BIG TUNNELS IN BAD ROCK

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(The Thirty-Sixth Karl Terzaghi Lecture)

ABSTRACT: Tunnels of 10 to 16 m span are frequently constructed for hydroelectric or transportation projects, and many of these tunnels are excavated in rock masses of very poor quality. When the ratio of rock mass strength to in situ stress falls below 0.2, squeezing of the rock mass becomes a problem that can cause instability of both the tunnel and the face. A method for predicting squeezing conditions is presented and the practical options for preinforcing the face and supporting the tunnel to deal with these problems are discussed. Two case histories are included to illustrate how these preinforcement and support measures can be incorporated into a tunnel design. Brief discussions are also given on water problems in tunneling, the use of tunnel boring machines (TBMs) in squeezing ground, and the construction costs for large tunnels in varying ground conditions.

INTRODUCTION

“Rock defects and loads on tunnel supports,” by Karl Terzaghi (1946), was a landmark paper in tunneling literature, and for many years it provided the basis for the rational design of tunnels, particularly those constructed in North America. There are still many valuable lessons to be learned from this work, and it is recommended reading for anyone seriously interested in the practical aspects of tunnel design and construction.

The “tunnel supports” discussed by Terzaghi were primarily steel sets, and these were designed to support the “rock load” due to the weight of the broken ground resulting from the excavation of the tunnel. This concept is illustrated in Fig. 1, and Terzaghi developed a set of guidelines for estimating the rock load for different geological conditions.

An alternative tunnel-support design method was developed in Europe; its origins can be traced to a paper by Fenner (1938). This method is based upon the development of a “plastic zone” in the rock mass surrounding a tunnel, as illustrated in Fig. 2.

The support pressure $p_i$ in Fig. 2 is that provided by the rock mass through which the tunnel is being advanced. At a distance of approximately one diameter ahead of the tunnel, the rock mass is not influenced by the presence of the tunnel and the support pressure $p_i$ equals the in situ stress $p_o$, corresponding to point A on the ground response curve. As the tunnel advances, the support provided by the rock mass diminishes and the rock mass responds elastically up to point B, at which plastic failure of the rock mass initiates. The radius $r_p$ of the plastic zone and the radial convergence $\delta$ both increase as the support pressure decreases, as illustrated in Fig. 2. Eventually, about two tunnel diameters behind the face, the support pressure $p_i$ provided by the face has decreased to zero and the radial convergence $\delta$ reaches its final value.

Tunnel support, which may be steel sets, rockbolts, shotcrete, or some combination of these, is installed after the tunnel has converged a distance $\delta$. This support acts like a spring with the support that it provides to the tunnel increasing with convergence of the tunnel. The system reaches equilibrium at point C, where the ground response curve and the support reaction line intersect.

Terzaghi (1925) published one of the first solutions for the elastoplastic stress distributions around a cylindrical underground opening, but he did not apply his calculations to the design of tunnel support systems. Between the publication of Fenner’s paper in 1938 and 1983, there were at least another 21 papers describing alternate solutions for the rock-support interaction analysis. Brown et al. (1983) reviewed these solutions and also published their own analysis. There have been several additional rock-support interaction solutions published since 1983.

The behavior of the tunnel face was not considered in either the Terzaghi “rock load” design method or the rock-support interaction analysis. This omission was not important for relatively shallow, small tunnels, since it was usually possible to devise some practical means for supporting the face if this proved to be necessary. However, as both the size of tunnels and their depth below surface increased, the stability of the face became a serious issue. This is illustrated in Fig. 3, which shows the plastic extrusion of a tunnel face as determined by means of an axisymmetric finite-element model. Lunardi (2000) has suggested that understanding and controlling the behavior of the “core” ahead of the advancing tunnel face is the secret to successful tunneling in squeezing ground conditions.

TUNNEL FACE STABILITY

The ground reaction curves plotted in Fig. 4 were calculated for an 8-m-diameter tunnel using the parameters defined in the figure. Displacements were measured at a point 20 m behind the tunnel face (A) and in the center of the face (B), and these values define the ground response curves illustrated. It is clear that, for this example, the tunnel face follows the same general deformation pattern as the tunnel itself although the displacements are about 30% smaller. Note that, for support pressures of greater than 6 MPa, the rock mass behaves elastically and both of the displacement curves follow straight lines to the point (0, 12).

The practical consequence of the observation that both the tunnel face and the tunnel walls behave in a similar manner is that, when it becomes necessary, the tunnel face has to be stabilized in order to provide safe working conditions and to ensure that the tunnel can be advanced. It would clearly be of great benefit to the tunnel designer to know the conditions that can give rise to instability of the face and the tunnel and how much effort has to be expended to stabilize both.

