Chapter 64

AN APPROACH TO THE CLASSIFICATION OF WEAK ROCK FOR TUNNEL PROJECTS

Steve Klein

Jacobs Associates

ABSTRACT

Weak rock is an important consideration in tunnel design and construction due to the difficult ground conditions that are often encountered. Nevertheless, when conditions are favorable high advance rates can be achieved leading to economical tunnel construction. Successful construction of a tunnel in weak rocks depends on correctly anticipating the behavior of these unique materials. The objective of this paper is to provide guidelines that can be used to accurately classify the performance of weak rock in tunnels.

INTRODUCTION

Weak rock conditions are an important consideration in tunnel design and construction because of the numerous challenges in dealing with them. Weak rocks are often overstressed at low stress levels as a result of their low strength and high deformability. These characteristics can lead to yielding, slabbing and spalling, raveling, and squeezing conditions. Another key factor is that the strength of many weak rock formations is not constant and they may deteriorate when exposed in a tunnel. In addition, some weak rocks are susceptible to time-dependent deformations due to squeezing or swelling. On the other hand, when conditions are favorable high tunnel advance rates can be achieved with modest ground support requirements.

The behavior of weak rocks in tunnels is not always correctly anticipated in advance of construction and this has led to problems during the construction of a number of projects. This is partly due to difficulties in accurately classifying the behavior of these materials. The objective of this paper is to describe an approach for classifying the behavior of weak rocks in tunnels. Other classification approaches such as RMR (Rock Mass Rating) and Q systems apply to jointed rock masses whose behavior is controlled by discontinuities. These classification systems do not specifically address some of the unique characteristics of weak rocks such as the potential for overstressing or deterioration of the rock material. The classification approach discussed in this paper focuses on the two categories of behavior that must be evaluated for weak rocks: the immediate response, and the long-term behavior of the rock mass. Important factors that need to be evaluated are the rock type, mineralogy, strength, in situ stress levels, stress-strain characteristics, discontinuities, and groundwater conditions. This paper also discusses the special geologic exploration and laboratory testing methods needed to properly characterize the physical properties of these materials and some examples of the performance of weak rocks in tunnels.

Table 1. Uniaxial compressive strength of shale and sandstone as related to porosity

Porosity, n (%)	Unconfined Compressive Strength (MPa)	
	Shale (Mudstone) (after Hoshino, 1981)	Sandstone (after Dobereiner and de Freitas, 1986)
10	7 to 15	50 to 120
20	4 to 10	10 to 30
30	1 to 5	1.5 to 5

DEFINITION OF WEAK ROCK

Most approaches that have been used to define weak rock from an engineering point-of-view are based on the uniaxial compressive strength (UCS) of the intact rock. For example, the International Society for Rock Mechanics (ISRM) describes rock with an UCS in the range of 0.25 to 25 MPa (about 35 to 3,600 psi) as “extremely weak” to “weak” (ISRM, 1981). A more appropriate upper bound strength limit for weak rocks may be 20 MPa (about 3,000 psi) because there appears to be a difference in the way rock weaker than this limit behaves when sheared. Strength test data for sandstones indicate rocks with a UCS below about 20 MPa generally contract when sheared whereas stronger rocks tend to dilate (Dobereiner and de Freitas, 1986). Materials that dilate when sheared tend to resist the strains imposed on them and therefore, are less deformable than materials that tend to contract when sheared. Therefore, for the purposes of this paper, weak rock is considered to be rock with an UCS in range of 0.25 to 20 MPa (about 35 to 3,000 psi).

Another important factor influencing the strength of weak rocks is the porosity, or the amount of void space in the rock. In general, high porosity correlates with low strength. Low porosity and high strength is a result of a dense arrangement of grains and/or cementing agents filling the void space between grains. Table 1 summarizes some available strength data for mudstone and sandstone, generally indicating that mudstone and sandstone with a porosity higher than about 10 and 20 percent, respectively, will most likely be considered weak rock, if 20 MPa is taken as the upper strength limit. Figure 1 is a plot of porosity and UCS data for samples from a tunnel project in Northern California in weak sedimentary rocks. This figure indicates the influence of porosity on rock strength as the strength increases by a factor of two from about 1 to 2 MPa for a small decrease in porosity from 0.30 to 0.25.

