other than the shotcrete itself) is required, openwork steel plate such as the Bernold sheets, or steel liner plate, should be specified. Timber can rot away. The use of timber with shotcrete is always unacceptable.

9-5.8 Basic Design Procedure

Figures 9-1(a) and (b) show shotcrete linings for tunnels. Figure 9-1(a) shows a simplified bigger triangular rock block being supported by the shotcrete lining than that of figure 9-1(b). Rock wedge sizes are usually

determined by the geometrical characteristics of the discontinuities present in the surrounding rock mass (see chapter 2). Joint sets 1 and 2 in figure 9-1 are assumed to have shear strengths and irregularities on their surface that are not sufficient to keep wedge ABC from displacing into the tunnel. In this case, the displacement reduces the normal force, P_N , along the joints, increasing the load to be supported by the liner. A limiting load condition, assuming firm rock surrounding the tunnel, will be that in which the total weight of the moving wedge, w , is acting on the support liner.

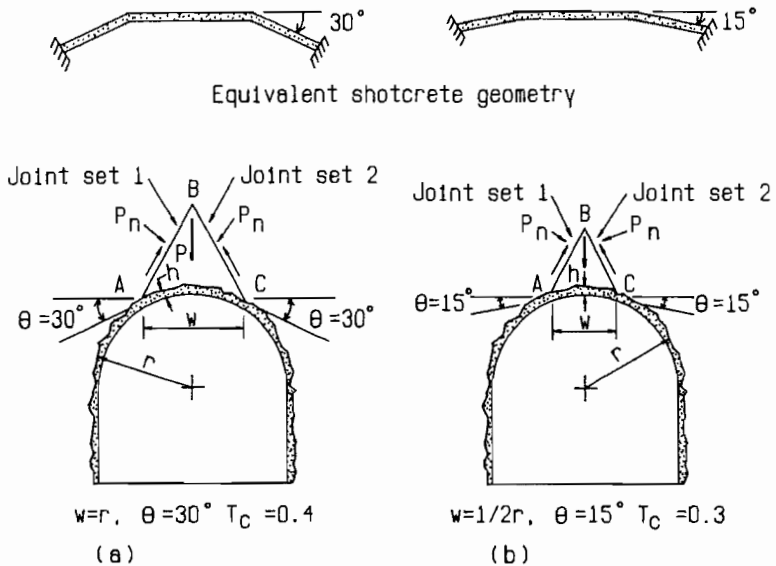


Fig. 9-1. Shotcrete Supporting Rock Wedges.

To support the rock wedge, the thickness of required shotcrete lining in inches is given by equation 9-1 (Fernandez-Delgado et al., 1981).

$$h = P / [2T_C(\sin\theta)(f'_{c28} \cdot L)]$$

9-1

where T_C , θ are defined in figure 9-1

P = weight of rock block in pounds

f'_{c28} = the 28-day unconfined compressive strength in psi of the shotcrete mix cylinder

L = the length in inches of the rock block perpendicular to the plane of figure 9-1.

When rock bolts are used in conjunction with shotcrete, the rock bolts are designed to carry the full rock wedge load with a factor of safety of not less than 2 (see chapter 4). In that case, the shotcrete is used as a nonstructural member.

When the shotcrete forms a closed ring of circular or horseshoe shape, it should be designed as an indeterminate structural member using methods of analysis described in chapter 5.

In general, shotcrete designers use intuition and judgement and do not rely heavily on calculations because such calculations are only as good as their basic assumptions. Design assumptions, especially those involving the rock load, are necessary to leave a paper trail for those interested in such trails. However, any designer who comes up with calculations requiring much more than two 2-in-thick layers of well-constructed shotcrete in an ordinary tunnel, had best do his calculations over again until he gets them right.

9-6 STEEL-FIBER-REINFORCED SHOTCRETE + MICROSILICA

9-6.1 Steel-Fiber-Reinforced Shotcrete (SFERS)

Shotcrete/concrete is a brittle material, and early shotcrete designs followed the European practice of including Welded Wire Fabric (WWF) between the two 2-in shotcrete layers for "steel reinforcement," and/or "to hold the shotcrete together after cracking, if any occurs, and keep the pieces from falling out." Neither of these ideas is correct.

Experience has shown that the WWF "reinforcement" is located at the neutral axis and does little work in resisting bending until long after the lower layer of shotcrete has cracked. Further, experience shows that vibration of the WWF during shotcreting typically causes poor bond between the two shotcrete layers and with the WWF. Thus, if bending or cracking of the shotcrete does occur, the poorly bonded lower layer quickly falls off the WWF, exposing it to view. People sometimes see the WWF exposed and think it is bravely "holding up the shotcrete," when actually it has caused such poor bond that the lower layer has failed and fallen off wholesale as a result! If the WWF performed well, one would never see it.

Field placement of the WWF is labor intensive (i.e., expensive), and the WWF drapes loosely over irregularities in the rock or shotcrete surface, so that shotcreting behind draped WWF requires added quantities of material. Thus, the designer who specifies WWF has inadvertently specified voids and a more

expensive product.

In the 1970's, steel-fiber-reinforced shotcrete (SFERS) began to be used in North America for underground support (Rose, 1985). While early steel fibers were simply straight, at the present time a number of improved, deformed fibers of various shapes are commercially available.

The actual amount of steel in a 6 by 6 WWF placed between two 2-in layers of shotcrete is about 2 lb/SY or about 20 lb/CY. Modern specifications for steel-fiber-reinforced shotcrete (SFERS) call for 80 to 120 lb/CY, i.e., 4 to 6 times as much steel as with WWF. The steel fibers are distributed evenly throughout the two 2-inch layers (see X-ray photographs in Parker et al., 1975) and act to inhibit cracking. During shooting, the air carrying the shotcrete and fibers turns ninety degrees when it hits the wall, and serves to turn and orient the fibers with a large percentage parallel to the wall, i.e., oriented in the most favorable direction. Large scale tests by Little (1979) and Opsahl (1985) and others show that using deformed steel fibers, the post-crack behavior of SFERS is superior to shotcrete with WWF.

Costs bid for SFERS appear to be comparable to, or lower than, costs bid for WWF. In 1983, at the Stanford Linear Collider tunnel, bidders could choose either WWF or SFERS; the two low bidders and three of the lowest four bidders chose SFERS (Rose, 1986), even though at that time most bidders had no experience in using SFERS. WWF is expensive: for the 18-ft Atlanta subway tunnels in good rock, in 1978, WWF alone was bid at \$5/SY, i.e., about \$25.55 per LF of tunnel, or say \$2.75/lb steel WWF furnished and installed. Steel fibers at 100 lb/CY, at that time costing \$0.45/lb, would have been \$25.56 per LF tunnel, with negligible labor costs. Furthermore, less shotcrete volume would be needed with SFERS compared to the quantities placed behind the draped WWF. A limited length of the Atlanta subway used SFERS, but political problems prevented use of SFERS throughout. It appears that with SFERS, for about the same or less cost, a superior shotcrete is produced, with a large amount of steel in the shotcrete and good bond between the layers.

9-6.2 Microsilica

As discussed earlier, a number of factors in modern tunneling with Roadheaders and TBM's sometimes conspire to make it impossible to shotcrete promptly. Certainly, contractors will attempt to shotcrete only when absolutely necessary, to avoid interrupting the excavation work. Loosening of the rock can occur, thus increasing ground load. The addition, microsilica adds enormously to the strength of shotcrete, and is recommended for use with SFERS.

Microsilica, or "silica fume," is composed of amorphous silica some 50,000 times smaller than cement grains. The microsilica not only fills voids between

cement grains, but also enters into limited chemical reactions with calcium hydroxide (the weakest link in the chain of shotcrete ingredients). Compressive strengths of 10,000 psi (70 MPa) or more are easily obtained by adding 8 to 10 percent microsilica by dry cement weight. Other properties are proportionately improved.

Microsilica takes up a large quantity of the free water in the shotcrete. This will reduce the apparent slump of Wet Mix microsilica shotcrete to such an extent that dismay is sometimes expressed in the field. To counteract this water demand, plasticizers are often added. Wet Mix microsilica shotcrete behaves generally the same as Wet Mix conventional shotcrete with respect to rebound and layer thickness. However, in Dry Mix microsilica shotcrete, where the water is added to the mix at the last split second at the nozzle, the applied shotcrete is enormously "sticky". Dry Mix microsilica shotcrete will adhere even to smooth or damp surfaces. Overhead shotcreting with Dry Mix microsilica shotcrete has been done to 12-in-thick layers, all in one pass. Thus, while engineers applaud the improved properties of microsilica shotcrete, contractors may save significant costs by reducing rebound.

9-7 PRACTICAL SHOTCRETING

9-7.1 Preconstruction Testing

Experience on major projects has shown that it is impossible to get good quality shotcrete without specifying careful preconstruction testing in the laboratory and field. For instance, on one major project some years ago, many tens of thousands of dollars were lost by the contractor because the shotcrete could not meet requirements, due to undetected incompatibilities between the cement and additives. Detailed discussions on practical shotcrete specifications are part of the Atlanta Research Chamber report (see Mason and Lorig, 1981).

9-7.2 Shotcrete Equipment

The cement, aggregate, and additives (accelerators, retarders, plasticizers, etc.) may be brought from the cement plant to the site in various ways, including ReadyMix trucks. This material must then be brought down into the tunnel using agitator cars, ReadyMix trucks, pipe, hose or other means. Delivery to the shotcrete crew must be made in a short time, so that unanticipated hardening or other chemical reaction of the mix does not occur, especially with Wet Mix and/or microsilica shotcrete.

(i) Batchers. Volumetric batchers are popular, but commonly cannot be kept calibrated in the field. Hence the proportions of the mix actually placed in the field may vary wildly. If volumetric batchers are used, specifications and field staff should insist on daily calibration with known weights of materials,

a nuisance in the field but a necessary requirement.

(ii) Air compressors. Shotcrete is simply concrete placed using compressed air, and conventional air compressors are used. As is well known, in many cases these compressed air rigs deliver air at variable rates due to age, wear and tear. On almost all shotcrete jobs, there is no gage on the compressed air rig to show actual air pressure and quantity of air pumped. This introduces a variable into all shotcrete work which is difficult to evaluate and, to date, impossible to avoid.

(iii) Liquid accelerator. On Wet Mix projects, liquid accelerator is often pumped from a barrel to the nozzle, where the accelerator is added to the shotcrete just a few instants prior to placement. In many cases these pumps have no gage to show the quantity of accelerator pumped and therefore added to the mix. Frequently, the only measurement is with a stick dipped into the barrel to estimate the total quantity of accelerator pumped out during the whole day (and later compared to the cubic yards of shotcrete placed that same day). This introduces another variable.

(iv) Shotcrete pump. The concrete/shotcrete pump is a vital part of the Wet Mix shotcrete equipment, and quality pumps which can move the materials hundreds of feet will cost several tens of thousands of dollars. Shotcrete pumps have a capacity of up to 20 CY/hr or more; actual placement of shotcrete is usually much less than this. Many pumps, unfortunately, have only one cylinder, so that the shotcrete is forced into the delivery hose in "surges" as the single cylinder fills with shotcrete and then empties. Some models have a nitrogen chamber designed to reduce the "surge" in the line. Newer and more expensive model concrete pumps, such as the Sidewinder, have two cylinders and a mechanism which swivels from one cylinder to the other, to deliver the shotcrete more uniformly. However, even the Sidewinder-type system results in a short but definite moment when there is no shotcrete pumped into the hose. Other ingredients (especially accelerators) are being added with other equipment at other rates. These surges and pulses can be felt easily by the nozzleeman, who must decide when the resulting variability in proportions is significant and/or unacceptable.

It is easy to appreciate that if the shotcrete materials "pulse" and vary from zero to 100 percent through the hose, while other materials such as water and/or accelerator are fed in at a constant rate from other sources, the water/cement ratio and the percent of accelerator will also vary, and the properties of the resulting shotcrete will vary. This is a disconcerting thought and, fortunately, the act of spraying on the shotcrete helps to even out the product. Testing of cores taken from in-place shotcrete is obviously necessary.

(v) Nozzle. The shotcrete nozzle varies in detail with different

manufacturers, but is commonly about 18 to 24 inches long. The nozzle is clamped to the hose, and other lines attached to the nozzle introduce compressed air and accelerator (in Wet Mix shotcrete), or water to the dry materials (in Dry Mix). The movement of the shotcrete mix and its aggregates (and sometimes steel fibers) through the nozzle erodes the end of the nozzle and enlarges it, changing the shape of the nozzle spray pattern. After a certain length of time, the nozzle is so enlarged that it must be replaced. On one tunnel project, a 2- to 3-ft length of delivery hose was used without any nozzle at all (King and Pease, 1980). Many modern nozzles are plastic, and are replaced every 40 CY or so.

(vi) Summary. To summarize: the proportions of the shotcrete mix ingredients, the air compressors, accelerator pumps, concrete pumps, hose and nozzles, all have certain variations quite impossible to monitor. Surges inadvertently caused by the equipment and/or obstructions in the lines may make local zones in the applied shotcrete heavy or deficient in water, accelerator, cement and materials and/or velocity of application. For this reason, intelligent and dedicated workmen, especially at the nozzle, are absolutely essential for successful shotcrete work.

Fortunately, shotcrete is apparently a "forgiving" material, even with this disconcerting and unscientific variation in percentage of ingredients. Careful measurement has shown that good shotcrete can be produced by good workmen even with very low velocity of application: as low as 30 to 40 ft/sec as opposed to the more normal 100 ft/sec or so (Opsahl, 1985). Experienced workmen and inspectors, good field control and a serious field testing program are necessary (see Mason and Lorig, 1981).

9-7.3 The Shotcrete Crew

After excavation has taken place, and all loose material has been barred down and the tunnel walls cleaned with a jet of compressed air and/or water, the tunnel is ready for shotcreting. If necessary, scaffolding or mobile platforms to raise the workmen up to the crown of the tunnel have been assembled (although many shotcrete crew members sometimes seem to prefer to perch hazardously atop various pieces of nearby equipment). Shotcreting should start at the invert, because shooting from the top down would result in a large pile of undesirable rebound waste material accumulating at the invert, making good invert footing impossible. Shotcrete is placed by the nozzlemen holding the hose with the nozzle about 3 to 4 feet from the surface to be shotcreted. Overhead shooting usually requires more accelerator, and sometimes the nozzle is inadvertently a greater distance from the crown than it is from the walls. Overhead shooting also involves greater rebound hazards, and consequent nozzleman discomfort. The effect of gravity on the stream of shotcrete, and on

the green in-place layers just beginning to bond to the crown, added to these other factors, make crown shotcrete typically less dense and strong than wall shotcrete.

The nozzleman is the key to the success of the whole job. He must carry the heavy, wriggling hose, control the placement of the shotcrete, adjust the amount of water (in Dry Mix), or accelerator (in Wet Mix) at the nozzle as needed, while shooting evenly (usually in small circular motions) at the same time. The rest of the crew is employed in assisting the nozzleman, cleaning up the rebound and moving lights, air/water lines and equipment.

Specialty shotcrete contractors (swimming pools, retaining walls, etc.) may use as few as three men on a shotcrete crew, and under good conditions may place up to 20 CY/hr, or as much as 120 CY/day. In tunnel work, the whole tunnel mining crew stops work while shotcreting is going on; and shotcreting is done by these same relatively inexperienced miners. The work is intermittent and the shotcrete equipment must be set up and torn down again and thoroughly cleaned several times a day. About 30 CY/shift would be typical good production on tunnel work, i.e., about 4 CY/hr.

9-7.4 Shotcrete in Soft Ground

For soft ground, the designer is liable to specify the same shotcrete layer thickness and other properties as for hard rock, but field procedures are different. Allowing soft ground to stand open for a long time may promote drying out at the surface, creating a loose and unstable layer which will be difficult to shotcrete. It must be assumed that "Standup Time" in soft ground is limited, so specifications must provide that excavation be halted intermittently and shotcrete placed often (for instance, every four hours) as appropriate. Probably, cleaning with a water jet would simply erode the soft ground, and cleaning with an air jet is more suitable and practical.

In soft ground, the shotcrete **MUST** be placed from the invert up, so that the hardened shotcrete can act as a free-standing arch if bond to the soft ground fails or if the ground itself fails in tension just behind the shotcrete (King and Pease, 1980; Rose, 1985). The soft ground must not be asked to "hold up the shotcrete" by shooting the crown but not the walls, as is often done in hard rock. The invert footing must be sound, and probably the footing should be somewhat enlarged in soft ground. Traffic must not be allowed to disturb or destroy the footing. The face should be kept shotcreted during shutdowns to prevent sloughing.

A few contractors spray a thin coat of sodium silicate on soft ground, which covers the ground with a strong coating to which shotcrete can easily adhere. Designers should consider specifying this coating, thus relieving the contractor of the cost of this desirable step.

It may be advisable to specify Dry Mix microsilica shotcrete for very soft and/or wet ground. Dry Mix microsilica shotcrete has such unusually "sticky" properties that it may solve problems of obtaining rapid support in poor ground conditions. Dry Mix microsilica shotcrete is so "sticky" it can be shot directly overhead to 12 inch thickness.

9-8 CASE HISTORY

The case history briefly described here is for a 16-ft-diameter Wet Mix tunnel excavated with a Roadheader. The rock at the station considered in this case history was a locally wet, badly broken sandstone, although the remainder of the tunnel was usually in good rock. Mucking out was done with one CY capacity load/haul/dump (LHD) units.

9-8.1 Excavation with Roadheader

The Roadheader operator is located about 20 feet behind the Roadheader cutter head, and hence the operator is protected from roof fallout by the previous day's shotcrete for up to 20 feet of new excavation. In soft ground, or in broken rock which can be plucked away easily, Roadheaders can excavate 12 to 15 CY/hr.

In this case, when the foreman decided shotcreting was prudent, usually after about 10 LF advance in the badly broken rock, the track-mounted, self-propelled Roadheader was halted and backed up a few tens of feet. The tunnel was then cleaned of muck and debris using hand labor and LHD units, and then air blasted. Lights and air lines were re-strung. This mucking and preparatory operation took about 45 minutes.

9-8.2 Shotcreting the first 2-inch layer

The Wet Mix was brought to the site by ReadyMix trucks, and dumped into an LHD loader with a one-cubic-yard capacity bucket, which brought the Wet Mix carefully inside the tunnel to the concrete pump, at about 3/4 CY per trip. The concrete pump was placed immediately behind the Roadheader; the LHD dumped the Wet Mix into the concrete pump hopper and the shotcreting started.

For the 10 LF excavated, the initial 2-in layer was shotcreted on both walls and crown. Including 20 percent waste and rebound losses, 3 CY shotcrete was required for 10 LF walls and crown. About 10 CY/hr was actually shot through the nozzle by the crew, but the preparatory work and final cleanup of all the shotcrete equipment added approximately another two hours to each shotcrete operation. The remainder of the day was spent in excavation.

Typical events for the FIRST shotcrete layer were as shown in Table 9-1, following.

TABLE 9-1
Shotcrete time study.

| Time | Activity: Excavate + First Shotcrete Layer |
|---|---|
| 7:30 a.m. | Start excavating with Roadheader. |
| 11:30 | Lunch. |
| 12:00 noon | Resume excavating with Roadheader. |
| 2:20 p.m. | Halt excavation after 10 LF, back up, clean up. |
| 3:08 | Cleanup complete, lights strung, shotcrete pump in. |
| 3:18 | Still waiting for ReadyMix truck to arrive. |
| 3:20 | ReadyMix truck arrives at portal with 4 CY. |
| 3:26 | LHD arrives at pump with 3/4 CY Wet Mix load. |
| 3:28 | Assemble nozzle, test lines, START SHOOTING #1. |
| 3:30 | END SHOOTING #1, smooth rough spots with trowel. |
| 3:34 | LHD arrives with 2nd load, START SHOOTING #2. |
| 3:38 | END SHOOTING #2, smooth rough spots with trowel. |
| 3:41 | LHD arrives with 3rd load, START SHOOTING #3. |
| 3:46 | END SHOOTING #3, trowel smooth, bring platform in. |
| 3:51 | Complete attaching platform to loader bucket. |
| 3:52 | Raise nozzleman up to crown, move lights up. |
| 3:53 | LHD arrives with 4th load, START SHOOTING #4. |
| 3:56 | END SHOOTING #4, trowel smooth, clean platform. |
| 4:00 | LHD arrives with 5th load, START SHOOTING #5. |
| 4:05 | END SHOOTING #5, trowel smooth, clean platform. |
| 4:06 | Roof shotcrete fallout 2-ft by 2-ft by 2-in thick. |
| 4:10 | START SHOOTING to repair fallout area. |
| 4:11 | END REPAIR SHOOTING. |
| 4:12 | Platform down, lights removed, all done shooting. |
| 5:28 | Complete cleanup of nozzle, lines, all equipment. |
| 5:30 | Go home. |
| SUMMARY: Excavate 10 LF (85 CY) = 7:30 a.m. - 2:20 p.m. = 380 minutes | |
| Clean out and prepare = 2:20 p.m. - 3:08 p.m. = 48 minutes | |
| First 2 in. + rebound = 3:08 p.m. - 4:12 p.m. = 64 minutes | |
| Cleanup lines & equip. = 4:12 p.m. - 5:28 p.m. = 76 minutes | |
| PRODUCTIVITY, FIRST LAYER: note 4 CY purchased but only 3.75 CY used | |
| A. Based on 2 inch thickness shown on drawings, i.e. pay quantity (Does NOT include rebound and falloff) | |
| = 2.5 CY / (64 + 76) minutes | |
| = 1.1 CY/hour | |
| B. Based on quantity passing through nozzle | |
| = 3.75 CY / (64 + 76) minutes | |
| = 1.6 CY/hour | |
| C. Based on actual shooting time only | |
| = 3.75 CY / (2+4+5+3+5+1) minutes | |
| = 3.75 CY / (20 minutes) | |
| = 11.25 CY/hour | |

9-8.3 Placing Welded Wire Fabric (WWF) in crown and walls

On this project, the designers had specified the old-fashioned WWF, and the contractor had bid \$10/SY for furnishing and installing the WWF. This generous price included a hefty profit unsuspected by the design engineer, and so the contractor did not propose any Value Engineering change in the design (to steel-fiber-reinforced shotcrete, for example). He just did the work as shown on the drawings.

After the first shotcrete layer was placed and hardened, the WWF was draped on walls and crown. Sags in the WWF were criticized by the unusually attentive inspector in the field, and after much discussion, the WWF was suspended about every 4 to 5 feet from studs. While the FIRST shotcrete layer followed the rock irregularities without conspicuous waste, the draped WWF could not follow the actual rock line and was hung loosely over recesses, and so required extra shotcrete. Vibration of the WWF and attempts to fill recesses behind the WWF caused extra rebound. Voids which were created were spotted by the inspector and he required their correction before their existence was simply covered over by subsequent shotcreting. The contractor's high price for WWF was in fact approached, due to the unusual expertise and diligence of the field inspector.

9-8.4 Shotcreting the SECOND 2-inch layer

On this job, with the concurrence of the experienced field inspector, the SECOND 2-in layer of shotcrete was placed much later, after the tunnel was holed through. For the second layer, preparatory and cleanup operations were required only once a day, but shotcreting was almost continuous all day. Up to 10 CY/hr were shotcreted, with 8 CY/hr not uncommon.

In areas where the rock was good, only the crown was shotcreted with the second 2-in layer. On this 16-ft-diameter tunnel, 10 CY of shotcrete covered about 56 LF of tunnel crown with the 2-in second layer. Moving lights, air/water lines and keeping up with the shotcrete nozzle man (who was advancing down the tunnel at 40 to 60 LF per hour) was a critical matter for the crew. Up to 80 CY could be placed in one 10-hr shift, i.e., one 2-in layer over the crown of up to 450 LF in a 16-ft tunnel. Except for operator fatigue (on this job, two nozzle men shared the work normally done by one), this second layer was highly efficient and profitable, as well as uniform and workmanlike.

9-9 COST STUDY

9-9.1 Assumptions and basic data

The cost study on the following pages assumes a 16-ft-diameter tunnel curved in plan with numerous side adits and alcoves, in sound but soft sandstone.

This complex configuration makes a Roadheader the best machine for the job. Dry Mix shotcrete with 100 lb/CY steel fibers and 10% microsilica were used, so that a single 4-in-thick shotcrete layer can be successfully placed in one pass (instead of two 2-in layers).

This estimate assumes the Roadheader excavates uniformly at 12 CY/hr and that shotcrete is placed uniformly at 4 CY/hr (pay shotcrete placed on the wall as shown on the drawings, i.e., not including rebound, falloff and waste). Shotcrete is purchased at \$50/CY; deformed steel fibers at 100 lb/CY @ \$0.70/lb = \$70/CY; and microsilica at 10% of cement dry weight = 66 lb/CY microsilica shipped from the mainland and delivered on site in Honolulu @ \$0.30/lb which adds another \$20/CY. The materials cost is thus \$140/CY. The direct labor, equipment and supplies cost (not shown here) is \$465/hour.

The simplified cost study assumes the same direct cost for all work done (i.e., labor + equipment + supplies, plus permanent materials) either excavating or shotcreting; does NOT take into account reasonable variations in productivity; and includes a flat 33% markup over direct cost for overhead and profit. It does not compare the relative merits and overall productivity of two 2-in layers versus one 4-in layer of shotcrete.

This parametric cost study shows that under these assumptions, if the Roadheader can only advance a short distance before shotcrete is required, the cost of EXCAVATION + SHOTCRETE is very high. Conversely, in good rock where the Roadheader can excavate many tens of feet before shotcrete is required, the contractor could produce a shotcreted tunnel for several hundred dollars per LF less.

The cost of time-related items (labor, equipment daily charges) far outweighs the cost of materials (shotcrete, fiber, microsilica), by a factor of five to eight. Therefore, the contractor will attempt to reduce his costs by completing the work faster, and by shotcreting as seldom as possible. This being the case, the designer should prudently specify better materials, such as steel-fiber-reinforced shotcrete with microsilica, to ensure a successful project.

The economic facts of life put great pressure on the contractor's foremen to advance the tunnel as far as possible before shotcreting. Designers should be aware of these economic pressures, and specifications should be fairly and realistically written. The use of steel-fiber-reinforced shotcrete with microsilica will help satisfy both the contractor's desire for rapid excavation and infrequent shotcrete placement, as well as the designer's concerns about stability.

TABLE 9-2

Parametric cost estimate.

| | |
|--|--------------|
| 1. Excavate (X) LF of tunnel with Roadheader | |
| Quantities: 16-ft-diameter tunnel | = 8.5 CY/LF |
| Productivity: Roadheader (70 ton) | = 12.0 CY/hr |
| *2. Stop excavating; muck out, clean up and prepare | = 1.0 hour |
| 3. Shotcrete place on (X) LF of tunnel | |
| Quantities: 4-in shotcrete walls and crown | = 0.5 CY/LF |
| Add 20% rebound and losses | = 0.1 CY/LF |
| | 0.6 CY/LF |
| Productivity: = 4 CY/hr pay shotcrete placed on the wall | |
| = 5 CY/hr through nozzle including rebound/waste | |
| *4. Clean up and move out | = 1.0 hour |
| 5. <u>Cost per LF completed tunnel:</u> for (*), refer to item 2 and 4 | |

| Dist. (X) | Exc. CY | Shot. CY | Exc. hr. | Shot. hr. | (*) | Tot. hrs. | L/E/S @\$465 | Mat'l @\$140 | L/E/S +mat'l | W/OH+ profit | \$/LF BID |
|--------------|------------|-------------|-------------|--------------|-----|--------------|-----------------|-----------------|-----------------|-----------------|--------------|
| 3.5 | 30 | 2.1 | 2.5 | 0.5 | 2.0 | 5.0 | \$2,325 | \$294 | \$2,619 | \$3,483 | 995 |
| 7.0 | 60 | 4.2 | 5.0 | 1.1 | 2.0 | 8.1 | 3,766 | 588 | 4,534 | 5,790 | 827 |
| 14.0 | 119 | 8.4 | 10.0 | 2.1 | 2.0 | 14.1 | 6,557 | 1,176 | 7,733 | 10,285 | 735 |
| 28.0 | 238 | 16.8 | 20.0 | 4.2 | 2.0 | 26.2 | 12,183 | 2,352 | 14,535 | 19,332 | 690 |
| 56.0 | 476 | 33.6 | 40.0 | 8.4 | 2.0 | 50.4 | 23,436 | 4,704 | 28,140 | 37,426 | 668 |

9-9.2 Summary

As can be seen from Table 9-2, if the rock is good enough (or the foreman bold enough) to excavate long distances before shotcreting, the tunnel cost is much reduced. Increasing the length excavated before halting in order to shotcrete, from 3.5 feet up to 7.0 feet, means a savings of \$165/LF to the contractor!

From the designer's point of view, short excavation advance and frequent shotcreting are desirable. However, if the designer seriously wants frequent halts in the work, he must expect costs to go up and tensions to exist in the field. Use of good materials (deformed steel fibers and microsilica) is a prudent solution to this age-old problem.

9-10 LIST OF ABBREVIATIONS

| | |
|----------------------|---|
| CIP = cast in place | PSF = pounds per square feet |
| CY = cubic yard | PSI = pounds per square inch |
| hr = hour | SFRS = steel-fiber-reinforced-shotcrete |
| LF = linear feet | TBM = tunnel boring machines |
| LHD = load/haul/dump | WWF = welded wire fabric |

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Chapter 10

WATER CONTROL

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10-1 INTRODUCTION

Groundwater control both during construction and in the completed tunnel is one of the most common, but most challenging problems faced by tunnel designers and constructors. To quote Schmidt (1977),

"More tunneling problems and hazards are associated with the appearance of groundwater than with any other single factor."

and Bartlett (1979),

"Subterranean water remains the tunneler's worst enemy."

This chapter summarizes current methods for both control of groundwater during construction of bored and cut-and-cover tunnels and for leakage control in completed structures. Subaqueous tunnels constructed by the sunken tube method are not discussed. Issues discussed include investigatory methods, description and selection criteria for water control methods, contractual considerations, and cost issues.

Tunnel excavation is uniquely susceptible to groundwater difficulties. In tunneling all activity is focused at the heading, and improperly controlled water will impair project schedule and can stop all forward progress because there is no alternate focus of construction activity while water problems are resolved.

The state-of-the-art for quantitative prediction of groundwater behavior is imprecise at best. Soil and rock transmissivities typically vary over many orders of magnitude, whereas other engineering properties such as shear strength or compressibility may vary over one order of magnitude or less. Compounding the issue, subsurface information is often inadequate to allow for a proper assessment of how the ground conditions will respond to particular groundwater control measures. Design phase investigations often are either inadequately detailed with respect to groundwater control issues or are focused on other design factors such as ground support, excavation method, and structural design of the lining. Because of the imprecision of analytical methods,

experienced engineering judgement is essential in both the design and construction organizations to successfully control groundwater without unwanted side effects.

The uncertainties associated with groundwater control can lead to serious legal and contractual dilemmas that require open communication to resolve. Innovative approaches to the resolution of such disputes are discussed in Section 10-2.6.

Improper groundwater control, i.e. leakage, in completed structures, can create major operational, structural, and maintenance difficulties for the owner. Most tunnels constructed below the permanent water table leak to some degree. Recent advances in waterproofing barrier systems show promise of improving the situation, but leakage should be anticipated in many instances. Water intrusion is particularly troublesome in transportation and wire conduit tunnels. The presence of water is the initial cause of many structural, architectural, and operational problems, i.e. electrical short circuits, corrosion, and ice formation on rails and pavements.

The state-of-the-art of groundwater control during construction has not changed appreciably for many years with the exception of chemical grouting. The basics are still valid with respect to water extraction techniques, i.e. wells and wellpoints. Cement grouting methods remain essentially unchanged except for the recent introduction of extremely finely ground cements which result in a low viscosity grout similar to some of the chemical grouts.

Leakage control in completed tunnels has changed with the introduction of effective, economical grouts such as water reactive urethane, acrylate which is a substitute for the neurotoxic acrylamide (AM-9), micro-fine cement and new multi-layered sheet waterproofing barrier systems. All of these and others are discussed herein. AM-9 is no longer manufactured in the USA, but it can still be obtained from Japan.

10-2 WATER CONTROL DURING CONSTRUCTION

10-2.1 Problem Evaluation

Identification and evaluation of groundwater problems is essential to the successful completion of any tunnel project. Definition of geohydrologic conditions must be a top priority of any subsurface investigation. It often appears that definition of groundwater conditions is secondary to other issues being evaluated in the investigatory program. Groundwater level measurements seem to be an afterthought, and there never seems to be enough money in the exploration budget to install a sufficient number of observation wells or piezometers or to conduct thoroughly instrumented pumping tests.

The primary issues that must be evaluated relative to groundwater control during construction of tunnels are:

- Distribution of soil and rock types, i.e. definition of aquifer geometry.
- Transmissivity of individual soil and rock units.
- Gradation of cohesionless soils.
- Soil compressibility.
- Regional and seasonal groundwater patterns.
- Water quality including both natural constituents and chemical contamination.
- Identification of facilities which could be adversely affected by groundwater control measures.
- Possible water discharge sites or facilities.

Table 10-1 summarizes recommended exploratory program guidelines for evaluation of groundwater control measures. Subsurface exploration programs are often undertaken in phases, increasing in complexity as the project design evolves.

(i) Preliminary Investigations. Preliminary programs typically consist of compilation and evaluation of previously existing data supplemented by relatively widely spaced borings. Reliable groundwater measurements, i.e. in observation wells or piezometers, should be made in every preliminary boring unless it is known that water will definitely not be encountered, which is rare. Laboratory testing of soil is usually limited to classification testing, primarily gradation analyses of cohesionless soils and Atterberg Limit determinations for cohesive soils. Soil unit transmissivities and compressibility can be estimated respectively from these index properties.

Preliminary water quality tests should be performed, particularly if the project is in an urban area. Some guidelines for water tests are summarized in Table 10-2. Environmental laws and regulations are becoming stricter every year and the consequences, particularly with respect to disposal of groundwater, can have very large financial and schedule impacts. It is not unusual that water treatment and/or special handling will be required as part of the groundwater control methods (Guertin, 1982). Environmental issues can control a project more than technical issues because they frequently have the force of law behind them.

(ii) Design Investigations. Design phase programs are formulated to fill the knowledge gaps identified in the preliminary work. They include more detailed and more closely spaced field explorations with the significant addition of field tests such as pumping tests. Full-scale performance tests may be warranted to evaluate the feasibility of innovative groundwater control methods, i.e. grouting, freezing, etc., or pilot water treatment studies.

TABLE 10-1. SITE INVESTIGATION GUIDELINES FOR EVALUATION OF GROUNDWATER CONTROL

PRELIMINARY PHASE - SOIL

| BORINGS AND WATER OBSERVATIONS | | | | LABORATORY TESTS | |
|--|--|--|---|--|---|
| Spacing | Depth | Type | O.W./Pz | Soil/Rock | Water |
| 1,000 ft. (305 m.) to 2,000 ft. 610m.) | A minimum of 2 tunnel diameters below probable invert. | 2-1/2 in. (6.4 cm.) diameter wash boring, split spoon samples. | In every completed boring (see note 3) if groundwater is encountered or expected to be encountered. | Limited number of soil classification tests. | Use available data. If none available, a limited number of tests may be required for evaluation of corrosion and encrustation potentials and contamination. |

NOTES:

1. Research all existing data.
2. Full-time quality field observation of explorations is essential.
3. Observation wells with surface seals are normally adequate. If perched water or a confined aquifer is suspected, install piezometers in critical horizons.
4. Special tests usually not required.

PRELIMINARY PHASE - ROCK

| BORINGS AND WATER OBSERVATIONS | | | | LABORATORY TESTS | |
|--------------------------------|------------|--|---|------------------|------------|
| Spacing | Depth | Type | O.W./Pz | Soil/Rock | Water |
| See above. | See above. | 3-1/2 in (8.9cm) diameter wash boring, split spoon samples. ----- Continuous NX size rock core. See above. | Usually no laboratory tests of rock cores required. | See above. | See above. |

TABLE 10-1. SITE INVESTIGATION GUIDELINES FOR EVALUATION OF GROUNDWATER CONTROL (CONT'D.)