PREDICTION OF TUNNEL AND FACE INSTABILITY

In order to analyze the behavior of the tunnel and its face under a variety of conditions, some means of estimating the
properties of the rock mass is required. The system proposed by Hoek and Brown (1980, 1997) is one of the most widely accepted means for assessing rock mass properties, and this system will be used here. Hoek and Marinos (2000) have described recent modifications to this system and its application to rock mass of poor quality.

Hoek and Marinos (2000) showed that a plot of tunnel strain against the ratio of rock mass strength to in situ stress provides a basis for estimating the potential for tunnel instability. In this context, strain is defined as the percentage ratio of tunnel wall deformation to tunnel radius. The plot referred to was produced by carrying out Monte Carlo analyses, using the two-dimensional closed-form analytical solutions by Duncan-Fama (1993) and Carranza-Torres and Fairhurst (1999), for a very wide range of rock mass properties and in situ stress conditions. This analysis was for the tunnel only, since neither of these solutions consider the stability of the face.

Similar curves relating strain to the ratio of rock mass strength to in situ stress can be generated for the three-dimensional case by means of the axisymmetric finite-element model that was used to produce the results presented in Figs. 3 and 4. (Note: All of the finite-element studies reported in this paper were carried out using the model Phase2. Details of this model are available from www.rocscience.com.) These curves are plotted in Fig. 5 and they show that the strain increases asymptotically when the ratio of rock mass strength to in situ stress falls below about 0.2. This indicates the onset of severe instability, and without adequate support the tunnel and the face would both collapse.

The influence of internal support pressure $p_s$ upon the strain of the tunnel and the face was also investigated by means of the axisymmetric finite-element model. This was done for a range of different rock masses, in situ stresses, and support pressures. Curve fitting to the results of these analyses gave the following approximate relationships for the strain of the tunnel $e_t$ and the face $e_f$ and the ratio of support pressure to in situ stress:

$$
\frac{e_t}{\%} = 0.15(1 - \left(\frac{p_s}{p_i}\right)) \frac{\sigma_{ct}}{p_s} \left[\left(\frac{p_s}{p_i}\right)^{1.38} + 0.04 \left(\frac{p_s}{p_i}\right)^{0.54}\right]
$$

(1)

$$
\frac{e_f}{\%} = 0.1(1 - \left(\frac{p_s}{p_i}\right)) \frac{\sigma_{cf}}{p_s} \left[\left(\frac{p_s}{p_i}\right)^{1.38} + 0.04 \left(\frac{p_s}{p_i}\right)^{0.54}\right]
$$

(2)

where $\sigma_{ct}$ and $\sigma_{cf}$ are the triaxial compressive strength of the tunnel and face, respectively.
Note that these relationships probably represent lower bound conditions since they were all derived from axisymmetric finite-element analysis assuming zero dilation of the rock mass. This assumption is considered appropriate for the very poor quality rock masses considered here. On the other hand, the curves published by Hoek and Marinos (2000) were derived for a wider range of mass properties, including, in some cases, significant dilation. In addition, for a given set of rock mass properties and in situ stresses, the two-dimensional closed-form solutions predicted larger displacements than the corresponding axisymmetric finite-element analyses.

CASE HISTORIES OF SQUEEZING TUNNELS

Based on field observations and measurements, Sakurai (1983) suggested that tunnel strain levels in excess of approximately 1% are associated with the onset of tunnel instability and with difficulties in providing adequate support. Field observations by Chern et al. (1998), plotted in Fig. 6, confirm Sakurai’s proposal.

Note that some tunnels which suffered strains as high as 4% did not exhibit stability problems. All the tunnels marked as having stability problems were successfully completed, but the construction problems increased significantly with increasing strain levels. Hence, the 1% limit proposed by Sakurai is only an indication of increasing difficulty and it should not be assumed that sufficient support should be installed to limit the tunnel strain to 1%. In fact, in some cases, it is desirable to allow the tunnel to undergo strains of as much as 5% before activating the support.

Apart from observations published by Lunardi (2000) there are very few reliable measurements of tunnel face extrusion. Hence, all of the field observations included in this discussion are for radial strain of the tunnel.

One of the problems in interpreting field observations of tunnel squeezing is that of estimating the influence of the tunnel support. It is particularly difficult when the support capacity is exceeded and where steel sets buckle, shotcrete cracks,
or rockbolts yield. The best that can be done under these circumstances is to plot the observations and to compare them with strain curves for a range of support pressures. This has been done in Fig. 7, which shows observed closures for a number of tunnels in Venezuela, Taiwan, and India. Details of the observations are tabulated in the Appendix.

Fig. 7 shows that the observations are in reasonable agreement with the squeezing behavior predicted by (1). The points marked 1, 2, 5, and 7 are for tunnels in which severe squeezing occurred and where extraordinary steps had to be taken to stabilize the tunnels.