Examples of weak rocks include sedimentary rocks (sandstone, siltstone, shale, claystone or mudstone, clay-shale, marl, and chalk), some volcanic rocks (tuff, agglomerate, and breccia), and weathered and altered (hydrothermal or chemical) rocks of all types. In addition, weak rock conditions can also be produced by close jointing, shear zones, or faults in the rock mass.

IMMEDIATE RESPONSE OF THE ROCK

The immediate response of weak rock in a tunnel is largely a function of the stresses imposed on the rock surrounding the tunnel and the strength of the rock. Although the behavior of many weak rock formations is controlled by the low strength and high deformability of the rock material, discontinuities in the rock mass and groundwater conditions can also influence the observed behavior.

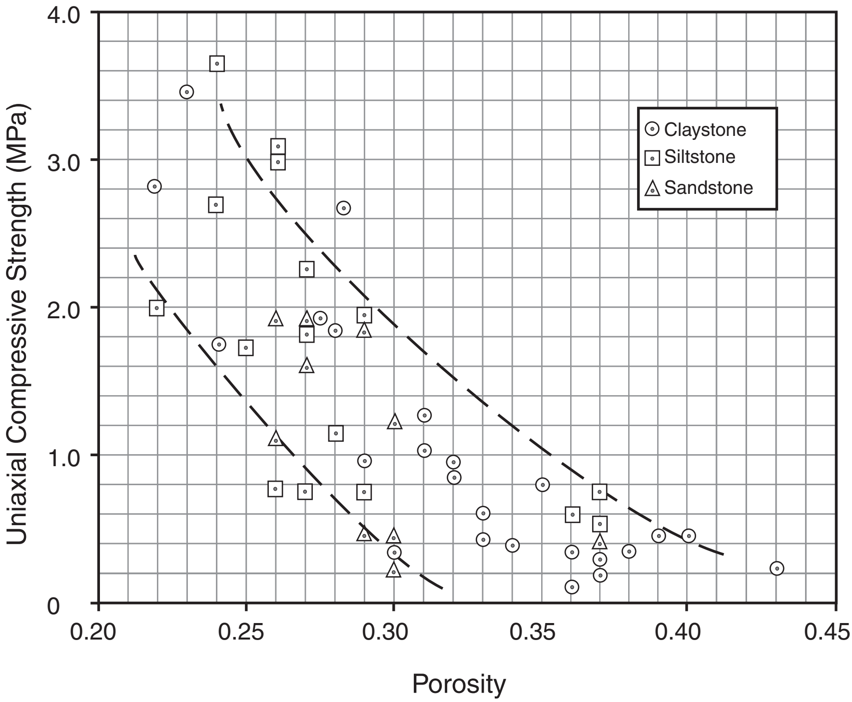

Figure 1. Influence of porosity on the uniaxial compressive strength of three weak sedimentary rocks

Influence of Stresses

One key aspect that is fundamental to the behavior of weak rock in a tunnel is the tangential stresses that develop around the tunnel excavation as compared to the strength of the rock material. The rock around the tunnel will yield or fail when the tangential stresses induced by the excavation exceed the strength of the rock mass. The magnitude of the induced tangential stresses around the tunnel excavation depends on the vertical overburden stress at the depth of the tunnel (P_z) and the in situ horizontal stress field, typically expressed by the in situ horizontal to vertical stress ratio (K_o). The following equations can be used for estimating the tangential stresses (σ_θ) around a circular tunnel (after Deere et al. (1969):