DESIGN PHASE - SOIL AND ROCK

| BORINGS AND WATER OBSERVATIONS | | | | LABORATORY TESTS | |
|---|--|--|---|--|---|
| Spacing | Depth | Type | D.W./Pz | Soil/Rock | Water |
| 300 ft (91.5m.) to 500 ft. (152.5m) (see note 3). | A minimum of one tunnel diameter below probable invert. | *3-1/2 (8.9cm.) dia. wash boring split spoon samples (See notes 1&2). *Continuous NX size rock core in rock. | *Installed in com- pleted borings at spacings on the order of 1,000 ft. (305m.). *A minimum of two at each major underground struc- ture. | *Classification tests of every major soil unit. *Engineering property tests of fine grained, compressible soils. *Certain sedimen- tary rocks suscep- tible to solution may require minera- logic testing to identify soluble components, i.e. carbonates. | *Total hardness *Total Fe *Total Np *Alkalinity *Chlorides *Sulphates *Nitrates *pH *Hydrogen sulphide *Carbon Dioxide *Dissolved oxygen *Total dissolved solids *Organic Chemicals |

NOTES:

1. Continuous soil sampling is advisable at tunnel depth to identify stratification details.
2. Undisturbed samples of soft compressible soils should be obtained from each major soil unit.
3. Closer spacing on the order of soft 50 ft (15.2m) to 200 ft (61.0m.) may be required in station areas.
4. Full-time qualified field observation of explorations is essential.
5. Testing to support water discharge permits may be required.
6. Falling and constant head borehole permeability tests in soil to identify major differences in soil permeability. Special care required in test interpretation.
7. Packer tests in rock. Limited objective pumping tests.
8. Long-term pumping tests to investigate significant aquifers.
9. Special performance tests to evaluate innovative groundwater control techniques.
10. Geophysical techniques may prove helpful in augmentation of boring program, but should be used with particular care in urban areas.

TABLE 10-2
Summary of Groundwater Constituents Affecting Dewatering Systems

| <u>Constituents</u> | <u>Effect</u> |
|----------------------------------|--|
| Calcium Carbonate | - Encrustation due to precipitation as amount of dissolved carbon dioxide decreases with decreasing pressure. |
| Iron Compounds (>0.5 ppm) and | - Encrustation as with calcium. - Jelly-like iron hydroxide deposited due to increased velocities. |
| Manganese Compounds | - Black ferrous oxide, red ferric oxide, and white ferrous hydroxide precipitates due to increased velocity and exposure to oxygen. - Severe corrosion can result if water contains less iron than it is capable of carrying. |
| Bacteria | - Iron bacteria feed on carbon compounds including bicarbonates and carbon dioxide producing a well clogging slime. |
| Hydrogen Sulphide | - Corrodes steel and copper based alloys. Copper sulphide encrustation can result. |
| Carbon Dioxide | - Dissolved carbon dioxide can form corrosive carbonic acid. |
| Oxygen | - Dissolved oxygen accelerates corrosion. |
| Total Dissolved Solids | - Accelerates corrosion due to increased electrical conductivity. |
| Hydrogen Ions | - A pH less than 7 is acidic and results in corrosion. |
| Silica | - Silica combines with iron and manganese to form encrusting silicates which are insoluble in acid. |

The analysis of pumping tests and full scale field performance tests has been facilitated in recent years by the availability of numerous micro-computer based programs, primarily through improved data logging and management. These automated techniques greatly reduce the time required to analyze test data and to apply the data to project geometries, but they do not replace the need for experienced judgement when making final design decisions.

(iii) Construction. Investigation usually continues during construction. All ambiguities relative to subsurface conditions are rarely fully defined prior to construction. Subsurface condition variations observed during construction should lead to modifications to the groundwater control system. For example, well or wellpoint spacing may have to be adjusted or a different grout type be selected depending on observed soil gradation or rock fracturing.

10-2.2 Groundwater Control Methods - An Overview.

There are two generic approaches to groundwater control during construction, extraction, and exclusion. Extraction methods, often referred to as dewatering techniques, are probably the oldest and most commonly used groundwater control methods. They remove water from the soil and rock in advance of excavation and include:

- Open sump pumping.
- Submersed pump wells.
- Lift wellpoints.
- Venturi ejectors.

Exclusionary methods include a wide variety of methods to create barriers to groundwater movement in the ground thereby excluding the water from the excavation. They include:

- Cutoff walls, i.e. steel sheeting, cast-in-place slurry walls, soil/bentonite slurry trench cutoffs.
- Cement and chemical grouts.
- Ground freezing.
- Compressed air shields.
- Closed face shields.
- Specialized shields, i.e. slurry shields, earth pressure balance (EPB) shields, combination shields.

The above-listed methods are summarized in Table 10-3 and described more fully below.

10-2.3 Dewatering Methods.

Dewatering is typically accomplished by open pumping, well casings containing electrical or pneumatically operated submersible pumps, wellpoints, or ejector wellpoints. The techniques are intended to remove water and to

TABLE 10-3 SUMMARY OF GROUNDWATER CONTROL METHODS DURING CONSTRUCTION

| METHOD | APPLICATION IN TUNNEL DEWATERING | ADVANTAGES | LIMITATIONS |
|------------------------------|---|--|--|
| Predrainage Deep Wells | Sand and Gravel | Wide Spacing effective effective for deep dewatering | May result in pumping a greater quantity than necessary. |
| Well Points | Sand and Gravelly Sand | Spot control possible | Lift limited to 15 to 20 feet |
| Ejectors | Sand, Silty Sand, Sandy Silt | Minimize quantities pumped | Inefficient. Susceptible to encrustation. Difficult to maintain. Complex. |
| Open Sump Pumping | In stable waterbear- ing strata | Inexpensive. Simple. | Can lead to unstable soils due to seepage pressures. |
| Cutoffs Steel Sheetpiling | Nearly any pervious or semi-pervious soil through which it can be driven | Materials reusable. Easily installed. Effective in pervious soils. | Sheetpiling must be driven intact. Cobbles or obstructions may damage Sheetpiling through interlocks can limit effectiveness in semi-pervious soil. |
| Slurry Walls | Any soil which will contain slurry, i.e. gravelly sand to | Wall can be used as part of structure. Leaks can be repaired during excavation. Many variations in basic technique. | Boulders difficult to excavate. Messy operation. |

TABLE 10-3 SUMMARY OF GROUNDWATER CONTROL METHODS DURING CONSTRUCTION (CONT'D.)

| METHOD | APPLICATION IN TUNNEL Dewatering | ADVANTAGES | LIMITATIONS |
|--|---|--|---|
| Thin Wall Cutoff | Any soil through which probe can be driven. | Economical. | Must penetrate impervious stratum. Thin cutoff. |
| Grout | Gravel to Sandy Silt | Very effective in eliminating flow into tunnel. Stabilize soils unstable due to seepage pressures. | Difficult to monitor. Environmental concerns with chemical grouts. |
| Compressed Air | Pervious Sands, Silty Sand Sandy Silt | Very effective in silt and fine sands Limits settlements outside tunnel. | Large air losses in very pervious soils (gravels). Strict regulation of working environment. Limited work shifts in pressures over 12 psi (0.083 mpa). Possible "blowout". |
| Specialized shields, (slurry, earth pressure, combination) | silt and silty sand and sand and Gravel | Stable in water bearing sands. Minimizes surface settlement. Minimal tunnel hazards. Accurate tunnel alignment. | Problems in mixed face conditions. Large cobbles a problem. Cohesive soils can be a problem. |

reduce seepage pressures to create "workable conditions" in the tunnel. "Workable conditions" will vary depending on ground conditions from essentially dry for sensitive fine sands or silty sands to relatively wet, but stable rock conditions. Dewatering can be accomplished either in advance of tunneling by predrainage or by pumping from within the tunnel. It is common to combine predrainage with internal pumping or other control methods.

(i) Sump Pumping. Open pumping from the tunnel excavation has been employed for centuries. It is used in soils which are difficult to predrain completely, but it is only practical in instances where the soil or rock is stable under seepage pressures. Open pumping creates a flow pattern through the soil to the point of pumping. If the limits of excavation are within the flow pattern established, the soil may become unstable resulting in difficult working conditions, low advance rates or, in the extreme case, collapse of the tunnel excavation. Sump pumping is often used as a secondary method to handle residual seepage not controlled by the primary extraction or exclusion system.

(ii) Wells. Wells are individual extraction units which employ a drilled-in permeable well casing containing a submersible pump to push the water up and out of the well casing. Often referred to as "deep well systems," these units can be used to pump groundwater from almost any depth limited only by the capacity and power of the pump. The pumps are usually electrically powered, but pneumatically operated pumps may be used. The pump motor is usually submersed, but may be surface mounted on a vertical pump drive. Fig. 10-1 is a photograph of one of three submersible pumps installed in a single well for a week-long, 2,000 gallon per minute (125 l/s) pumping test for the Light Rail Rapid Transit tunnels constructed in Buffalo, New York. Well systems can be used to pump large quantities of water and can therefore be installed at relatively large spacings, i.e. 50 ft. (15.2 m) or more up to hundreds of feet. The spacing and pump capacity must be designed based on aquifer characteristics and required drawdown. They are typically used in pervious soils where moderate to large, i.e. greater than 20 ft. (6.1 m) drawdowns are required.

Well systems are flexible in that wells can be installed at variable spacing with relative ease to control local anomalous conditions. They also have the advantage that they can be located away from the immediate work area as is required with most other dewatering systems because of the relatively large area of influence of individual wells installed in pervious soil.

There is typically a large investment at each well location due to the depth and size of the well casing, i.e. typically on the order of 1 ft. (0.3 m) or more and individual pumps. It is therefore prudent to carefully design the system to maximize capacity and life span, and to minimize operating and maintenance costs.

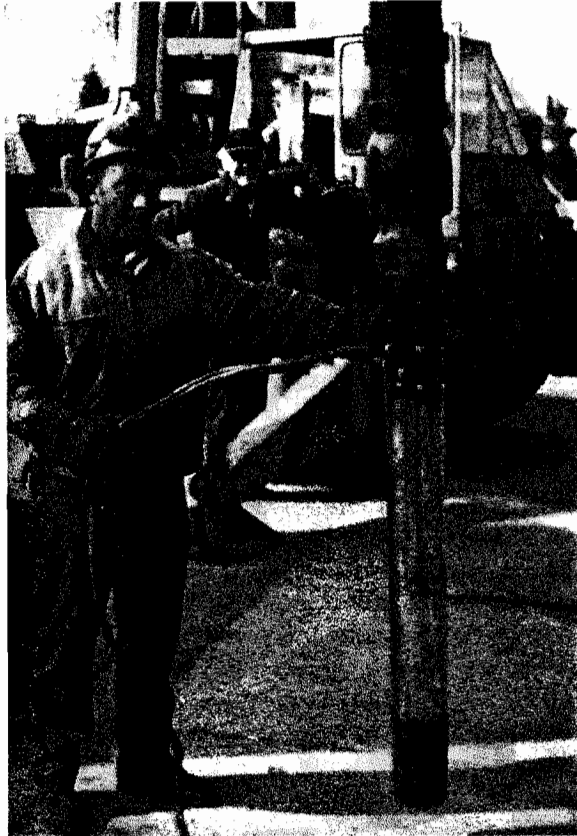
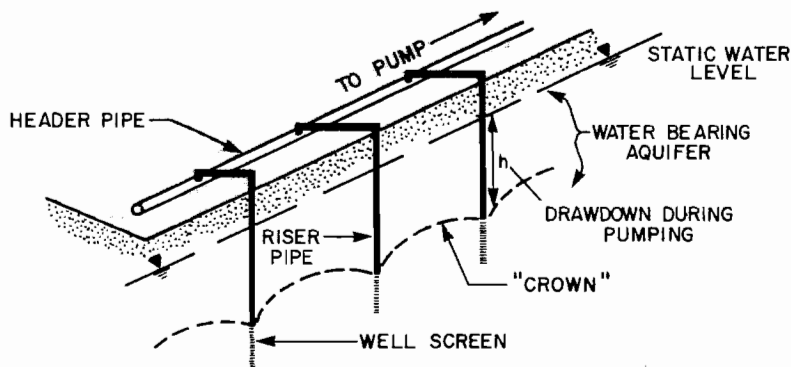


Fig. 10-1 Electric Submersible Pump (Buffalo Subway Tunnels)

Union work rules in some cities require an operator for small numbers of individual wells, i.e. one operator for five wells in New York City. This tends to increase the cost of well systems disproportionately to other methods such as wellpoints.

(iii) Wellpoints. Wellpoints are typically used when the required drawdown is 15 to 20 ft. (4.6 to 6.1 m) or less and the anticipated water quantities are moderate because of finer grained soil or less required drawdown. Wellpoints are connected to a common suction header which is evacuated by a pump as illustrated in Fig. 10-2. The system relies on atmospheric pressure to lift the water, therefore the maximum practical lift is about 15 ft. (4.6 m); however, this can be increased by careful sealing of the evacuation system and pump to maximum of about 20 ft. (6.1 m). This is

sometimes referred to as a "vacuum wellpoint" even though a vacuum pump is not used.



a) SINGLE STAGE

Fig. 10-2 Wellpoint System Installed (Guertin, 1982)

Wellpoints are most applicable to shallow bored tunnels and to open excavations, i.e. cut-and-cover tunnels. They are useful only for the shallowest of bored tunnels because of the limited lift capability. They can be used with deeper bored tunnels being excavated under compressed air to lower the water head at the face and thereby lower the required air pressure. Wellpoints can be installed from inside the tunnel, but it is not commonly done because of interference with other tunneling operations. In open excavations, multiple stages can be installed to dewater to relatively large depths, but considerable construction site space is required for the header pipes and the multiple wellpoint installations. Because of the relatively low cost per installed well, wellpoints are effective in control of water in stratified soil where close spacing is required.

(iv) Ejectors. Ejectors remove water from the ground by means of a water driven jet pump venturi nozzle inside of a wellpoint screen (Fig. 10-3). Water is supplied to the jet pump under pressure by a centrifugal pump attached to a water storage tank on the surface. The water is distributed to the wellpoints through a header pipe and the combined flow of supply water plus groundwater is pushed up a riser pipe to a collector manifold which returns to the storage tank. Excess groundwater is overflowed from the tank and pumped to

discharge. Ejectors require the installation of two pipes in a borehole either as two separated pipes or as concentric pipes. Installation of two separate pipes requires a larger borehole than the concentric arrangement.

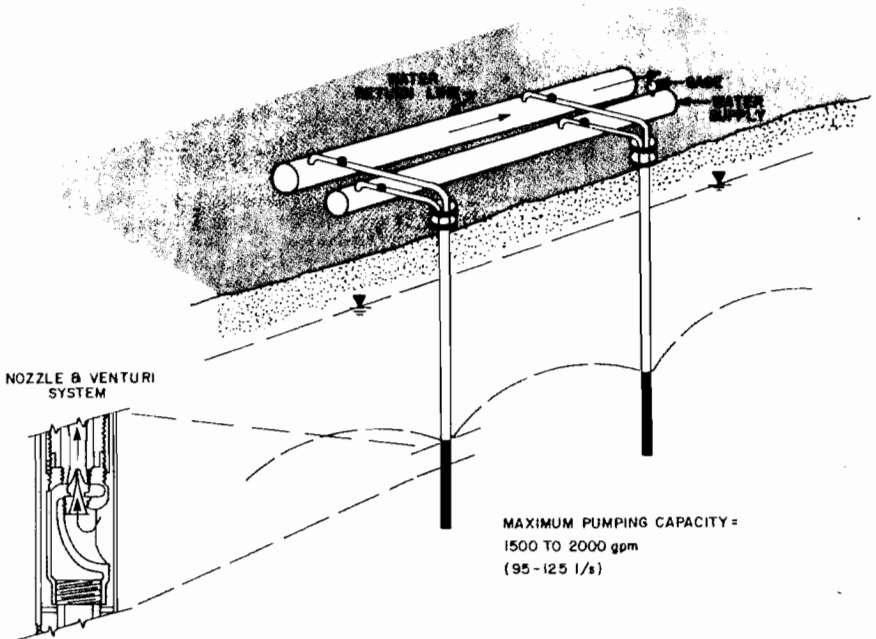


Fig. 10-3 Typical Ejector System

Ejectors are capable of removing water at depth without multiple stages, and they can be economically installed at close intervals when required due to soil stratification or permeability. Ejectors have a maximum efficiency of about 25 percent, therefore the required horsepower is higher by a factor of four or five than for wells or wellpoints at the same head. These economic considerations, therefore, limit the applicability of ejectors to low yield situations, i.e. 5 to 10 gpm (0.3 to 0.6 l/s) per ejector. A typical application is usually in stratified fine sands and silty sand where the vacuum effect created by the jet pump, which pumps air as well as water, can create negative

pore pressures in the soil having a dramatic stabilization effect on unstable saturated fine sand.

Ejector systems are highly susceptible to iron or manganese encrustation. If the water contains high concentrations of dissolved iron or manganese, continuous maintenance can become costly.

(v) Unwanted Side Effects. As reported by Powers (1985) dewatering methods can have unwanted side effects on adjacent properties, the work itself, and the environment. Among the side effects are:

- Ground settlement due to loss of ground from improperly designed water extraction wells and wellpoints.
- Ground settlement due to consolidation of compressible soils resulting from increased effective stresses from water table lowering.
- Depletion of adjacent groundwater and/or surface water supplies.
- Deterioration of untreated timber piles by exposure to air resulting from lowering groundwater levels below pile butts.
- Salt water intrusion.
- Expansion of contamination plumes.
- Release of contaminated water into the environment.
- Harmful effects on vegetation in wetlands.
- Development of sinkholes.

The possibility of any of these effects being a problem on individual projects must be evaluated on a case-by-case basis. Of the above-listed effects, the most common issues are those related to settlement, deterioration of wood pile foundations, and contamination. Increasing environmental regulations makes consideration of water quality issues one of the most important variables to be evaluated prior to implementing a dewatering system, particularly in urban areas. Disposal of contaminated groundwater even at relatively low levels can become very costly. The unit cost for disposal of contaminated groundwater from a Boston project built in 1987 was approximately \$0.80 per gallon.

10-2.4 Exclusionary Methods

There are numerous techniques for excluding water from open and tunnel excavations as summarized in Table 10-3. They may be used in instances where the side effects of dewatering are not acceptable to a specific situation such as anticipated settlement due to consolidation of soft organic soils underlying fill materials as is the case in many coastal cities or induced infiltration of contaminated groundwater. Exclusionary methods are often used in combination with dewatering techniques to reduce the water which must be removed. Barrier techniques, such as cutoffs, grouting or freezing greatly reduce active

dewatering requirements, but do not always eliminate them completely. For instance, water stored within the cutoff must be controlled.

(i) Cutoff Walls and Trenches - An Overview. These methods are not frequently used in connection with bored tunnels, but are commonly used for cut-and-cover projects. Cutoffs must either intercept an impervious layer or extend deep enough to reduce flow quantities and exit gradients. The most common cutoff walls used in tunnel construction are steel sheeting walls and cast-in-place slurry walls. Soil freezing is used in connection with both open excavations and around tunnels, but has been used more in Europe than in the United States. Soil-bentonite slurry trench cutoffs and grouted cutoffs are commonly used in dam designs, but are not common in cut-and-cover tunnel projects. Grouting is, however, commonly used in bored tunneling, particularly for water control in rock.

Soil bentonite cutoffs have not been used particularly in urban areas because of the messy site conditions created and areas needed for mixing. The method is, in fact, more applicable to large sites such as dams.

Steel sheeting, cast-in-place slurry walls, and slurry trenches are discussed below.

(ii) Steel Sheeting. Steel sheeting is probably the oldest and still the most common cutoff method used in construction (Fig. 10-4).

The primary advantages of steel sheet pile cutoffs are the ease of installation, applicability as a structural ground support system, ability to withstand high driving stresses, long-term durability, and reuse potential. Steel sheeting is most effective as a cutoff in relatively pervious soils as opposed to silty or clayey sands where the effective permeability of the sheeting system can be near that of the surrounding soil. Water leaks through the individual sheet interlocks. Studies of typical sheet interlocks indicate that spaces of 1/32 in. to 3/32 in. (1 to 2 mm) occur at the interlocks and can correspond to nearly 0.5 percent of the total area of sheeting. Additional openings in the sheeting caused by driving in difficult ground can increase the degree of imperfection to 1 percent or more which effectively reduces the effectiveness of the cutoff to zero. Cutoff efficiency increases as opening sizes decrease due to increasing stresses from soil pressures as excavation proceeds and as interlocks become clogged with soil particles due to water migration. Artificial means to plug interlocks using bituminous products or hay have been effectively implemented. If very difficult driving conditions are anticipated such as into glacial till containing cobbles and boulders or weathered bedrock, steel sheeting may not be the best form of cutoff because of the interlock damage which is likely to occur from hard driving.



Fig. 10-4 Steel Sheet Pile CutOff (Guertin, 1982)

(iii) Cast-in-Place Slurry Walls. The application of cast-in-place slurry wall cutoffs to tunneling is generally restricted to cut-and-cover projects. The wall may or may not be incorporated into the final structure, although recent trends are to do so. Fig. 10-5 illustrates the most common construction procedure. The procedure was developed in Europe and consists of excavation of a narrow, i.e. 1 to 3 ft. (0.3 to 0.9 m) wide trench to required depth using a modified clamshell bucket similar to that illustrated in Fig. 10-6 or a backhoe. The depth of panel is limited only by the excavating equipment. Depths in excess of 100 ft. (30.5 m) are not uncommon.

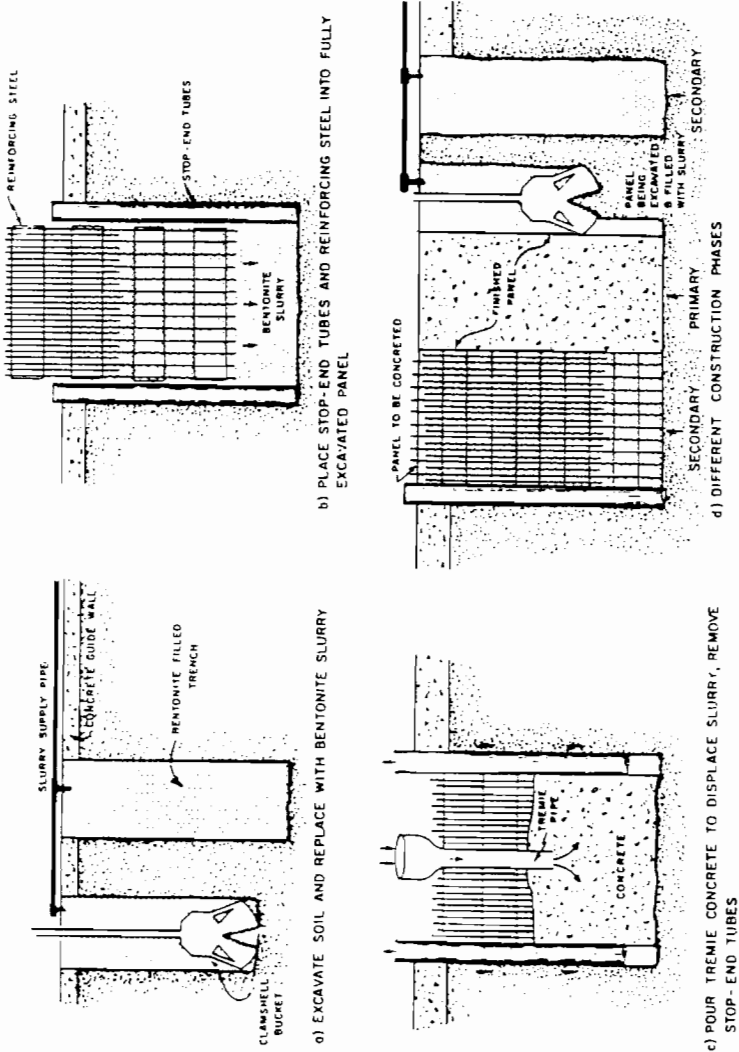


Fig. 10-5 Schematic of Slurry Wall Procedure

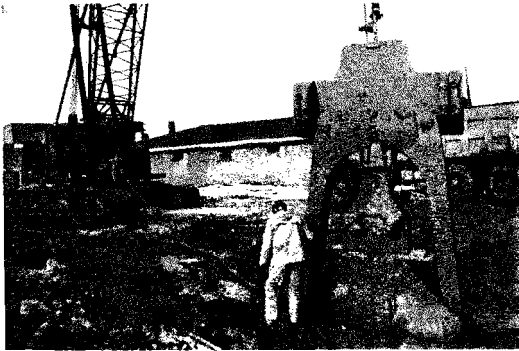


Fig. 10-6 Typical Slurry Wall Clamshell Bucket

The clamshell is the most common excavation technique because greater depths and better dimensional control are possible with this equipment than with a conventional backhoe. The trench is kept open by keeping it filled with a bentonite slurry. Reinforced concrete guidewalls are constructed at the surface prior to trench excavation to provide geometric control and to keep the top of the trench from caving. Hollow stop-end tubes, which are the most common joint form, are placed at the ends of each panel to form a uniform joint between adjacent sections. Once a panel is fully excavated, a preassembled reinforcing cage is lowered into the slurry and positioned from the guidewalls. Concrete is then tremied into the excavation displacing the slurry which is pumped off and recirculated for use on subsequent panels. Alternate panels are constructed using this procedure and the stop-end tubes removed before the concrete has set completely. Secondary panels are then excavated between the primary panels to complete the wall. Typical panel lengths are on the order of 10 to 20 ft. (3.0 to 6.1 m).

Variations on this procedure exist including:

1. Continuous construction using a single stop tube.
2. Use of soldier piles at panel ends incorporating them into the final wall. This wall is known as the soldier pile tremie concrete method (SPTC).
3. Bored secant or secant piles drilled under slurry to form a continuous wall.
4. Pre-cast wall panels placed in the slurry trench in lieu of a cast-in-place reinforced wall.

Limitations of the cast-in-place slurry wall method are generally limited to excavation difficulties. Obstructions such as rubble fill or boulders can seriously impair excavation efficiency. Special excavating tools such as drop chisels or rotary or percussive drills may be necessary to deal with such conditions. When such equipment is used, problems with wall irregularity often result necessitating secondary excavation of concrete upon final excavation.

Another construction problem that can develop is loss of slurry stability and therefore trench stability due to groundwater/slurry incompatibility. This is a problem with low pH groundwater such as saline water. A pH of 4 should be considered the lowest practical limit for slurry stability.

Where the walls are to be part of the completed structure, secondary architectural walls may be necessary in areas of finished space such as subway stations. The final wall surfaces are rough reflecting the texture of the soil within which the wall is built.

It is not uncommon for slurry wall construction joints to leak. The common remedial method is to use one of several grouts. Acrylate based grouts and acrylamide grouts have been used particularly where flowing water must be controlled. Both of these chemical grouts are toxic and should be used with great care, particularly acrylamide which is many times more toxic than acrylate which was developed as an alternative to the acrylamide. Cement or slag based grouts are non-toxic and should be used wherever conditions will permit.

(iv) Injection Beam Cutoff Walls. Injection beam cutoff walls are often considered as slurry walls even though the procedure is not the slurry wall construction procedure. They are categorized as slurry walls because the resultant cutoff is primarily bentonite slurry. The method is not common in the United States, but has been used more extensively in Europe.

The procedure involves insertion of a steel H-pile into the ground by driving or vibration with concurrent insertion of slurry through tubes welded to the web. After insertion of the pile to the required depth, it is withdrawn while cement grout is pumped to fill the void left by the pile. There are two common procedures for forming the cutoff wall as illustrated in Fig. 10-7. The first involves continuous insertion of a single pile. Each insertion of the structural member typically overlaps the previous one by 4 to 6 in. (10.2 to 15.2 cm.). The second method uses a series of piles where members withdrawn at the end of the section are redriven at the head of the line. Thin wall cutoffs installed by these techniques are often very thin, i.e. as much as 1 to 4 in. (2.5 to 10.2 cm), and are typically irregular, but are capable of providing an effective cutoff.

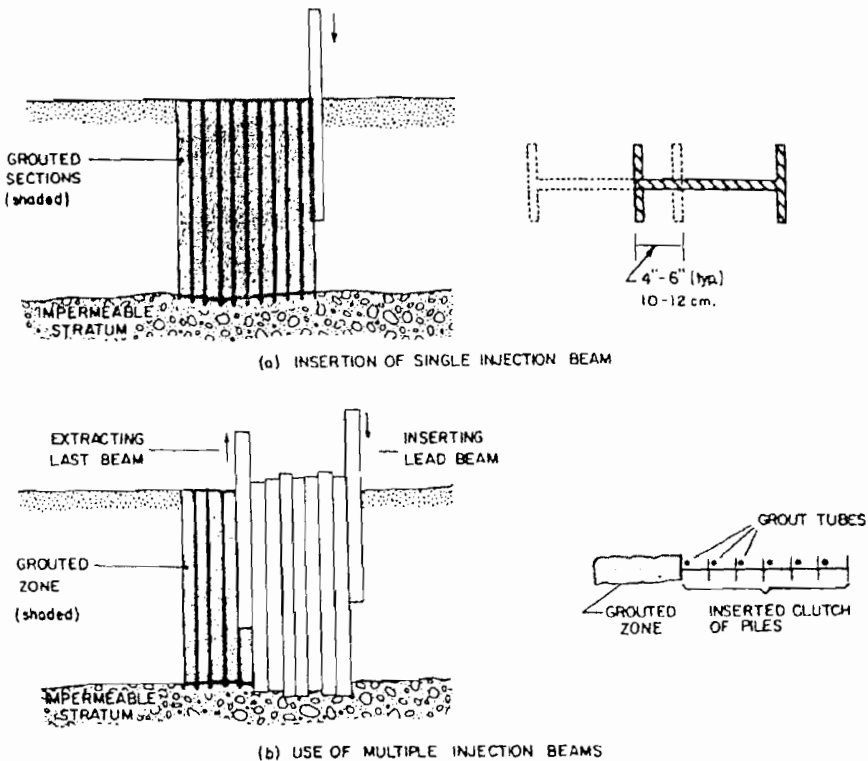


Fig. 10-7 Injection Beam Grouting (Guertin, 1982)

(v) Compressed Air Tunneling. Compressed air has been used as an aid in tunnel and caisson construction for over one hundred years. It was first used on tunnel projects in 1879 concurrently in Antwerp, Belgium, and in New York City beneath the Hudson River. Compressed air is used both for control of water inflow and to improve heading stability in relatively impermeable cohesive soils. While still a valid method, it is used less frequently as new specialty shields which permit free air conditions in the work space are being refined and adopted resulting in less expensive and safer tunneling in difficult ground conditions. A typical air shield is illustrated in Fig. 10-8.

(vi) Slurry, Earth Pressure Balance and Combined Shields. Slurry shields and earth pressure balance shields are designed to advance tunnels through soil under conditions which would normally require compressed air for groundwater control and heading stability. The shields are designed to permit crews to work in free air while supporting the face. Slurry shields achieve this by means of a slurry filled chamber located behind the cutter wheel. Earth pressure balance shields maintain pressure by means of stored excavated soil in a chamber behind the cutter wheel. Pressure on the face is regulated by controlling the volume of soil or slurry removed from the sealed chamber. Schematic representations of slurry and earth pressure balance shields are illustrated in Fig. 10-9 and Fig. 10-10.

It appears that slurry or earth pressure balance shields will be more widely used in lieu of compressed air. Earth pressure balance shields, and variations thereof, are simpler machines than slurry shields and, therefore, more flexible. It is anticipated that earth pressure balance shields will see more frequent application than slurry shields in the future.

10-2.5 Selection of Appropriate Methods

Selection of the appropriate groundwater control method for cut-and-cover and bored tunnels is based on the general descriptive information presented in previous sections. However, in an attempt to systematize the process, a series of selection matrices has been developed (Guertin, 1982) to guide the engineer or contractor through this process. The matrices presented below are based on idealized simplified ground conditions and as such are presented to serve as initial guidelines to method selection. It is recognized that each individual project has variables which dictate a case-by-case final decision.

Fig. 10-11 illustrates the systematic geometry and explanatory notes by which each case is judged. Both cut-and-cover and bored tunnel geometries are illustrated. Fig. 10-12 and Fig. 10-13 are selection matrices for both cut-and-cover and bored tunnels, respectively.

10-2.6 Contractual Considerations

Successful groundwater control during construction is probably as dependant on the form of the contract documents as it is on technical details of any particular method. Groundwater control is always a high risk activity and it is the function of the contract documents to define how this risk is to be shared. Ultimately, it is the owner who should bear the costs of the risk because it is the owner who ultimately benefits from the project. The objective of the legal and contractual elements of a tunneling project should be to achieve the minimum cost for the groundwater control through a proper sharing of risk.

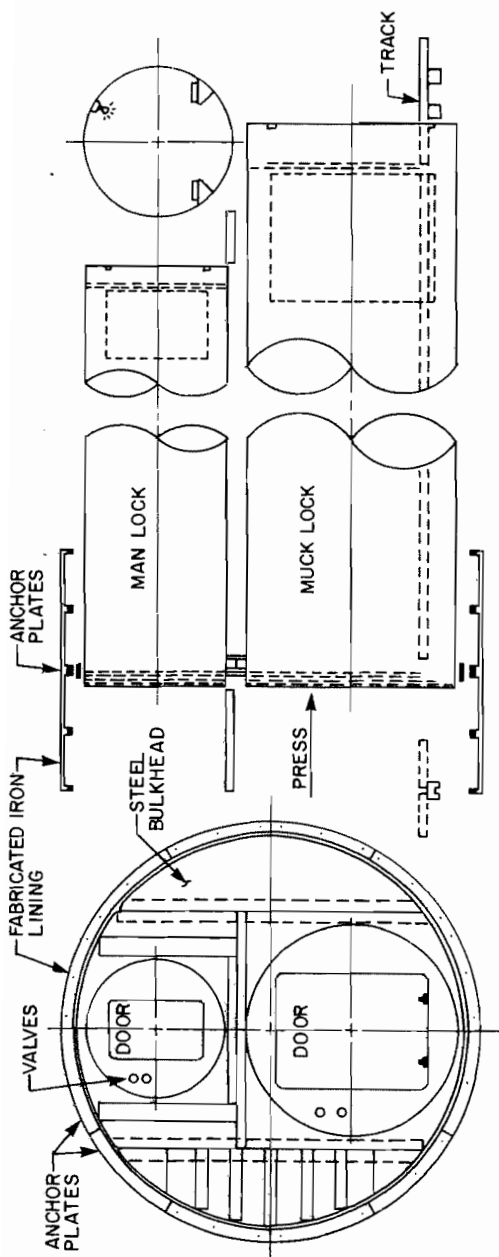


Fig. 10-8 Typical Compressed Air Shield

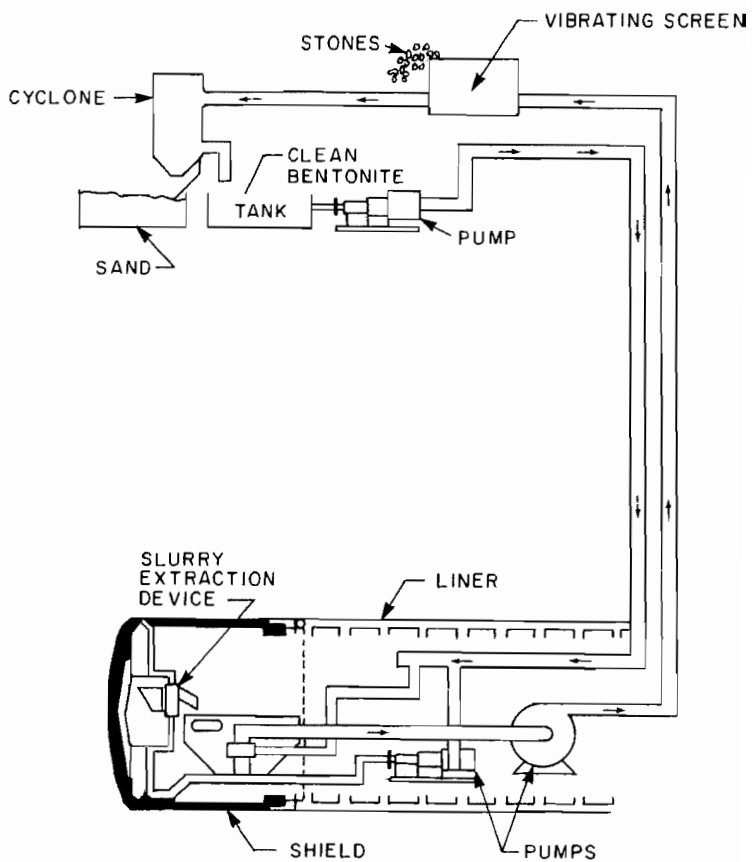
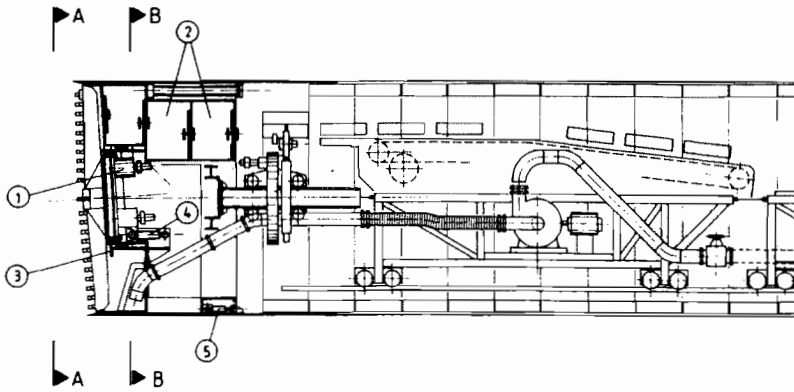
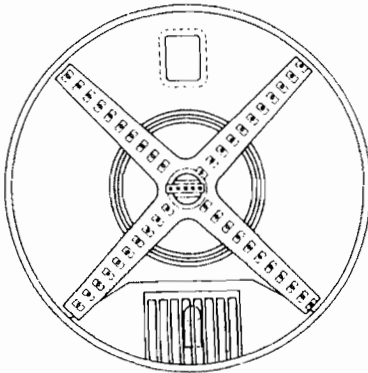


Fig. 10-9

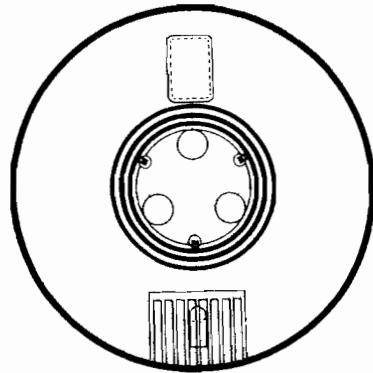
Slurry Shield Schematic Clamshell Bucket



- | | | |
|-------------|--------------------------|------------------|
| 1. gear | 3. diaphragm wall | 5. steering rams |
| 2. air lock | 4. cutting wheel advance | |



cross-section A-A



cross-section B-B

Method Ratings

1 - Very low; 2 - Low; 3 - Medium; 4 - High

** Groundwater control method not generally applicable, but may merit in special situations.
 ■ Method not applicable.

No groundwater control required.