The Yacambú-Quibor tunnel (Fig. 7, point 1) is a 5.5 span, 25-km-long water transmission tunnel being excavated through the Andes near the city of Barquisimeto in Venezuela. The maximum cover on this tunnel is 1,270 m and a significant proportion of the rock through which it is mined is graphitic phyllite. Construction of this tunnel, regarded by many as the most difficult tunnel in the world, commenced in 1975 and approximately 5 km remains to be excavated. In 1979 a tunnel boring machine was trapped by squeezing rock during a stoppage in the drive. The machine could not be restarted and the squeezing rock gradually filled all the cavities in the machine structure. The remains of this machine were removed several years later—Fig. 8 shows this excavation in progress. Complete closure of this tunnel occurred in several locations and a technique was eventually developed to control the stability of the tunnel by installing steel sets with sliding joints that locked after the tunnel had converged about 0.3 m (equivalent to a strain of approximately 6% after the installation of the sets). These sets were fully embedded in shotcrete except for 1-m-wide gaps over the sliding joints. After the tunnel face had advanced 5 to 10 m, the joints had closed and the sets began to accept load. The gaps were filled with shotcrete to complete the lining. Convergence measurements have shown that these sections are stable, and the long-term behavior of this support has been excellent (Sánchez Fernández and Terán Benítez 1994).

Point number 2 in Fig. 7 is for a 10-m-diameter hydropower headrace tunnel on the Nathpa Jhakri project in India (Hoek 1999). In a section of the tunnel passing through a wide fault zone under a cover of 300 m, severe deformations occurred, as illustrated in Fig. 9. The deformed section was remined.
under the protection of a forepole umbrella. The remainder of the fault zone was successfully traversed using this method. The installation of the forepoles is illustrated in Fig. 10.

Point number 7 in Fig. 7 is for the 16-m-span Mucha highway tunnel, in Taiwan (Chern et al. 1998). Inward displacements of the roof and sidewalls of approximately 1.2 m (equivalent to a strain of about 15%) occurred when a fault zone was encountered. The reduction in cross section of the tunnel meant that remining to the original tunnel profile was necessary. This involved heavy support using long tensioned grouted cables that were used to support the failed rock mass while it was mined section by section. The remined section was stabilized by additional tensioned grouted cables, and the final concrete lining was placed as soon as possible after completion of the remedial work (Fig. 11).

SUPPORT OPTIONS FOR SQUEEZING GROUND

On the basis of the preceding discussion, the curve defined by (1) can be used to give a set of approximate guidelines on the degree of difficulty that can be encountered for different levels of strain. Since these strain levels are associated with specific ranges of the ratio of rock mass strength to in situ stress, the curve given in Fig. 12 can be used to give a first estimate of tunnel squeezing problems.

For example, during site investigations for a tunnel, one of the rock masses through which the tunnel will be excavated is identified as having strength $\sigma_{cm} = 1.5$ MPa (based upon the methodology described by Hoek and Marinos 2000). The tunnel will pass through this rock mass at an average depth of 500 m, which means that the vertical in situ stress will be $p_v = 13.5$ MPa. The usual assumption for very weak rock masses is that they are incapable of sustaining significant differential stresses and that failure occurs until the in situ horizontal and vertical stresses have been equalized. This assumption has been verified, for squeezing rocks, in a few cases in which the tunnel deformations have been back analyzed to estimate the ratio of horizontal to vertical in situ stresses.

For a rock mass strength $\sigma_{cm} = 1.5$ MPa and an in situ stress $p_v = 13.5$ MPa, the ratio $\sigma_{cm}/p_v = 0.11$. Fig. 12 shows that this corresponds to a strain of approximately 10%, and the tunnel designer should therefore anticipate having to deal with very severe squeezing problems in this section.
tunnels needs to be addressed since such tunnels, with spans ranging from 10 to 16 m, are becoming increasingly common in hydroelectric and transportation projects around the world.

From the preceding discussion it can be appreciated that the stability of the face of a tunnel is a critical factor in driving large tunnels through squeezing ground. Instability of the face not only creates extremely dangerous conditions for the workmen in the tunnel, but it also has a major impact on the subsequent behavior of the tunnel. Unless this instability is dealt with in an appropriate manner, significant damage may occur in the rock mass surrounding the tunnel due to the formation of cavities from collapse of material at the face or through gaps in the support system. This damage may require time-consuming and expensive treatment once the face has advanced through the fault or, if left untreated, it may cause problems later during the operating life of the tunnel.

If the fault is anticipated—for example, by probe drilling ahead of the face—and a well-designed plan of attack is developed by the tunnel designer and the contractor, the squeezing problems can usually be overcome. The rock mass surrounding the tunnel may be improved by grout injection, placement of grouted pipe forepoles, or reinforcement with grouted fiberglass dowels. While this treatment will be slow and expensive, it is more likely to succeed and to minimize subsequent problems than the more typical approach where no prereinforcement is used and where problems are dealt with as they are exposed in the face.