$$\sigma_\theta = (3\,K_o - 1)\,P_z\text{, at the crown and invert} \tag{1}$$

$$\sigma_\theta = (3 - K_o)\,P_z\text{, at the springline} \tag{2}$$

When K_o is 1 the stresses are uniform around the tunnel and the maximum tangential stress is 2 P_z. Tensile stresses develop at the crown and invert for K_o less than 1/3 and at the springline for K_o greater than 3. Table 2 summarizes the tangential stresses for K_o values of 0.5, 1.0, and 1.5. This table indicates the significant influence that K_o has on the tangential stresses around the tunnel. Maximum tangential stresses for K_o values of 0.5 and 1.5 are 25 and 75 percent higher, respectively, than when K_o is 1.0 (Table 2). When K_o is greater than one the maximum tangential stresses are at the crown and invert of the tunnel. Instability of the ground above the tunnel crown due to overstressing when K_o is greater than one is a serious concern in terms of ground

Table 2. Tangential stresses as a function of K_o

In Situ Stress Ratio, K_o	Tangential Stress at Crown and Invert	Tangential Stress at Springline	Average Tangential Stress
0.5	0.5 P_z	2.5 P_z	1.5 P_z
1.0	2 P_z	2 P_z	2 P_z
1.5	3.5 P_z	1.5 P_z	2.5 P_z

support and safety. Therefore, it is critical to accurately determine K_o in order to assess the behavior of weak rocks in a tunnel with confidence. The most reliable methods are probably in situ tests performed in boreholes utilizing dilatometer, pressuremeter, or hydraulic fracturing methods.

The rock around the tunnel will be overstressed when the tangential stresses around the tunnel exceed the strength of the rock, although some cracks may begin to form when the tangential stresses are greater than about 50 percent of the UCS. Deere et al. (1969) developed a factor called the modified overload factor (OFM), which can be used to evaluate the potential for overstressed conditions:

$$OFM = \sigma_\theta / UCS \tag{3}$$

Overstressed rock conditions develop around a tunnel when the modified overload factor is greater than one. The behavior of the rock under overstressed conditions depends on the stress-strain characteristics of the rock. Rock that fails in a brittle manner (Figure 2) will fracture when overstressed. Rock that exhibits ductile characteristics (Figure 2) will yield resulting in a plastic zone around the tunnel. Typically, coarse-grained rocks (sandstone, conglomerate, and possibly chalk) exhibit brittle characteristics and fine-grained rocks (shale, claystone, marl, and weathered/altered rock) exhibit ductile behavior. Figure 3 shows typical stress-strain curves for sandstone and claystone samples from the Riverside Badlands Tunnel in Southern California (DMJM/WCC, 1997). The sandstone fails in a brittle manner and the claystone exhibits ductile behavior. Another important point to recognize is that brittle materials typically fail at strains of about 1 to 2 percent whereas ductile materials fail at higher strains of 3 to 5 percent or more (Figure 3).

Brittle rocks that are overstressed will fracture leading to stress slabbing or spalling of the rock surrounding the tunnel. Stress slabbing and spalling will result in detached slabs and blocks of rock around the tunnel that can loosen and fall into the tunnel excavation, unless timely support is provided. Materials that exhibit ductile behavior will yield leading to convergence around the tunnel and squeezing conditions. In evaluating the potential for squeeze, the average tangential stress around the tunnel (see Table 2), estimated as $P_z\ (1 + K_o)/2$, is normally used to determine the overload factor.

Influence of Discontinuities

Discontinuities (bedding planes, joints, foliation, shears, and faults) will affect the behavior of the rock mass under overstressed conditions. Overbreak and loosening around the tunnel due to fracturing and stress slabbing will be much more extensive when steeply-dipping joints or shears strike sub-parallel with the tunnel alignment (within about 25 degrees). Another adverse discontinuity is flat-lying joints or bedding planes (dipping from 0 to 30 degrees) at or just above the crown of the tunnel. This type of discontinuity can lead to significant loosening and overbreak above the crown when stress-induced fracturing results in detached slabs of rock that buckle and fall out. A single set of discontinuities without intersecting discontinuity sets will only result