Notes

1. Ratings based on assumption of a single method being successful.
2. Ratings assume methods are applied external to the tunnel.
3. Minimal sump pumping is likely to be required with any method.

Strata Interface Legend

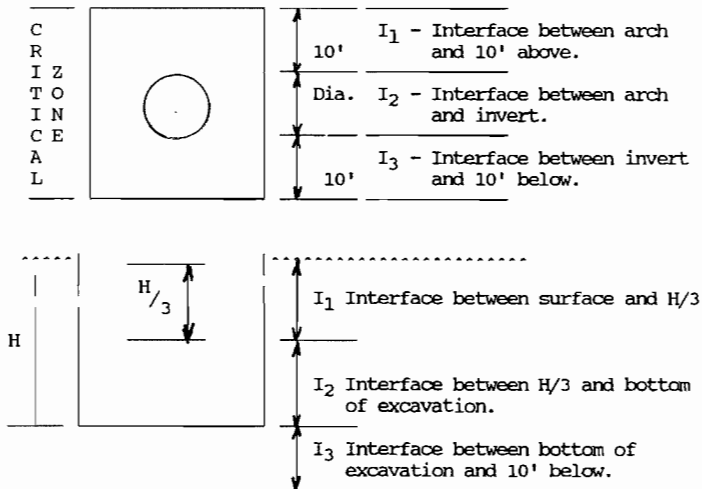


Fig. 10-11 Method Selection Matrices Legend

UNIFORM PROFILE

| CRITICAL ZONE IN | | WELLPOINTS | EJECTORS | WELLS | SUMPING | GROUTING | COMPRESSED AIR | SLURRY/EPB SHIELDS |
|------------------------|--|-------------------------|----------|-------|---------|----------|----------------|--------------------|
| | | MED. SAND OR LGR. K_1 | 4 | 2 | 4 | 1 | 3 | |
| FINE SAND K_2 | | 4 | 3 | 3 | * | 1 | | |
| V.F. SAND & SILT K_3 | | 4 | 4 | 1 | * | * | | |
| CLAY K_4 | | | | | | | | |
| FRACTURED ROCK K_5 | | | | 4 | 4 | 4 | | |

2 LAYER PROFILE: SAND/___

| SOIL TYPE IN CRITICAL ZONE | INTERFACE LOCATION | WELLPOINTS | EJECTORS | WELLS | SUMPING | GROUTING | COMPRESSED AIR | SLURRY/EPB SHIELDS |
|-----------------------------------|--------------------|-----------------------------------|----------------|-------|---------|----------|----------------|--------------------|
| | | $\frac{K_6}{K_3}$ SAND SILT | 1 ₁ | 4 | 3 | 1 | * | * |
| 1 ₂ | 4 | | 3 | 2 | * | * | | |
| 1 ₃ | 4 | | 3 | 3 | 1 | 1 | | |
| $\frac{K_6}{K_4}$ SAND CLAY | 1 ₁ | 3 | 3 | 1 | 2 | 3 | | |
| | 1 ₂ | 3 | 3 | 2 | 2 | 2 | | |
| | 1 ₃ | 3 | 3 | 2 | 2 | 1 | | |
| $\frac{K_6}{K_5}$ SAND FR | 1 ₁ | 3 | 2 | 2 | 2 | 3 | | |
| | 1 ₂ | 4 | 3 | 2 | 2 | 2 | | |
| | 1 ₃ | 3 | 3 | 3 | 2 | 2 | | |

* $K_5 < K_6$ (COARSE FINE)
 Δ $K_6 < K_5$ (FINE COARSE)

2 LAYER PROFILE: SILT /___

| SOIL TYPE IN CRITICAL ZONE | INTERFACE LOCATION | WELLPOINTS | EJECTORS | WELLS | SUMPING | GROUTING | COMPRESSED AIR | SLURRY/EPB SHIELDS |
|-----------------------------------|--------------------|-----------------------------------|----------------|-------|---------|----------|----------------|--------------------|
| | | $\frac{K_3}{K_6}$ SILT SAND | 1 ₁ | 3 | 2 | 4 | * | 2 |
| 1 ₂ | 3 | | 3 | 4 | * | 2 | | |
| 1 ₃ | 3 | | 3 | 4 | * | | | |
| $\frac{K_3}{K_4}$ SILT CLAY | 1 ₁ | 3 | 4 | | | | | |
| | 1 ₂ | 3 | 4 | | * | | | |
| | 1 ₃ | 3 | 4 | | * | | | |
| $\frac{K_3}{K_5}$ SILT FR | 1 ₁ | | | 4 | 3 | 2 | | |
| | 1 ₂ | | | 4 | 2 | 2 | | |
| | 1 ₃ | 1 | 1 | 4 | * | | | |

2 LAYER PROFILE: CLAY /___

| SOIL TYPE IN CRITICAL ZONE | INTERFACE LOCATION | WELLPOINTS | EJECTORS | WELLS | SUMPING | GROUTING | COMPRESSED AIR | SLURRY/EPB SHIELDS |
|-----------------------------------|--------------------|-----------------------------------|----------------|-------|---------|----------|----------------|--------------------|
| | | $\frac{K_4}{K_6}$ CLAY SAND | 1 ₁ | 4 | 2 | 4 | | 2 |
| 1 ₂ | 4 | | 2 | 4 | | 3 | | |
| 1 ₃ | 3 | | 2 | 4 | | 3 | | |
| $\frac{K_4}{K_3}$ CLAY SILT | 1 ₁ | 4 | 4 | | | * | | |
| | 1 ₂ | 4 | 4 | | | * | | |
| | 1 ₃ | 4 | 4 | | | | | |
| $\frac{K_4}{K_5}$ CLAY FR | 1 ₁ | 2 | 2 | 4 | | 4 | | |
| | 1 ₂ | 2 | 2 | 4 | | 4 | | |
| | 1 ₃ | 2 | 2 | 4 | | 4 | | |

Fig. 10-12

Groundwater Control Method Matrices for Cut-and-Cover Tunnels

UNIFORM PROFILE

| CRITICAL ZONE IN | UNIFORM PROFILE | | | | | | |
|-------------------------|-----------------|----------|-------|---------|----------|----------------|--------------------|
| | WELLPOINTS | EJECTORS | WELLS | SUMPING | GROUTING | COMPRESSED AIR | SLURRY/EPB SHIELDS |
| MED. SAND OR LGR. K_1 | 2 | 4 | 1 | 3 | 2 | 2 | |
| FINE SAND K_2 | 3 | 3 | * | 1 | 3 | 3 | |
| V.F. SAND & SILT K_3 | 4 | 1 | * | * | 4 | 4 | |
| CLAY K_4 | | | | | | | |
| FRACTURED ROCK K_5 | 2 | 4 | 4 | 4 | | | |

2 LAYER PROFILE: SAND/_____

| SAND/_____ | | 2 LAYER PROFILE: SAND/_____ | | | | | | |
|-----------------------------------|--------------------|-----------------------------|----------|-------|---------|----------|----------------|--------------------|
| SOIL TYPE IN CRITICAL ZONE | INTERFACE LOCATION | WELLPOINTS | EJECTORS | WELLS | SUMPING | GROUTING | COMPRESSED AIR | SLURRY/EPB SHIELDS |
| $\frac{K_6}{K_3}$ SAND SILT | 1 ₁ | | 3 | 1 | * | * | 4 | 4 |
| | 1 ₂ | | 3 | 2 | * | * | 4 | 3 |
| | 1 ₃ | | 3 | 3 | 1 | 1 | 3 | 2 |
| $\frac{K_6}{K_4}$ SAND CLAY | 1 ₁ | | | | | | | |
| | 1 ₂ | | 3 | 1 | * | 2 | 4 | 3 |
| | 1 ₃ | | 3 | 2 | 1 | 1 | 3 | 2 |
| $\frac{K_6}{K_5}$ SAND FR | 1 ₁ | | 2 | 1 | 2 | 4 | 2 | * |
| | 1 ₂ | | 3 | 1 | 2 | 4 | 2 | * |
| | 1 ₃ | | 3 | 1 | 4 | 2 | 2 | * |

* $K_5 < K_6$ ($\frac{\text{COARSE}}{\text{FINE}}$)



Δ $K_6 < K_5$ ($\frac{\text{FINE}}{\text{COARSE}}$)

2 LAYER PROFILE: SILT/_____

| SILT/_____ | | 2 LAYER PROFILE: SILT/_____ | | | | | | |
|-----------------------------------|--------------------|-----------------------------|----------|-------|---------|----------|----------------|--------------------|
| SOIL TYPE IN CRITICAL ZONE | INTERFACE LOCATION | WELLPOINTS | EJECTORS | WELLS | SUMPING | GROUTING | COMPRESSED AIR | SLURRY/EPB SHIELDS |
| $\frac{K_3}{K_6}$ SILT SAND | 1 ₁ | | 2 | 4 | * | 2 | 4 | 4 |
| | 1 ₂ | | 3 | 4 | * | 2 | 4 | 4 |
| | 1 ₃ | | 3 | 4 | | * | 4 | 4 |
| $\frac{K_3}{K_4}$ SILT CLAY | 1 ₁ | | | | | | 1 | 1 |
| | 1 ₂ | | 4 | | * | * | 4 | 4 |
| | 1 ₃ | | 4 | | * | * | 4 | 4 |
| $\frac{K_3}{K_5}$ SILT FR | 1 ₁ | | 4 | 3 | 4 | | | |
| | 1 ₂ | | 2 | 4 | * | 3 | 4 | |
| | 1 ₃ | | 2 | 4 | * | * | 4 | |

2 LAYER PROFILE: CLAY/_____

| CLAY/_____ | | 2 LAYER PROFILE: CLAY/_____ | | | | | | |
|-----------------------------------|--------------------|-----------------------------|----------|-------|---------|----------|----------------|--------------------|
| SOIL TYPE IN CRITICAL ZONE | INTERFACE LOCATION | WELLPOINTS | EJECTORS | WELLS | SUMPING | GROUTING | COMPRESSED AIR | SLURRY/EPB SHIELDS |
| $\frac{K_4}{K_6}$ CLAY SAND | 1 ₁ | | 2 | 4 | | | 2 | 4 |
| | 1 ₂ | | 2 | 4 | | | 3 | 4 |
| | 1 ₃ | | 3 | 4 | | | 3 | 2 |
| $\frac{K_4}{K_3}$ CLAY SILT | 1 ₁ | | 4 | | | | * | 4 |
| | 1 ₂ | | 4 | | | | * | 4 |
| | 1 ₃ | | 4 | | | | | 1 |
| $\frac{K_4}{K_5}$ CLAY FR | 1 ₁ | | 2 | 4 | 4 | 4 | 4 | |
| | 1 ₂ | | 2 | 4 | 4 | 4 | 4 | |
| | 1 ₃ | | 2 | 4 | 1 | 4 | 2 | |

Fig. 10-13

Groundwater Control Method Matrices for Bored Tunnels

(i) Contract Documents. Basic elements of contract documents are:

1. Disclosure of all subsurface information, both factual and interpretive.
2. A clear statement of the contractor's responsibilities in terms of practical, realistic results to be achieved.
3. Clear statement of the measurement means by which the specified results will be determined.
4. Minimum restraints on the contractor's ingenuity to produce specified results.
5. Procedural provisions should be provided to prevent unskilled practitioners or inadequate methods for groundwater control. These provisions should be such so that the adequacy of the proposed methods can be demonstrated in advance of construction and that there will be adequate monitoring during construction.
6. Payment provisions should allow for fair compensation, but should not provide an incentive for unnecessarily expanding system elements such as water pumped or number of wells.
7. There should be provisions for changed soil and groundwater conditions or changed behavior of the groundwater system such as well encrustation or compressed air leakage.
8. There should be provisions for quick settlement of disputes such as use of Disputes Review Boards (UIRC, 1987), mediation, arbitration, mediation-arbitration, or some variation on these ideas. The primary objective is to provide for efficient settlement of disputes, and avoidance of formal legal proceedings.

(ii) Execution. Several approaches to contracting for groundwater control have been adopted including:

- No Separate Payment for Groundwater Control Provisions. - This method places all of the risk on the contractor by not providing any mechanism for payment based on work performed. While it might appear to be an easy way out for the owner, the realities of contracting practice are that if costs for groundwater control exceed the Contractor's bid allowance a claim for extra payment will probably be made based on changed conditions. The owner will probably pay the costs in the end.

- Unit Prices for all Elements of the System. - This method places all of the risk on the Owner and provides little or no control on the Contractor. This procedure has some merit, but in practice difficulties do develop because the contractor's incentive may be to install as many elements, i.e. wells, as he can depending on his bid unit price. Well efficiency is a function of the Contractor's expertise, particularly in variable soil conditions. Therefore an inexperienced Contractor may install many more wells than

are really necessary. It is usually not in the owner's best interest to provide a contract which gives the Contractor the incentive to install as many groundwater control system elements as possible.

- Owner Designed Minimum System. - This alternate approach is not commonly adopted, but it is one which has merit, particularly in situations where groundwater control is a major element of the work and the consequences of inadequate groundwater control could be catastrophic. Following this approach, the Owner or his engineer designs and carefully specifies a minimum system which the Contractor must build. The method assures that a reasonable minimum system will be installed and it minimizes the risk of an inexperienced Contractor's attempting the construction with unsuitable methods. In the course of installing and operating the minimum system, an experienced Contractor will gather data upon which he can base his needs for supplemental work if needed. Minimum expected performance of the system must be described in the contract documents as well as the final groundwater control results so that the need for additional work can be judged. Payment provisions for additional system elements must be provided.

(iv) Submittals. The primary objective of the specifications is to protect the Owner from unrealistic pricing of bids as a result of inadequate or unskilled approaches to groundwater control. A procedure which has proven useful in this regard is the two-stage submittal. Prior to the start of work, the Contractor submits a detailed plan of his groundwater control method to the Engineer for review. Review of the plan by the Engineer does not relieve the Contractor of his responsibility for adequate groundwater control, but it does give the Engineer an idea of the thoroughness of the Contractor's approach to groundwater control.

A second submittal is required after the Contractor has installed the system, but prior to the start of excavation. This submittal will contain detailed data gathered during system installation including performance testing and is intended to demonstrate that the system, as installed, is capable of adequately controlling the groundwater during tunneling. Risks of tunneling problems associated with unanticipated groundwater conditions are, therefore, reduced.

10-3 MINIMIZING AND CONTROLLING WATER INFILTRATION INTO COMPLETED TUNNELS

10-3.1 Problem Definition

Water intrusion affects all underground structures to varying degrees (Parks, 1986). Fig. 10-14 is a photograph of typical leakage in a subway tunnel during construction. However, in recent years, new technology including multiple layer barrier systems, chemical, and micro-fine cement grouts have greatly alleviated water intrusion and, therefore, the associated problems.

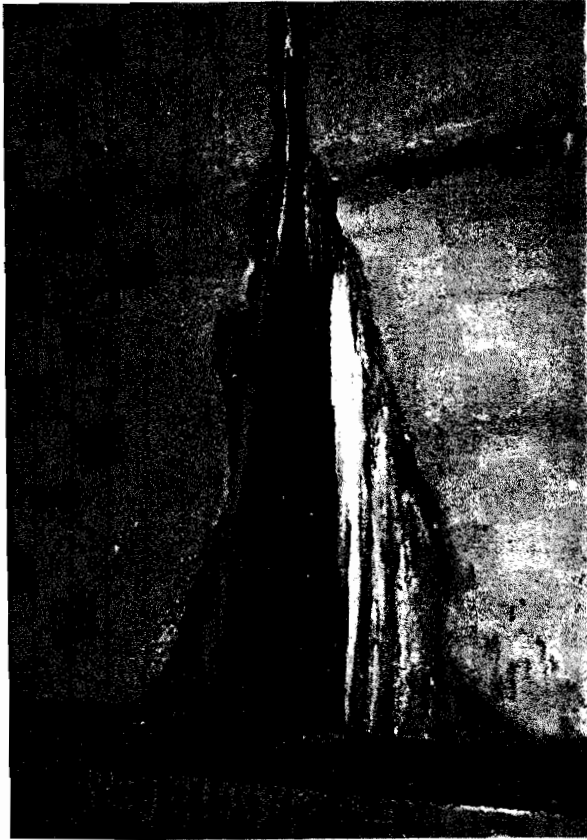


Fig. 10-14 Typical Leakage in a Subway Tunnel During Construction

In the case of rail and highway tunnels, water intrusion can be particularly troublesome as it affects electrical systems and can cause ice build-up in cold regions. The presence of water can be the initial cause of structural and architectural problems. Structural deterioration such as spalling and cracking concrete, reinforcement corrosion, metal fastener corrosion, invert deterioration, and deterioration of architectural finishes, i.e. tiles in particular, would generally not be a problem in dry tunnels. Water intrusion into electrical and communication conduit tunnels is also especially troublesome for all the reasons cited above. Water intrusion into water and sewer tunnels is a much less severe problem, but in the extreme can be a factor in the capacity of wastewater treatment facilities, e.g. substantial money is spent by wastewater

agencies to reduce infiltration into sewers. Intrusion into clean water tunnels can be a problem with the security of the water supply if groundwater is contaminated.

10-3.2 Allowable Infiltration Rates

Typical allowable infiltration rates as specified in recent rapid transit and sewer tunnels are summarized in Table 10-4. (O'Rourke, 1984, pp 69-74). These data are for recent construction with the oldest being the Bay Area Rapid Transit System in San Francisco which was completed in the late 1960's. The other systems cited were built during the 1970's and early 1980's. Many older systems in Boston, New York, and Chicago have experienced significantly more leakage which requires continuous expensive maintenance.

TABLE 10-4
Maximum Allowable Specified Infiltration Rates for Rapid Transit and Wastewater Conveyance Tunnels

| <u>Authority/Tunnel and Location</u> | <u>Infiltration Rate as Stated in Contract Specifications</u> | <u>Infiltration Rate per Ft. of Tunnel (gal./ft²/day) (l./m²/day) ^a</u> |
|--|---|--|
| <u>Rapid Transit Tunnels</u> | | |
| Bay Area Rapid Transit San Francisco, CA | 0.2 gpm/250 ft. | 0.020 (0.82) |
| Washington Metropolitan Area Rapid Transit Authority (WMATA) Washington, D.C. | 0.2 gpm/250 ft. w/not more than 0.1 gpm in any 100 ft. | 0.020 (0.82) |
| Metropolitan Atlanta Regional Transit Authority (MARTA) Atlanta, GA | 0.2 gpm/250 ft. | 0.020 (0.82) |
| Baltimore Regional Rapid Transit Authority (TA) Baltimore, MD | 0.07 gpm/100 ft. w/not more than 0.05 gpm for any 10 ft. | 0.018 (0.71) |
| Massachusetts Bay Transit Authority (MBTA) Boston, MA | 10 gal./hr./100 ft. w/no single leak > 0.25 gal./hr. | 0.042 (1.70) |
| Niagara Frontier Transit Authority (NFTA) Buffalo, NY | a) 0.005 gal./ft ² per 24 hrs. b) 0.01 gal/ft ² per 24 hrs. over any 30 ft. | 0.005 (0.19) |

(Note: ^aEstimated based on a 18 ft. [5.5 m] I.D.)

TABLE 10-4 (CONT'D.)
Maximum Allowable Specified Infiltration Rates for Rapid Transit and Wastewater Conveyance Tunnels

| <u>Authority/Tunnel and Location</u> | <u>Infiltration Rate as Stated in Contract Specifications</u> | <u>Infiltration Rate per Ft. of Tunnel (gal./ft²/day) (l./m²/day)^a</u> |
|--|---|---|
| <u>Wastewater Conveyance Tunnels</u> | | |
| Tunnel and Reservoir Plan Chicago, IL | 500 gal./in. of dia./mile/day | 0.362 (14.74) |
| Calumet Tunnel System Chicago, IL | 500 gal./in/ of dia./mile/day | 0.362 (14.74) |
| Culver-Goodman Tunnel Rochester, NY | 4 gal./min./ 1000 ft. | 0.115 ^a (4.67) |
| Milwaukee Water Pollution Abatement Program Milwaukee, WI | 200 gal./in. of dia./mile/day | 0.145 (2.13) |

(Note: ^aEstimated based on a 16 ft. [4.9 m] I.D.)

10-3.3 An Overview of Water Control in Completed Tunnels

Many possible ways of preventing and controlling water infiltration are possible as summarized in Table 10-5; however, the most common approaches in U.S. practice includes:

- High quality concrete placement to minimize shrinkage.
- Synthetic water stops.
- Internal drainage.
- Multilayered sheet drainage/membrane systems.
- Sheet and fluid applied membranes.
- Bentonite, sheets, sprays, and panels on cut-and cover tunnels.
- Cement grouting through the concrete lining.
- Acrylate grouting through the concrete lining.
- Polyurethane grouting of concrete cracks.
- External drainage/groundwater control.

Parks (Parks, 1986, p. 125) concluded after observing and studying tunnel leakage in five U.S. Rapid Transit systems that, "Thus it becomes quite obvious that the principal approach to the water problem solution has to rely on improving the quality of initial concrete construction." This comment is based on his observations of leakage primarily through shrinkage cracking in the concrete; however, his comments also refer to design and installation of

TABLE 10-5 SUMMARY OF METHODS TO EXCLUDE WATER FROM COMPLETED TUNNELS

| METHOD | ADVANTAGES | DISADVANTAGES |
|---|---|---|
| Cast-in-place Concrete Linings | <p>Resistant to environmental deterioration.</p> <p>Defects easily repaired.</p> <p>Flexibility to deal with special field situations.</p> <p>Self healing.</p> | <p>Difficult to place in wet confined areas.</p> <p>Attention to details of quality concrete placement required to minimize shrinkage and maximize impermeability.</p> <p>Must be kept dry during installation.</p> |
| Bentonite | <p>Bridges cracks.</p> <p>Withstands alternate wetting and drying.</p> <p>Can be backfilled immediately. Insensitive to temperature.</p> <p>High strength.</p> <p>Easily applied.</p> | <p>Does not perform as an exposed coating.</p> <p>Acids, brines and alkalies may impair impermeability.</p> <p>Not self healing; won't seal cracks.</p> |
| Cementitious Coatings | <p>Adapts to irregular surfaces.</p> <p>Flexible/resilient over wide temperature range.</p> <p>Resistant to chemicals.</p> <p>May be laid loose.</p> | <p>Contact adhesives make repositioning difficult during installation</p> <p>Adhesives may delaminate when submerged for extended periods.</p> <p>Careful attention required for watertight field seams.</p> <p>Must be installed at temperatures above 40 degrees F.</p> |
| Vulcanized Rubber and Plastic Sheet Membranes | <p>May be fully bonded to substrate.</p> <p>No adhesives required.</p> | <p>Substrate must be very well prepared and dry.</p> <p>Protection board required.</p> |
| Modified Bituminous Sheet Membranes | | |

TABLE 10-5 SUMMARY OF METHODS TO EXCLUDE WATER FROM COMPLETED TUNNELS (CONT'D.)

| METHOD | ADVANTAGES | DISADVANTAGES |
|------------------------|---|--|
| Liquid Applied Systems | <p>Conforms to complicated substrate configurations.</p> <p>Bridges cracks up to 1/16 inch.</p> <p>Relatively easy leak detection.</p> <p>Elastomeric membranes remain flexible.</p> <p>Rubberized membranes are self healing and remain flexible.</p> <p>Rubberized membranes not effected by rain, snow or frost.</p> | <p>Difficult to achieve uniform thickness.</p> <p>Require skilled applicators, particularly two component systems.</p> <p>Protection required.</p> |

water stops between pours. Special attention to concrete technology during construction is recommended because experience in rapid transit operations has shown that it is more cost effective to prevent water intrusion at the earliest possible time of construction than to treat the problem later.

The next most effective way for creation of a dry tunnel is through use of various membrane systems. While relatively expensive, they are effective. Recently a multi-layer drainage membrane system has been used on several rock tunnel projects with success. The system which is manufactured by several primarily European manufacturers consists of a fleece-backed impervious membrane which is installed behind the permanent concrete lining. Many other membrane products are available, but are applicable primarily to cut-and-cover tunnels. More detail is presented in following sections.

Most of the other methods listed above are for control or elimination of leakage after completion of the lining. It has generally been considered for years that all tunnels constructed below the water table will leak to some degree and measures must be implemented to control this leakage, the most common being injection of cement grout through holes drilled in the lining into the soil or rock outside of the lining. In recent years, the cement grouts have been supplemented with chemical grouts such as acrylate which was developed in response to the toxicity of acrylamide. Fig. 10-15 is a photograph of acrylate grouting of lining leakage in a subway tunnel. Very finely ground cements are also finding usage in place of the chemical grouts.

For sealing cracks in concrete linings, polyurethane grouts are being widely used by rapid transit system operators to eliminate leakage. The materials are very expensive, but small quantities can effectively seal structural cracks.

Finally there is drainage, both external and internal, which effectively controls leakage. External drainage is used primarily outside of rock tunnels or soil tunnels where compressible soils are not present. Reduction of pore pressures in compressible soils by drainage can result in intolerable settlement of the ground outside the limits of the tunnel project. In highly permeable soils the cost of pumping can be prohibitive. However, the New York City Transit Authority does extensive pumping for control of external water levels in pervious soils in the Borough of Brooklyn. Internal drainage control is common in transportation tunnels and electrical utility tunnels. It is usually achieved through use of drip pans, gutters, and sumps.



Fig. 10-15 Acrylate Grouting in a Subway Tunnel

10-3.4 High Quality Concrete - The Primary Defense




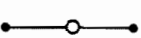

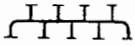

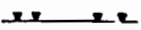
The primary defense against water infiltration into underground structures should be high quality concrete. Everything else is a second line of defense.

High quality, dense concrete is easier to achieve in cut-and-cover structures than in bored tunnels. It is also easier to install exterior drains and membranes in cut-and-cover situations than in bored tunnels. Therefore, it is much more difficult to achieve dry tunnel conditions in bored construction. Concrete is often placed under less than ideal conditions. Rich concrete mixtures, wet placement, and high curing temperatures all serve to encourage cracking of the final liner resulting in leakage which requires remedial action for control.

Water passes into underground structures through joints, wall penetrations, cracks, honeycomb, and the pores of the concrete itself. To produce impervious concrete, all joints and penetrations must be properly sealed, cracks prevented from forming, and honeycomb formation prevented during placement. Experience has shown that the most difficult problem to control is cracks which forms as a result of concrete shrinkage. Sealing of joints and penetrations with waterstop details is relatively easy to achieve; however, special care is required during construction to see that waterstops are

properly placed. Table 10-6 (Greenstreak Plastic Products Co., 1987) is summary of basic waterstop design considerations. Prevention of porous honeycombed concrete requires proper placement techniques with respect to mixing, transportation, free fall, and vibration.

TABLE 10-6
Waterstop Selection Criteria (Greenstreak, 1987)

| Type | Schematic | Application |
|-----------------------------|---|---|
| Serrated |  | Where movement is not expected. |
| Serrated w/ Center Bulb. |  | Expansion joints where movement is expected. The most versatile joint. |
| Dumbbell |  | Where little or no movement is expected. Expansion joints less than 1" wide. |
| Dumbbell w/ Center Bulb |  | Expansion joints where movement is expected. Longitudinal and transverse movements can be accommodated. |
| Splits |  | To eliminate split formwork. |
| Labyrinth |  | Vertical and horizontal construction joints. Eliminates split formwork. |
| Tear Web |  | Accommodates large movements. Web will tear upon movement. |
| Base Seals |  | Installed on bottom of slabs on grade or walls to be backfilled to prevent vertical seepage. |

Shrinkage cracks occur as a result of temperature changes during curing and the drying out of the concrete. Concrete typically shrinks in the range of 300 to 800 units per million which translates to approximately 1/8 inch (3.2 mm) in 20 ft. (6.1 m). To prevent shrinkage cracks, temperature changes must be controlled during the curing process and the mix should be as dry as possible consistent with placement requirements. Low water cement ratios can be achieved more easily with the use of plasticizers such as Plastocrete by Sika or if the available aggregates are deficient in fines by the addition of finely divided materials such as microsilica.

Thermal shrinkage occurs because the temperature of the plastic concrete is higher than the service temperature of the final product. This is usually due to the heat of hydration created as the concrete sets. It may also be due the temperature of the aggregate and water when mixed and placed. The heat changes and, therefore, shrinkage can be reduced by:

1. Chilling the plastic concrete prior to placement.

2. Reducing the amount of cement consistent with requirements for strength, impermeability, and workability.
3. Replacing some of the cement with a material such as pozzolan, which acts like cement but does not give off as much heat of hydration.
4. Spraying cool water on the forms.
5. Curing with cool water sprays.
6. In extreme cases, piping cooling water through the concrete mass could be considered as was done by the Dutch (Janssen, 1979, pp 25-29) for sunken tube fabrication. There is no known case of this technique for a cast-in-place tunnel lining.

Use of expansive cements may also help to reduce shrinkage and, therefore, produce more watertight concrete. They are not widely used in tunnel construction but, in principle, should help to counteract shrinkage effects.

While complete elimination of shrinkage cracking is not likely, it can be significantly reduced by careful attention to basics of concrete technology. Other factors to be considered for improved watertightness are:

1. Chemical attack on the concrete. The most common problem has been sulphate rich groundwater which may dictate use of sulphate resistant cement. In urban areas, chemical contamination of water may be an issue and should be investigated. During design of a portion of the subway expansion in Boston, heavily contaminated groundwater having a pH of 2 was encountered and had to be considered in design.
2. Prevention of corrosion of reinforcing steel which can create water pathways through the concrete. Proper minimum concrete cover is essential.
3. Creation of a dense low porosity concrete through use of finely ground cements, air entrainment additives, and non-porous aggregates.

Creation of essentially impervious concrete is possible, but is more likely in cut-and-cover construction than bored tunnels. This was observed during construction of the Buffalo, New York subway system where leakage into portions of the tunnels driven through rock was much more prevalent than in the cut-and-cover portions of the project.

10-3.5 Waterproofing Cut-and Cover Tunnels

The most common waterproofing techniques for cut-and -cover tunnels after consideration of high quality concrete are:

1. Bentonite
2. Cementitious Coatings

3. Membranes

- Vulcanized Rubber
- Plastic Membranes
- Modified Bituminous or Composite Laminated Membranes

4. Fluid Applied Systems

- Cold-Applied elastomeric membranes
- Hot-poured rubberized elastic asphaltic membranes

(i) Bentonite. Bentonite swells when subjected to water thereby forming an essentially impervious barrier to water. Bentonite waterproofing can be applied in the following ways:

- Bentonite in Flutes of Cardboard Panels - This is the most common application of bentonite for waterproofing cut-and cover tunnels (Fig. 10-16). The bentonite is contained in the voids of corrugated paper units which are usually 4 ft. (1.2 m) by 4 ft. (1.2 m) by 3/16 in. thick (4.8 mm) containing about one pound (0.45 kg) of bentonite per square foot. The paper deteriorates soon after installation allowing the bentonite to form a continuous water barrier. Protection board is used to prevent damage to the panels and polyethylene film or waxed corrugated paper may be used to prevent premature wetting of the bentonite.
- Bentonite Adhered to Geotextile in the Form of a Mat - Bentonite is sandwiched between a top tough layer of polypropylene fabric and a biodegradable paper secondary backing paper. Mat rolls are either 13-1/2 ft. wide (4.1 m) by 82 ft. long (25.0 m) or 4-1/2 ft. wide (1.4 m) by 13-1/2 ft. long (4.1 m) and contain about 0.8 pounds (0.36 kg) of bentonite per sq. ft. Mats have the distinct advantage that they do not require protection board and are fastened with nails or mechanical fasteners.
- Bentonite Adhered to High Density Polyethylene in Sheet and Panel - This product is similar to the geotextile form consisting of about 1/8 in. (3.2 mm) of granular bentonite adhered with a polymer adhesive to a 20 mil (0.5 mm) high density polyethylene (HDPE) sheet. It is packaged in 4 ft. wide (1.2 m) by 24 ft. long (7.3 m) rolls or 4 ft. (1.2 m) by 4 ft. (1.2 m) panels. It contains about 0.75 pounds (0.34 kg) per square foot (0.09 sq m) and does not require protection board.

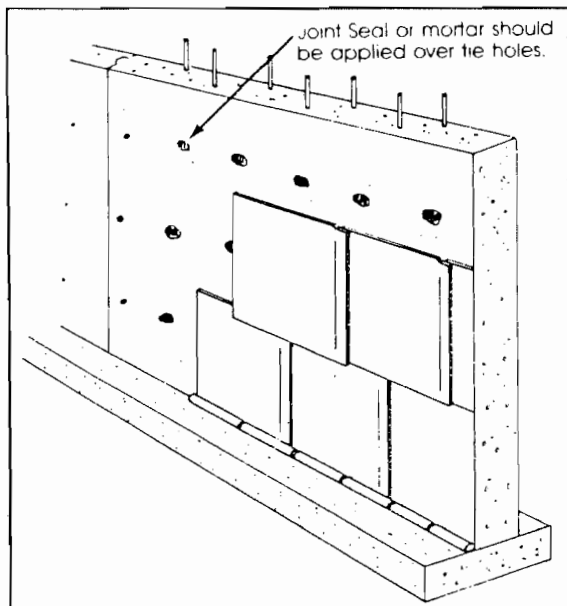


Fig. 10-16 Bentonite Panel Installation (American Colloid Co.)

- Trowel or Spray Grade Bentonite in Liquid/Gel - Powdered bentonite is premixed with chemical additives and the viscosity can be modified appropriately for spraying or trowel on applications. Approximately 0.5 pounds (0.22 kg) per square foot (0.09 sq m) is applied in a 3/16 inch (4.8 mm) thick layer. It requires one to two days curing and must be covered with polyethylene prior to backfilling.
- Bentonite with Asphalt Binder - Dry bentonite is mixed with an asphalt binder and sprayed on at a rate of approximately 1.5 pounds (0.68 kg) per square foot (0.09 sq m).
- Spray Polymer Enhanced Bentonite - Bentonite is sprayed in a two-part nozzle system where dry bentonite and an adhesive are mixed to form a 95 percent bentonite/5 percent adhesive mix. It is typically applied at a rate of 1.5 pounds (0.68 kg) of bentonite per square foot (0.09 sq m). Protection board is advised.
- Bulk Bentonite (Can be mixed with Sand.) - This bulk use of the bentonite is confined to horizontal surfaces.

(ii) Cementitious Coatings. Cementitious coatings are of three types:

- High strength, dense surface coatings.
- Integral surface coatings with penetrating characteristics.
- Latex-based surface coatings combined with water repellants.

There is debate among engineers, contractors, and material suppliers as to whether these coatings are true waterproofing measures or are merely dampproofing. Waterproofing materials will prevent water passage for long periods of time against hydrostatic head while dampproofing products resist water flow, but do not stop it under a sustained head.

High strength dense surface coatings can be applied on either the positive, i.e. water source side, or negative side, i.e. side in area to be protected, of walls. The bond strength between the coating and the concrete should exceed the tensile strength of the concrete if the coating is on the negative side to prevent water pressure rupture. Dense material is typically created through a mixture of special cement, fine silica sand, water and chemical admixtures. Purified iron particles and oxidation promoting agents are sometimes added to form a dense coating.

Dense surface coatings are applied by brush, trowel, and spray-on methods. They are not well-suited in situations where cracking is anticipated because they have no flexibility once a crack is formed. Leakage is likely if cracks form after placement of the coating. Typically 30 percent of the total shrinkage will occur within 30 days if it is moist cured at 50 percent relative humidity. Six to nine months are required to achieve 85 percent of the total shrinkage. Application of waterproofing should be delayed as long as practical to minimize the possibility of rupture from shrinkage cracking.