Methods for dealing with face stability in squeezing ground

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<td>Safety rockbolts in crown with 50 mm thick shotcrete</td>
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<td>Safety rockbolts, 50 mm thick shotcrete and face buttress</td>
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<td><strong>MINOR SQUEEZING</strong></td>
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<td>Steel sets in shotcrete with elephant foot and invert lining</td>
<td>Lattice girders, shotcrete, fiber-glass dowels grouted in face</td>
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<td>Partial face excavation, 150 mm thick shotcrete lining and invert</td>
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<td><strong>EXTREME SQUEEZING</strong></td>
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<td>Central pillar, lattice girders embedded in 250 mm thick shotcrete lining, no rockbolts</td>
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<td>Split into two smaller tunnels and use steel sets with sliding joints in 250 mm shotcrete</td>
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**FIG. 13.** Face Excavation and Support Options for Large Tunnels
have been developed mainly in Europe to deal with tunneling through the Alps (Schubert 1996). These methods can be divided into three distinct categories. One of these involves driving small-sized headings in advance of other portions of the face. This method, which tends to be favored by tunnel designers north of the Alps, relies on the fact that smaller faces require less support and the sequential construction process results in the creation of a very strong shotcrete shell. The alternative approaches, typically used by tunnel designers from south of the Alps, are to drive a tunnel full-face or by top heading and bench excavation, and to rely on reinforcement of the face and the rock mass surrounding the tunnel to stabilize the tunnel. Fig. 13 presents a brief summary of representative options for the control of tunnel face stability and the subsequent installation of support for the tunnel, while Fig. 14 shows a typical field installation in which many of the support elements have been incorporated.

All of the approaches illustrated in Fig. 13 have advantages and disadvantages, and there are no simple rules for deciding which method is better for a particular set of circumstances. For relatively mild squeezing conditions rockbolts and shotcrete are used as the primary elements in all of these support systems. In the case of the multiple heading method, the face is divided into a larger number of headings as squeezing becomes more severe. This ensures that the outer reinforced shotcrete shell is not overstressed at any stage in the excavation process. The stability of the smaller faces is also easier to control.

For the top heading and bench and full-face excavation options, heavier and more closely spaced steel sets are added as the severity of squeezing increases. For very severe squeezing conditions, grouted fiberglass dowels are added for face reinforcement and forepoles or similar reinforcing elements are used to prereinforce the rock mass ahead of the advancing face.

While this prereinforcement is very effective in protecting the rock core ahead of the face, it can become a liability once it is exposed in the tunnel. As illustrated in Fig. 14, the exposed ends of the forepoles have to be supported by steel sets installed as close to the face as possible. It is particularly important that foundation failure of the bases of the steel sets is prevented by the provision of some form of footing or anchoring system. A frequent design error is the use of excessively large forepoles that, while they provide good support for the rock mass ahead of the face, tend to overload the steel sets behind the face.

Eventually a point is reached where it is difficult to provide support of sufficient capacity, particularly if extremely severe squeezing is associated with very poor quality rock masses in which rockbolts are ineffective. In such cases it may be necessary to allow the support to yield in a controlled manner so that its capacity is only mobilized after significant displacement. As illustrated in Figs. 2 and 4, this results in a reduction in the support pressure required to stabilize the tunnel and the face.

In the case of the 5.5-m-span Yacambú-Quibor tunnel described earlier, very large deformations were accommodated by using sliding joints in the steel sets (Sánchez Fernández and Terán Benítez 1994). An alternative system is to use support elements that are designed to deform plastically in a controlled manner, as described by Schubert (1996).

When a large tunnel is required to deal with high-volume hydraulic flow or a two-lane transportation system, the tunnel can be split into two smaller tunnels, which are generally easier to support. This method has been used successfully for crossing wide fault zones.

In very poor ground it is difficult to keep drillholes open and it may be necessary to use self-drilling rather than conventional rockbolts. These are rockbolts fitted with disposable drill bits that are left in place at the bottom of the holes. In extremely poor quality ground, particularly where clay minerals are present, self-drilling rockbolts may be ineffective because of failure of the bond between the grout and the surrounding rock.

The multiple heading method tends to be safer than the full-face method, but it places high demands on careful design of details in the support system and on the quality of workmanship required to implement the design. The full-face method

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**FIG. 14.** Top Heading Excavation with Steel Sets Supporting Undermined Portion of Forepole Umbrella. Enlarged Footings (“Elephant’s Feet”) Have Been Used to Prevent Foundation Failure at Base of Sets; Sets and Forepoles Are Fully Embedded in Shotcrete to Form Very Strong Structural Shell; Face Is Supported by Means of Grouted Fiberglass Dowels

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carries a relatively high risk since failure of any part of the support system can result in collapse of a large volume of material. On the other hand, when implemented correctly, the method can be very effective.