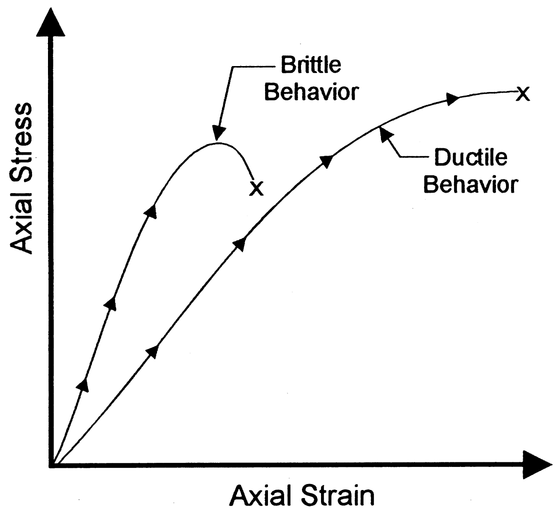

Figure 2. Stress-strain curves for brittle and ductile behavior

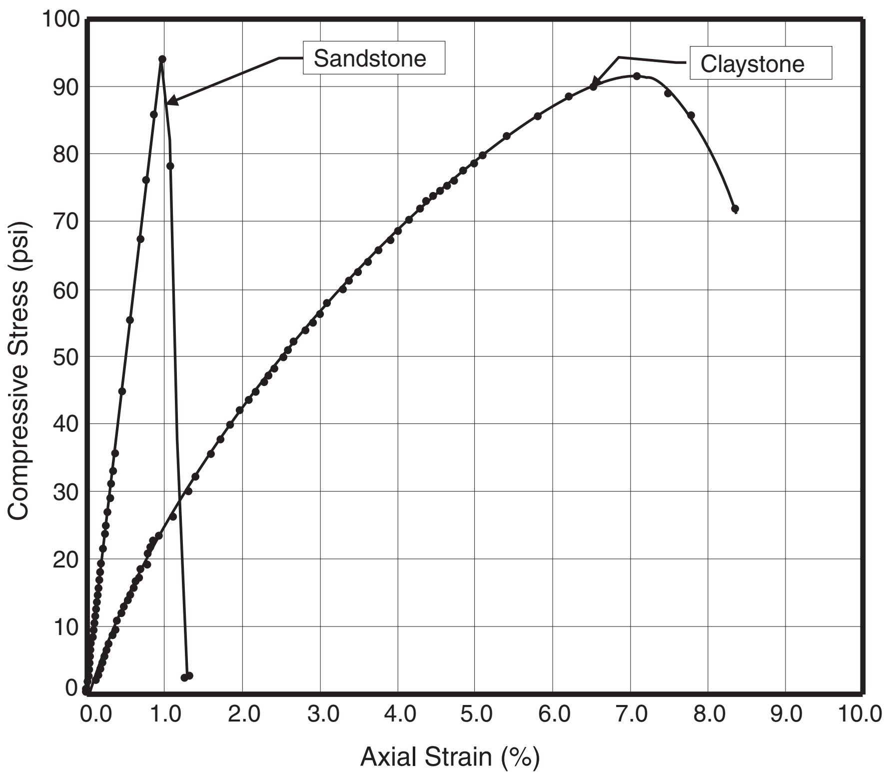

Figure 3. Typical stress-strain curves for sandstone and claystone, Riverside Badlands Tunnel