Integral surface coatings with penetrating characteristics are similar to the dense surface coatings except that manufacturers claim that a chemical reaction takes place at the coating/concrete interface. The two theories as to why crystals form between the coating and the concrete are discussed in detail by Anderson (Anderson, 1983, pp 60-61) and will not be presented herein. However, the crystal formation is effective in sealing microcracks which form after application of the coating. Large cracks can be sealed as crystal growth continues in the presence of water, but these instances are variable.

Latex-based surface coatings are formed using cement, silica sand, water, and a water repellent such as stearic acid combined with latex. The latex base gives the system more flexibility than the other two types of cementitious coatings, but it is not elastic and still may be ineffective in sealing cracks which form after application of the coating. The system is non-penetrating because of the latex base, and there are many who argue that these mixtures are only effective as dampproofing agents.

Of the three types of cementitious coatings described, the dense surface coatings seem to be the most effective, although there have been demonstrated success with the penetrating types. They are typically applied in minimum 1/8 inch thicknesses (3.2 mm) and are most commonly used for repair of cracks and spalls in existing concrete.

(iii) Sheet Membranes. The common sheet membranes are: vulcanized rubber, plastic, and modified bituminous coated. Systems that are completely bonded to the substrate are preferable to unbonded systems with regard to leak detection because they prevent water migration at the substrate interface when water enters from a membrane rupture. Leaks show at the leak location in bonded systems, whereas in unbonded systems the evidence of leakage may be nowhere near the actual leak.

Whether bonded or unbonded, continuous protection should be provided to the membrane particularly at changes in direction where punctures and sagging can occur. Substrate movement through shrinkage or structural movements should be expected. Sufficient expansion joints must be provided particularly with bonded systems to prevent rupture. Unbonded systems bridge substrate cracks more effectively than do bonded systems. Therefore, expansion joints are less critical. While both bonded and unbonded systems require substrate support, bonded systems require more substrate preparation than do unbonded loose laid systems which can be a significant cost consideration. Membrane failure is frequently caused by improper substrate preparation. Waterproofing of tunnel inverts using membranes requires a subgrade support slab beneath the membrane.

The vulcanized rubber and plastic materials used in waterproofing systems are very similar precured non-fabric reinforced sheets. The manufacturing process for these sheet materials results in a nearly perfect waterproofing membrane free of pinholes and holidays which makes them particularly applicable where relatively high water pressures, i.e. 50 ft. (15.2 m) heads, are anticipated. Typical thicknesses vary from 30 to 60 mils (0.8 mm to 1.6 mm); however, some products are manufactured as thin as 20 mils (0.5 mm) to as thick as 120 mils (3.0 mm).

Vulcanized rubber materials used are butyl rubber, EDPM (ethylene propylene-diene monomers) and chloroprene (neoprene). Plastic materials used are vinyl (poly [vinyl chloride]) (PVC), high density polyethylene (HDPE), ethylene copolymer, chlorinated polyethylene (CPE), and chlorosulphonated polyethylene (CSPE or "Hypalon").

The systems may be fully bonded or mechanically fastened to the substrate or may be laid loose. Vulcanized rubber sheets are joined using elastomeric based adhesives whereas plastic sheets may be joined using solvent welding, thermal welding or adhesives (Fig. 10-17, Sarna Polymer, Inc., 1985). The adhesive materials are typically contact-bond type adhesives which preclude

repositioning sheets once contact has been made. Attempts at repositioning are likely to damage the sheets. Also adhesives may delaminate when submerged for extended periods.



Fig. 10-17 Thermal Welding Waterproofing Sheet (Gnilsen, 1986)

Butyl is a synthetic rubber which has been used extensively for waterproofing underground structures. It is difficult to bond to the substrate and is therefore usually loose laid. It stretches under its own weight and is therefore difficult to work with on vertical surfaces.

EDPM is similar to Butyl, but is more resistant to chemicals and ultraviolet radiation. EDPM is used frequently as an underground flashing in combination with bentonite products.

Chloroprene known under the trade name of Neoprene is also a synthetic, rubber-like butyl and EDPM. It is seldom used underground, but is commonly used as a flashing material.

PVC is a thermoplastic polymer synthesized from vinyl chloride. It is very temperature sensitive. Its anticipated life underground is 5 to 25 years (Anderson, 1983, p.53) and it is usually installed unbonded.

All of the polyethylene products, i.e. HDPE, CPE and CSPE, are chemical resistant membranes which are usually loosely laid. A relatively thin CPE, i.e. 20 mil (0.5 mm), membrane can be fully bonded using a water-based synthetic resin adhesive. They behave similarly to the vulcanized rubber products, but are more temperature sensitive.

Modified bituminous coated and composite laminated membranes are factory fabricated membranes consisting of a rubberized asphalt element and a high density polyethylene film laminated to the outer face. The membrane is usually 1/16 in. (1.6 mm) thick and should provide long-time protection underground. These membranes are usually fully bonded to the substrate, but can be mechanically fastened. Seals and laps between sheets are accomplished by bonding of the tacky rubberized asphalt core to the high density polyethylene outer film. The membranes can be installed only at temperatures above 40 degrees F. on a cured dry well prepared substrate. Grinding of the concrete surface is common before installation of rubberized asphalt membranes. The outer film must be protected from sunlight and damage.

(iv) Fluid Applied Systems. Fluid applied systems can be applied at ambient temperatures or preheated. The preheated products are the rubberized asphalts and hot mopped asphalts and cold tars, i.e. built up waterproofing. Ambient temperature products include:

- Rubberized/asphaltic products:
 - Single component asphalt/coal tar modified urethane
 - High grade polymeric asphalt
 - Asphalt/coal tar dampproofing
 - Rubbers (butyl, EPDM, Neoprene, CSPE)
- Elastomeric products:
 - two component urethane
 - silicon elastomers and penetrants
 - plastics and vinyls (PVC)

Liquid applied systems are suitable to irregular substrates, but require a high degree of applicator skill and quality control to create a uniform watertight barrier. They will bridge cracks up to 1/16 in. (1.6 mm) and have the leak detection qualities of fully bonded membranes.

The rubberized products tend to be self-healing when penetrated. They can be applied in cold weather, i.e. 20 degrees (F), and are generally not affected by rain, snow, or frost after application, but they should be protected from prolonged exposure to sunlight. They are usually applied in heavier thicknesses than the elastomeric products because their elastomeric behavior is less.

The elastomeric membranes are available in one- and two-component formulations. The two-component formulations are better suited to varying conditions, but require skilled knowledgeable applicators to achieve a satisfactory waterproofing membrane. They remain flexible over a wide range of temperatures. Consistency of product has been a problem and the mixing

instructions of the two-component systems must be followed carefully. Cured elastomeric membranes should be protected from physical damage and sunlight as soon as practical after application.

(v) Selection Considerations. Selection from the long list of available waterproofing products of the appropriate waterproofing membrane for cut-and-cover tunnels requires careful consideration of many variables. In an attempt to systematize the process, Table 10-7 has been prepared to identify the key issues and to rate various generic products. The basis for this table is work by Anderson (Anderson, 1983) and the Construction Specifications Institute (CSI, 1987). The apparent large variations in ratings for different generic waterproofing methods reflects variations in detailed methods. For example, under cementitious coatings are included trowel-on coatings which are quite effective as well as brushed-on products which are much less effective. Also, under the other products, those which are a derivative of asphalt in some form are, in general, more effective than other synthetic products.

TABLE 10-7
Selection Criteria for Waterproofing Cut-and-Cover Tunnels

| <u>Criteria</u> | <u>Membranes</u> | | | | | |
|---------------------|------------------|-------------|----------------------|-------------------|----------------------|-----------------------|
| | <u>Cement</u> | <u>Clay</u> | <u>Fully Adhered</u> | <u>Laid Loose</u> | <u>Heated Liquid</u> | <u>Solvent Liquid</u> |
| Crack bridging | 1 | 4/5 | 3/4 | 5 | 3/4 | 3/5 |
| Resealability | 2 | 4/5 | 1 | 1 | 3/4 | 1/3 |
| Leak Detection | 4 | 4/5 | 3 | 1 | 4/5 | 4/5 |
| Resist chemicals | 5 | 1 | 5 | 5 | 3/4 | 1/5 |
| Resist Puncture | 5 | 1/3 | 3/4 | 3/4 | 3 | 1/3 |
| Resist pressure | 1/5 | 4/5 | 4/5 | 5 | 3/4 | 1/3 |
| Skill required | 4/5 | 4/5 | 1/3 | 4 | 1/3 | 3/5 |
| QC ease | 1/5 | 4/5 | 4/5 | 5 | 3 | 3 |
| Equip. complexity | 5 | 3/5 | 3/5 | 5 | 3 | 3/4 |
| Adapt complex surf. | 4/5 | 1/4 | 1/3 | 1 | 3 | 4/5 |
| Horiz. application | 5 | 4/5 | 3 | 5 | 4/5 | 5 |
| Vert. application | 4/5 | 1/4 | 1 | 4/5 | 1 | 4/5 |
| Protection | 5 | 3/5 | 4/5 | 4 | 3/4 | 3 |
| Availability | 1/5 | 1/5 | 4/5 | 5 | 3/5 | 1/5 |
| Cost | 1/5 | 3/4 | 3 | 4 | 3 | 1/5 |

Legend: 1=Poor; 3=Fair; 4=Good; 5=Excellent

10-3.6 Waterproofing Bored Tunnels

The primary defense against water intrusion into bored tunnels in rock is high quality concrete as previously discussed. For tunnels in soil there may be another line of defense in the form of a steel segmented liner as illustrated in Fig. 10-18. Other options to prevent water intrusion during and after bored tunnel construction, whether in rock or soil, are more limited than

for cut-and-cover tunnels. These options are limited to grouting and multi-layered drainage/membrane systems. Grouting is not discussed herein, but new multilayer systems will be.

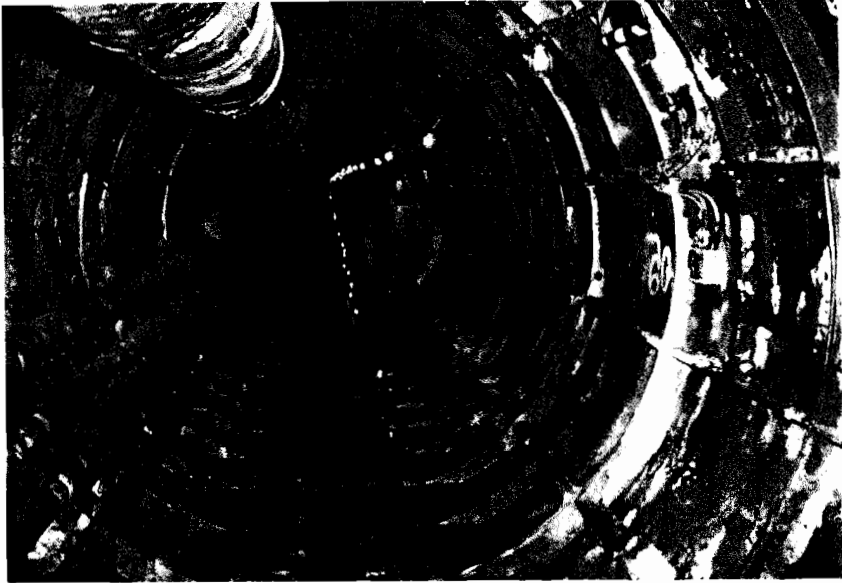


Fig. 10-18 Steel Segmented Tunnel Liner, Staten Island, New York

Cast-in-place concrete technology has been discussed; however, a second option for bored tunnels, usually in soil, is precast concrete tunnel liner segments. Depending on the use of the tunnel, these segments may serve as the final lining, as is the case for subway tunnels, or a cast-in-place liner may be installed usually to maximize hydraulic conductivity. The concrete quality in the segments is usually much higher than cast-in-place concrete because it is manufactured under much tighter quality control and; therefore, leakage through the concrete is usually not an issue. However, there are many more joints with a segmented liner and therefore, many more potential leakage paths as illustrated in Fig. 10-19.

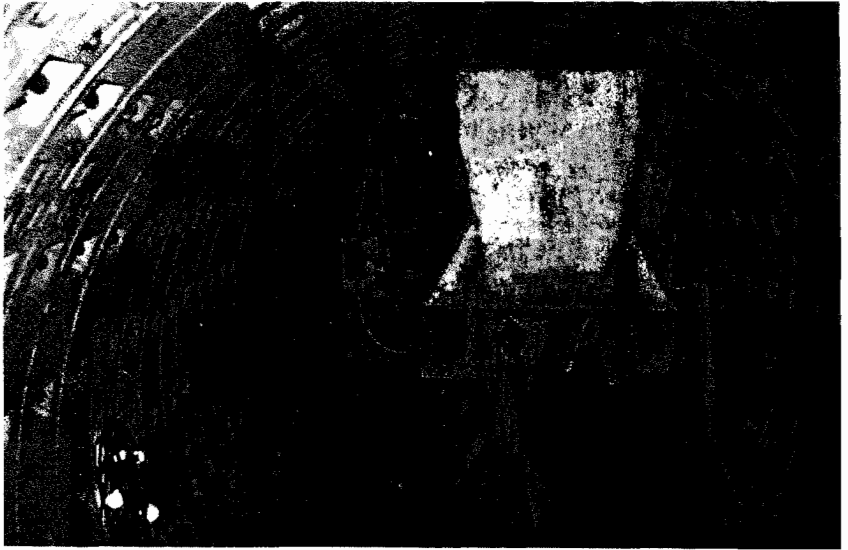


Fig. 10-19 Precast Concrete Segment Lined Tunnel, Baltimore, Maryland

Close casting tolerances, i.e. dimensions controlled to an accuracy of 0.016 in. (10.4 mm) and gasket designs make it possible to achieve prescribed allowable infiltration specifications. A typical gasket is illustrated in Fig. 10-20. If a cast-in-place liner is required, a second line of defense is provided and grouting may be necessary to control infiltration to desired limits depending on the tunnel function.

Prevention of groundwater infiltration into completed bored tunnels, particularly in rock, has been a challenge that designers and builders had not been able to completely master until the 1970's in Europe and the 1980's in the U.S. The challenge has been mastered with the use of multilayered, geomembrane systems first used by Swiss and Austrian engineers in the construction of road tunnels through the Alps and later in subway and railroad tunnels (Sauer, 1987, pp. 461-478).

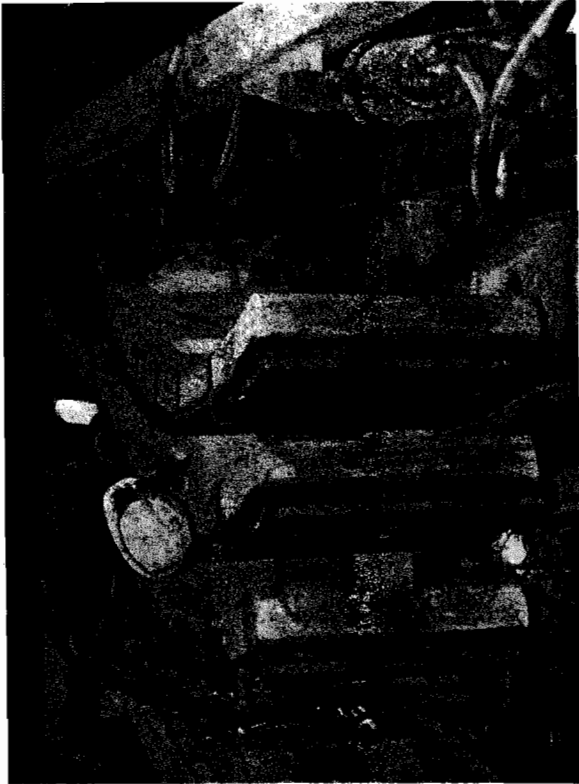


Fig. 10-20 Precast Concrete Segment Gasket

The membrane system was first used in the U.S. for the Washington Metropolitan Transit Authority's (WMATA) contract B-10a. A detail of the system as implemented is illustrated in Fig. 10-21 (Sauer, 1987, pp. 469). It is a two-layer system consisting of a "fleece" layer nailed to the shotcrete to serve as a drainage medium with a plasticized PVC unbonded membrane over the "fleece". The entire system is then encased by the final cast-in-place concrete lining. The system has proven to be very effective in totally controlling groundwater infiltration where it has been properly installed.

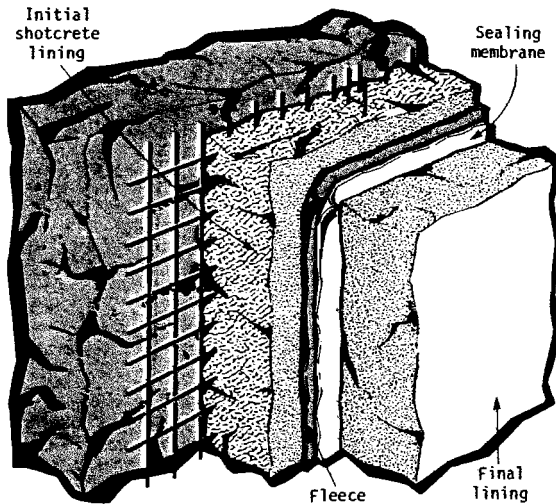


Fig. 10-21 Tunnel Membrane Detail (Martin, 1987)

The "fleece" which is a needle-punched or bonded, non-woven polypropylene geotextile serves to protect the PVC liner from sharp protrusions on the shotcrete surface and to promote drainage down to lateral invert drains. It is fixed to the shotcrete lining by PVC disks which are held in place by fixing nails gunned into the shotcrete as illustrated in Fig. 10-21. The PVC disks are designed so that the PVC liner can be supported by them.

The PVC membrane used in Washington was a plasticized (softened) PVC; however, high density polyethylene (HDPE) and ethylene copolymer bitumen (ECB) have been used (Sauer, 1987, p.471). The liner serves to prevent intrusion of water and to serve as a slip layer between the final lining and the shotcrete rock support which is believed to reduce shrinkage cracking. The lining in place on contract B-10a is illustrated in Fig. 10-22. This system has subsequently been specified in at least four other transportation tunnels under design or construction in the U.S. in 1988.



Fig. 10-22 Tunnel Membrane in Place, Washington, D.C. (Strasser, 1988)

10-3.7 Waterproofing Costs

Some typical costs for waterproofing reported by Anderson (1983, pp. 12) and Sauer (1987, pp.468) are summarized in Table 10-8. These costs have been corrected to 1988 using the Engineering News Record construction cost index. They are intended as guidelines only and should not be used for actual cost estimating. Individual conditions for a single project can cause these approximate figures to vary widely.

Table 10-8
Approximate Waterproofing Prices per Square Foot of Surface Area

| <u>Waterproofing Method</u> | <u>\$/Sq. Ft.</u> | <u>Reference</u> |
|-----------------------------|-------------------|------------------|
| Dampproofing | 0.30 - 0.35 | Anderson, 1983 |
| Dampproofing w/ Poly | 0.35 - 0.50 | Corrected to |
| Rubberized Sheet Asphalt | 1.25 - 1.50 | 1988 assuming |
| Polyurethane(moisture cure) | 1.10 - 1.40 | 22 percent |
| PVC Sheets | 1.30 - 1.45 | inflation. |
| CPE Sheets | 1.45 - 1.85 | |
| Bitumen Sheet | 1.40 - 1.60 | |
| Bentonite Panels | 1.25 - 1.45 | |

Table 10-8 (CONT'D.)
Approximate Waterproofing Prices per Square Foot of Surface Area

| <u>Waterproofing Method</u> | <u>\$/Sq. Ft.</u> | <u>Reference</u> |
|-----------------------------|-------------------|------------------|
| Bentonite Spray | 1.25 - 1.50 | |
| Cementitious (2 Coat) | 1.15 - 1.90 | |
| Fleece/PVC liner | 1.75 - 2.50 | (Sauer, 1987). |

10-3.8 Contractual Considerations

Waterproofing specifications and acceptance criteria for bored tunnels and cut-and-cover tunnels will be very different reflecting the difference in the two types of tunnels. Typically driven tunnel specifications will be of the performance type wherein an acceptable infiltration rate must be achieved by the Contractor. Typical acceptance criteria were presented in Table 10-5. This approach may change with the wider acceptance of the membrane system first introduced to the U.S by Austrian engineers on WMATA contract B-10a.

Waterproofing of cut-and-cover tunnels is typically similar to specifications for building foundations. The waterproofing must not permit any leakage and the materials and methods are tightly specified.

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Chapter 11

INSTRUMENTATION

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11-1 INTRODUCTION

Tunneling is a difficult and hazardous art. Locations, alignments, and dimensions are usually dictated by demographic considerations, rather than by costs, safety, or construction efficiency. Sites are often quite inaccessible, either because of terrain or because of urban development. Geologic information is usually incomplete, and sometimes virtually non-existent. Considering also that in typically restricted tunnel working spaces even a minor difficulty can involve major safety hazards and cost overruns, it is perhaps not surprising that tunneling is one of construction's most hazardous and economically precarious undertakings.

Many tunneling problems are caused by unexpected changes in the strength or deformability of the rock or soil mass in which the tunnel is constructed. When such a mass is disturbed, for example by excavation of a tunnel, it undergoes a redistribution of stress, accompanied by a change of shape. The change is typically inward, tending to close the opening and restore a triaxially confined medium in which a stable stress field can be reestablished. Shape changes can be either inconsequential or catastrophic, depending on the distribution of stresses in the mass, its strength, deformability and anisotropy, the design and orientation of the tunnel, the effect of associated structures, and the extent to which hazardous or costly changes can be detected promptly and precautionary or remedial measures taken.

Fortunately, most shape-changes are reflected in geologic and structural displacements, stresses, strains, and pressures which can be measured using existing instrumentation. Early detection of changes in any of these parameters is of great value not only in the identification of potential hazards, but in devising remedial measures and confirming their effectiveness. The same instrumentation can be used to evaluate the adequacy of support, the behavior of associated structures, and the efficiency of construction methods and their implementation.

Instrumentation need not necessarily be elaborate or costly. It must however, be carefully organized, and program requirements must be foreseen

far enough in advance so that instruments and accessories are on hand when needed and so that provisions for their installation, operation, and maintenance can be incorporated into the contract documents.

11-2 APPLICATIONS

11-2.1 Initial Considerations

Most rock instrumentation is undertaken for one or more of the following reasons:

- (i) Identification of rock and soil mass properties such as strength, deformability, anisotropy, and alterability.
- (ii) Identification of the state of stress in a mass.
- (iii) Measurement of the response of a mass to natural or man-made disturbances.
- (iv) Measurement of the response of a man-made structure to changes in an associated mass.
- (v) Measurement of the response of a mass, an associated structure, or both, to conditions related to the operation of the completed structure.
- (vi) Measurement of the stability of preexisting structures or services located in the zone or potential zone of influence of disturbances.
- (vii) Detection of hazards, either existing or potential.
- (viii) Identification of remedial measures.
- (ix) Verification of the effectiveness of remedial measures.
- (x) Long-term monitoring, both of geologic masses and of associated man-made structures.

Table 11-1 is a list of instruments for typical measurements. Important advantages and disadvantages of each of the principal instruments and applications are indicated by a letter code.

11-3 HARDWARE

11-3.1 Requirements in Common

Regardless of type, style, or design, all instruments have certain general requirements in common. These are:

- (1) Range: The need for adequate measuring range is often overlooked, possibly because load and deformation magnitudes cannot usually be determined in advance of construction and because these values are underestimated more often than overestimated. Range must be adequate, but not excessive because

TABLE 11-1.

Geotechnical Instruments and Instrumentation

| <u>Parameter</u> | <u>Generic Instrumentation</u> | <u>Advantages/Disadvantages</u> |
|------------------|--------------------------------------|--|
| Deformability | Plate Jacking | A b g k |
| | Radial Jacking | A b g k |
| | Dilatometer | B g <u>p</u> |
| | Flat Jack | O k |
| | Pressuremeter | B |
| | Plate Loading | A b k |
| | Torsional Shear | b k |
| Stress | Overcoring | B N a c d e f k p s |
| | Slotting | a b d s |
| | Hydrofracturing | B s |
| Stress Changes | Stress Meters | F J <u>M</u> <u>N</u> d n q |
| | Total Pressure Cells | F <u>J</u> <u>M</u> <u>N</u> d n q |
| | Pore Water Pressure Cells | F <u>J</u> <u>M</u> <u>N</u> d q |
| | Other Piezometers | M l s |
| | Hyd./Pneum. Borehole Cells and Jacks | B <u>H</u> <u>L</u> c d |
| Load Measurement | Load Cells | A H J K b d e f j |
| | Flat Jacks | A O a b d j |
| | Strain Gauges | E a c d f j |
| Load Application | Flat Jacks | A O a b d j |
| | Rams | K P b k |
| | Packers | B |
| | Dead Weight | A b k |
| Deformation | Borehole Extensometers | A B C D <u>F</u> <u>H</u> <u>I</u> J K L f s |
| | Borehole Inclometers | A B C D <u>H</u> <u>I</u> J <u>K</u> <u>L</u> e f h l s |
| | Borehole Deflectometers | A B G <u>H</u> <u>I</u> J <u>K</u> <u>L</u> f i <u>l</u> s |
| | Strain Meters | F H K <u>M</u> N d e f q |
| | Strain Gauges (surface) | F a c d f j |
| | Plumb Lines | A C D K L P |
| | Collimated Optical Gauges | l |
| | Tilt Meters | A I J K L b e |
| | Settlement Gauges | A B C <u>D</u> l |
| | Photoelastic Gauges | P b d j l s |
| | Brittle Devices (failure indicators) | B <u>H</u> <u>f</u> <u>s</u> |

Key to Advantages ^{1/}

- A - Averages response of large volume of material.
- B - Useable at depth.
- C - Data can have high order of reliability.
- D - Data tends to be self-checking.
- E - Requires little maintenance.
- F - Some dynamic capability.
- G - Useable in holes of any inclination.
- H - Some versions well-suited to monitoring and alarm applications.
- I - Can be calibrated or recalibrated while in use.
- J - Use or installation can be accomplished without special space provisions and with little or no delay to construction.
- K - Automated readout is practicable at low or moderate expense.
- L - Can be retrofitted with remote readout.
- M - Inherently long-lived.
- N - Extreme resolution (sensitivity) possible.
- O - Capable of producing or measuring large forces distributed over large areas.
- P - Hardware comparatively inexpensive.

Key to Disadvantages ^{1/}

- a - Subject to large sampling error.
- b - Practicable for near-surface use only.
- c - Data may be of qualitative value only.
- d - Spurious data difficult to recognize.
- e - Unusually vulnerable to mechanical damage.
- f - Requires good waterproofing.
- g - Very expensive.
- h - Useable only in vertical holes or--when manufactured--can be made for single narrow band of inclinations other than vertical.
- i - Error accumulates exponentially in solution. Special procedures required.
- j - Cannot be checked or recalibrated after initial installation.
- k - Use or installation requires special chamber or creates substantial interference with construction.
- l - Instrument or hole collar must be accessible for readout.
- m - Remote readout usually not practicable.
- n - When used in massive concrete, special provisions are necessary to avoid errors due to hydration temperatures and honeycombing.
- o - inherently short-lived.
- p - High-cost components at risk in borehole.
- q - Subject to calibration errors due to creep in transducer.
- r - Subject to errors due to zero shift.
- s - Involve significant drilling costs.

Note: ^{1/} Comments underlined (n) apply only to certain instruments in category.

range must often be obtained at the expense of some other essential characteristic, such as sensitivity.

(ii) Sensitivity: Sensitivity, also called "resolution" denotes the smallest change of measurement an instrument can detect (but not necessarily repeat). Good resolution is essential for early detection of hazards. Excessive sensitivity, like excessive range, should be avoided as it may involve undesirable trade-offs of range and repeatability.

(iii) Repeatability: Repeatability denotes the smallest reading which can be consistently reproduced provided the parameter being measured has not changed. Perhaps more than any other characteristic, repeatability determines measurement quality.

(iv) Accuracy: The concept of instrument "accuracy" is not a simple and straightforward one. The relevant hardware essentials are repeatability and calibration consistency, and most commercial instruments are repeatable and well-calibrated. Problems arise, however, when the conditions of operation are appreciably different than conditions under which the instruments were calibrated, and when the characteristic anisotropy of a geologic mass cannot be defined with the same precision required of the measurement hardware. A not infrequent result is that instruments are blamed for what in fact are deficiencies in their selection and application. Excessively restrictive hardware specifications may be unrealistic unless all of the aspects of their application, as well as the anisotropy of the mass, can be established and controlled with more or less equivalent precision.

(v) Survivability: Instruments must be able to survive for the duration of their prospective use. This is not a simple requirement, as experience indicates beyond question that field conditions are almost invariably more severe than anticipated.

11-4 TUNNEL INSTRUMENTATION

11-4.1 Initial Considerations

In choosing parameters to be measured, and instruments for the measurements, four main factors have to be considered:

(i) Measurements should be capable of providing information on an entire mass or structure, rather than on "typical" or "test" sections.

(ii) Measurements should provide good sample distribution, even under the conditions of limited or otherwise restricted site accessibility.

(iii) Measurements should be redundant enough so that important decisions need not be made on the basis of inconclusive or fragmentary information.

(iv) Costs must not be excessive or unreasonable.

These requirements can usually be met more easily by measurements of shape-dependent than of stress-dependent parameters. This is because shape changes are typically larger in scale, greater in extent and more systematic than equivalent changes in most stress-related parameters. Moreover, shape change measurements can be made with consistent levels of precision using many different types of instruments in widely varying applications and under widely differing conditions of operation.

Three types of shape change measurements have proven to be particularly useful in tunnel instrumentation. These are convergence measurements using tape and bar extensometers, axial borehole deformation measurements using borehole extensometers, and transverse borehole deformation measurements using borehole inclinometers. These instruments are described in the following sections, together with comments on instrument selection and installation, data representation and, and at least the preliminary interpretation of the data obtained.

11-4.2 Convergence Measurements

Good, quick, and economical indications of rock or soil mass stability and of support system effectiveness can be obtained by simple measurements of wall-wall and roof-floor convergence, using tape extensometers or bar extensometers. The measurements can be made in constricted areas (for example in the space between a TBM head and shield) and the number and distribution of measuring points can be easily increased, decreased, changed or otherwise modified to adapt to changing conditions and requirements.

A typical convergence measurement station consists of one or more pairs of reference points, typically arranged to permit measurements of distance across a tunnel (or other opening or excavation (Fig. 11-1)). The distances between opposing pairs of points are measured periodically, using a tape extensometer or bar extensometer, and noted in relation to time, face distance or other parameters. Time is particularly useful, as it facilitates the comparison of convergence data with information from other sources.

(i) Selection and installation. Fig. 11-2 shows a tape extensometer and one type of reference point. The extensometer consists of a tensioning head, a steel tape, and two snaps or hooks for fastening the end of the tensioning head to one reference point and the far end of the tape to the other. Reference points can be grouted or mechanically anchored in short drill holes, or attached to linings, ribs, or other structures.

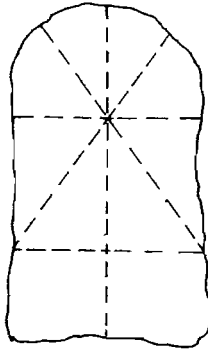


Fig. 11-1. Typical Convergence measuring lines. Lines may be modified as necessary to avoid interference with construction services and to protect measuring points in vulnerable locations.

In use, the far end of the tape end is snapped onto one reference point and the tensioning head onto the other. The actual measurement consists of the sum of a coarse increment of length, measured by hooking the tensioning head into a perforation at an even-numbered tape graduation and a fine increment of length, measured using a micrometer or dial gauge incorporated into the instrument head. Prior to measuring the fine increment, the head mechanism is adjusted to apply a predetermined, carefully controlled spring tension to the tape. Deformation is detected by comparing successive measurements between the same reference point pairs.

Fig 11-3 shows another convergence measuring device, the bar extensometer, and a typical reference point. A bar extensometer consists of two telescoping sections, either or both of which can be lengthened or shortened by adding or subtracting extensions. The outward tips of the two telescoping sections terminate in either hardened steel points, to engage punch marks in reference points, or in snap fasteners to engage mating fasteners on reference points. A two-piece micrometer carriage is fixed in place at the inner end of the larger diameter telescoping section and engaged by means of a clamping bolt to the inner end of the smaller telescoping section.

In use, the bar extensometer is engaged between a pair of reference points, using snap couplings or hardened steel tips. Two increments of length are measured; a coarse increment, made using graduations on the smaller diameter telescoping tube and a fine increment, made using the carriage micrometer. The

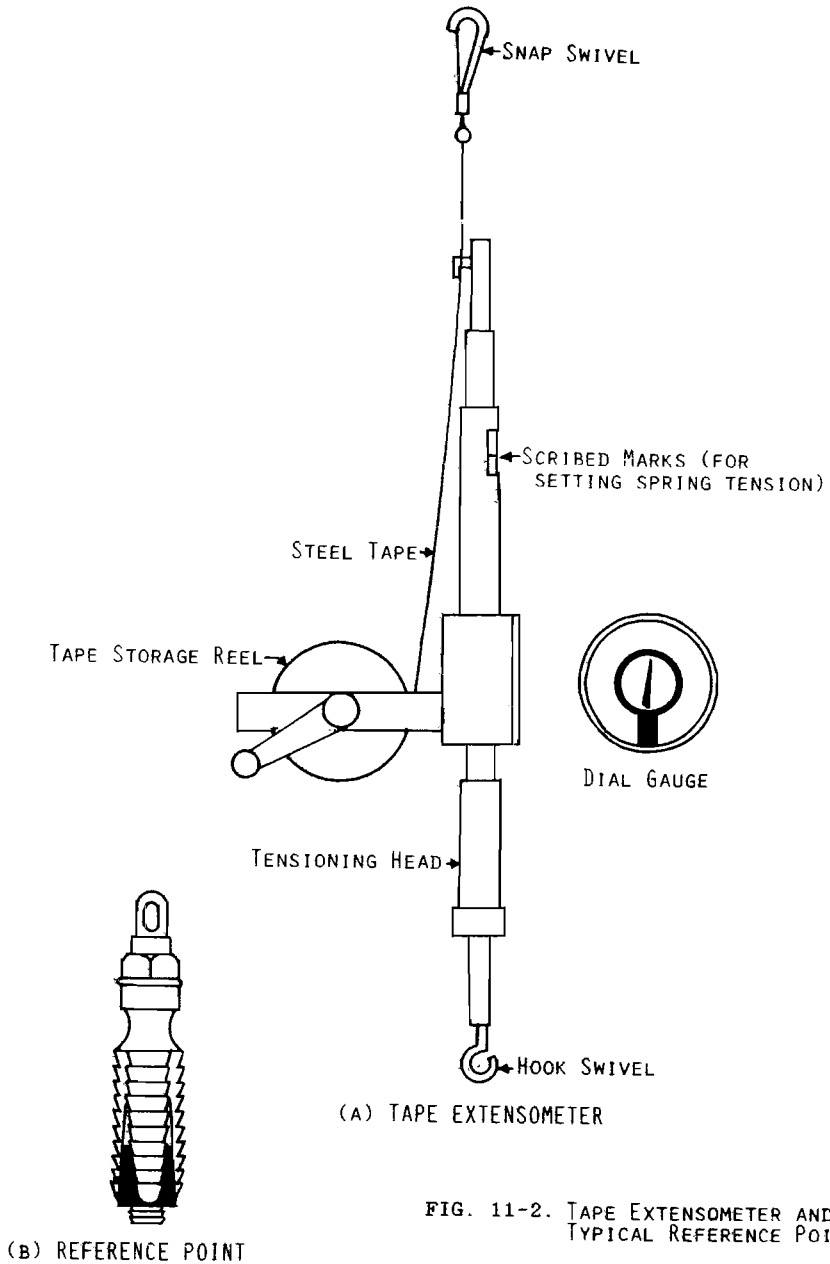


FIG. 11-2. TAPE EXTENSOMETER AND TYPICAL REFERENCE POINT

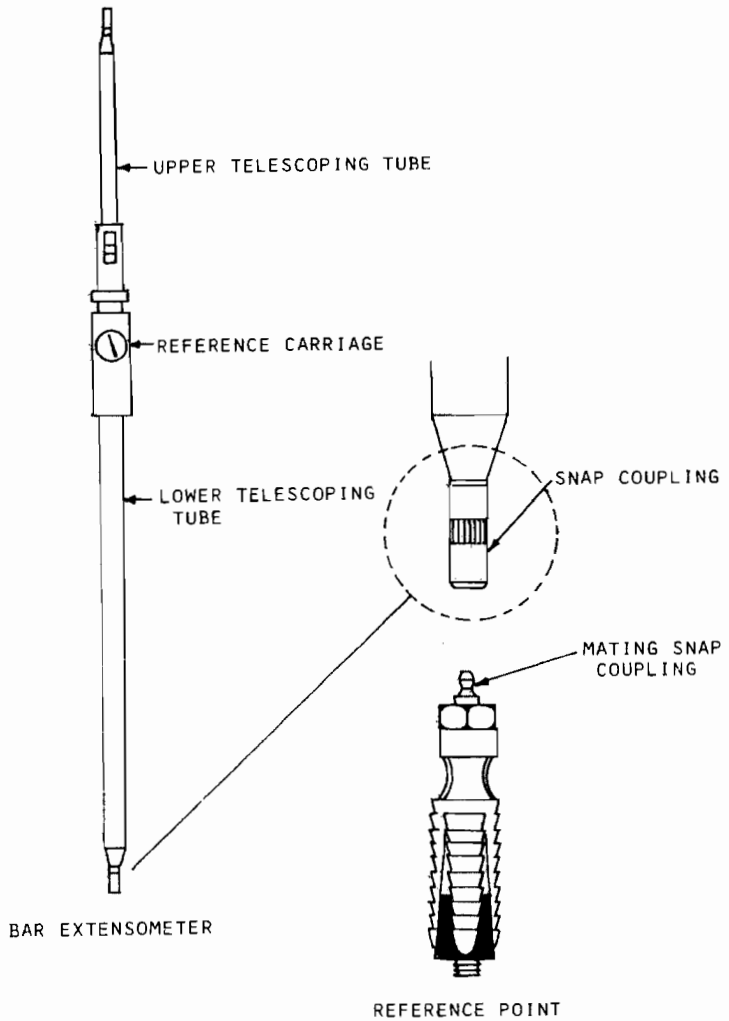


FIG. 11-3. BAR EXTENSOMETER AND TYPICAL REFERENCE POINT.

two increments are summed, and the resulting measurement is recorded as a function of time, face distance, or other parameter. Successive measurements are compared to identify deformation.