The final choice of the method to be used for a specific situation depends upon the complex interaction of a number of factors. In addition to safety, cost, and schedule considerations, these factors also include the relevant experience of the contractor, the designer, and consultants engaged to assist in the project. The successful implementation of the methods illustrated in Fig. 13 depends more on experience-based judgment than on theoretical calculations. In particular, the experience of and the authority given to the individual directing the work at the face is crucial, since there is seldom time for lengthy academic discussions when dealing with unstable tunnel face problems. Wherever possible this individual should be an engineer, because it is not only experience but also an understanding of the mechanics of rock-support interaction that will dictate the choice of the most appropriate course of action.

**EXAMPLES OF SUPPORT DESIGN**

Fig. 15 shows one possible method for driving a tunnel through squeezing ground. In order to illustrate the design procedure that would be used in such a case, a set of calculations is described and comments are given on the assumptions made and the reliability of each calculation step.

The tunnel has an excavated span of 12.15 m and an 11 m span, measured inside the final concrete lining. The cover over the tunnel crown is 44 m, and the poor-quality flysch in which it is to be excavated has a friction angle $\phi = 23^\circ$, a cohesive strength of 0.06 MPa, and a deformation modulus of 308 MPa. The estimated rock mass strength is $\sigma_{cm} = 0.17$ MPa and, for an in situ stress of $p_o = 1.35$ MPa, this gives a ratio of rock mass strength to in situ stress $\sigma_{cm}/p_o = 0.13$. Eq. (1) gives an estimated strain of 7% under these conditions, and this suggests that very severe squeezing and face instability problems are likely unless appropriate support measures are implemented.

In this example the friction angle is $23^\circ$, and this suggests that the rock mass has a low clay mineral content and that its...
behavior is sufficiently frictional to justify the use of 6-m-long, 32-mm-diameter fully grouted untensioned rockbolts for the tunnel arch and sidewalls and of 12-m-long grouted fiberglass dowels for face support. Because of the anticipated face stability problems, a forepole umbrella consisting of 114-mm-diameter pipes at 500 mm center-to-center spacing will be used over an arc of about 140°. These forepoles are 12 m long and successive umbrellas are installed at 8 m spacing, giving an overlap of 4 m between umbrellas.

The pipe forepoles are installed one by one, after each hole has been drilled, and they are fully grouted as soon as possible after installation. Under no circumstances should the contractor be permitted to drill all the holes before installing the forepoles. This can weaken the rock mass and achieve exactly the opposite effect of that desired.

A 0.6-m-thick final concrete lining, reinforced where necessary, will be placed after completion of the excavation. This lining will be surrounded by an impermeable plastic membrane that will be drained by means of geotextile drainage layers leading the water to drainage pipes on either side of the base of the tunnel excavation.

Fig. 16 illustrates a finite-element model that was constructed to investigate the proposed excavation sequence for this case. This model involved eight stages of excavation and support installation or activation, as defined in the table included in the figure.

At this time there are no generally accepted methods for designing forepole umbrellas. These forepoles form a shell that reduces the gravitational stress acting on the rock core ahead of the advancing tunnel face. The correct way to approach this problem is by means of a full three-dimensional numerical analysis in which the forepoles are installed as structural elements embedded in the rock mass. Suitable programs for such analyses are available, but they have not been used yet for

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**Fig. 16.** Finite-Element Model for Investigation Construction Sequence and Effectiveness of Support
systematic studies of the stress distribution and progressive failure of the rock mass under the umbrella. A few studies have been carried out using simpler axisymmetric models.

The fact that no reliable means for designing forepoles exists represents a challenge for the geotechnical research community. Three-dimensional numerical analyses, supported by physical model studies and field measurements of the performance of installed umbrellas, would provide a basis for understanding the complex interaction between these support elements and the deforming rock mass.

In the meantime, in order to provide a logical basis for the design of the many forepole umbrellas being installed, some form of two-dimensional approximation is generally used. A very crude approach is to assume that a zone of "improved" rock can be used to simulate the forepole arch, and this is the approach that has been followed in the example given in Fig. 16. The improvement of rock mass properties is estimated by considering the weighted average (based on cross-sectional areas) of the strength and deformation properties of the steel forepoles, the grout filling, and the original rock mass. While this model does not correctly represent the three-dimensional bending strength of the umbrella, it does permit the construction of a two-dimensional model that is numerically stable and which appears to model the physical behavior of the reinforced tunnel in a realistic manner. More importantly, the observed (but as yet unpublished) performance of tunnels constructed with forepole umbrellas designed in this manner confirms that the improved strength estimates appear to be reasonable.