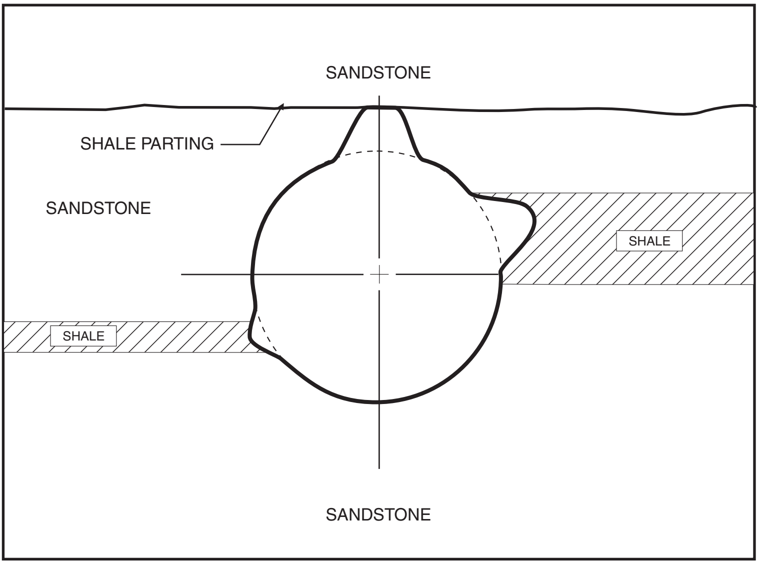

Figure 4. Some typical overbreak at the Navajo Tunnel No. 3 (after Sperry and Heuer, 1972)

in minor overbreak, unless the rock mass is overstressed enough for stress-induced fractures develop.

Case Histories of Tunnels in Weak Rock

Overstressed weak rocks have been encountered in several tunnels. The following case histories illustrate some examples of the observed performance of weak rocks under overstressed conditions. In the Navajo Tunnel No. 3, in New Mexico, extensive slabbing and spalling was observed in the 6.4 m diameter tunnel, excavated with a tunnel boring machine in weak sandstone, siltstone, and shale (Sperry and Heuer, 1972). The overload factor was in the range of 1 to 2.5 and slabs of rock, 15 to 20 cm thick and 60 to 90 cm in lateral dimension, developed at the crown and sometimes around the entire tunnel perimeter. In some cases these unstable slabs loosened and become detached immediately behind the cutterhead. Figure 4 illustrates the influence of the weak shale beds and flat-lying shale partings on the overbreak and rock failures that developed around the tunnel in overstressed conditions.

Figure 5 indicates the types of rock mass failure that developed in weak claystones, siltstones, and sandstones around the Delivery Tunnel of the North Lesotho Highlands Water Project for K_o less than and greater than one (Richards et al., 1992). Similar spalling was observed at the Northside Storage Tunnel in Sydney due to the combination of high horizontal stresses (K_o >1) and weak sandstone (Wallis, 2000). The higher horizontal stresses in the sandstone caused severe spalling and instability in the crown of the 6 m diameter tunnel increasing substantially the amount of tunnel support required.

One tunnel that had significant problems due to overstressing of the rock mass was the Stillwater Tunnel in Utah (Phien-wej and Cording, 1991). The closely jointed, sheared shale exhibited extensive raveling and squeezing in the 3.5- to 4-m excavated

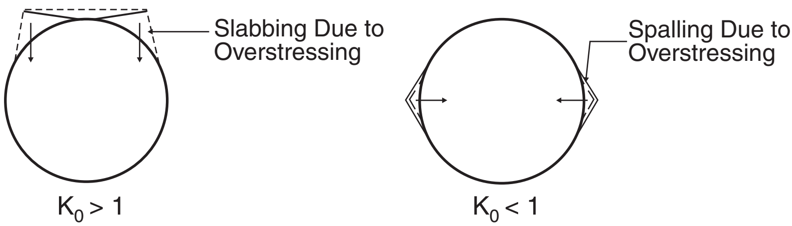

Figure 5. Typical failure mechanisms in massive rock

diameter tunnel. The shale tended to fracture into thin layers of rock, less than 5 cm in thickness and up to 15 cm deep into the tunnel wall. Significant raveling developed due to the combined effect of stress slabbing in the thinly-bedded shale and loosening along steeply dipping joints oriented nearly parallel to the tunnel. Significant squeezing was only observed in sheared shale with large amounts of clay infilling.