(ii) Data representative and interpretation. The apparent relationship between convergence (radial deformation) and certain other tunnel parameters is shown in Fig. 11-4 (after Daley and Abramson, 1985; Daemen, 1977; Fenner, 1938; Pacher, 1964 and others).

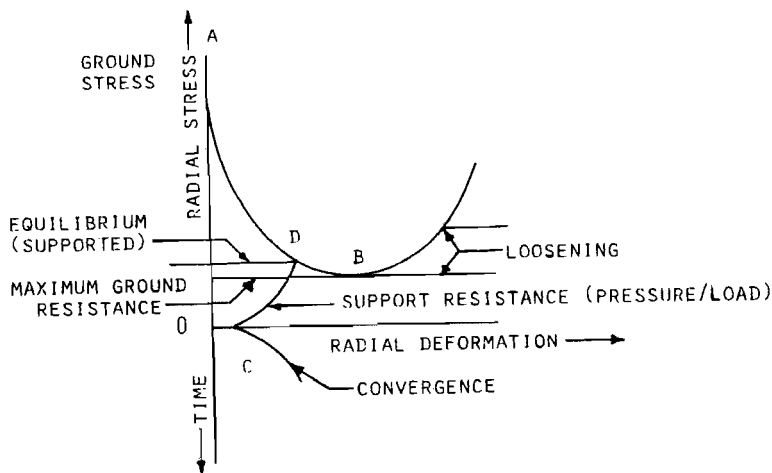


FIG. 11-4. HYPOTHETICAL GROUND-SUPPORT REACTION CURVE.
AFTER DALEY AND ABRAMSON, 1985.

Convergence-confinement relationships have been described in detail in Chapter 2, and readers are referred to Sections 2-3 and 2-11, and Fig. 2-1 for that discussion. Briefly, it is postulated that when an opening is excavated in a mass, the stress in the mass immediately adjacent to the opening is reduced by an amount which at least initially tends to be proportional to the radially inward deformation (convergence) of the opening walls. At some point, continued convergence may significantly disrupt and loosen material in the tunnel walls, initiating a new cycle of adjustments which may result in the transfer of potentially dangerous and damaging additional load to the support system. An efficient support system is one which utilizes the maximum ground resistance; i.e., one which is installed and fully coupled to the surrounding material before any significant loosening has developed.

The apparent relationship between convergence and radial deformation should be used somewhat cautiously, as it is subject to some influence by irregularities in the shape of the tunnel, by local geologic features, and by the effect of associated structures.

Tape or bar extensometer convergence information can be represented in graphs showing changes of distance versus time, elapsed time, face distance, or other parameter. Time is particularly useful, as it permits the direct comparison of convergence information with data from other sources. In tunneling, face distance is a useful secondary variable because it permits the comparison of responses in different intervals, and possibly in different materials to possible influences of conditions in unexcavated material ahead of the face.

Fig. 11-5 shows two graphs of wall-wall convergence versus face distance in a small-diameter (12 ft) tunnel in a predominantly granitic rock mass. Curve "A" shows convergence in a moderately supported section (4-I-7.7 ribs 2.0 ft O.C.) while Curve "B" shows convergence in unsupported ground. The principal geologic difference is fracture spacing, which averaged approximately 0.2 foot at the site of Graph "A" and 1.6 feet at the site of Graph "B".

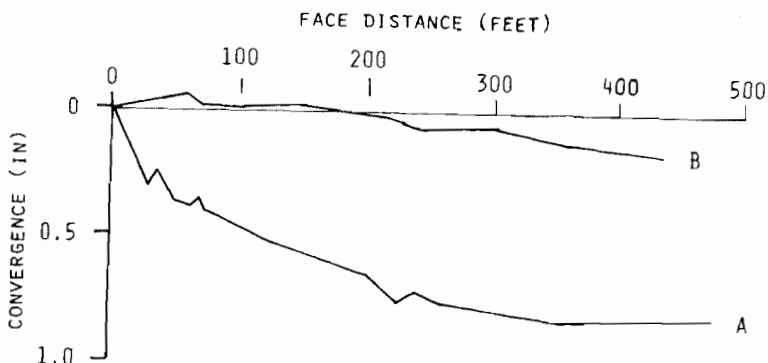


FIG. 11-5. TUNNEL WALL-TO-WALL CONVERGENCE, VERSUS FACE DISTANCE. CURVE "A" REFLECTS MEASUREMENTS IN A STEEL RIB SUPPORTED TUNNEL SECTION. CURVE "B" REFLECTS MEASUREMENTS IN UNSUPPORTED GROUND. AFTER DUTRO AND PATRICK, 1982.

In graphs of convergence versus time, the actual traces show magnitude (of change), the slope of the traces show change rate, and variations (curvature) in slope show acceleration. In the interpretation of the data, magnitude and rate are parameters which are strongly dependent on the specific nature of the geological materials. Large magnitudes and high rates are not in themselves indicative of possible failure. Acceleration, on the other hand, is largely independent of materials characteristics, and so is an invaluable indicator of existing or imminent problems. Acceleration, if continued, inevitably ends in failure. In the measurement of convergence, therefore, as in the measurement of many other geotechnical parameters, it is essential that acceleration be detected at the earliest possible stage in order to provide a maximum amount of time for warnings to personnel, for safeguarding equipment, services and structures, and for remedial action.

Convergence measurements are subject to systematic errors due to variations in sag and variations in temperature. The effects of sag can be minimized by being sure, in successive measurements, to occupy sets of reference points in the same directions (i.e., left-to-right or right-to-left). Temperature is not usually a large source of error in most tunnel applications. Its effect can, however, be adjusted out of the data using correction factors either supplied by the instrument manufacturers or determined experimentally.

11-4.3 Borehole Extensometers

Disturbances in rock and soil masses may be influenced by geologic or structural factors outside the immediate periphery of a tunnel. Simple measurements, such as convergence, may detect the disturbance but fail to provide much information on its exact nature, or on its causes and potential precautionary or remedial measures. In such instances, instruments installed in boreholes can be used to test ground outward for distances of as much as many hundreds of feet into a mass, and to provide good sample distribution and data redundancy even under conditions of limited site accessibility.

Fig. 11-6 shows one such instrument, the multiple position borehole extensometer. A typical borehole extensometer consists of an instrument head, usually mounted at the collar of a drill hole, and one or more in-hole anchors, each fixed in position at a known initial depth in the hole. Each anchor is connected by means of a rod or wire to an individual transducer in the instrument head. As the rock or soil mass is deformed, the distance between each in-hole anchor and the instrument head changes, and the changes are measured by the individual transducers. Various types of transducer can be used, ranging from mechanical devices such as vernier calipers and depth micrometers to electronic

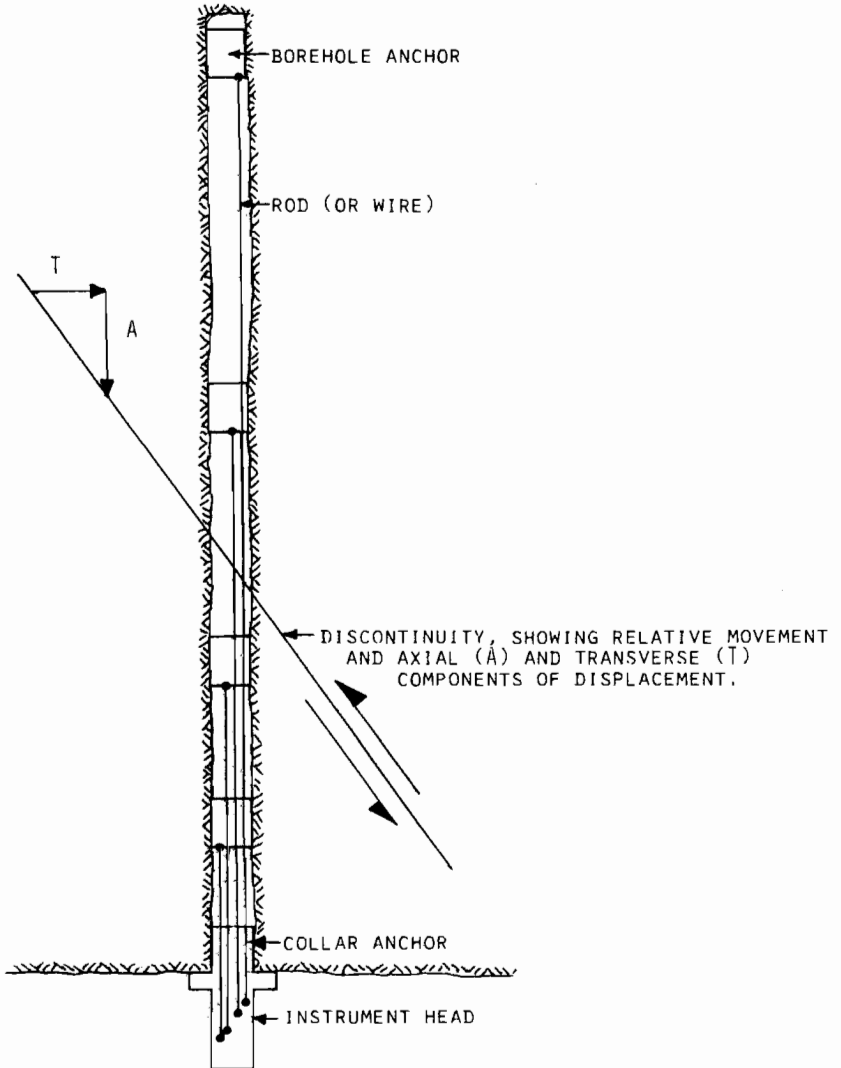


FIG. 11-6. TYPICAL BOREHOLE EXTENSOMETER (DIAGRAMMATIC).

sensors such as bonded and weldable strain gauges, potentiometers, and linear variable differential transformers (LVDT). Mechanical transducers have the advantage of simplicity and low cost. Electronic transducers permit remote readout, the use of automatic and semi-automatic data systems, and simple interfacing with computers.

Extensometers provide direct measurements of displacement magnitude, usually noted in relation to time. Changes in time and magnitude are used to calculate deformation rate (time rate of change of magnitude) and acceleration (time rate of change of the deformation rate).

(i) Selection and installation. The selection of an extensometer for a particular application requires consideration of the nature of the geologic mass, desired hole depth, number of anchors, probable deformation magnitude, required sensitivity, readout mode (mechanical or electronic), probable useful life of the installation, and any special requirements (such as measures to limit water migration along extensometer boreholes).

Extensometers measure only that component of displacement which is acting parallel to the axis of the borehole. Holes should be drilled to intersect zones of probable deformation at angles which will insure that a measurable component of displacement is oriented parallel to the hole and still cross geologic structure (rather than narrowly intersect a single zone or discontinuity). An optimum intersection angle is approximately 35 degrees, at which the measurable component comprises approximately 82% of the total displacement and the extensometer crosses approximately 57 feet of structure perpendicular to a discontinuity for every 100 feet of hole length. Good results are also possible in holes intersecting zones and discontinuities at greater or lesser angles, subject to the general limitation that intercepts of less than about 20 degrees or more than about 60 degrees are vulnerable to special problems.

(ii) Data representation and interpretation. Extensometer measurements are in the form of displacement magnitude, usually noted in relation to time, face distance, pool elevation, or other parameter. Time is generally the most useful independent variable, and it facilitates the comparison of extensometer data with information from other sources.

The information necessary for at least a preliminary evaluation of safety and stability can usually be extracted quite easily from the raw (field book) data, or from simple graphs of displacement versus time. Hazards are usually reflected in unprocessed data in stepwise or exponential changes in successive readings, and all data should be scanned routinely for any such indications.

Perhaps needless to say, early detection is extremely important in providing a maximum amount of time for precautionary or remedial actions.

To prepare displacement graphs, a reference datum must be identified and a format selected. In most extensometer applications, the instrument head is located in the part of the geologic mass which is being most actively deformed. If possible, extensometer holes should therefore be drilled deep enough to place the deepest in-hole anchor well beyond the "zone of influence" of the deformation. The furthestmost anchor or anchors are then in locations which are least comparatively fixed in space. The graph showing displacement of the deepest anchor can then be plotted as a straight line and used as a reference datum or ordinate for the calculation of displacements measured at successively shallower anchor depths. Fig. 11-7 show two displacement graphs plotted using this format.

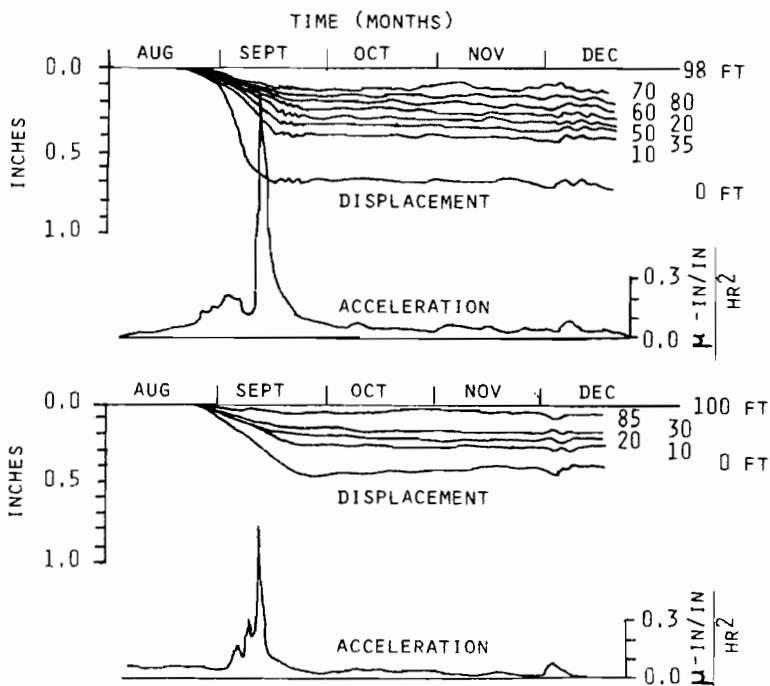


FIG. 11-7. MULTIPLE POSITION BOREHOLE EXTENSOMETER DATA. GRAPHS OF DISPLACEMENT AND ACCELERATION VERSUS TIME. DOWNWARD DISPLACEMENTS REFLECT AXIAL BOREHOLE EXTENSION. ACCELERATION MAY BE EITHER POSITIVE ("ACCELERATION") OR NEGATIVE ("DECELERATION"). ACCELERATION IS IN A SINGLE INTERVAL COMPRISING THE ENTIRE DEPTH OF THE BOREHOLE. AFTER DUTRO AND DICKINSON, 1974.

In the graphs, displacement magnitude is plotted, for each in-hole anchor and for the instrument head, as a function of time (months). Rate is indicated by the slope of a curve and acceleration (sometimes called "strain change rate") by its curvature. For each graph, a separate acceleration curve shows the average or net acceleration in the single interval comprising the entire length of the hole. Single-interval graphs are useful for comparing events and trends in different holes, for comparing conditions in the same hole at different times, and for correlating deformation with specific events, activities, or other influences.

In interpreting extensometer data, deformation magnitude and rate are materials-dependent parameters, and may not therefore necessarily be useful in indicating the likelihood of a specific failure. Acceleration, on the other hand, is essentially independent of mass properties. Acceleration, if continued, invariably and inevitably ends in a failure. The most important objective in extensometer data interpretation is the early detection of acceleration.

Another important objective is the detection of inelastic rock or soil mass behavior, particularly in masses such as dams and slopes, which may be subject to seasonal, cyclic, or periodic disturbances. Yet another is the identification of potential failure mechanisms by comparing current displacements and trends with evidence from past or previous failures.

A related application is the use of borehole extensometers in the investigation of rock and soil mass mechanical properties. Extensometers can be used, for example, to measure the deformation produced by controlled loads applied by dead weights, borehole jacks, hydraulic pressure, or heat.

Extensometers are versatile, economical, and in many ways unique in their usefulness in the observation of large masses and structures. Used either alone or in conjunction with other instruments, they permit measurements not only of the effects of a variety of natural and man-made disturbances, but of other design construction, and operational factors and influence as well.

11-4.4 Borehole Inclinometers

In the previous section, borehole extensometers were described as being most effective in boreholes intersecting potential deformation zones or discontinuities at moderate to comparatively low angles. This suggests the need for a complementary measurement system, specifically one which would be most effective in intersections at high or comparatively high angles. Such a system does in fact exist. Just as extensometers measure most efficiently that deformation component which is axial to the instrumented borehole, borehole

inclinometers measure most effectively that component which is transverse to the hole. Used in combination the two instruments make it possible to develop programs for which neither instrument by itself would be completely effective. Together, these versatile instruments can be used to test large or remote masses from a limited selection of sites, to provide good sample distribution and data redundancy, and in general permit a wide variety of useful practical applications.

A borehole inclinometer is a pendulum-actuated instrument in which the deflection of a mass from the vertical is detected and measured in relation to the axis of a borehole. Inclinometers are available as either vertical or horizontal traversing instruments (sondes) or stationary ("in place") arrays made up of interconnected sonde-like elements. Fig. 11-8 shows a typical traversing inclinometer. The inclinometers are designed to be traversed in grooved aluminum or plastic casing. Wheels on the inclinometer engage the casing grooves to maintain the sonde in the desired plane or planes of measurement. "In place" inclinometers are similar in principle, but consist of a succession of interconnected elements, designed to be installed in the casing and left in place until measurements are no longer necessary. Traversing inclinometers are economical, because one sonde can be used in any number of cased or embedded casings. On the other hand, a considerable amount of time is required for traverses, and the instruments can be somewhat cumbersome to transport from site to site. Moreover, if hazardous conditions are detected, access to the site for further measurements may not be possible.

"In place" inclinometers, on the other hand, can be read out quickly and remotely and in fact are often outfitted with automatic alarm circuits and set up to operate unattended. In addition, they can be easily interfaced with a variety of automatic and semi-automatic readout apparatus, telemetry apparatus, and directly with computers. They are therefore particularly useful in many monitoring applications. Unfortunately, they are quite complex and expensive, and this limits their use in what might be termed "operational" instrumentation.

Most inclinometers are designed to operate in vertical or near-vertical holes. A less common version is designed for horizontal holes and embedded arrays.

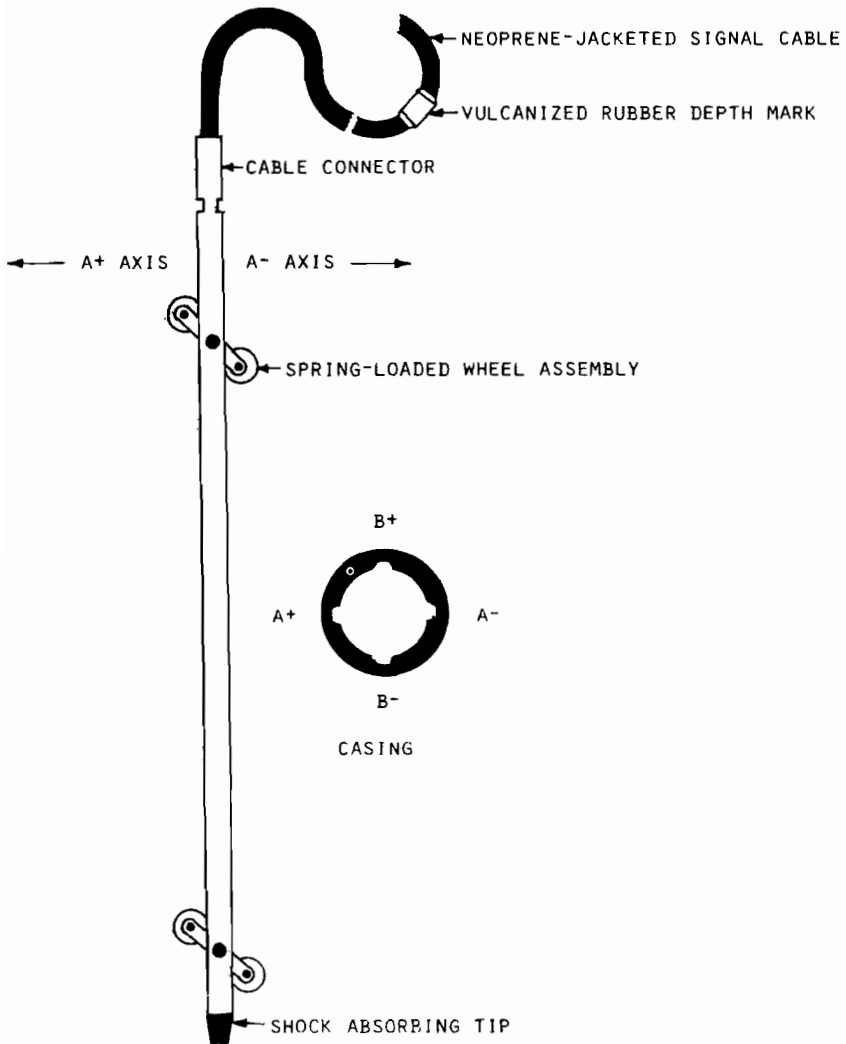


FIG. 11-8. TYPICAL BOREHOLE INCLINOMETER

The transducers in most commercial inclinometers are electronic servo-accelerometers. Each servo-accelerometer consists fundamentally of a mass (pendulum), a circuit to sense the position of the mass in relation to the housing of the inclinometer, and an electromagnet which can be energized to restore the mass to its undeflected position (relative to the accelerometer housing). The mass-sensor-electromagnet system is "looped", so that whenever the inclinometer is turned "on", the sensor-electromagnet subcircuit is energized, thus maintaining the mass in its undeflected position at all times. At any moment, the electrical current necessary to maintain or restore the undeflected position is the analog of the angle of pendulum inclination. A uniaxial accelerometer contains one mass-sensor-electromagnet system, acting in one vertical plane. A biaxial accelerometer contains two such systems. The long axis of each system is parallel to the axis of the probe, with the measuring planes rotated 90 degrees apart to provide X-X' and Y-Y' measurements in the usual grooved inclinometer casing. Most vertical traversing inclinometers and some vertical in-place inclinometers use biaxial accelerometer packages. Most horizontal inclinometers, on the other hand, use uniaxial packages set to provide inclination measurements in one vertical plane coincident with the axis of the casing. The output of the horizontal instrument, however, is scaled to indicate the inclination of the casing from the horizontal, rather than from the vertical as in the other inclinometers.

Sonde-type inclinometers are usually traversed from the far end of a casing toward the near end. At intervals of exactly one probe length ("gauge length"), the transducer is read out and the data either recorded for later processing or processed on the spot to indicate inclinations in the plane or planes of measurement. The inclinations are summed up over the length of the casing to indicate the attitude of the casing in relation to the measurement plane or planes. The results of successive traverses are compared to detect and measure deformation. In stationary arrays the summation procedure is the same but all elements are read out more or less simultaneously.

(i) Selection and installation. Inclinometers can be manufactured in a variety of lengths and diameters, generally in the range of one to several feet in length and two to several inches in diameter. As a rule, the smaller diameters are more suitable for measurements in competent materials. The larger instruments, however, may be used in either competent or incompetent masses.

Readout apparatus ranges from battery-operated hand portable instruments to elaborate data systems, sometimes with integral alarm, data reduction, and telemetry capabilities. Most systems are more or less interchangeable, and less sophisticated equipment can be upgraded as necessary.

Plastic and aluminum casing is available in diameters ranging from about 2 inches to about 3.5 inches. Accessories include special "settlement" couplings, designed to telescope and thus accommodate a certain amount of axial deformation of the casing line while limiting casing distortion. Other accessories include special flanges to maximize coupling between the casing and soft wall or embankment materials, and a variety of grouting, hoisting, and casing handling tools and apparatus.

Casing and borehole diameters are usually selected to provide adequate clearance in the hole for both the casing and any accessories such as hoisting lines, signal cable, and grout tubes. In some applications casing can be suspended loosely in the hole or allowed to rest on the hole walls or bottom, usually so that it can be removed at some later time. Most installations, however, are permanent, and the casing should be well-coupled to the surrounding material. In borehole installations, coupling is insured by sand packing, gravel packing, or grouting the annular space around the casing. Grouting is the preferred means, particularly when water migration along the casing must be minimized. In embedded arrays, good coupling is usually developed when the surrounding material is compacted, and water migration can be controlled by mixing small quantities of bentonite in the backfill at intervals along the casing.

Whenever possible, both borehole and embedded installations should be planned so that at least one end of the casing is located outside the zone of anticipated deformation. That end can then be used as at least a temporary reference for the identification of subsequent deformation.

(ii) Data representation and interpretation. Inclinator data is initially in the form of a succession of vectors, each representing a component of casing inclination between two points one gauge length apart along the casing in the plane of measurement. For each traverse, the vectors are summed to provide an approximate casing "profile", and particularly, the deflection of one end of the casing relative to the other. In most installations, the results of the initial traverse are used as a base line, with which subsequent traverses are compared to detect deformation.

Inclinator data can also be processed to yield deformation rate and acceleration, identify trends, and correlate inclinometer data with other information.

Fig. 11-9 shows both a "quick" plot of changes in reading versus depth and a "profile" developed from the same data. Both are from one traverse, and reflect inclination magnitudes measured in a single vertical plane. Some caution is necessary in interpreting "profiles". In some instances the results of an initial (first active) traverse are plotted by convention as a straight line to facilitate the identification of subsequent deformation. Subsequent plots then reflect only the cumulative deformation magnitudes, and not necessarily the actual physical shape of the holes.

Data from successive traverses can be processed or "profile" compared to identify deformation rate and acceleration. Magnitude and rate are primarily materials-dependent parameters and hence not necessarily good indicators of either actual or potential hazards. Acceleration, on the other hand, is largely independent of materials properties, and so is an extremely useful indicator. Acceleration, if continued, invariably ends in failure. As in the interpretation of other instrumentation data, the primary objective of inclinometer data interpretation should only be early recognition of acceleration in order to provide as much time as possible for warnings and for remedial action.

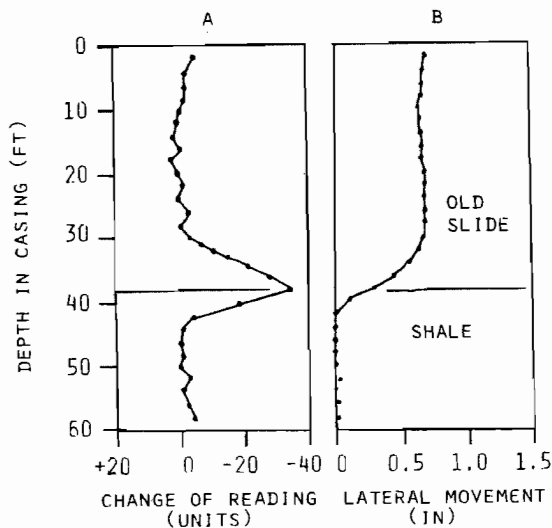


FIG. 11-9. BOREHOLE INCLINOMETER DATA. "A" IS A "QUICK" PLOT OF CHANGES IN READING. "B" IS A "PROFILE", IN ONE PLANE, OF REDUCED DATA PLOTTED AS CUMULATIVE CASING OFFSET. AFTER WILSON AND MIKKELSEN, 1978.

Inclinometers can also be used for the identification of inelastic behavior in geologic masses and structures, for the measurement of rock and soil mass mechanical properties, and for a variety of other measurement and monitoring applications.

11-4.5 Other Instrumentation

In addition to the measurements described in the foregoing sections, other more specialized instruments can be used to define specific rock and soil properties, to observe details of behavior in disturbed masses, to detect or investigate specific problems, to confirm the effectiveness of specific remedial measures, and for long-term monitoring. These include measurements of strength, deformability, load, stress, pressure, deflections, rock noise, permeability, alterability and sonic velocity, as well as the many geophysical techniques such as resistivity, self potential, density, and others. Used singly or in combination, these tests can provide valuable detail in support of the more broadly-based "operational" instrumentation.

In tunnels, load can be measured on ribs and struts, on bolts and on either active or passive reinforcing tendons and anchors. Stress, and stress changes, can be measured in geologic masses, linings, and structures, as can internal or external pressure, thrust, and water pressure. Strain can be measured, either in a geologic mass, a support system, or an associated structure. Although each of these measurements can undoubtedly provide valuable information, their usefulness for day to day operational and monitoring applications tends to be limited by their characteristic variability, and hence the difficulty and expense of obtaining good samples and sample distribution and reasonable sample redundancy.

11-5 INSTRUMENTATION CASE HISTORIES

Each of the following seven case histories consists of a brief explanatory text and one or more illustrations. Although not all of the case histories are drawn strictly from tunneling applications, each illustrates points which are important in tunnel instrumentation. Three of the examples (Libby Dam, the Tehachapi No. 1 - North Tunnel, and Vaiont Dam) show conditions leading up to actual failures. Another (Cabin Creek Pumped Storage Project) shows deformation associated with a probable near failure, as well as the effect of a series of cyclical temperature changes on the measurements. One history (Grand Gulf Nuclear Generating Station) shows rebound induced by removal of material in a deep surface excavation. The final example (Jeffrey Pit) shows the behavior of a well-designed slope in competent rock.

All of the graphs are of deformation versus time, with rate and acceleration evident in the slopes and curvatures of the traces. Effects which appear to reflect dilatancy, compaction, surficial deformation in altered rock, blasting, and instrument response to substantial temperature changes can also be identified.

11-5.1 Case History No. 1

(i) Nature of case history. Case History No. 1 is a record of rock displacements leading up to a failure in the left abutment of Libby Dam, Montana. The data on which the case history is based is courtesy of the U.S. Army Corps of Engineers, Seattle District.

The material is metasedimentary rock, primarily argillite with some quartzite. The slope was underlain by a succession of rock wedges defined by the intersection of a set of moderately inclined bedding plane faults and one or more sets of more steeply-dipping faults or joints.

(ii) Instrumentation. The instrumentation consisted of multiple position borehole extensometers, installed in holes ranging up to many hundreds of feet in depth. The program, now (1989) in its 23rd year, has used many different extensometer styles, readout modes, anchor configurations.

(iii) Summary of record shown. Fig. 11-10 shows displacements versus time

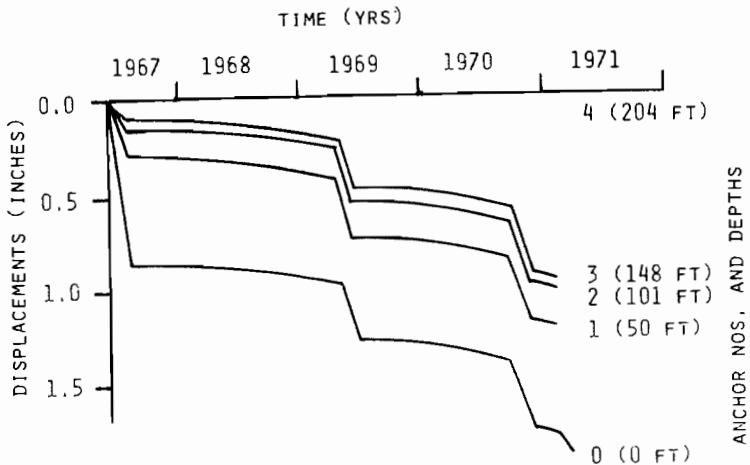


FIG. 11-10. CASE HISTORY NO. 1 - LIBBY DAM LEFT ABUTMENT. DOWNWARD DISPLACEMENTS REFLECT AXIAL BOREHOLE EXTENSION.

(years) for instrument head at surface and anchors at depths of 50, 101, 148, and 204 feet in a 204-foot hole inclined approximately 10 degrees from the horizontal. Instrument was installed in mid-1967 and destroyed by rock slide in early 1971.

Initial displacements (1967) show adjustments to excavation of slope. Mid-1967 to mid-1969 displacements show low rates (slopes) and very low accelerations (downward curvatures), followed by stepwise adjustments. Mid-1969 to mid-1970 displacements again show low rates (slopes) and very low accelerations (downward curvatures), followed by stepwise adjustments. Late 1970 to early 1971 displacements once again show low rates (slopes) and very low accelerations (downward curvatures), culminating in a failure of the slope on January 31, 1971.

11-5.2 Case History No. 2

(i) Nature of case history. Case History No. 2 is a record of the displacements leading up to the failure of a section of the roof and wall of the Tehachapi No. 1-(North) Tunnel, California, during construction. The data is courtesy of the California Department of Water Resources.

The material involved is a lightly weathered, strong, hard, foliated to locally moderately jointed diorite gneiss.

(ii) Instrumentation. Instrumentation consisted of eight position multiple position borehole extensometers equipped for mechanical readout using a dial gauge depth micrometer.

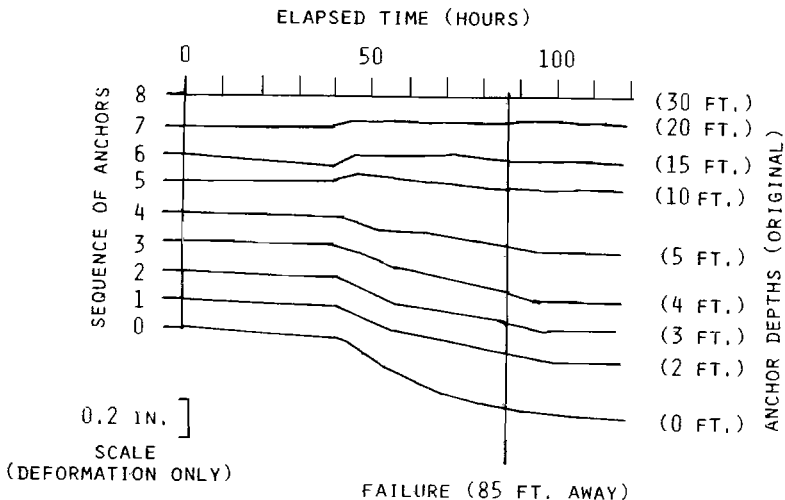


FIG. 11-11. CASE HISTORY NO. 2 - TEHACHAPI TUNNEL NO. 1 NORTH. DOWNWARD DISPLACEMENTS REFLECT AXIAL BOREHOLE EXTENSION.

(iii) Summary of the record shown. Displacement graph showing conditions leading up to the failure of the roof in a small-diameter shotcreted tunnel under construction. Material is described as a lightly weathered, strong, hard, foliated to locally moderately jointed diorite gneiss.

(iv) Summary of record shown. Fig. 11-11 shows displacements versus time (hours) for instrument head at the surface and anchors at depths of 2, 4, 4, 5, 10, 15, 20, and 30 feet in a 30-foot hole drilled vertically upward in the tunnel roof. Early record (0-45 hr.) shows downward deflections in zones defined by Anchors 0 (instrument head) through 4, and by Anchor 6. Divergent trends noted by Anchors 5 and 7. At 45 hr., acceleration (downward curvature) noted in traces of Anchors 0, 1, 2, 3, and 4. Coinciding upward deflections apparent in traces of Anchors 5, 6, and 7. Subsequent data showed increased rates (slopes) and accelerations (curvatures) culminating in a nearby roof failure at 86 hr. The failure initiated 85 feet away in the direction of the face, and propagated in the direction of the extensometer station, stopping approximately 1 foot away. Divergent trends of Anchors 5 and 7 appear to reflect separation of discontinuities with cohesion, causing some rebound of overlying materials.