The results obtained from this model are illustrated in Fig. 17, which shows the extent of rock mass failure around the tunnel. The displacements of the crown and invert, for both the tunnel and the concrete lining, are given in the table included in Fig. 17. It can be seen that the excavation support...
system comes under full load in stage 4 and that displacements of 17 mm and 39 mm occur at points A and D in the roof and floor, respectively. Without support, the same model indicated a displacement of more than 4 m at point A in the roof and shows that the failure extends to the ground surface, resulting in a surface settlement of almost 2 m above the tunnel. Hence, the installed support is clearly necessary and, as demonstrated by this analysis, it is effective.

This analysis indicates overstressing of the shotcrete in the two “corners” at the ends of the invert in step 4 of the loading process. The extent of this overstressing does not increase during subsequent loading stages. This behavior is common where a sudden change of curvature occurs, and it does not usually give rise to any significant practical problems. In any case the shotcrete will be reinforced, either with reinforcing bars or with steel fiber reinforcement in the shotcrete itself. Hence, the overstressing will result in the formation of plastic hinges with a high residual load-carrying capacity. The heavy concrete lining section in these locations can easily accommodate any weakness in the shotcrete lining.

This analysis does not include the contribution of the fiber-glass face reinforcement that would probably be used in this case, as illustrated in Fig. 17. While it is possible to make crude estimates of the effectiveness of this reinforcement on the basis of the axisymmetric analysis used to produce Fig. 4, this is seldom done, because there is no simple way to incorporate these estimates into a two-dimensional analysis. The current method used to choose the capacity and density of the face support is to make it roughly equivalent to the rockbolt pattern used in the tunnel walls. The length is generally the same as that of the forepoles.

In this example, the final concrete lining is unreinforced and the analysis indicates that the stresses induced in the lining are well within allowable working loads, even under the condition of full external water pressure. In finalizing the lining design, it would also be necessary to check for any possible adverse effects from eccentric loading or thermal stresses.

Another example of a large excavation in very poor quality rock is illustrated in Fig. 18. This is a 16.5-m-span excavation for one of the underground stations of the Athens Metro, described in a paper by Kavvadas et al. (1996). The cover over the excavation crown is 15 to 20 m, and the principal problem is one of surface subsidence rather than failure of the rock mass surrounding the openings. The rock mass is poor-quality Athenian schist, and the multiple excavation and support stages illustrated in Fig. 18 are designed to ensure that a continuous reinforced shotcrete shell is created. This ensures that the deformation of this rock mass is kept to a minimum.

The Academia, Syntagma, Omonia, and Olympion stations were constructed using the method illustrated in Fig. 18. During the construction of the Omonia station, the maximum vertical displacements of the surface above the centerline of the station amounted to 51 mm. Of this, 28 mm occurred during the excavation of the side drifts, 14 mm during the removal of the central pillar, and a further 9 mm occurred as a time-dependent settlement after completion of the excavation. Ac-
According to Kavvadas et al. (1996), this time-dependent settlement is due to the dissipation of excess pore water pressures that were built up during excavation.

The appearance of the rock mass in one of the Olympion station side drift excavations is illustrated in Figs. 19 and 20.

HANDLING WATER IN TUNNEL CONSTRUCTION

The presence of water can cause significant problems during tunneling as a result of strength reduction due to either physical deterioration of the components of the rock mass or the reduction of effective confining stress due to pore water pressure. The physical deterioration of the rock mass can usually be estimated by tests carried out under conditions as close as possible to those anticipated underground. Appropriate support measures can then be designed to accommodate this strength reduction or to seal rock types, such as shales, that are particularly sensitive to moisture change.

In the case of water pressure, the tunnel itself generally acts as a drain during construction and, hence, these pressures are not usually a problem. However, if water is trapped behind an impermeable fault zone or similar geological barrier, the water pressure may cause a rock mass strength reduction as described above. In addition, a sudden release of this water pressure can give rise to dangerous operating conditions in the tunnel. Where such water barriers are anticipated, it is generally prudent to use probe-holes ahead of the face in order to gain as much advance warning as possible.

The design of tunnels in impermeable rock masses should include a full effective stress numerical analysis.

As illustrated in Fig. 21, a plastic membrane backed by a geotextile drainage layer generally surrounds the final concrete linings for most transportation tunnels. This layer leads water into drainage pipes in the base of the tunnel, from where it flows under gravity to one of the portals. This system usually works well and the design of the concrete lining can be carried out on the assumption that there is no external water pressure. There are, however, special circumstances in which drainage is not permitted and where the water table has to be allowed to return to its pretunneling level. For example, in tunneling under national parks or environmentally sensitive areas, this is a common requirement. Under these conditions the final concrete lining must be designed to carry the full water pressure. This type of design consideration was included in Fig. 16.

In water transmission tunnels the concept of a “leaky” lining is sometimes accepted. In areas of very high external groundwater pressure, the lining is intentionally punctured by drain holes so that the pressure difference across the lining is minimized. This is obviously only possible where the internal pressure is lower than the external pressure and where the quality of the external water entering the system is acceptable.