Influence of Groundwater

Groundwater is an important issue in a tunnel in weak rocks due to its influence on the physical properties of the rock and the behavior of the ground. Most rocks become weaker when exposed to water and this phenomenon is more pronounced in weak rock formations. Groundwater pressures may build up behind rock blocks around the tunnel excavation (due to overstressing or existing discontinuities) resulting in lower stand up time, reduced stability, and fall out into the tunnel excavation. Large groundwater inflows may occur where fault zones are encountered. These inflows can carry gouge and rock fragments into the tunnel creating voids outside the tunnel and causing instability of the rock mass. Groundwater can also cause slaking and swelling of the rock mass, as discussed below for the long-term behavior of the ground.

One rock that is particularly susceptible to groundwater is friable sandstone. Friable sandstone is an extremely weak rock that is only poorly cemented and sometimes uncemented and cohesionless. Friable sandstone is so poorly cemented that it completely disintegrates when placed in water. Fast raveling or flowing behavior is likely to occur, depending on the rock strength and groundwater pressures, because these rocks are too weak for stress slabbing and spalling to develop. Flowing ground was encountered in the Riverside Badlands Tunnel in Southern California in a Pleistocene/Pliocene-age friable sandstone with a UCS less than 0.1 MPa and a silt and clay fines content generally less than 10 percent (DMJM/WCC, 1997). High groundwater levels up to about 100 meters above the tunnel were considered a major factor in the ground response. The flowing conditions were stabilized by lowering groundwater levels with large dewatering wells installed from the ground surface. Similar ground conditions were observed in another poorly cemented sandstone in the Los Rosales Tunnel in Columbia (Dolcini, et al., 1990). The rock was so weak and friable that it was very unstable in the tunnel face and sidewalls crumbling away under the TBM cutters and requiring immediate ground support. Sand and water poured into the tunnel in several locations when drilling holes for contact grouting. High groundwater inflows were also encountered, up to 22,000 liters/minute at the tunnel face. Test results for the Cretaceous-age sandstone yielded an UCS of 18 to 64 MPa, however, these results proved to be misleading because in many instances samples were cohesionless and could not be tested.

Proposed Classification System

Figure 6 is a flow chart that was developed to summarize the evaluation of immediate response of the ground in a tunnel in weak rock. Terms used to describe ground conditions are defined in Deere et al. (1969), and also Terzaghi (1946) and Bieniawaski (1984). As indicated in Figure 6, potentially firm ground conditions are anticipated when the overload factor is less than one. If the ground is classified as firm, the influence of discontinuities must be considered to determine the behavior of the ground. Table 3 summarizes a description of rock mass conditions based on the spacing of the discontinuities. As indicated, the rock mass can be described as solid, massive, blocky/seamy, fractured, or crushed/shattered. The rock mass is moderately blocky and seamy when individual blocks are larger than 60 cm and very blocky and seamy when the blocks are smaller than this (Deere, et al., 1969). Alternatively, when the ground is firm and significantly jointed (i.e., discontinuity spacing less than about 1/5 of the tunnel span) it can be classified using the RMR or Q systems (Bieniawski, 1984; Barton et al., 1974).

Firm ground and solid rock (Table 3) has good stand up time and represents a stable tunnel excavation that does not require a great deal of ground support. Overstressed rock conditions that produce stress slabbing and spalling will most likely exhibit raveling behavior, particularly if there are adverse discontinuities present (see discussion of discontinuities, Navajo Tunnel No. 3, and Stillwater Tunnel above) or the rock mass is blocky/seamy. For rocks that will squeeze when overstressed, stand up time and face stability are usually good provided that the overload factor is less than about 2 to 3 (Deere et al., 1969). An exception is the raveling that develops in conjunction with squeezing when the rock mass is closely jointed and sheared, as discussed above for the Stillwater Tunnel. Yielding conditions that significantly affects face stability and stand up time do not develop unless the overload factor exceeds about 5 to 6. Massive and blocky/seamy rock (that is not overstressed) can be characterized as loosening ground. Support is required to stabilize rock blocks immediately surrounding the tunnel to prevent further loosening and larger volumes of rock from displacing into the tunnel. Fractured rock conditions will most likely be raveling ground requiring immediate and full-perimeter ground support. Crushed/ shattered rock, fault gouge, and heavily sheared rock will probably be weak enough to be considered soft ground and swelling, squeezing, raveling, or flowing behavior are likely in these materials, as discussed by Howard et al., (1975), depending on whether the materials are sand or clay.