11-5.3 Case History No. 3

(i) Nature of case history. Case History No. 3 is a record of displacements leading up to a probable near-failure of a highly unstable slope at the Cabin Creek Pumped Storage Project, Colorado, during construction. The data is from Dutro and Dickinson (1974).

The material involved is a fractured and hydrothermally altered rock described as generally hornblende gneiss. One pronounced set of joints was oriented approximately parallel to the valley walls, forming slabs of varying thicknesses. A previous rock slide appeared to have been caused by an excavation-induced disturbance of a similar block.

(ii) Instrumentation. Instrumentation consisted of electronic multiple position borehole extensometers equipped for remote readout.

(iii) Summary of record shown. Fig. 11-12 shows displacements versus time (months) for instrument head at the surface and anchors at depth of 10, 20, 35, 50, 60, 70, 80, and 98 feet in a 98-foot hole. The hole was inclined upward 7 degrees from the horizontal, so that it could also function as a drain.

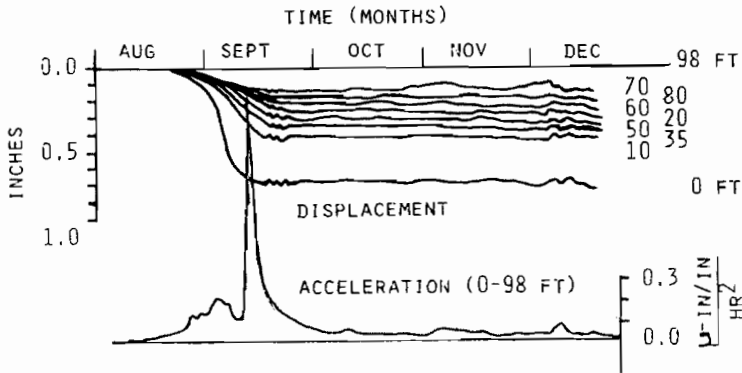


FIG. 11-12. CASE HISTORY NO. 3 - CABIN CREEK PUMPED STORAGE PROJECT. DOWNWARD DISPLACEMENTS REFLECT AXIAL BOREHOLE EXTENSION. AFTER DUTRO AND DICKINSON, 1974.

Fig. 11-12. Case History No. 3 - Cabin Creek Pumped Storage Project. Downward displacements reflect axial borehole extension. After Dutro and Dickinson, 1974.

Initial displacements mid-August to early September reflect adjustments in the slope following excavation of material disturbed by previous slide. In early September, slope received 4 inches of rainfall in a few hours time, leading to the pattern of acceleration (curvature) and changed rates (slopes) shown. The adjustments did not, however, result in a failure. Note the negative acceleration (sometimes incorrectly termed "deceleration") apparent in the traces prior to mid-September.

A separate acceleration graph is shown, indicating the net or average acceleration over the entire 98-foot length of the hole. The graph indicates that the most pronounced acceleration was negative acceleration leading toward stabilization, and not acceleration during development of the displacements.

Saw tooth displacements in October reflect the effect of daily temperature variations of approximately 55 degrees F which were automatically adjusted out of the data during processing but left where shown in the example to provide a record of temperature effects and their potential influence on the measurements.

Somewhat similar expressions in the data in December reflect mechanical interference due to the freezing of drain water in the instrument head, a condition subsequently corrected by the addition of a heat tape.

11-5.4 Case History No. 4

(i) Nature of case history. Case History No. 4 is a record of roof rock deformation in heavy ground in the Straight Creek Tunnel Pilot Bore, Colorado, during construction. The data is from Dutro and Patrick (1982).

The Straight Creek Tunnel Pilot Bore was driven to investigate the site of the present (1989) Eisenhower Memorial Highway Tunnel on U.S. Interstate Highway 70. In the section described, measurements were made of displacements in the roof of a 11-foot (high), 12-foot (wide) tunnel in heavy ground. The rock was described as "80% granite, 15% metasediments with locally as much as

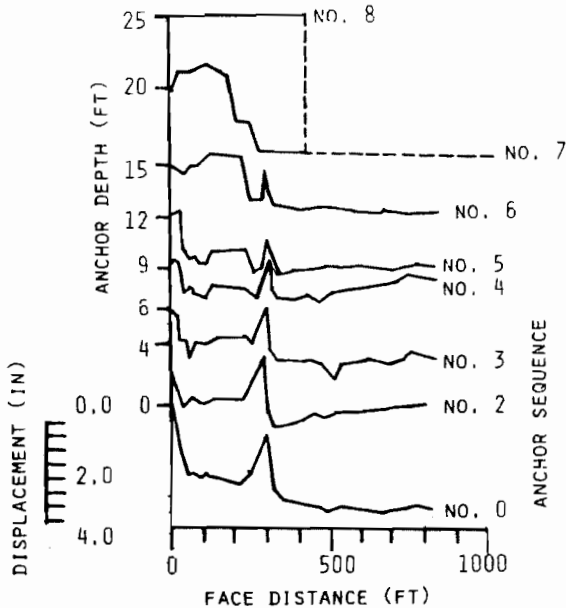


FIG. 11-13. CASE HISTORY NO. 4 - STRAIGHT CREEK TUNNEL PILOT BORE. DOWNWARD DISPLACEMENTS REFLECT AXIAL BOREHOLE EXTENSION. AFTER DUTRO AND PATRICK, 1982.

75% gouge clay". Support was 6 H 25 three-piece (including invert strut) ribs on 1.5 foot centers. Peak rock loads on steel rib support system were approximately 33,000 psf vertical and 51,000 psf horizontal. Subsequent "stabilized" rock loads were approximately 14,000 psf vertical and 49,000 psf horizontal. These loads were substantially greater than the rib design load. As a result, nearby ribs were severely distorted prior to at least the apparent "stabilization" of rock deformation and support loading.

(ii) Instrumentation. The measurements reported were made using electronic multiple position borehole extensometers. Other instrumentation consisted of single position borehole extensometers, prop load cells, and bar extensometers.

(iii) Summary of record shown. Fig. 11-13 shows displacements versus face distance (ft). Face measured using an instrument head at the surface and seven (later six) anchors at depths 4, 6, 9, 12, 15, 20, and 25 feet in a 25-foot hole vertically upward in the tunnel roof. Initial displacements reflect adjustments of the rock immediately following excavation, followed by apparent stabilization in intervals measured by the 4-foot anchor and deeper anchors. Some continuing deformation apparent in the near-surface interval throughout period of measurement. Inverse deformation peak at a face distance of approximately 240 feet is defined by more than one measurement in each trace, virtually eliminating the possibility of spurious data. The peak probably reflects a temporary disturbance related to construction activity--probably repair or reblocking of a nearby rib or ribs.

11-5.5 Case History No. 5

(i) Nature of case history. Case History No. 5 shows rebound induced by excavation of deep foundation during construction of the Grand Gulf Nuclear Generating Station, Mississippi. The data shown is after Blendy and Boisen (1978).

The material involved is a sandy to silty marl.

(ii) Instrumentation. Instrumentation consisted of electronic multiple position borehole extensometers installed prior to construction and read out as the foundation cut was excavated, and as the generating plant facilities were subsequently constructed. Purpose was to identify rebound resulting from removal of load in the excavation and recompaction under subsequent structural loading.

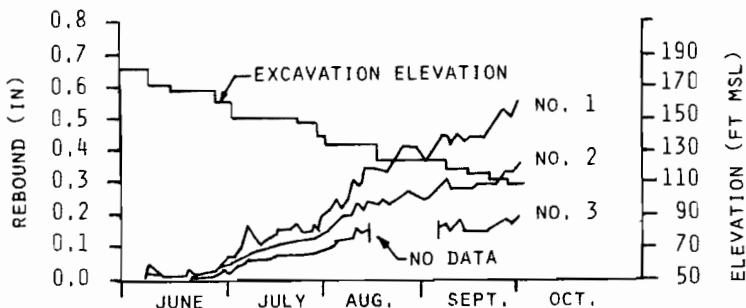


FIG. 11-14. CASE HISTORY NO. 5 - GRAND GULF NUCLEAR GENERATING STATION. REBOUND IN FOUNDATION EXCAVATION. UPWARD DISPLACEMENTS OF ANCHORS 1 (ELEV. 75 FT MSL), 2 (ELEV. 30 FT MSL) AND 3 (ELEV. -20 FT MSL), IN RELATION TO A DEEP REFERENCE ANCHOR (ELEV. -70 FT MSL) REFLECT UPWARD REBOUND. AFTER BLENDY AND BOISEN, 1978.

(iii) Summary of record shown. Fig. 11-14 shows displacements versus data (months) and excavation depth (Elevation, ft. above MSL) for instrument with a common anchor point at Elevation -70 ft. MSL and three up-hole transducer packages anchored at Elevation -20 ft. MSL, +30 ft. MSL, and +75 ft. MSL. Each transducer package was connected to the common anchor by a steel rod. The instrument was read out via a signal cable extending up the hole, initially to the original ground surface at approximately Elevation +182 ft. MSL, and subsequently at varying elevations as the excavation was deepened. The traces show rebound in response to the progressive deepening of the excavation between about Elevation +182 ft. MSL and Elevation +94 ft. MSL as measured at the three transducer package elevations relative to the common anchor point (-70 ft. MSL). The transducer packages identified as Nos. 1, 2, and 3 were situated, respectively, approximately 12.5, 57.5, 107.5 feet below the final excavation grade at Elevation 87.5 ft. MSL.

The total measured rebound was substantially less than predicted.

11-5.6 Case History No. 6

(i) Nature of case history. Case History No. 6 shows displacement measured in the slope of an open pit in competent rock at the Jeffrey Pit, Quebec. Data is courtesy of the Manville Corporation.

The Jeffrey Pit is approximately one mile in maximum diameter and 1000 feet deep. The rock is metamorphic, locally called "slates".

(ii) Instrumentation. Primary instrumentation consisted of multiple position borehole extensometers installed in near horizontal holes drilled in newly excavated benches to monitor stability of slope above. Typical hole depth is approximately 300 feet. Extensometers were mechanically read out using dial gauge depth micrometers.

(iii) Summary of record shown. Fig. 11-15 shows displacements versus time (months) as measured by an extensometer with an instrument head at the surface and in-hole anchors at depths of 16, 46, 96, and 296 feet in a 296 feet near-horizontal hole. Response is typical of that of relatively competent rock following disturbance resulting from excavation of the pit to

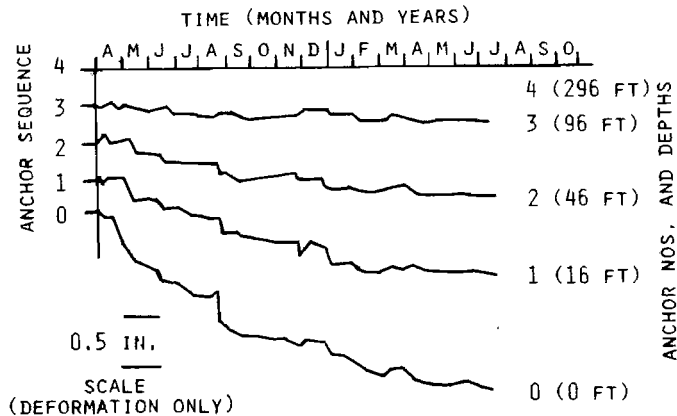


FIG. 11-15. CASE HISTORY NO. 6. - JEFFREY OPEN PIT.
DOWNWARD DISPLACEMENTS REFLECT AXIAL BOREHOLE
EXTENSION.

a new bench elevation. More or less uniform relaxation, with perhaps some dilatancy, totaling approximately 0.169 inch during the period shown. Gradual negative acceleration (curvature) of traces suggests imminent stabilization. Minor stepwise adjustments affecting some or all of the traces at common times appear to correlate with blasts on nearby benches. Some of these effects appear to be confined to the intervals above the 96-foot anchor, suggesting the influence of a discontinuity located in the interval between the 46-foot and 96-foot anchors.

11-5.7 Case History No. 7

(i) Nature of case history. Case History No. 7 shows surficial displacements leading up to the massive failure of the left reservoir slope at Vaiont Dam, located on a tributary of the Piave River approximately 90 km north of Venice, Italy. The data is from Mueller (1964 and 1968).

The Vaiont slide is remarkable because of the size of the slide mass (approximately 250 million cubic meters) and the extent of the damage produced. The slide mass filled much of the reservoir, displacing water which overtopped the dam in a wave several hundred meters high. The wall of water continued downstream about 2 km, destroying the town of Longarone and causing over 2000 deaths.

(ii) Instrumentation. Because of the height of the dam (approximately 800 feet), a considerable amount of instrumentation was installed in its immediate abutments to observe their performance under load. However, instrumentation in the reservoir area was limited to surface observation points distributed along the toe of the slope and for about 0.5 km to the south (upslope), and to observation wells. Part of the reason for lack of instruments was the comparative unavailability of instruments for deep borehole instrumentation at the time of construction (CA 1957-1960).

(iii) Summary of the record shown. Fig. 11-16 consists of two graphs of displacement versus time (months) at Observation Point No. 2, located near the toe of the slope at about the center of the slide mass. One graph shows displacements of the point during a period of instability associated with the first filling of the reservoir in 1960. At that time, a peripheral crack developed around the same general area involved in the later slide, which occurred during the third filling, in 1963. The other graph shows displacement of the same point versus time (months) in the interval immediately preceding the slide (October 9, 1963).

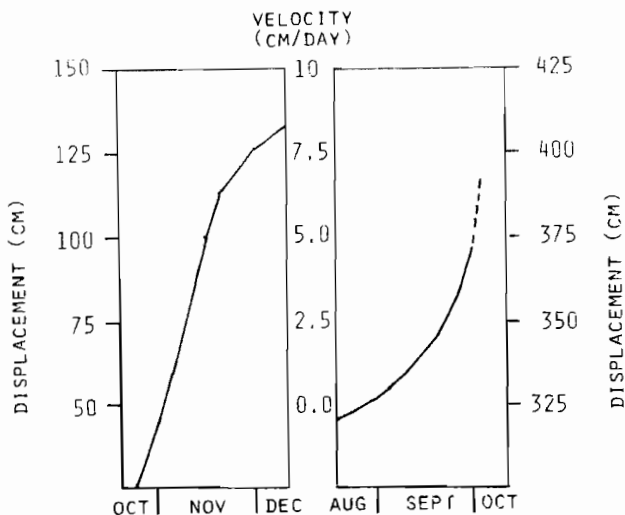


Fig 11-16. Case History No. 7 - Vaiont Dam left reservoir slope. Upward displacements of the reference point. After Mueller, 1964.

In the sources listed above, the similarity of rates in the 1960 and 1963 curves is cited as the reason the slide was not anticipated until it actually occurred. The 1964 reference states, in relation to the 1963 displacements, "The peak velocities increased progressively during the early days of October. According to the report of the 'Commissione di Inchiesta' the velocity had reached 20 cms per day by October 9. Compared with the final velocity of the sliding mass (about 25 m/sec), all movement, even in the last phase, must be considered a creeping movement up to the very instant of the slide itself".

The graphs shown are exactly as represented in the 1964 reference in support of the foregoing statement. At no point in either the 1964 or 1968 references is it noted that the 1960 graph shows an interval culminating in negatively accelerating ("decelerating") displacements, while the displacements in the 1963 graph are CLEARLY UNDERGOING POSITIVE ACCELERATION ("accelerating").

In fairness, it is necessary to recognize that in 1960-1963, concepts in instrumentation were not as advanced as they are at the present time (1988).

At present, it is almost axiomatic that acceleration of deformation is a more critical failure indicator than are either deformation magnitude or rate. ACCELERATION, IF CONTINUED, INVARIABLY AND INEVITABLY ENDS IN FAILURE, a point well illustrated by the two Vaiont displacement graphs.

11-6 SUMMARY AND CONCLUSIONS

The behavior of disturbed geologic masses can be observed using existing instruments and techniques. In addition to providing information for the early recognition of actual or potential hazards, the identification of remedial measures, and confirmation of their effectiveness, the systems described can be used to evaluate the mechanical properties of geologic masses and for long-term safety and performance monitoring.

Instrumentation should be planned in a way which will insure that it meets several important basic requirements. It should permit observation of entire masses and structures, rather than "representative" or "test" sections. It should provide good sample distribution, even under conditions of limited site accessibility. Data should be redundant enough so that important decisions need not be made on the basis of isolated or fragmentary bits of information. Costs must not be excessive or unreasonable.

In tunneling these requirements can be met using a combination of convergence measurements, using tape extensometers or bar extensometers, and borehole instrumentation, using borehole extensometers and borehole inclinometers. Supplementary information, if required, can be obtained using other, more specialized, devices and techniques.

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Chapter 12

TUNNELING IN SOFT GROUND

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12-1 INTRODUCTION

Soft ground tunneling is an art, no other application of geotechnical and structural engineering is more so. The nature of the tunneling process, a relatively crude excavation initiated by brute force, hastily erected support systems, performed under highly unfavorable conditions has made the study of tunneling difficult and imprecise. The complex ground-structure interaction between lining and surrounding ground had, until recent times, defied a rigorous definition and formulation despite the fact that such interaction had been appreciated and understood by the miners and tunnelers for generations. Few if any other structural designs are as dependent on the timing, sequence and workmanship of the imposed support system and lining installed during the tunneling process.

Soft ground tunnels generally are defined as those in which the ground may be excavated or dug by conventional means. These means may include: picks, shovels, spades, digger, or backhoes and similar earth excavating equipment. Generally, as opposed to rock tunnels, soft ground tunnels require more or less immediate support to maintain the opening. Soft ground tunneling in civil construction is performed at relatively shallow depths and is most often found in our most urban areas. In many areas the construction of soft ground tunnels is a traditional craft skill and often the miners, sandhogs, or local equivalent, are a special breed, generally bearing a special sense of danger and accomplishment. Few construction projects are more costly.

The objectives of any tunneling project, whether in soil or rock are fundamentally the same (Bickel, et.al., 1982):

- (1) Maintain a safe and stable opening to protect tunnel workers and minimize ground movements until an initial, initial/final lining is installed.
- (2) Minimize tunneling effect on the surrounding or overlaying utilities and structures.

- (3) Meet the user requirements.
- (4) Remain relatively maintenance-free and operational for the useful service life of the structure - which for tunnels generally means 50 - 100's of years.
- (5) Provide for economical/practical construction.

One or several of these often times dictate the final configuration, construction method, and/or techniques.

To maintain a safe and stable opening generally requires support both during the excavation process and thereafter. The support of the excavation immediately after excavation is termed initial support, and will be described in some detail later in this chapter. This initial support system is generally erected within a temporary movable support system, a shield.

The avoidance of distress of surface structures and utilities and minimization of ground surface movements and subsidence during the tunneling process, especially for near surface soil tunnels, has become increasingly important in our urban areas where many of the other forms of construction, typically cut and cover and trench types, have such adverse impacts. Loss of critical utilities and damage to neighboring structures has important legal and economic consequences when assessing project costs and impacts. The minimization of ground movements as induced from ground losses into the tunnel excavation and from induced changes of soil state during the tunneling process are an important consideration in the engineering process of design because the elements of construction, support system design and construction technique are all interrelated and may have the overriding effect on the project outcome and success.

The meeting of user requirements is in many instances not a trivial pursuit. Many owners and users agencies do not understand or fully realize their needs or requirements, especially if they have had no tunneling experience. Some requirements or "ideals" imposed by users may likewise be too costly, if not impossible, to obtain. The results of these unattainable user requirements results in users, engineers, and contractors with unfilled expectations and needs.

It is the understanding of the user requirements and the need to produce a structure with a lifetime of 50 to 100's of years that oftentimes becomes the ultimate challenge. The ability to fully describe the extent of ground conditions and these variations from the average conditions during construction and the range of operating conditions may present a very complex and extended

set of design parameters, especially since the construction process itself almost exclusively dictates the induced ground loadings and distortions to which the structure will be subjected and many times may be the largest loads which the structure may be required to sustain.

12-2 CONTRAST WITH ORDINARY DESIGN PROCESS

The steps for the tunnel design process are not unlike those followed for the design of any other more common civil structure, i.e., highway bridge, office structure, etc.

- (1) Establishing material behavior and likely range of values.
- (2) Developing a model of system and interactions.
- (3) Determining stresses and strains for the range of expected load values.
- (4) Sizing of the lining/members.
- (5) Comparing behavior to historical precedent/reasonableness and modify and revise as required to meet objectives.
- (6) Monitoring of behavior.

However, a comparison with the case of a highway bridge of structural steel beams and a tunnel quickly reveals some basic differences. Inherent in the bridge design is the assumption that the steel is a homogeneous, isotropic, linear elastic, that the limitation of small deflection theory are applicable. For the soil structure, this is not the case, the material is no longer linear elastic, homogeneous, or isotropic.

The very nature of the soil media, its variability, and relatively low strength generally leads to difficulties during tunneling. The deformation and complex interaction between the soil media and the linings are not easily defined and are not analogous to those generally used to design more conventional civil engineering structures such as buildings or bridges. The interaction is such that the soil media provides both the load and sustaining reactions. The deformations and induced strains within the soil mass, which is generally unloaded during the excavation process, have a direct bearing on the excavation process, and the stability of the excavation parameter and face.

Normally soil is fairly heterogeneous and may be fractured, jointed, layered or exhibit other anisotropy which complicates behavior and its subsequent characterization. The soil's constitutive relationships and behavior may be drastically altered when in the presence of a simple substance, water, and as importantly, its behavior may be affected by its insitu state of internal stress. In contrast, the basis of design for the steel bridge is found in AASHTO or state highway department bridge design manual complete with load factors, load placement, and combination, etc. The design loading conditions for the steel bridge are rigorously defined and the values for such simple parameters as unit weights, wheel loadings, etc., are defined by code or through the use of standardized design vehicles. No such standard exists for loading on tunnel lining or for defining soil/structure interactions, where the surrounding soil is both load and reaction and the state of stress change drastically because of the excavation sequence or time of installation.

Perhaps most elusive of all is the determination of factors of safety and serviceability. Codes and manuals of practice define the limit states and the needed criteria for stability of most conventional structures. While engineers often attempt to apply the same standards and criteria for tunnel structures, the highly redundant nature and complicated interaction process makes such analogy doubtful. The codes and standards which provide the factors of safety for the design of common structures offer no guidance for tunnel or underground structures. The normal structural steel and concrete codes are almost totally devoid of any reference to underground structures. The definition of factors of safety must be developed by the designer through application of engineering judgement and based on past performance of similar tunnels in similar ground conditions. Often the only criteria precedent provides for tunnel structures is if the previous attempt collapsed. The true factor of safety for standing tunnel structures is rarely if ever known except for the fact that most tunnel structures built at the turn of the century and even earlier structures in Europe are still in use today.

12-3 TYPES OF GROUND

The type and extent of the soil mass is critical in the determination of the conditions and techniques of tunneling to be applied. The variability and heterogeneity of the soil deposits must be accurately and as thoroughly uncovered. It is the recognition of these differences and probable variations in ground behavior to be encountered that directs the design and subsequent construction. A site investigation must include an understanding of the geologic history and forces that have acted on the area.

Soils are generally formed from some form of degradation process, either mechanical/physical or chemical in nature. Most soils are derived from the degradation of rock materials or decomposition of organic material. Residual soils remain in place where they are formed. Soil transported from where it is formed, is called transported soils. During the course of geologic time, transport, deposition, and weathering may continue to occur.

12-3.1 Residual Soils

The decomposition of near surface or surficial rock is a continuing process. Many soils are direct results of such degradation and many times the insitu change from rock to soil strata are transitional and gradational. Physically weathering processes generally result in coarsely grained deposits in relatively thin deposits. Chemical weathering induced by the local hydrologic, climatic, vegetated, and other environmental factors results in varied weathering products. Sandstones form sandy soils, shales form silty and clayey soils.

12-3.2 Transported Soils

Transported soils may be classified according to the method of transport and subsequent deposition and include: alluvial, glacial, aeolian, lacustrine, marine, and colluvial soils.

Residual soils formed from igneous, metamorphic, and limestone rocks are often clayey and the depth to sound rock may be very irregular. Also, there are likely to be stones and boulders within the soil matrix.

Alluvial soils are those transported and deposited by rivers and streams. In general, they may be subdivided as braided stream deposits and meander belt deposits.

Overladen with sediment, braided stream deposition and erosion occurs all along younger stream channels. Large numbers of interconnecting channels separated by bars and islands are constantly shifting as deposition and erosion occurs in an ever changing stream pattern. A particular channel may be filled with sediment and the water overflows to cut new channels. The resulting soil deposits generally consist of an agglomeration of fingers and lenses of soils of varying particle sizes, primarily sand and gravel usually cross bedded with individual beds or deposits tending to be finer grained at the top. The type and density of the soil may vary considerably from one lens to another depending on stream velocities at the time of deposition. Braided stream deposits

may occur as small local deposits such as alluvial fans, etc., or as large scale deposits such as glacial outwash plains.

Meander belt deposits are laid down by more mature sediment laden streams which meander in a relatively broad valley. Locally, material is eroded from the outer banks where the velocity is greater and is deposited along the inner banks as point bars, blanket bars. The primary soil types are silts and sands which remain after the finer grained soil are carried downstream.

The meanders migrate downstream. If a more resistant material is encountered a meander may be cut off and abandoned. During flood these fill with water to form "oxbow lakes" from which fine sediments settle to eventually form clay plugs which may eventually become numerous enough to restrict the area in which the stream can flow. The changing environment during deposition, and the change in stream channel locations, commonly cause stratified soil deposits.

When a sediment laden stream floods and overflows its banks the velocity decrease causes coarser materials to be deposited along the banks to form natural levees of fine sands and silts. The finer grained contents stay in suspension to spread across the flood plain and eventually settle from flood waters trapped in low areas when the flood recedes, forming backswamp deposits of clays and organics.

Glacial soils result from transporting and deposition by glacial action. All materials deposited by glacial action is called drift, and includes till, glacial-fluvial, and glacial-lacustrine soils.

Till, or unstratified drift, are materials deposited directly by the ice as it melts to form such features as terminal morrains, recessional morrains, and ground morrains. Terminal morrains form at the maximum extent of glacial advance, forming a characteristic knob and kettle topography. Terminal morrains are normally unstratified with a wide range of particle sizes gradation but it may also be quite uniform. Locally, stratification may occur in places once having melt water pools and on the side away from the ice there may be irregular outwash deposits of silt-sand-gravel. Recessional morrains are similar to terminal morrains but are formed where a temporary nearly stationary ice front occurs during recession.

Ground morrains consist of unstratified till of varying composition laid down at the base of retreating glacial ice. The topography is rolling with an internal drainage which results in marshes, ponds, and lakes. Drumlins may also occur as elongated very regular till hills of silts and clays.

Glacial-fluvial deposits are formed by glacial outwash streams and by streams and pools within the ice. Glacial outwash streams form outwash plains

of braided stream deposits. Kames and eskers are hills or ridges of primarily silt, sand, and gravel deposited from pools or streams within the ice mass.

Glacial lacustrine soils are formed from deposition from glacial lakes. Wave action and shore currents form sand and gravel beach deposits that may be of quite uniform grain size. Distribution, changing water levels, and soil deposition may alternate erratically with organic silt and peat deposits. Delta deposits of granular materials and silts may form at the mouths of inflowing streams. Deposits of uniform or varved clays and silts are formed on the lake bottom during seasonal and longer climatic conditions and change the suspended solids within the water course.

Aeolian soils are those deposited by wind action and include sand dunes and loess. The sorting action of wind causes a high degree of uniformity, with particle sizes decreasing with distance from the source. Sand dunes may occur as transverse or longitudinal dunes. The leeward side is generally less dense than the windward side. Dunes tend to migrate in the direction of the prevailing winds.

Loess consists of wind blown fine silts and clays, with clay content increasing with distance from the source. Loess deposits may vary from hundreds of feet thick near the source to just a few feet at greater distances. The materials are very uniform characterized by pipestems (carbonate coated root holes) and unless it becomes saturated, the material is more stable as a vertical cut than on a slope. Structural collapse with resultant large volume change may occur if loess is overloaded or becomes saturated.

Marine soils include beaches, deltas, uplifted deep ocean deposits, and estuary deposits. Wave action and shore currents form uniform sand and gravel beaches. Marine deltas generally consist of finer grained materials and organics. Sea bottom deposits may range from clean sands to soft clays depending on the environment of deposition. Estuary deposits occur at river mouths and coastal areas where tides play an important part in sedimentation. They tend to have highly variable properties and to be very irregular both vertically and horizontally. Coarser materials occur in tidal-creek channels and silts, clays, and organics in tidal flats and marshes.

Colluvial soils are those deposited due to rock falls, landslides, and mud flows. Talus consists of fallen rock and debris at the foot of steep rock slopes and consists of irregular gravel to boulders. Landslide deposits are the distorted soil mass caused by a slide; properties depend on the original soil material but some mixing of materials may occur. Mud flows occur when loose sandy soils on slopes become saturated and flow like water and are redeposited in a more dense condition. The material is likely to be quite heterogeneous.

12-4 TUNNEL EXCAVATIONS

12-4.1 Excavation Problems and Standup Time

Soft ground tunnels may be excavated by a variety of means, either mechanized or by hand:

- (1) Hand mining - clay spades, knives, and shovels
- (2) Shield - open face, closed face, with or without breasting tables or boards, forepoles
- (3) Tunnel boring machine (TBM)
- (4) Earth pressure balance (EPB) or slurry face machine

Generally tunneling in soil is performed with the protection of a shield. Shields are provided to support the excavated perimeter and to, when necessary, support the tunnel face until the initial or initial/final support system can be erected.

The criteria for selecting the appropriate tunneling method and lining system are based on the properties and expected behavior of the ground and are based primarily on practical experience. A descriptive name is given to various ground conditions which correspond to the face stability and working conditions at the face for various soil types. Table 12-1 presents these various classifications which are traditionally called the Tunnelman Classification (Terzaghi 1950).

Table 12-1. TUNNELMAN'S GROUND CLASSIFICATION

| No. | Classification | Tunnel Working Conditions | Representative Soil Types |
|-----|----------------|---|--|
| 1 | Hard | Tunnel heading may be advanced without roof support. | Very hard calcareous clay; cemented sand & gravel. |
| 2 | Firm | Tunnel heading can be advanced without roof support, and the permanent support can be constructed before the ground will start to move. | Loess above the water table; various calcareous clays with low plasticity such as the marls of South Carolina. |
| 3 | Slow Raveling | Chunks or flakes of material begin to drop out of the roof or the | Fast Raveling occurs in residual soils or in sand |

Table 12-1 (Continued)

| No. | Classification | Tunnel Working Conditions | Representative Soil Types |
|-----|---------------------|---|---|
| | | sides sometime after the ground has been exposed. | with clay binder below the water table. Above the water table the same soils may be <u>Slow Raveling</u> or even <u>Firm</u> . |
| 4 | Fast Raveling | In <u>Fast Raveling</u> ground the process starts within a few minutes; otherwise it is referred to as <u>Slow Raveling</u> . | |
| 5 | Squeezing | Ground slowly advances into tunnel without fracturing and without perceptible increase of water content in ground surrounding the tunnel. (May not be noticed in tunnel but cause surface subsidence.) | Soft or medium-soft clay. |
| 6 | Swelling | Like Squeezing Ground, moves slowly into tunnel, but the movement is associated with a very considerable volume increase in the ground surrounding the tunnel. | Heavily precompressed clays with a plasticity index in excess of about 30; sedimentary formations containing layers of anhydrite. |
| 7 | Cohesive Running | The removal of the lateral support on any surface rising at an angle of more than 34° to the horizontal is followed by a "run", whereby the material flows like granulated sugar until the slope angle becomes equal to about 34°. If the "run" is preceded by a brief period of raveling, the ground is called <u>Cohesive Running</u> . | <u>Cohesive Running</u> occurs in Clean, fine, moist sand. |
| 8 | Running | | <u>Running</u> occurs in clean, coarse or medium sand above the water table. |
| 9 | Very Soft Squeezing | Ground advances rapidly into the tunnel is a plastic flow. | Clay and silts with high plasticity index. |
| 10 | Flowing | Flowing ground moves like a viscous liquid. It can invade the tunnel not only through the roof and the sides but also through the bottom. If the flow is not stopped, it continues until the tunnel is completely filled. | Any ground below the water table that has an effective grain size in excess of about 0.005 mm. |
| 11 | Bouldery | Problems incurred in advancing shield or in forepoling; blasting or hand-mining ahead of machine possibly necessary. | Boulder glacial till; riprap fill; some landslide deposits; some residual soils. The matrix between boulders may be gravel, sand, silt, clay or combinations thereof. |

Many case histories in the literature describe tunneling behavior by these using the classifications as soil description has become more uniform and modern geotechnical investigations include soil classification generally given in terms of the Unified Classification System, some correlation between these two soil classification systems is necessary. Table 12-2 presents a correlation between the typical soil descriptions in the Unified System and the Tunnelman Classification. It should be noted that depending on a soils consistency, heterogeneity, and groundwater table position within the soil mass, a given soil may correspond to more than one Tunnelman's classification.

12-4.2 Stability of the Tunnel Face

Stability of the tunnel face is a function of many variables of which the more important appear to be (Heuer, 1974, 1976):

- (1) type of soil and variability,
- (2) size and geometry of opening,
- (3) existing hydrostatic condition,
- (4) past and existing state of stress,
- (5) excavation method and support.

The stability of the unsupported tunnel excavation and the face of the excavation determine the methods and means of construction and generally dictate the time at which tunnel support must be applied. The construction of every soft ground tunnel is associated with some change in the state of stress in the ground surrounding the tunnel with corresponding induced strains and displacements. These induced strains and displacements are not necessarily bad for they allow the mobilization of the soils inherent strength to support the excavation. However, if they exceed the strength of the soil, they can result in excessive movements or failure of the soil itself if these movements are allowed to continue without the support of the opening, threatening the stability of the excavation allowing large movements of surrounding ground. In less extreme cases the instability of the face and sidewalls may manifest itself as cave-ins or as a slow creep and plastic flow into the excavation.

Table 12-2 CORRELATION BETWEEN UNIFIED CLASSIFICATION AND TUNNELMAN'S CLASSIFICATION

| Typical names in Unified Soil Classification System | Tunnelman's Ground Classification | | | | | | | | | | |
|---|-----------------------------------|------|---------------|---------------|-----------|----------|------------------|---------|---------------------|---------|----------|
| | Hard | Firm | Slow Raveling | Fast Raveling | Squeezing | Swelling | Cohesive Running | Running | Very Soft Squeezing | Flowing | Bouldery |
| Gravel and coarser | | | | | | | | | | | |
| Sand | | | | | | | | | | | |
| Silt | | B | | | | | | | F | | |
| Clay | | C D | | | E D | | | | F | | |
| Gravel, with clay binder | | | | | | | | | | | |
| Gravel, with silt binder | | C D | | | | | | | | | |
| Sand, with clay binder | | | | | | | | | | | |
| Sand, with silt binder | | C D | | | | | | | | | |
| Cemented sand and gravel | | | | | | | | | | | |
| Highly organic soils | | | | | | | | | | | |

- A - moist, above water table
 B - loess
 C - stiff to very hard
 D - stiff to hard
 E - soft to medium
 F - very soft
- Notes: 1. The typical soil names refer to the dominant soil type with regard to their behavior in a tunnel
 2. The shaded areas indicate the soil types that usually cause the ground conditions described by the tunnelman's terms.

The stability of the face (standup time) may be examined in terms of four principal groupings of soil; granular soils with little or no actual cohesion, cohesive granular soils, nonswelling stiff to hard clays, and stiff to soft saturated clays.

12-4.2.1 Cohesionless Granular Soils

The stability of a tunnel face in cohesionless materials such as uncemented sand, silts, and gravels is essentially controlled by the groundwater conditions and effects of the construction method used. Excavations in this material can be carried out only by providing complete protection to the face and excavated parameter of the tunnel. Above the groundwater table these soils will not generally stand unsupported but will ravel until a stable slope is formed at the face with a slope equal to the angle of repose of the soil material in such a loose state. In many instances granular soils above the moisture table contain enough soil moisture to create a small apparent cohesion which may be sufficient to allow erection of an initial support system if the erection time is small enough to prevent drying and the mechanically induced vibration of the construction process from destroying this effect, otherwise full breasting or forepolling may be required to support the tunneled face. Failure to do so will allow runs to develop.

Runs may also develop into cavities and unfilled voids outside the initial/final support system which remain open temporarily but may collapse following excavation and lead to surface subsidence at a later time. Unless groundwater is adequately drained from ahead of such soil masses, even small seepage gradients may induce large ground movements or runs which completely invade the heading. The control of groundwater then becomes paramount. Dewatering may be applied to drain the soil, however, the general stratified and lenticular nature of most soil deposits makes the complete drainage of all zones unlikely allowing imperfect drainage of others. Coarser and more permeable zones will be well drained and tunneling advance satisfactory until a poorly drained area is encountered and a run may develop. Fine grained soils may also be easily transported through even the smallest of cracks in lagging or initial support systems or poorly cast joints in the final lining for the smallest of flows. Loss of such ground around the lining system once erected may be result in a run which may deprive the support system of the necessary restraining ground reaction, inducing failure of the lining system.