One of the most difficult problems to deal with in tunneling is that of very large quantities of water, such as is illustrated in Fig. 22. The geotechnical problems discussed above are not an issue here, but rather the problem is simply dealing with the piping and pumping capacity required to handle the volume of water. This problem can be even more severe when the water temperature is high as a result of geothermal activity and when it is essential to pipe the water away from the face as quickly as possible in order to reduce the temperature and humidity at the working face. It may also be necessary to supply cooling in the form of refrigerated air or loads of ice dumped at the face to lower the temperature to an acceptable level. There are no theoretical solutions to these problems; it is a matter of organization and having appropriate and adequate resources available on site.
SQUEEZING AND TUNNEL BORING MACHINES

The photograph reproduced in Fig. 8 may give the impression that tunnel boring machines (TBMs) are unsuitable for use in squeezing ground conditions. This is by no means the case since, if the squeezing conditions are recognized in advance, machines can be designed to deal with this problem. Dowden and Cass (1991) and Babendererde (1989) have described machines where the outer shield consists of a number of blades, each one of which is supported on hydraulic rams so that the blade can move independently in both axial and radial directions. The machine advances by a “shuffle-shoe” process that is capable of accommodating squeezing. A machine of this type, illustrated in Fig. 23, was used to excavate a pilot tunnel for the Freudenstein tunnel in Germany.

Face instability is not generally a problem because, when the machine is stationary, the presence of the cutting head provides effective face support. When the machine is advancing, any squeezing is excavated as part of the cutting process. The final lining generally consists of precast concrete segments that are placed directly behind the machine. If necessary, compressible elements can be incorporated into the joints of the segmental lining in order to accommodate squeezing of the tunnel behind the machine.

TUNNEL EXCAVATION AND SUPPORT COSTS

It can readily be appreciated that the costs associated with these special tunnel excavation and support methods can be high. Fig. 24 summarizes typical tunnel costs (in 1999 U.S. dollars) for large tunnels in difficult ground. These costs are for excavation and support only and do not include the cost of final concrete lining or the tunnel fittings (ventilation, lighting, rails, roadway, etc.). These costs were incurred in major projects currently under construction in various parts of the world and, while every care was taken to ensure that the information is as accurate as possible, the reader should exercise care in using this information since local conditions may give rise to significant variations in costs. In particular, costs associated with change orders and claims can give rise to very much higher costs than those indicated in Fig. 24.

CONCLUSIONS

The demands of infrastructure development have increased demands for the construction of large tunnels in rock masses of very poor quality. Problems with stability of the tunnels occur when rock mass strength is less than one-fifth of the in situ stress level. A method for estimating the severity of squeezing problems that occur under these conditions has been presented. Case histories of tunnel squeezing were discussed and various support options for controlling the stability of the face and the tunnel were summarized. Two practical examples were included to demonstrate how some of these support options can be incorporated into a tunnel design. The use of tunnel boring machines (TBMs) in squeezing ground and the costs of large tunnels in various ground conditions were briefly reviewed.
APPENDIX. CASE HISTORIES OF SQUEEZING TUNNELS