LONG-TERM PERFORMANCE OF THE ROCK

The second part of a classification system for weak rock involves evaluating the long-term performance of the ground in a tunnel. Some weak rocks will deteriorate, swell, or continue to deform (squeeze) after exposure in the tunnel environment. Most of the rocks that exhibit these behaviors have a high clay mineral content such as

Table 3. Rock mass condition based on discontinuity spacing (after Bieniawski, 1984)

Description	Discontinuity Spacing	Rock Mass Condition
Very Wide	> 2 m	Solid
Wide	0.6 to 2 m	Massive
Moderately Close	0.2 to 0.6 m	Blocky/Seamy
Close	60 to 200 mm	Fractured
Very Close	< 60 mm	Crushed/Shattered

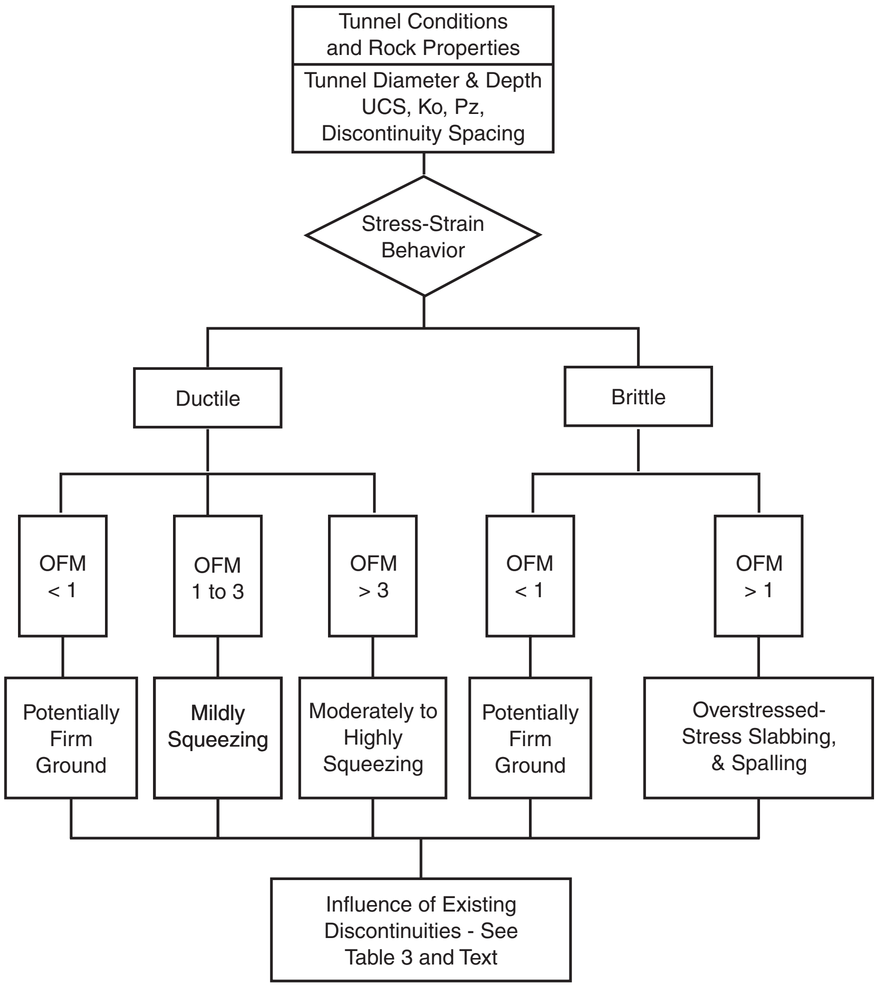

Figure 6. Immediate response of weak rocks in tunnels

shale, claystone, mudstone, clay-shale, sheared rock, some fault gouge, and weathered/altered rocks. Groundwater is a significant impact on the long-term behavior of the rock particularly the potential for softening, slaking, and swelling.