12-4.2.2 Cohesive Granular Soils

Soils of this category include soil types ranging from clayey sands to sandy clays to cohesive silts. Residual soils and hydrochemically altered soils possessing cohesive bonds or cementation may also be included. In most instances these soils behave admirably with sufficient standup time to allow support of the excavation. Losses of ground are typically associated with the infilling of the annular space behind the tunneling shield once the support system is erected and emerges from the shield. Raveling may develop if support is not provided or installed in sufficient time. The use of a shield is considered a prudent precautionary measure to forestall such problems. Where standup time is long enough to allow erection of an initial support system behind the shield, expansion or immediate grout injections may be beneficial to minimize the infilling of the annular space.

Raveling of such ground may, if allowed to begin, continue to persist and where these soils exhibit a sensitivity to adverse seepage gradients, they must be predrained in advance of tunneling to minimize catastrophic ground losses. Ground losses due to raveling may be further troublesome because settlements so induced may be delayed for years as overlying soils ravel or slough slowly into the voids created during tunneling.

12-4.2.3 Nonswelling Stiff to Hard Clays

These ground have the desirable properties of those in the preceding category and are not likely to be adversely influenced by seepage gradients towards the tunnel face or subjected to raveling. Some such ground does possess relic or secondary structures such as slickensided joints. These soils are similar to the London and Chicago clays and have historically been mined by conventional hand mining means from within shielded excavations. Precast concrete, metal segmental and ribs and lagging have all been used in these tunnels where standup time is satisfactory. Major ground losses associated with these tunnels appear from in back of the shield as ground moves inward into the annular space between the lining and outside of the shield perimeter. Loss of ground may also occur by the inward squeezing of soil.

12-4.2.4 Soft to Stiff Saturated Clays

These soils are characterized by undrained shear strengths ranging from 200 to 2,000 psf (1.4 to 14 Mpa) and comprise a relatively large number of

naturally occurring clay strata generally found at shallower depths and which are also generally impervious. Movement of the ground during tunneling occurs as a longitudinal or inward movement into the shield without any visible signs of distress or soil raveling. This process continues for the duration of the jacking and excavation process itself and generally seem to diminish once these activities are stopped. The minimization of ground movements may best be established by filling the annular void created during the tunneling process. Some tunnels in similar soils have been pushed blind (Hudson River Tunnel, Lincoln Tunnel, and Holland Tunnels), allowing only modest amounts of soil intake into the tunnel shield and displacing the remainder.

12-5 THE TUNNEL SHIELD

12-5.1 General

Much has been said in the preceding sections about the control of excavation and the measure of tunnel face stability. The use of tunnel shield has been advocated in a number of instances. This device is often compared to a cookie cutter turned horizontally and propelled from the rear. The shield, besides providing a cutting head of the correct geometry, provides safety and a stable opening for the miner until the initial support system can be erected. In times past, tunneling was performed without the aid of a shield, using cribbing, heavy timbering, and lagging. Such tunneling proved both time consuming and cumbersome and all too often restricted the heading, limiting excavation and subsequent advance rate.

The tunnel shield, to be used to the fullest advantage, must offer the support required in the most timely of manners and must be arranged so that there is sufficient access to the face in front of the shield, where the excavation takes place to allow support of the face while at the same time allowing forward progress. The tunnel shield consists essentially of a horizontal cylinder called the skin, the forward part of the shield is called the cutting edge, and an internal structure or structural frame with hydraulic jacks as a propulsion system and a rear section or hood. See Figure 12-1.

The tunneling procedure or advance in shield tunneling is first to excavate a length ahead of the shield, the length of which is the width of one ring or initial support element length, then push the shield forward the distance excavated by means of the jacks reacting against the previously erected lining.

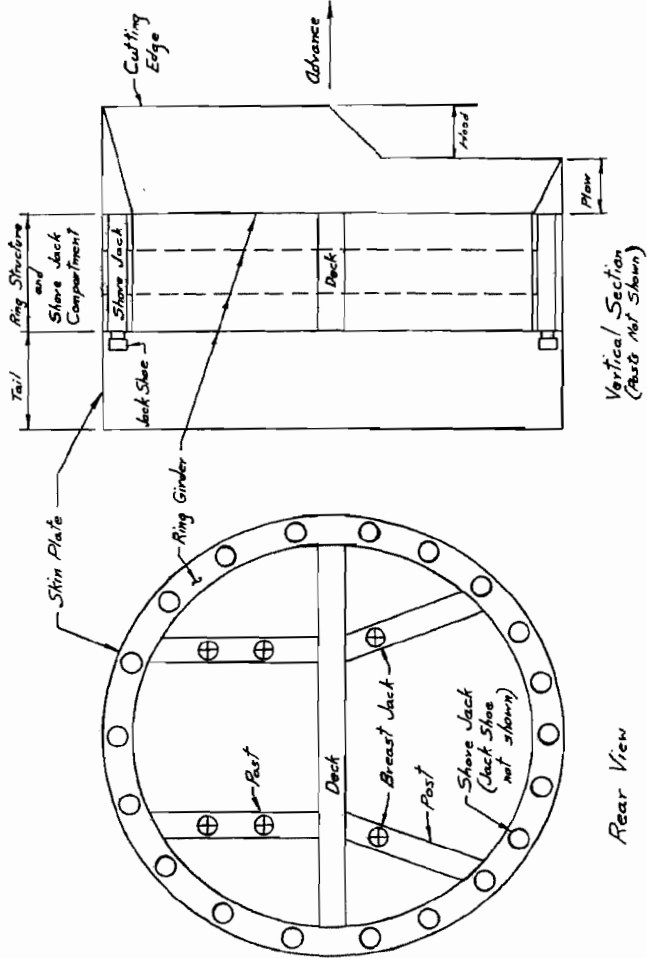


Fig. 12-1. Typical Shield Section

Once extended the jacks are withdrawn within the shield to make room for the erection of the lining and finally to erect the lining in the clear space left in the tail of the shield. This process is repeated again and again in sequence.

In order for the shield to be satisfactory it must:

- (1) Support all soil loads either radially applied or at the face.
- (2) Minimize soil disturbance and vibration.
- (3) Be of the correct geometry and size for the tunnel section.
- (4) Facilitate lining.
- (5) Allow construction to line and grade and required tolerances.

In order for the shield to be effective and perform the functions intended it is imperative that the full range of tunneling conditions to be encountered are known and that the shield and tunneling technique chosen allows for means of coping with this range of conditions. While for the bulk of a tunnel's length the ground may be "good standup" ground, the small length of bad ground with its attendant problems and delays cause the bulk of the economic and legal nightmares for both Contractor and Owner alike.

If running or raveling ground is possible, appropriate timely face control measures should be part of the tunneling system requiring provisions for installation of breasting jacks, poling plates, sliding table or whatever else means is required to ensure face stability. Similarly if induced water flow and soil transport into the heading are likely to be a problem, then face control techniques may be required, but more appropriately dewatering used in advance of the shield to forestall such problems. The cost of tunneling is determined by its labor intensive nature. Adding slightly more to the initial capital cost of equipment provides the necessary tools to handle all the tunneling conditions, that cost is usually small compared to the time delays and extra labor that it takes to "bull" your way through a problem once construction starts.

In the United States where a myriad of public agencies and utilities are required to take the low bidder, there is no guarantee that shields are adequately equipped even if the full range of tunnel conditions are clearly

delineated in a geotechnical design summary report or are clearly indicated by the borings and factual data provided. Failure of the Contractor to adequately provide for the "worst" case is not only a problem for and of contractors themselves but often the problem of the Owner and his Engineers who place the risk and judgement on the contractor failing to realize that they to have the same common goal to ensure a timely, economic completion of the project. It is a mistake for Owner and Engineers to think that shield type, tunneling techniques and methodology are Contractor's alone, and the ground conditions exposed and actually encountered are as much the Owner's as they are the Contractor's risk and the assignment of risk are properly a shared responsibility and the subject of a chapter of its own.

12-5.2 Detail of Shield Structure

i) Annular Void - The diameter of the skin plate is determined by the type of tunnel lining and required clearance to maintain steering and control of the line and grade. This requires an annular space between the outside of the lining and the inside of the tail. This space facilitates the erection of the lining and allows deviation of the diameter of the axis of the shield from that of the lining. These erection clearances are generally a function of judgement and experience, however the allowance for deviation is a function of shield size, shield lead, and the lining element length. Generally the average value of the total clearance is about 0.8 percent of the outside diameter or in other words, it is the usual practice to make the inside diameter of the tunnel tail of the shield equal to approximately 1.008 times the outside diameter of the lining. The annular space then formed between the lining and excavation perimeter generally ranges from 2-5 percent of the mean excavation. (Cording, et al., 1975) The smaller the clearance, the smaller the subsequent overexcavation and the size of the annular space. The use of an expanded lining system, which allows increase of tunnel lining size upon emergence from the shield is used to advantage to minimize the size of the annular void.

ii) Tail Length - Length of the tail depends on the length of the initial support element width. The tail should be long enough to allow erection of one complete ring or unit and still provide overlap with the previously erected unit. While this would generally favor the use of a long tail (some instances 2 segments or units long) the tail is the weakest element of the shield and the longer its length, the greater the clearances required to provide steering

iii) Hoods - The hood of the shield is a forward extension of the circumferential structure of the shield. The hood may be detachable, and in some cases extendable, composed of a series of plating plates, allowing a change in the slope of the shield hood. The slope of the hood should be adjusted to conform to the natural angle of repose of the ground to be encountered to minimize the need for supporting the face of the excavation. For many soil types, this angle of repose becomes large especially in granular soils and is impractical. Support of the face is then required.

iv) Shield Length - Length of the shield is the sum of the length of the hood and cutting edge, the tail and the internal structure length required to stow the jacks and hydraulics. The smaller the ratio of the length to the diameter the easier the shield is to steer. It is preferable to keep this ratio below 80 percent. In recent years the advent of tunnel boring machines with mechanical excavators has made it almost impossible to keep machines within this acceptable range because of the size of the excavation equipment itself. In such cases the ratio of length to diameter may become larger; however, when this ratio approaches one, an articulated shield should be used. In such shields, the shield is provided with an articulated joint which enables the forward section of shield to be aligned independently from the rear section. This allows the shield to maintain horizontal and vertical alignment without extremely larger clearances.

v) Face Control Methods - The tunnel shield should be equipped to control the exposed face of the tunnel, especially if forward advance is to be accomplished with speed and with minimum losses of ground. This may be accomplished by a number of means (Richardson, et al., 1941, 1975) including: (1) face jacks - which are jacks placed horizontally to the axis of the shield which can be extended forward to contact the face, usually used in conjunction with breasting plates or boards. During a forward shove of the shield, the hydraulic pressure on the face jack is retained but the greater forces of the shove jacks forces them to close up. The shield operator then maintains a positive face pressure or support allowing enough force to prevent overstressing of the breasting members. Application of too little pressure will allow breasting members to fall. (2) Breasting or sliding platforms usually consist of steel sliding platforms attached to forward extending face jacks. These platforms divide the face into a number of small vertical increments which for unstable soils with large angles of repose then allows a modest hood overhang to prevent inundation of the entire shield as well as providing protection for the workers on lower level. (3) Mechanical doors have been used on modern tunnel boring machines, many consist of orange peel or segmental doors which rotate

downward and outward from within the shield from positions around the inside of the cutting edge. While these doors do provide some support of the face, when fully deployed often allow forward movement of the shield only by relief of the doors which may allow soil movement into the center of the shield as the doors rotate. To be effective radial doors must be attached to move within the machine and along the axis of the shield as breast boards do.

vi) Mechanical Excavators - Mechanical excavators have been successfully installed on a number of tunneling machines. In some instances these excavations are large enough to be used to control localized raveling but in many instances these devices are too large and bulky to allow easy access to the face to control large scale raveling or running ground. Excavator size should be kept in proportion with the face size to allow access to the face. Provision should be made on any such machines to be able to mobilize additional face support including face jacks or tables.

vii) Shield Shove Jacks - Shield jacks are required to propel the shield forward once the erection of a completed lining element or ring has been completed. In order to move the shield forward a number of forces must be overcome, including: the friction of the ground on the shield's exterior surface, the friction of the lining in the tail of the shield and the resistance to the displacement of the ground in front of the shield. The required jacking resistance may vary depending on the amount of face control used and the nature of the ground. Generally the hydraulic system is designed to provide hydraulic capacity of varying degrees and to individual jacks as well as to ensure steering and control of alignment. Working pressure generally of most hydraulic systems is about 3,500 psi (24 MPa) with a maximum hydraulic pressure of 5,000 psi (34 MPa). The jacks are placed in the shield in such a manner that their cylinders move forward with the shield while the piston rods or plungers remain stationary.

viii) Jacking Ring - In order to bear against the lining and minimize eccentric loadings the jacks are placed circumferentially as close to the skin as possible. Jacks may be uniformly spaced around the circumference but more commonly jacks are placed with more jacks placed below the horizontal diameter than above because of the natural tendency of the shield to dive and the need for greater force at the bottom than the top to forestall such rotation. Jack rams and heads should be equipped so that the induced jacking load is equally distributed over as large an area as possible of the tunnel lining. Special jacking shoes and jacking rings are often used to ensure equal pressure distribution. Often the induced jacking stresses are more detrimental and of larger magnitudes than the ground loadings.

12-5.3 Tunnel Boring Machines

Tunnel shields were initially used with hand mining (Elgood-Mayo, 1976) and a number of face control and stability measures ranging from full breasting to open face mining depending on the nature of the ground were used in conjunction with them. The time taken for an advance of the shield was directly proportional to the time necessary to excavate the face. Full breasting required removal of boards one at a time allowing excavation of only small portions of the face at one time. Today, mechanical excavating equipment has been incorporated into tunnel shields to aid in the speed of excavation. Gradually a wide variety of tunneling machines have been developed to cope with differing ground conditions by incorporating differing excavation methods. A significant feature of the tunnel boring machines is their relative high cost and generally specialized nature derived for a particular ground type on a particular job. This individualized nature may not only require specialized excavation equipment, but muck handling, lining erection arms or equipment and specialized face support equipments as well.

12-6 LINING DESIGN

12-6.1 STRUCTURAL DESIGN MODELS

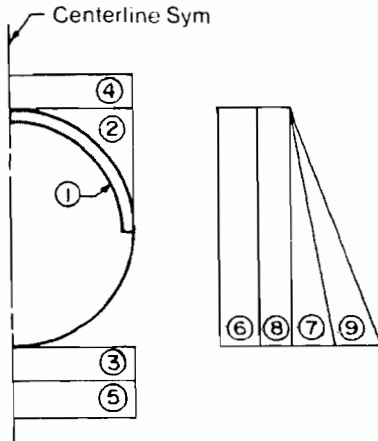
The stability of the tunneled face and the surrounding excavated parameter is understood by applying some simple principle of strength of materials. Imagine the state of stress within the soil mass, at tunnel depth. This stress is composed of two components, one vertical and the other horizontal. The vertical stress, is, unless past geologic conditions indicate otherwise, assumed to be a direct function of the overburden depth γh . The horizontal stress is usually expressed as some function of the vertical stress $K\gamma h$. Upon the excavation of the tunnel opening, a change of stress is induced. The theory of elasticity predicts an increase of tangential and radial stress around the opening and at some distance from the excavated surface as well. The stress concentration factor or stress increase for elastic materials is a function of the opening geometry, and the magnitude of the resultant stress a function of the lateral confining stress provided by the soil. For a soil element on the horizontal centerline immediately on the excavated surface, the internal confining stress is zero and the stress concentration factor approximately two. If the ratio of maximum hypothetical tangential stress at the tunnel wall and

the undrained shear strength are less than unity then the ground behaves elastically. If this ratio is greater than unity then failure of the ground occurs as plastic flow. Soils as a general rule do not behave as ideal, elastic-plastic material but have curved stress-strain relationships. Considerable strain redistribution and movement may occur even if failure of the ground does not occur or plastic flow is not initiated. (Bjerrum et al., 1956, Broms et al., 1967) Even if the resultant state of stress does result in a plastic zone forming around an opening, the opening may be stable if the soils plastic strength is not considerably less than its peak strength. Measurements on actual tunnels in impermeable clays have actually shown that the ratio of insitu vertical stress to the peak shear strength may reach magnitudes of five or six without initial instability. Where the strength of the clays is not sufficient, an internal air pressure can be used to increase the stability.

The design and the analysis of the initial and final support systems has resulted in a number of proposed models and methods. In the earliest attempts the design of soft ground tunnel lining began as a structural design problem by assuming a series of hypothetical soil loads applied to the lining. Methods proposed by Hewett and Johannesson (1922) and others, applied loadings induced from self weight of the lining, soil load above the tunnel and a series of horizontal pressures obtained by assigning an assumed value to the lateral earth pressure coefficient K . These loading systems were then applied to the liner represented as an elastic ring with shears, moments and thrusts computed (within the ring) by elastic theory. (See Figure 12-2.) Terzaghi (1950) suggested that the soil above the tunnel is only partially supported by the liner and the remainder is transferred by arching to the soil at the sides of the tunnel. This method allowed some reduction of load and a reduction in lining size, but these early attempts did not adequately account for the complex soils/lining interaction nor for the large reduction in ground stresses that occur as part of the deformation process around the tunneled excavation, both at the tunnel face and in the vicinity of the face before a lining is erected. The actual ground stresses acting on the tunnel once the lining is installed then bear no resemblance to those assumed in the design analysis. (Terzaghi 1968).

A comprehensive analysis of design models is given by Duddeck and Erdman (1985), Craig and Muir Wood (1976, 1978), Einstein H. et al (1979-80), and will not be repeated here.

In recent years many authors have developed models and techniques for the more appropriate analysis of the soft ground tunnel linings (Newmark (1942)



LOADS

- ① The weight of the upper half of the tunnel.
- ② The weight of the earth within the area marked 2.
- ③ A uniform upward force balancing 1 and 2.
- ④ The weight of the loading above the top of the tunnel.
- ⑤ A uniform upward reaction balancing 4.
- ⑥ The horizontal pressure due to the water above the top of the tunnel.
- ⑦ The horizontal pressure due to the water from top to bottom of the tunnel.
- ⑧ The horizontal pressure due to the earth above the top of the tunnel equal to the product of the weight of earth (buoyant unit weight if submerged) above the top of the tunnel and the factor K .
- ⑨ The horizontal pressure due to the earth between the top and the bottom of the tunnel. At any point, the pressure is the product of the weight of soil between that point and the top of the tunnel and the factor K . Soil weighed as in 8.

Fig. 12-2. Tunnel Loading
(After Hewett and Johannesson, 1922)

Hoeg (1968) Muir Wood (1976). From these studies, almost international consensus on the model to be applied has been achieved and was summarized by Duddeck and Erdmann (1985). Overall a consensus has developed which was summarized by Duddeck and Erdmann.

- (1) For the design model of the linings it may be sufficient to consider only a cross-section, assuming plane-strain conditions for the lining and the ground. The three-dimensional stress-strain effects close to the tunnel face are neglected.
- (2) The cross-section is circular. The stiffness of the lining is taken as constant along the circumference. Complete or restrained structural hinges may or may not be considered.
- (3) The active soil pressures on the lining are assumed to be equal to the primary stresses in the undisturbed ground because the ground is soft. Hence, it is assumed that for the final stage (years after construction), the ground will eventually return to the same condition as before the tunneling.
- (4) A bond exists between the lining and the ground, either for radial and tangential deformations or for radial deformations only. With this assumption, the model complies with the equilibrium conditions as well as with the compatibility conditions at the boundary between lining and the ground.
- (5) With the bond between lining and ground, the deformations of the lining result in reaction stresses in the ground. Continuum models include this effect automatically. Beam models must include bedding springs with appropriate bedding moduli. Bonding at every place around the lining results in a reduction of the "loading" ground pressure where the lining deflects inward.
- (6) The material behavior of ground and lining are generally assumed to be elastic. More refined theories may also include nonlinear and plastic material laws, which, however, in most cases require the application of numerical methods.

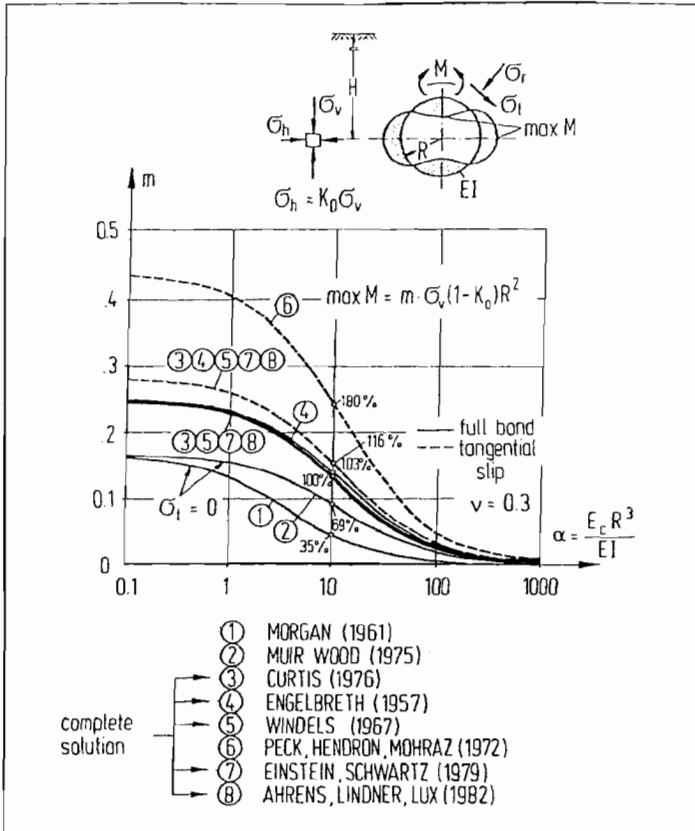


Fig. 12-3. Comparison of Maximum Calculated Bending Moment (After Duddeck and Erdmann, 1985)

As summarized by these authors, the varying design methods when compared critically over the same range of soil parameters generally predict lining thrusts and moments of the same orders of magnitude as indicated in Figure 12-3.

Generally it is concluded that for a relatively flexible lining the pre-dominate load is axial compression with bending stresses at very low levels.

12-6.2 Design Approach

The design of the lining flexible elements has generally evolved into the use of two different approaches, one an empirical, semi-analytical approach where limits on deformation and loading are assumed based on measurements and observations in installed lining systems and the other a more mathematical closed form relative stiffness method. Both of these methods assume the lining to be relatively flexible and generally minimize the importance of induced bending stresses within the lining.

Ground Lining Interaction

While complex, lining design may be approached as any composite structure analysis where the elements of the composite system share or assume the applied loads as a function of their relative stiffness to the other elements of the system. The surrounding ground is as much a part of the composite system as the concrete or steel elements of the lining. The tunnel lining is not then an independent structure acted upon by a well-defined series of loads, but instead the deformation and induced strains are a function of the ground/soil response to the excavation process. The lining elements generally may mitigate some of these responses and may tend to redistribute loads to the surrounding ground. The ground response to these redistributed forces generally tends to average the effects creating a more or less uniform state of stress about the lining. The response of the ground is highly dependent on the range and duration of the applied loads and is not generally a constant value, i.e., spring constant, or modulus of subgrade reaction.

A convenient and often used tool (Schwartz, et al., 1980) to describe the interaction is the characteristic curve which generally plots radial pressure vs. radial displacement relation for the ground mass and the tunnel support. As an example of the effect of two different support stiffnesses on the final support load is shown using such a curve in Figure 12-4.

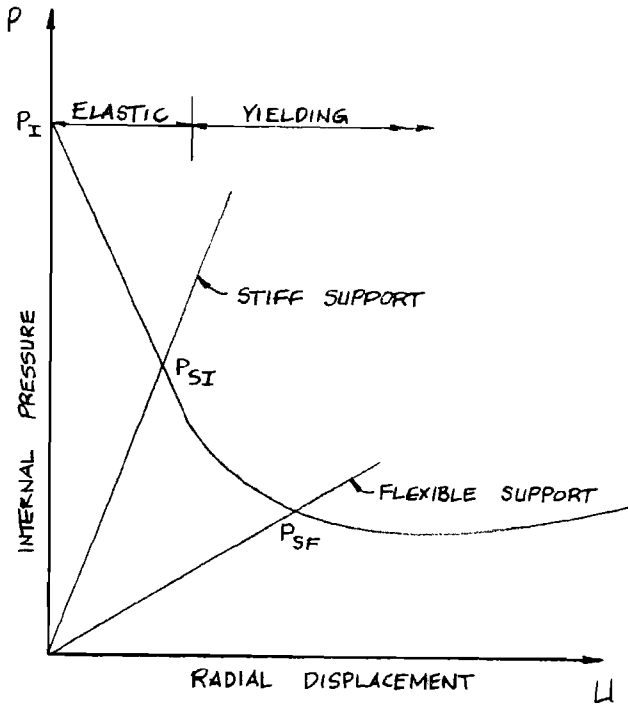


Fig. 12-4. Effect of Relative Stiffness on Support Loads - Characteristic Curves

The curve begins at the equilibrium, insitu pressure, P_1 , presumably the insitu hydrostatic ground pressure. As the excavation is introduced the ground lining interaction begins with the ground deformation inward, met by a corresponding lining resistance. The lining resistance is basically a function of its stiffness, as is the ground response. The initial straight line portion of the curve for the soil response, is the linear elastic range of the ground response, depending on the strength (elastic and plastic strength) parameters. This portion of the curve may be relatively small or for more competent soils may be large to the point for which near rock like ground the entire response of the ground may be well within this elastic range. More generally as the displacements and induced strains grow, plastic and inelastic limits of the ground are exceeded and the curve generally turns concave upward. The ideal point of placement of the perfect flexible support would be at the point of the minimum. An ideally flexible support in a nonuniform stress field will deform until the stress is uniform and accordingly eliminate the bending moments induced within the lining.

The equilibrium pressure (load) on a stiff, nonflexible support system is considerably larger than that on a flexible support system.

12-6.3 Flexible Linings

The flexibility or stiffness of the soil-liner system is normally divided into two separate types: the extensional stiffness which is a measure of the equal all around strain of the liner with no change in shape; the second of the non-uniform pressure necessary to cause a unit diametrical strain. (Paul, et al, (1983), Ranken, (1978)

The compressibility ratio, C , is a measure of extensional stiffness of the medium relative to that of the lining and is given by

$$C = \frac{ER(1-\nu_r^2)}{E_l A_l (1-\nu_l^2)}$$

The flexibility ratio, F , is a measure of the flexural stiffness and is expressed as

$$F = \frac{ER^3(1-\nu_r^2)}{6E_l I_l (1-\nu_l^2)} = \frac{\text{soil stiffness}}{\text{bending stiffness of lining}}$$

In most instances the Poisson's Ratio has only a small effect on the flexibility ratio and an approximate relation for the F is therefore

$$F = \frac{ER^3}{6E_x I_x}$$

The compressibility ratio may be expressed in a modified form neglecting Poisson's Ratio effects.

$$C = \frac{E}{\left(\frac{E_x t}{R}\right)}$$

In these equations

E = modulus of elasticity of the soil

ν = Poisson's ratio of the soil

E_l = modulus of elasticity of the liner

I_l = moment of inertia of the liner per unit length

ν_l = Poisson's ratio of the liner and

R = the radius of the tunnel lining

Interaction of the soil with a lining as deformations take place permits the large reduction of bending moments for the flexible lining. (Peck, et al, 1972) For tunnel lining systems with a flexibility ratio greater than 20, the resultant moments are less than one percent of γHR^2 or a moment coefficient of 0.01.

12-6.4 Tunnel Design - Empirical Method

The most realistic criterion for the formulation of lining designs by the empirical method is that originally formulated by Peck (1969) and Ramos et al (1972) which suggests that the circular lining be designed for a uniform ring compression corresponding to the overburden pressure at spring line plus an arbitrary imposed distribution usually measured as a percentage of change of radius. These criterion are based on field measurements and observations of actual performance of many soft ground tunnels. The amount of ring distortion may be varied depending on the stages of construction and lining type in similar ground conditions. Schmid (1984) has recommended ranges of distortion ratios to be used for verification of design. These are presented below:

Recommended Distortion Ratios for Soft-Ground Tunnels

| Soil Type | R/R - Range* |
|--|--------------|
| Stiff to Hard Clays, Overload Factor < 2.5-3 | 0.15-0.40% |
| Soft Clays or Silts, Overload Factor > 2.5-3 | 0.25-0.75% |
| Dense or Cohesive Sands, Most Residual Soils | 0.05-0.25% |
| Loose Sands | 0.10-0.35% |

Add 0.10-0.30% for tunnels in compressed air
 Add appropriate distortion for external effects,
 such as passing neighbor tunnel.

**values assume reasonable care in construction,
 and standard excavation and lining methods.

Thus the semi empirical design procedure consists of four separate steps (Deere 1969):

(i) provide adequately for the ring loads to be expected which is normally taken as the average radial pressure exerted by the soil on the lining multiplied by the external radius of the lining.

(ii) provide for the anticipated distortion due to $\Delta R/R$. The moment resulting in the lining may be computed from

$$M = \frac{3EI}{R_m} \left(\frac{\Delta R}{R} \right).$$

M = bending moment

E = modulus of elasticity of the lining

I = effective moment of inertia of the lining

R_m = average radius of the lining

$(\Delta R/R)$ = distortion ratio

(iii) provide adequate factor of safety against buckling

$$P_c = \frac{3EI_c}{R_m^2}$$

where E_c , I_c and R_m are as previously defined and,

(iv) provide allowance for any significant external handling or construction loads and jacking forces.

The accompanying design examples in Section 12.7 illustrate the general approach to lining design using these steps. See also Selander (1980) and Sgouros (1982).

12-6.5 Tunnel Design – Relative Stiffness Approach

This method is a simple analytical method (Schwartz et al, 1980) which reduces the complex ground structure interaction to a series of closed form analytical derivations and finite element numerical solutions for three components: (1) the decrease of support loads with decreasing stiffness of the support relative to the ground; (2) decrease of support loads with increasing delay of support construction behind the advancing tunnel face; (3) increase of support load by ground yielding. The effects of the relative support stiffness on the tunnel support loads are incorporated into the method through elastic plane-strain, relative stiffness solutions. The solution explicitly considers the effects of support stiffness and ground stress state (the lateral insitu stress ratio) on the support thrusts and moments at all points around the circumference of the opening.

12-7 DESIGN EXAMPLES

As indicated, examples have been chosen with a common set of design data including depth to springline, tunnel diameter, and soil type. The lining thickness and parameters are chosen from experience and for constructability based on typical sizes used with these design systems.

These designs indicate the tunnel lining size is often not a function of the required lining thickness but a function of the smallest possible practical lining thickness that can reliably and consistently be placed using modern tunnel concrete equipment. Rarely have tunnel linings been placed, especially when reinforced, in thicknesses much less than 8 or 9 inches.

As indicated by these design examples, the need for reinforcing in some tunnel linings is questionable especially at the more common shell depths found in civil construction. Condition of serviceability, long term creep and shrinkage induced effects, long term movements and distortion plus proximity to changing temperatures or hydrostatic condition generally preclude the use of non-reinforced tunnels in soft ground.

Internal pressures, points of hydraulic instability also warrant special consideration.

INITIAL SUPPORT
 "STEEL RIBS"
 SHT. 1 of 3

Structural Steel Ribs
 as Initial Support:

Assumptions

- $\gamma_{\text{soil}} = 120 \text{ pcf}$
- $z = 30 \text{ ft.}$
- ASTM A36 Steel
- Ribs placed on 3-Foot Centers
(Usually placed on Centers ≤ 6 Feet)
- Wood Lagging
- $\sigma_{\text{allowable}} = 1,200 \text{ psi}$
- Shear, $V = 120 \text{ psi}$
- I.D. = 14'-0" ; O.D. = 15'-0"
(Assume a 6" rib is req'd.)