<table>
<thead>
<tr>
<th>Tunnel name, location and rock type</th>
<th>Depth H - m</th>
<th>συ - MPa</th>
<th>συ/H</th>
<th>Tunnel span - m</th>
<th>Closure - m</th>
<th>Strain %</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Yacambu-Quibor, Venezuela (graphitic phyllites)</td>
<td>600</td>
<td>1.0</td>
<td>0.06</td>
<td>5.5</td>
<td>5</td>
<td>&gt;30</td>
<td>Extreme squeezing, stability controlled by yielding steel sets (Sánchez Fernández and Terán Benítez, 1994).</td>
</tr>
<tr>
<td>2. Nathpa Jhakri headrace tunnel, India (fault zone)</td>
<td>300</td>
<td>0.6</td>
<td>0.25</td>
<td>10</td>
<td>2</td>
<td>20</td>
<td>Severe squeezing, stability controlled by forepole umbrella (Hoek, 1999)</td>
</tr>
<tr>
<td>3. Maan headrace tunnel, Taiwan (sandstone / shale)</td>
<td>200</td>
<td>1.6</td>
<td>0.33</td>
<td>6.5</td>
<td>0.1</td>
<td>1.5</td>
<td>Mild squeezing with local shotcrete damage (Chern et al., 1998)</td>
</tr>
<tr>
<td>4. Maan project, Adit A, Taiwan (sandstone / shale)</td>
<td>200</td>
<td>0.7</td>
<td>0.14</td>
<td>6</td>
<td>0.22</td>
<td>3.7</td>
<td>Large squeezing with severe support damage (Chern et al., 1998)</td>
</tr>
<tr>
<td>5. New Tienlin headrace tunnel, Taiwan (fault zone)</td>
<td>400</td>
<td>0.7</td>
<td>0.07</td>
<td>6.5</td>
<td>0.9</td>
<td>14</td>
<td>Severe squeezing with local tunnel collapse (Chern et al., 1998)</td>
</tr>
<tr>
<td>6. Mucha tunnel, Taiwan (sandstone / shale)</td>
<td>110</td>
<td>1.4</td>
<td>0.49</td>
<td>16</td>
<td>0.16</td>
<td>1</td>
<td>Stable tunnel (Chern et al., 1998)</td>
</tr>
<tr>
<td>7. Mucha tunnel, Taiwan (fault zone)</td>
<td>120</td>
<td>0.28</td>
<td>0.09</td>
<td>16</td>
<td>2.4</td>
<td>15</td>
<td>Severe squeezing with local tunnel collapse (Chern et al., 1998)</td>
</tr>
<tr>
<td>8. Pengshan tunnel, Taiwan (sandstone / shale)</td>
<td>140</td>
<td>1.9</td>
<td>0.55</td>
<td>12</td>
<td>0.01</td>
<td>0.11</td>
<td>Stable tunnel (Chern et al., 1998)</td>
</tr>
<tr>
<td>9. Maneri-Uttarkashi power tunnel, India (metabasics)</td>
<td>800</td>
<td>2*</td>
<td>0.1</td>
<td>4.75</td>
<td>0.43</td>
<td>9</td>
<td>Severe squeezing, damage to sets and concrete lining (Goel et al., 1995)</td>
</tr>
<tr>
<td>10. Chhib-Khodri tunnel, India (crushed red shale)</td>
<td>280</td>
<td>0.7*</td>
<td>0.1</td>
<td>3</td>
<td>0.01</td>
<td>2.8</td>
<td>Moderate squeezing, stabilized by circular steel sets (Singh et al., 1992)</td>
</tr>
<tr>
<td>11. Giri-Bata tunnel, India (slates)</td>
<td>380</td>
<td>0.8*</td>
<td>0.08</td>
<td>4.2</td>
<td>0.3</td>
<td>7.6</td>
<td>Large squeezing with deformation of steel sets (Singh et al., 1992)</td>
</tr>
<tr>
<td>12. Giri-Bate tunnel, India (phyllites)</td>
<td>240</td>
<td>0.7*</td>
<td>0.1</td>
<td>4.2</td>
<td>0.38</td>
<td>9</td>
<td>Severe squeezing with buckling of steel sets (Singh et al., 1992)</td>
</tr>
<tr>
<td>13. Loktak tunnel, India (shale)</td>
<td>300</td>
<td>0.7*</td>
<td>0.1</td>
<td>4.8</td>
<td>0.34</td>
<td>7</td>
<td>Large squeezing, supported by rock bolts, shotcrete and sets (Singh et al., 1992)</td>
</tr>
<tr>
<td>14. Maneri Bhali Stage I, India (fractured quartzite)</td>
<td>350</td>
<td>1*</td>
<td>0.1</td>
<td>4.8</td>
<td>0.38</td>
<td>7.9</td>
<td>Large squeezing with buckling of steel sets (Singh et al., 1992)</td>
</tr>
<tr>
<td>15. Maneri Bhali Stage II, India (sheared metabasics)</td>
<td>410</td>
<td>3*</td>
<td>0.28</td>
<td>7</td>
<td>0.2</td>
<td>3</td>
<td>Mild squeezing (Singh et al., 1992)</td>
</tr>
<tr>
<td>16. Maneri Bhali Stage II, India (Metabasic rocks)</td>
<td>480</td>
<td>3*</td>
<td>0.24</td>
<td>2.5</td>
<td>0.06</td>
<td>2.5</td>
<td>Mild squeezing (Singh et al., 1992)</td>
</tr>
</tbody>
</table>

* Estimated from descriptions provided by authors and from personal experience of Indian rock types.

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REFERENCES


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NOTATION

The following symbols are used in this paper:

\( p_i \) = support pressure applied to surface of tunnel;
\( p_o \) = in situ stress in rock mass surrounding tunnel;
\( r_o \) = radius of tunnel;
\( r_p \) = radius of plastic zone surrounding tunnel;
\( \delta \) = radial displacement of tunnel wall;
\( \delta_0 \) = displacement of tunnel wall at which support is installed;
\( \phi \) = angle of friction of rock mass surrounding tunnel;
\( c \) = cohesive strength of rock mass surrounding tunnel;
\( \sigma_{cm} \) = uniaxial compressive strength of rock mass surrounding tunnel;
\( E \) = modulus of deformation of rock mass surrounding tunnel;
\( \varepsilon_r \) = percentage strain of tunnel perimeter = (radial displacement \( \delta \)/tunnel radius \( r_o \)) \times 100; and
\( \varepsilon_f \) = percentage strain of tunnel face = (displacement at center of tunnel face/tunnel radius \( r_o \)) \times 100.