Loads

- Ground Load; $p = \gamma z = (120 \text{ pcf})(30 \text{ ft.}) = 3,600 \text{ psf}$
 $= 3.6 \text{ Ksf}$
- Rib Ring Thrust
 per Foot of Tunnel; $T = p \cdot r = (3.6 \text{ Ksf})(7.5 \text{ ft.}) = 27 \text{ K/ft.}$
- Thrust per Rib; $T = (27 \text{ K/ft.})(3 \text{ ft.}) = 81 \text{ K/rib}$

Steel

Try WG x 25 ($A_g = 7.34 \text{ in}^2$)

$$\text{AXIAL STRESS, } \sigma_c = \frac{T}{A} = \frac{81 \text{ K}}{7.34 \text{ in}^2} = 11.04 \text{ Ksi}$$

$$= 11,040 \text{ psi}$$

$$* \text{ Moment, } M = \pm \frac{3EI}{R} \left(\frac{\Delta D}{D} \right), \text{ where } \frac{\Delta D}{D} = 0.5\%$$

$$\text{Bending Stress, } \sigma_B = \frac{Mc}{I}$$

* DEERE et al, 1969
 PECK, 1969

BENDING STRESS

$$\sigma_B = \pm \frac{3EI}{R} \left(\frac{\Delta D}{D} \right) \times \frac{c}{I}$$

$$\sigma_B = \pm \frac{3EC}{R} \left(\frac{\Delta D}{D} \right)$$

$$\sigma_B = \pm \frac{3(29 \times 10^6 \text{ psi})c}{(12 \text{ in/ft.})(7.5 \text{ ft.})} \times (0.005) = \pm 4,833c \text{ psi.}$$

$$c \approx 3 \text{ inches}$$

$$\therefore \sigma_B = (4,833 \text{ psi.})(3) = \pm 14,500 \text{ psi.}$$

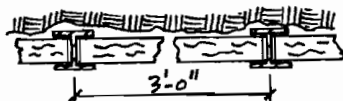
$$\sigma_{\text{Total}} = \sigma_B + \sigma_c$$

$$\sigma_{\text{Total}} = \pm 14,500 \text{ psi} + 11,040 \text{ psi}$$

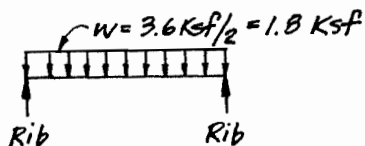
$$\therefore \sigma_{\text{max}} = 25,540 \text{ psi} \quad \& \quad \sigma_{\text{min}} = -3,460 \text{ psi}$$

For Temporary Support, $F_{\text{allowable}} = 0.75 F_y$
 $= 0.75(36 \text{ Ksi})$
 $= 27 \text{ Ksi}$

$$\sigma_{\text{max}} = 25,540 \text{ psi} < 27,000 \text{ Ksi ok}$$

Check Wood Lagging

Assume $\frac{1}{2}$ of Ground
Load Arches onto
Stiffer Steel Ribs



$$\text{Moment, } M = \frac{wL^2}{8} = \frac{(1.8 \text{ Ksf})(3')^2}{8} =$$

$$2.025 \text{ K.Ft.} = 24,300 \text{ in-lbs.}$$

$$\text{Req'd. Section, } S = \frac{M}{\sigma_{\text{allowable}}} =$$

$$\frac{24,300 \text{ in-lbs.}}{1,200 \text{ psi}} = 20.25 \text{ in}^3$$

$$S = \frac{bh^2}{6} = \frac{12h^2}{6} = 2h^2 = 20.25 \text{ in}^3$$

$$h = 3.18 \text{ in}$$

→ Use Nominal 4" x 4" Timber ok

INITIAL SUPPORT
"STEEL RIBS"
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Check Lagging Shear

$$V_{\text{actual}} = \frac{(1,800 \text{ psf})(3')(1')}{2} = 2,700 \text{ lbs.}$$

$$\text{Shear Stress, } V = \frac{2,700 \text{ lbs.}}{(4 \text{ in.})(12 \text{ in.})} = 56.3 \text{ psi}$$

$$\text{Shear } V = 56.3 \text{ psi} < \text{Allowable} = 120 \text{ psi ok}$$

Steel Liner Plate
as Initial Support:

INITIAL SUPPORT
"STEEL
LINER PLATE"
SHT. 1 of 4

Assumptions

In examples given below using Liner Plate as Initial Support, we will consider Liner Plate alone and Liner Plate with Steel Ribs. To illustrate these two examples, we shall make the following assumptions:

- Rib Steel is ASTM A36 Steel, $F_y = 36$ Ksi
- Use a 4 - Flange Liner Plate
- Liner Plate
 - Yield Strength = 28 Ksi
 - Tensile Strength = 42 Ksi
- Liner Plate Factor of Safety for
 - Buckling = 2.0
 - Seam Strength = 3.0
- Ground Load, $p = 2,750$ psf; ($\sim 23'$ depth)
- Diameter = 15'-0"
- $K_{soil} = 0.44$
- Ultimate Design Longitudinal Seam Strengths

| Plate thickness, inches(mm) | Ultimate strength, kips/ft. (kN/m) | |
|--------------------------------|---------------------------------------|---------------|
| | 2 Flange | 4 Flange |
| 0.075(2) | 20.0(292) | |
| 0.105(2.7) | 30.0(438) | 26.0(379.6) |
| 0.135(3.3) | 47.0(686.2) | 43.0(627.8) |
| 0.164(4.1) | 55.0(803) | 50.0(730) |
| 0.179(4.5) | 62.0(905.2) | 54.0(788.4) |
| 0.209(5.3) | 87.0(1270.2) | 67.0(978.2) |
| 0.239(6.0) | 92.0(1343.2) | 81.0(1182.6) |
| 0.313(8.3) | | 115.0(1679) |
| 0.375(9.5) | | 119.0(1737.4) |

* AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES,
Division 1, Section 13
AASHTO 1.13.4

$$\bullet \text{ Minimum Stiffness} = \frac{EI}{D^2}$$

where :

- D = Diameter in inches or (m)
 E = Modulus of elasticity, psi (29×10^6) or (199948 MPa)
 I = Moment of inertia, inches to the fourth power per inch (m^4/m)
- * For 2 Flange (EI/D^2) = 50 minimum
 For 4 Flange (EI/D^2) = 111 minimum

INITIAL SUPPORT
 "STEEL
 LINER PLATE"
 SHT. 2 of 4

• Section Properties (per inch of plate width)

4 Flange

| Gage | Thickness | | Area | Effective Area | | Moment of Inertia | |
|------|-----------|--------|---------------|-------------------------|-----------------------------|-------------------------|-------------------------|
| | In. | (mm) | | $in^2/inch$ (m^2/m) | $in^2/inch$ (m^2/m) | $in^4/inch$ (m^4/m) | $in^4/inch$ (m^4/m) |
| 12 | 0.108 | (2.67) | 0.133 (.0034) | 0.067 (.0017) | 0.042 (.67 $\times 10^4$) | | |
| 11 | 0.1196 | (3.04) | 0.152 (.0039) | 0.076 (.0019) | 0.049 (.78 $\times 10^4$) | | |
| 10 | 0.135 | (3.42) | 0.170 (.0043) | 0.086 (.0022) | 0.055 (.88 $\times 10^4$) | | |
| ** 8 | 0.144 | (3.15) | 0.209 (.0053) | 0.106 (.0027) | 0.070 (1.12 $\times 10^4$) | | |
| 7 | 0.179 | (4.55) | 0.227 (.0057) | 0.114 (.0029) | 0.075 (1.21 $\times 10^4$) | | |
| 6 | 0.209 | (5.31) | 0.264 (.0067) | 0.132 (.0033) | 0.087 (1.39 $\times 10^4$) | | |
| 5 | 0.238 | (6.07) | 0.309 (.0078) | 0.150 (.0038) | 0.120 (1.92 $\times 10^4$) | | |
| 1/4 | 0.250 | (6.40) | 0.309 (.0078) | 0.155 (.0039) | 0.101 (1.62 $\times 10^4$) | | |
| 5/16 | 0.3125 | (7.90) | 0.386 (.0098) | 0.193 (.0049) | 0.123 (1.97 $\times 10^4$) | | |
| 3/8 | 0.375 | (9.50) | 0.460 (.0117) | 0.230 (.0058) | 0.143 (2.28 $\times 10^4$) | | |

Example:

Ring Thrust

$$\text{Per Foot, } T = p \cdot r = (2,750 \text{ psf})(7.5 \text{ ft}) = 20,625 \text{ lb/ft.} \\ = 20.625 \text{ K/ft.}$$

Check Joint

Seam Strength

$$3.0T < \text{Joint Strength} \quad (\text{F.S.} = 3.0)$$

$$3.0(20.625 \text{ K/ft.}) = 61.875 \text{ K/ft.}$$

For Seam Strength, try 0.209" thick plate
 from AASHTO 1.13.4 Table

$$\text{Joint Strength} = 67.0 \text{ K/ft.} > 61.875 \text{ K/ft.} \quad \text{ok}$$

* AASHTO 1.13.5

** AASHTO 1.13.9

For 0.209" PL;

$$A = (0.264 \frac{\text{in}^2}{\text{inch}})(12 \frac{\text{in}}{\text{ft}}) = 3.168 \frac{\text{in}^2}{\text{foot}}$$

$$A_{\text{eff}} = (0.132 \frac{\text{in}^2}{\text{inch}})(12 \frac{\text{in}}{\text{ft}}) = 1.584 \frac{\text{in}^2}{\text{foot}}$$

$$I = (0.087 \frac{\text{in}^4}{\text{inch}})(12 \frac{\text{in}}{\text{ft}}) = 1.044 \frac{\text{in}^4}{\text{foot}}$$

$$r_{\text{eff}} = 0.44$$

INITIAL SUPPORT
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LINER PLATE"
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Check for Buckling

$$* \text{ Critical Diameter, } D_c = \frac{r}{K} \sqrt{\frac{24E}{f_u}} = \frac{0.44}{0.44} \sqrt{\frac{24(29 \times 10^6 \text{ psi})}{42,000 \text{ psi}}}$$

$$D_c = 128.73" = 10.73 \text{ ft.}$$

$$\text{Diameter} = 15'-0" > \text{Critical Diameter} = 10.73'$$

∴ Use Buckling Formula

$$f_b = \frac{12E}{\left(\frac{KD}{r}\right)^2} = \frac{12(29 \times 10^6 \text{ psi})}{\left(\frac{0.44(15')(12 \frac{\text{in}}{\text{ft}})}{0.44}\right)^2} = 10,740.74 \frac{\text{lb}}{\text{in}^2}$$

$$T_{\text{allowable}} = \frac{f_b A_{\text{eff}}}{F.S.} = \frac{(10,740.74 \text{ psi})(1.584 \frac{\text{in}^2}{\text{ft}})}{2.0}$$

$$T_{\text{allowable}} = 8,506.7 \frac{\text{lb}}{\text{foot}} = 8.507 \text{ K/ft.} > 20.625 \text{ K/ft.}$$

∴ Must use Ribs or Larger PL
to handle Buckling Loads

Try 0.375" PL

$$A = (0.460 \frac{\text{in}^2}{\text{inch}})(12 \frac{\text{in}}{\text{ft}}) = 5.52 \frac{\text{in}^2}{\text{foot}}$$

$$A_{\text{eff}} = (0.230 \frac{\text{in}^2}{\text{inch}})(12 \frac{\text{in}}{\text{ft}}) = 2.76 \frac{\text{in}^2}{\text{foot}}$$

$$I = (0.143 \frac{\text{in}^4}{\text{inch}})(12 \frac{\text{in}}{\text{ft}}) = 1.716 \frac{\text{in}^4}{\text{foot}}$$

$$r_{\text{eff}} = 0.52$$

* AASHTO 1.13.6

$$* D_c = \frac{r}{K} \sqrt{\frac{24E}{f_u}} = \frac{0.52}{0.44} \sqrt{\frac{24(29 \times 10^6 \text{ psi})}{42,000 \text{ psi}}}$$

$$D_c = 152.14" = 12.68 \text{ ft.}$$

$$D = 15' > D_c = 12.7'$$

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LINER PLATE"
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∴ Use Buckling Formula

$$f_b = \frac{12E}{\left(\frac{K D}{r}\right)^2} = \frac{12(29 \times 10^6 \text{ psi})}{\left(\frac{0.44(15')(12' / 1)}{0.52}\right)^2} = 15,000 \text{ psi}$$

$$T_{\text{allowable}} = \frac{f_b A_{\text{eff}}}{F.S.} = \frac{15,000 \text{ psi} (2.76 \text{ in}^2 / \text{ft})}{2.0} = 20,700 \text{ lb} / \text{foot}$$

$$T_{\text{allowable}} = 20.7 \text{ k} / \text{ft} > T = 20.625 \text{ k} / \text{ft.} \quad \text{ok}$$

** Check for Handling Strength

$$\text{Minimum Stiffness} = \frac{EI}{D^2} = \frac{(29 \times 10^6 \text{ psi})(0.143 \text{ in}^4 / \text{inch})}{[15'(12' / 1)]^2} = 128.0$$

$$128 > 111 \text{ (Min. Stiffness for 4-Flange PL)}$$

* AASHTO 1.13.6

** AASHTO 1.13.5

Precast Concrete Segments
as Initial Support:

INITIAL SUPPORT
"PRECAST CONCRETE
SEGMENTS"
SHT. 1 of 6

Assumptions

- $\gamma_{\text{soil}} = 120 \text{ pcf}$
- $\gamma_{\text{concrete}} = 150 \text{ pcf}$
- $z = 30 \text{ ft}$
- $f'_c = 5,000 \text{ psi}$
- radius, $r = 7.5 \text{ ft}$
- A615, Grade 60 Steel
- *- $E_{\text{conc}} = 57,000 \sqrt{f'_c}$
 $= 4.03 \times 10^6 \text{ psi}$
- Segments are 6-inches
thick in a 3-piece
ring x 4-foot wide

Loads

- Ground Load; $p = \gamma_z = (120 \text{ pcf})(30 \text{ ft}) = 3,600 \text{ pcf}$
 $= 3.6 \text{ Ksf}$
- Ring Thrust
per Foot; $T = p \cdot r = (3.6 \text{ Ksf})(7.5 \text{ ft}) = 27 \text{ K/ft}$

Moment

Cracked EI (for a 1-inch length)

$$I = \frac{bh^3}{12} = \frac{(1'')(6'')^3}{12} = 18 \text{ in}^4/\text{inch}$$

$$** \frac{(EI)_c}{2.5} = \frac{(4.03 \times 10^6 \text{ psi})(18 \text{ in}^4/\text{inch})}{2.5} = 2.9 \times 10^7 \text{ in}^2 \cdot \text{lb}/\text{inch}$$

* ACI 318-83
8.5.1

** ACI 318-83
10.11.5

* Moment, $M = \frac{3(EI)_c}{R} \times \frac{\Delta D}{D}$, INITIAL SUPPORT
 "PRECAST CONCRETE
 SEGMENTS"
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where $\frac{\Delta D}{D} = 0.5\%$

$$M = \frac{3(2.9 \times 10^7 \text{ lb} \cdot \text{in}^2 / \text{inch})}{7.5 \text{ ft.} (12 \text{ in} / \text{ft.})} (0.005)$$

$$= 4,833 \text{ in.} \cdot \text{lb} / \text{inch}$$

$$M = 4,833 \text{ in.} \cdot \text{lb} / \text{inch} = 58 \text{ in.} \cdot \text{kips} / \text{foot}$$

Check as a Column subjected to Axial Load and Bending with $f'_c = 5 \text{ Ksi}$ & $f_y = 60 \text{ Ksi}$ using Ultimate Strength Design

$$\frac{P_u}{f'_c b t} = \frac{(27 \text{ k})(1.7)}{(5 \text{ Ksi})(6")(12")} = 0.1275$$

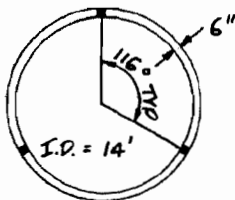
$$\frac{M_u}{f'_c b t^2} = \frac{(58 \text{ in.} \cdot \text{k})(1.7)}{(5 \text{ Ksi})(12")^2 (6")^2} = 0.046$$

** From Ultimate Strength Design Tables

$$P + m = 0 \quad m = \frac{f_y}{0.85 f'_c}$$

$\therefore P + m = 0$ No Steel Req'd.

Check Handling Loads



TYPICAL SEGMENT WEIGHT
 (over a 4' Width)

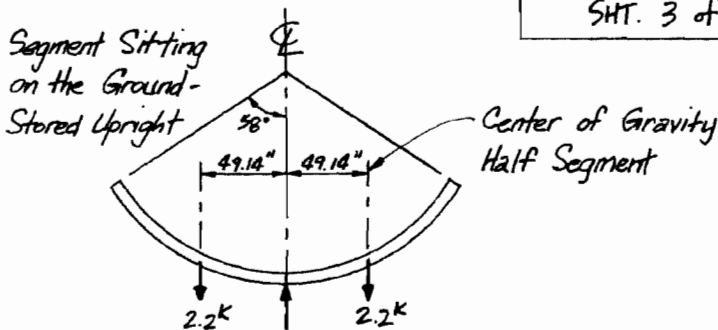
$$\text{Volume} = \pi D t (4') \left(\frac{116^\circ}{360^\circ} \right)$$

* DEGRE et al, 1969
 PECK, 1969
 ** ACI PUBLICATION
 SP-7

$$\text{Volume} = \pi (14.5') (0.5') \left(4' \left(\frac{116^\circ}{360}\right)\right) = 29.36 \text{ ft}^3$$

$$\text{Segment Wt.} = (29.36 \text{ ft}^3) (0.15 \text{ Kcf}) = 4.4 \text{ Kips}$$

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SEGMENTS"
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* Find Center of Gravity Location

$$A = 2 \Delta R t \quad \text{where } \Delta = 58^\circ \text{ in Radians}$$

$$A = 2 (1.012291) (7.5') (0.5') = 7.592 \text{ ft}^2$$

$$y_2 = R \left(\frac{\sin \Delta}{\Delta} - \cos \Delta \right) = 7.5' (0.307832) = 2.30874 \text{ ft.}$$

$$y_1 = R \left(1 - \frac{\sin \Delta}{\Delta} \right) = 7.5' (0.162249) = 1.21687 \text{ ft.}$$

$$R - 1.21687' = 7.5' - 1.21687' = 6.28313 \text{ ft.}$$

$$\text{Center of Gravity} = 4.0954 \text{ ft.} = 49.14 \text{ inches}$$

At Φ of Segment

$$\text{Shear Force, } V = 2,200 \text{ lbs.} \rightarrow \frac{2,200 \text{ lbs.}}{4 \text{ ft.}} = 550 \text{ lb./ft.}$$

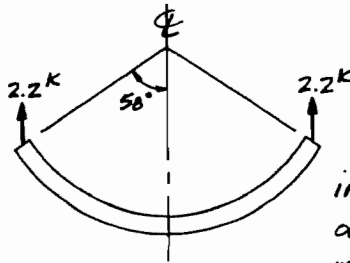
$$\text{Moment, } M = (2,200 \text{ lbs.}) (49.14 \text{ in.}) = 108,108 \text{ in.-lbs.}$$

$$\text{Design Moment, } M_u = 1.7 (108,108 \text{ in.-lbs.}) = 183,784 \text{ in.-lbs.}$$

↑ Conservative

* After Roark, 1965

"Formulas of Stress and Strain"
pg. 76, no. 12, 4th Edition

Check Segment Lifted at Ends

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inside radius = 7'
outside radius = 7.5'
median radius = 7.25'

At C of Segment

$$\text{Moment, } M = (2,200 \text{ lbs.})(7.25' \sin 58^\circ) = 13,526 \text{ ft}\cdot\text{lbs}$$

$$= 162,316 \text{ in}\cdot\text{lbs}$$

$$\text{Design Moment, } M_u = 1.7(162,316 \text{ in}\cdot\text{lbs.}) = 275,938 \text{ in}\cdot\text{lbs}$$

↙ conservative

Analysis of Section

$$h = 6" \therefore d = 4.75"$$

$$b = 12"$$

$$f'_c = 5 \text{ Ksi}$$

$$f_y = 60 \text{ Ksi}$$

$$\phi \text{ bending} = 0.90$$

$$M_u = 68,985 \text{ in}\cdot\text{lbs/foot}$$

$$\text{Assume } \#5\text{'s @ } 12" \text{ OC/face} \rightarrow A_s/\text{foot} = 0.31 \text{ in}^2/\text{foot/face}$$

Allowable

$$\text{Ultimate - } M_{uA} = FK_u \text{ where } F = \frac{bd^2}{12,000} = \frac{(12")(4.75")^2}{12,000}$$

Moment

$$F = 0.023$$

$$q = \frac{A_s}{bd} \cdot \frac{f_y}{f'_c} = \frac{0.31}{(12)(4.75)} \cdot \frac{60}{5}$$

$$q = 0.0653$$

$$K_U = \phi f'_c q (1 - 0.59 q)$$

$$= 0.9(5,000)(0.0653) [1 - 0.59(0.0653)]$$

$$= 282.53$$

$$\therefore M_{UA} = FK_U = (0.023)(282.53) = 6.5 \text{ K-ft}$$

$$= 78,000 \text{ in.-lbs/foot/face}$$

INITIAL SUPPORT
"PRECAST CONCRETE
SEGMENTS"
SHT. 5 of 6

$$M_{UA} = 78,000 \text{ in.-lbs/foot/face} > 68,985 \text{ in.-lbs/foot/face} \text{ ok}$$

$$\rho = \frac{A_s}{bd} = \frac{0.31 \text{ in}^2/\text{foot/face}}{(12 \text{ in})(4.75 \text{ in})} = 0.0054$$

$$* \rho_{\min} = \frac{200}{f_y} = \frac{200}{60,000 \text{ psi}} = 0.0033 < 0.0054 \text{ ok}$$

Check Shear Reinforcement for Complete Ring

** Shear Strength
Provided by Conc., $V_c = [1.9\sqrt{f'_c} + 2,500\rho] \frac{V_{ud}}{M_u} bd$

*** Maximum Shear, $V_{\max} = 0.5 \frac{\Delta D}{D} \left(\frac{12 EI}{R^3} \right) R$

with $\frac{\Delta D}{D} = 0.5\%$

$$E_{\text{conc.}} = 57,000 \sqrt{f'_c} = 57,000 \sqrt{5,000} = 4.03 \times 10^6 \text{ psi}$$

$$\therefore V_{\max} = \frac{0.5(0.005)12(4.03 \times 10^6 \text{ psi})(18 \text{ in}^4/\text{in})}{[7.5 \text{ ft.}(12 \text{ in}/\text{ft})]^3} \times (7.5 \text{ ft.})(12 \text{ in}/\text{ft})$$

$$V_{\max} = 268.7 \text{ lbs/foot} \quad V_u = 1.7(268.7 \frac{\text{lbs}}{\text{ft}}) = 456.7 \text{ lbs.}$$

* ACI 318-83
Eq'n. 10-3

*** SELANDER et al, 1980

** ACI 318-83
Eq'n. 11-6

Concrete

$$\text{Shear Strength } V_c = \left[1.9 \sqrt{5,000} + 2,500 \times \right. \\ \left. (0.054) \frac{456.7 (4.75 \text{ in})}{68,985 \text{ in}^2 \cdot \text{lbs}} \right] \\ \times (12 \text{ in}) (4.75 \text{ in}) \\ V_c = 7,682.2 \text{ lbs/foot}$$

INITIAL SUPPORT
"PRECAST CONCRETE
SEGMENTS"
SHT. 6 of 6

$$\text{Shear from Handling Loads, } \frac{2,200 \text{ lbs}}{4 \text{ ft.}} = 550 \text{ lbs/ft.}$$

$$V_c = 7,682 \frac{\text{lbs}}{\text{ft.}} \gg 550 \frac{\text{lbs}}{\text{ft.}} > 456.7 \frac{\text{lbs}}{\text{ft.}} \quad \text{OK}$$

→ No Shear Reinforcement is Req'd.; however, it will be provided to tie together the two mats of Steel

$$* \text{ Minimum } A_v = \frac{50 \text{ bs}}{f_y} = \frac{50 (12 \text{ in}) (12 \text{ in})}{60,000} = 0.12 \text{ in}^2/\text{foot}$$

$$\text{Provide: } \#3\text{'s @ } 12" \text{ OC } \quad A_s = 0.11 \text{ in}^2/\text{foot} \quad \text{OK}$$

Steel Segmented Liner
as Initial Support:

INITIAL SUPPORT
"STEEL
SEGMENTED LINER"
SHT. 1 of 3

Assumptions

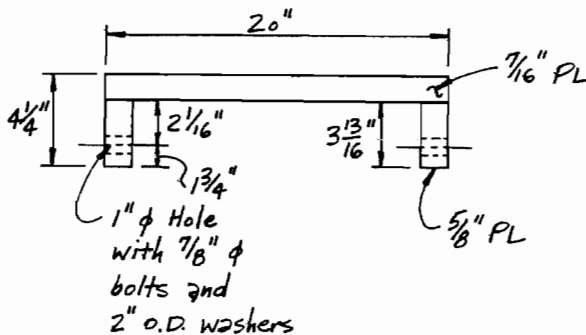
- $\gamma_{\text{soil}} = 120 \text{ pcf}$
- $z = 30 \text{ ft.}$
- Diameter = 14'-0"
- $K_{\text{soil}} = 0.22$
- ASTM A36 Steel, $F_y = 36 \text{ Ksi}$
- Allowable Stress = $0.6 F_y = 21.6 \text{ Ksi}$
- $E = 29 \times 10^6 \text{ psi}$

Loads

Ground Load, $p = \gamma_z = 120 \text{ pcf} (30 \text{ ft.}) = 3,600 \text{ psf}$
 Ring Thrust
 per Foot, $T = p \cdot r = (3.6 \text{ Ksf})(7 \text{ ft.}) = 25.2 \text{ K/ft.}$

Steel

Assume $7/16"$ PL with



Moment of Inertia

$$\text{SKIN}, I_o = \frac{1}{12} b h^3 = \frac{1}{12} (20") (4.375")^3 = 0.1396 \text{ in}^4$$

$$\text{Flange}, I_o = \frac{1}{12} (2) (0.625") (3.8125")^3 = 5.7724 \text{ in}^4$$

$$\text{Hole}, I_o = \frac{1}{12} (1.25") (1")^3 = 0.1042 \text{ in}^4$$

INITIAL SUPPORT

"STEEL

SEGMENTED LINER"

SHT. 2 of 3

— SEGMENT PROPERTIES —

| ITEM | b (in) | d (in) | A (in ²) | Y _s | A _{Y_s} | Y _n | A _{Y_n} ² | I _o | I = I _o + A _{Y_n} ² |
|--------|-----------|-----------|-------------------------|----------------|----------------------------|----------------|---|----------------|--|
| SKIN | 20 | 4.375 | 8.75 | 2.1875 | 1.9141 | .5933 | 3.080 | .1396 | 3.2196 |
| FLANGE | 1.25 | 3.8125 | 4.7656 | 2.34375 | 11.1694 | 1.53175 | 11.1811 | 5.7724 | 16.9535 |
| HOLE | 1.25 | 1.00 | -1.25 | 2.5 | -3.125 | 1.688 | -3.5417 | -.104 | -3.6657 |
| | | | 12.2656 | .812 | 9.9585 | | 10.6994 | 5.808 | 16.5074 |

Segment Property Summary:

$$\text{- Area} = 12.27 \text{ in}^2$$

$$\text{- I} = 16.51 \text{ in}^4$$

$$\text{- } C_1 = .812$$

$$\text{- } C_2 = 3.438$$

$$\text{- Area/foot} = 7.359 \text{ in}^2/\text{foot}$$

$$\text{- I/foot} = 9.904 \text{ in}^4/\text{foot}$$

$$\text{- } S_1 = I/C_1 = 9.904/.812 = 12.197$$

$$\text{- } S_2 = I/C_2 = 9.904/3.438 = 2.881$$

Stresses

Compressive Stress

Per Foot

$$\sigma_c = \frac{T}{A} = \frac{25.2 \text{ k/ft}}{7.359 \text{ in}^2/\text{ft}} = \sim 3,425 \text{ psi}$$

Bending Stress

Per Foot

$$\sigma_B = \frac{M_c}{I} \quad \text{where } M = \frac{3EI}{R} \times \frac{\Delta D}{D}$$

$$\sigma_B = \frac{3EI}{R} \times \frac{\Delta D}{D} \times \frac{C}{I} = \frac{3EC}{R} \times \frac{\Delta D}{D}$$

$$\sigma_B = \frac{3(29 \times 10^6 \text{ psi})C}{(7 \text{ ft})(12 \text{ in/ft})} \times (0.005) = 5,178.57c \text{ (psi)}$$

$$\text{Total Stress per Foot, } \sigma_T = \pm \sigma_B + \sigma_c$$

INITIAL SUPPORT
 "STEEL
 SEGMENTED LINER"
 SHT. 3 of 3

| * | CROWN & INVERT | | SPRINGLINE | |
|-------------|---------------------|----------------------|---------------------|----------------------|
| COMPRESSION | +3,425 psi | +3,425 psi | +3,425 psi | +3,425 psi |
| BENDING | -17,804 psi | +4,205 psi | +17,804 psi | -4,205 psi |
| TOTAL | -14,379 psi | +7,630 psi | +21,229 psi | -780 psi |
| | <i>inside fiber</i> | <i>outside fiber</i> | <i>inside fiber</i> | <i>outside fiber</i> |

$$\text{Max } \sigma_{\text{TOTAL}} = 21.2 \text{ Ksi} < 0.6 F_y = 21.6 \text{ Ksi} \quad \text{ok}$$

* AISC 1.5.1.4.4

FINAL SUPPORT
"EMPERICAL METHOD"
SHT. 1 of 2

Empirical Method:

* Assumptions

$$-\gamma_{\text{soil}} = 120 \text{ pcf}$$

$$-z = 30 \text{ ft.}$$

$$-f'_c = 4,000 \text{ psi}$$

$$-I.D. = 12'-0"$$

$$O.D. = 13'-6"$$

- Assume a 9" cast-in-place final lining

$$\therefore R = 6.75 \text{ ft.}$$

Loads

Ground load acts on O.D. of CIPC Final Lining: Tunnel is relatively shallow, therefore, Design for Full Overburden...

$$\begin{aligned} \text{Ground Load, } p = \gamma z &= (120 \text{ pcf})(30 \text{ ft}) = 3,600 \text{ pcf} \\ &= 3.6 \text{ Ksf} \end{aligned}$$

Ring Thrust

$$\text{per foot of Tunnel} \rightarrow T = p \cdot r = (3.6 \text{ Ksf})(6.75 \text{ ft}) = 24.3 \text{ K/ft.}$$

Bending Moment

$$** \text{ Moment, } M = \frac{+3EI}{R} \left(\frac{\Delta D}{D} \right) \quad \text{where } \frac{\Delta D}{D} = 0.5\%$$

FOR LONG TERM
MOVEMENT, ASSUME
 $\frac{1}{2}$ OF $\frac{\Delta D}{D} = 0.5\%$

However, Final Lining will be installed within an initial support system; Therefore assume:

$$\frac{\Delta D}{D} \approx \frac{.005}{2} = .0025$$

USE $\frac{\Delta D}{D} = .003$

* PECK, 1969

** DEERE et al, 1969
PECK, 1969

Uncracked Section

$$E = 57,000 \sqrt{4,000} = 3.6 \times 10^6 \text{ psi}$$

| |
|---|
| FINAL SUPPORT "EMPERCAL METHOD" SHT. 2 of 2 |
|---|

Cracked Section (for a 1" length)

$$I = \frac{bh^3}{12} = \frac{(1'')(9'')^3}{12} = 60.75 \text{ in}^4/\text{inch}$$

$$\frac{(EI)_c}{2.5} = \frac{(3.6 \times 10^6 \text{ psi})(60.75 \text{ in}^4/\text{inch})}{2.5} = 8.748 \times 10^7 \text{ in}^2 \cdot \text{lb}/\text{inch}$$

$$\text{Moment, } M = \frac{3EI}{R} \cdot \frac{\Delta D}{D} = \frac{3(8.748 \times 10^7 \text{ in}^2 \cdot \text{lb}/\text{inch})}{(6.75 \text{ ft.})(12 \text{ in}/\text{ft.})} \times 0.003$$

$$M = 3,240 \text{ in} \cdot \text{lb}/\text{inch} = 38.9 \text{ in} \cdot \text{K}/\text{foot}$$

Check as a Column Subjected to Axial Load and Bending with $f'_c = 4 \text{ Ksi}$ & $f_y = 60 \text{ Ksi}$ using Ultimate Strength Design.

$$\frac{P_u}{f'_c b t} = \frac{1.7(24.3 \text{ K}/\text{ft})}{(4 \text{ Ksi})(12 \text{ in})(9 \text{ in})} = 0.0956$$

$$\frac{M_u}{f'_c b t^2} = \frac{1.7(38.9 \text{ in} \cdot \text{K}/\text{ft})}{(4 \text{ Ksi})(12 \text{ in})(9 \text{ in})^2} = 0.017$$

* From Ultimate Strength Design Tables

$$p + m = 0 \quad m = \frac{f_y}{0.85 f'_c}$$

$$\therefore p + m = 0 \quad \text{No Steel Req'd.}$$

+ Closed Form Relative Stiffness Method :

Assumptions

- 9" Cast-in-place lining - $\gamma_{soil} = 0.4$
- I.D. = 12'-0" - $\gamma_{conc} = 0.15$
- o.D. = 13'-6" - $\gamma_{soil} = 120 \text{ pcf}$
- $z = 30 \text{ ft.}$ - $K_o = 0.5$
- $f'_c = 4 \text{ Ksi}$ - Radius, $R = 6.75 \text{ ft.} = 81 \text{ in}$
- $E_{soil} = 5,000 \text{ psi}$ (assume stiff clay)
- $E_{conc} = 57,000 \sqrt{f'_c}$; for long-term creep & cracking use $\frac{1}{2} E_{conc} \therefore E_{conc} = 1.8 \times 10^6 \text{ psi}$

FINAL SUPPORT
"CLOSED FORM
RELATIVE
STIFFNESS METHOD"
SHT. 1 of 4

Relative Stiffness Analysis

$$C^* = \frac{E_{soil} R (1 - \gamma_{conc}^2)}{E_{conc} A_{conc} (1 - \gamma_{soil}^2)} = \frac{(5,000 \text{ psi})(81 \text{ in}) [1 - 0.15^2]}{(1.8 \times 10^6 \text{ psi})(9 \text{ in}^2/\text{inch})(1 - 0.4^2)}$$

$$C^* = 0.0291$$

$$F^* = \frac{E_{soil} R^3 (1 - \gamma_{conc}^2)}{E_{conc} I_{conc} (1 - \gamma_{soil}^2)} = \frac{(5,000 \text{ psi})(81 \text{ in})^3 (1 - 0.15^2)}{(1.8 \times 10^6 \text{ psi}) \left[\frac{(1^4)(9 \text{ in}^4)}{12} \right] (1 - 0.4^2)}$$

$$F^* = 28.3$$

$$a_o^* = \frac{C^* F^* (1 - \gamma_{soil})}{C^* + F^* + C^* F^* (1 - \gamma_{soil})} = \frac{(0.0291)(28.3)(1 - 0.4)}{(0.0291) + (28.3) + (28.3)(0.0291)(1 - 0.4)}$$

$$a_o^* = 0.01714$$

+ After SCHWARTZ
and EINSTEIN, 1980

$$a_z^* = \frac{(F^* + b)(1 - \gamma_{soil})}{2F^*(1 - \gamma_{soil}) + 6(5 - 6\gamma_{soil})}$$

$$= \frac{(28.3 + 6)(1 - 0.4)}{2(28.3)(1 - 0.4) + 6[5 - 6(0.4)]}$$

$$a_z^* = 0.4153$$

FINAL SUPPORT
"CLOSED FORM"
RELATIVE
STIFFNESS METHOD"
SHT. 2 of 4

Full Slip Case, assume $K_0 = 0.5$

Thrust

$$\frac{T}{pr} = \frac{1}{2}(1 + K_0)(1 - a_0^*) + \frac{1}{2}(1 - K)(1 - 2a_z^*) \cos 2\theta$$

Moment

$$\frac{M}{pr^2} = \frac{1}{2}(1 - K)(1 - 2a_z^*) \cos 2\theta$$

Stresses in Crown ($\theta = 90^\circ$)

$$\frac{T}{pr} = \frac{1}{2}(1 + 0.5)(1 - 0.01714) + \frac{1}{2}(1 - 0.5)(1 - 2(0.4153)) \cos 180^\circ$$

$$\frac{T}{pr} = 0.695$$

$$\frac{M}{pr^2} = \frac{1}{2}(1 - 0.5)(1 - 2(0.4153)) \cos 180^\circ$$

$$\frac{M}{pr^2} = -0.0424$$

Tunnel is at \Rightarrow Depth of 30ft. ($z = 30'$)

$$\therefore \text{Ground in situ Stress} = 30'(120 \text{ pcf}) = 3,600 \text{ psf}$$

$\approx 25 \text{ psi}$

$$\text{Thrust, } T = (0.695)(25 \text{ psi})(81 \text{ in}) = 1,407.4 \text{ lbs/inch of tunnel}$$

$$\text{Moment, } M = (-0.0424)(25 \text{ psi})(81 \text{ in})^2 = -6,955 \text{ in.lbs/inch of tunnel}$$

For a 9" x 1" Section of Tunnel:

Crown Stresses

$$\begin{aligned}\sigma_{\max, \min} &= \frac{T}{A} \pm \frac{M_c}{I} \\ &= \frac{1,407.4 \frac{\text{lb}}{\text{inch}}}{(1 \text{ in})(9 \text{ in})} \pm \frac{(-6,955 \frac{\text{in} \cdot \text{lb}}{\text{inch}})(\frac{9 \text{ in}}{2})}{\left[\frac{(1 \text{ in})(9 \text{ in})^3}{12} \right]}\end{aligned}$$

$$\sigma_{\max, \min} = 156.4 \text{ psi} \pm -515.2 \text{ psi}$$

- $\sigma_{\max} = 671.6 \text{ psi}$ (compression)
- $\sigma_{\min} = -358.8 \text{ psi}$ (tension) $< 6\sqrt{f'_c}$

Check Stresses at Springline ($\theta = 0^\circ$)

$$\frac{T}{pr} = 0.779 \quad \frac{M}{pr^2} = +0.0424$$

$$\text{Thrust, } T = (0.779)(25 \text{ psi})(81 \text{ in}) = 1,577.5 \text{ lbs/inch of tunnel}$$

$$\text{Moment, } M = (+0.0424)(25 \text{ psi})(81 \text{ in})^2 = +6,955 \text{ in} \cdot \text{lbs/inch of tunnel}$$

For a 9" x 1" Section of Tunnel:

Springline Stresses

$$\sigma_{\max, \min} = \frac{T}{A} \pm \frac{M_c}{I} = \frac{(1,577.5 \frac{\text{lb}}{\text{inch}})}{(9 \text{ in})(1 \text{ in})} \pm \frac{(6,955 \frac{\text{in} \cdot \text{lb}}{\text{inch}})(\frac{9 \text{ in}}{2})}{\left[\frac{(1 \text{ in})(9 \text{ in})^3}{12} \right]}$$

$$\sigma_{\max, \min} = 175.3 \text{ psi} \pm 515.2 \text{ psi}$$

- $\sigma_{\max} = 690.5 \text{ psi}$ (compression)
- $\sigma_{\min} = -339.9 \text{ psi}$ (tension)

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"CLOSED FORM
RELATIVE
STIFFNESS METHOD"
SHT. 3 of 4

Relative Stiffness Analysis Summary

Crown:

$$T = 1,407.4 \frac{\text{lb}}{\text{inch}} \quad \sigma_{\max} = 671.6 \text{ psi (comp.)}$$

$$M = -6,955 \frac{\text{in} \cdot \text{lb}}{\text{inch}} \quad \sigma_{\min} = -358.8 \text{ psi (tension)}$$

Springline:

$$T = 1,577.5 \frac{\text{lb}}{\text{inch}} \quad \sigma_{\max} = 690.5 \text{ psi (comp.)}$$

$$M = +6,955 \frac{\text{in} \cdot \text{lb}}{\text{inch}} \quad \sigma_{\min} = -339.9 \text{ psi (tension)}$$

- Maximum Moment = $6,955 \frac{\text{in} \cdot \text{lb}}{\text{inch}} = 6.96 \frac{\text{in} \cdot \text{K}}{\text{inch}} = 83.5 \frac{\text{in} \cdot \text{K}}{\text{foot}}$
- Maximum Thrust = $1,577.5 \frac{\text{lb}}{\text{inch}} = 1.577 \text{ K/inch} = 18.9 \text{ K/foot}$

Use Ultimate Strength Design

$$\frac{P_u}{f'_c b t} = \frac{1.7 (18.9 \text{ K/ft})}{(4 \text{ Ksi})(12 \text{ in})(9 \text{ in})} = 0.0744$$

$$\frac{M_u}{f'_c b t^2} = \frac{1.7 (83.5 \text{ in} \cdot \text{K/foot})}{(4 \text{ Ksi})(12 \text{ in})(9 \text{ in})^2} = 0.037$$

* From Ultimate Strength Design Tables

$$p_t = \frac{0.033}{17.65} = \underline{0.002} ; \text{ where } p_t m = 0.033$$

$$\text{and } m = \frac{f_y}{0.85 f'_c} = \frac{60}{0.85(4)} = 17.65$$

** Will provide minimum flexural steel

$$p_{\min} = \frac{200}{f_y} = 0.0033$$

* ACI PUBLICATION SP-7

∴ Req'd. Steel, #4's @ 12" OC
each face ok

** ACI 318-83
10.5.1

FINAL SUPPORT
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RELATIVE
STIFFNESS METHOD"
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