Softening/Slaking Potential

Rocks with high clay content are often prone to softening and slaking due to moisture changes and some sandstones are also susceptible to slaking. Softening is a significant loss of strength and slaking results in the disintegration of the rock. Rocks that will soften considerably when exposed to water must be protected to maintain their strength. The potential for softening can be evaluated by soaking samples and performing strength tests at various time intervals to determine the time-dependent loss of strength. Mudstones and shales with a UCS greater than about 3.4 MPa have been found to lose less than 40 percent of their original strength and are not generally susceptible to significant softening (Morgenstern and Eigenbrod, 1974).

The potential for slaking is controlled by the mineralogic composition of the rock, in particular the presence of swelling clay minerals. Stress relief in clay-bearing rocks is another important mechanism producing cracks, which allow water to penetrate the rock mass more deeply. Franklin and Chandra (1972) indicate the potential for slaking depends on the following:

1. Permeability and porosity of the rock
2. Influence of water on the rock in terms of dissolving cementing agents, disruption of bonds, or generation of pore pressures
3. The capacity of the rock for swelling or softening

The potential for slaking can be determined by subjecting rock samples to alternate wetting and drying cycles following slake-durability testing procedures such as those described by ISRM (1981). Morgenstern and Eigenbrod (1974) developed correlations between slaking potential and the liquid limit. It has been found that rocks with a liquid limit greater than 50 will generally have a high potential for slaking and rocks with a liquid limit less than 20 usually have a low potential for slaking.

Squeezing Ground

The conditions resulting in squeezing behavior in a tunnel were previously discussed above. Squeezing is characterized by time-dependent deformations, which are associated with yielding, creep, or plastic behavior caused by overstressed conditions in the rock mass around the tunnel. Some of the effects of squeezing are evident immediately following excavation such as the convergence that occurs at the tunnel face, but there are long-term affects as well, including continued ground movements and a gradual build up of load on the tunnel support system. The magnitude of the ground movements (i.e., tunnel convergence) associated with squeezing, the extent of the yielding zone around the tunnel, ground loads, and support requirements depend on the rock type, rock mass strength and deformation properties, and the in situ stress conditions. Stress analyses and ground-lining interaction analyses are usually carried out to address these issues. Approaches have been developed to do this and are discussed elsewhere (Deere et al., 1969; Jewtha et al., 1984). It is beyond the scope of this paper to cover design analyses for squeezing ground, however, in characterizing the ground it is important to recognize the special requirements for these analyses. For the analyses discussed above, it is critical that the deformation properties of the rock mass be accurately determined. In situ tests using a dilatometer, pressuremeter, or Goodman jack can be performed to determine these properties. In addition, triaxial tests may be necessary to estimate strength parameters for these analyses.

Swelling Potential

Swelling ground displaces into the tunnel as a result of an increase in volume of the rock mass surrounding the tunnel due to the adsorption of water. When the volume increase is resisted significant swelling pressures can develop. Swelling is mainly limited to the fine-grained rocks mentioned above that contain an appreciable amount of swelling clay minerals like montmorillinite (or smectite). Swelling can also occur due to the chemical or hydrothermal alteration of minerals such as feldspar, which produces montmorillinite, and also due to the hydration of anhydrite, which produces gypsum. Stress-induced cracking may facilitate access to water and promote swelling.

The potential for swelling can be evaluated from the results of laboratory tests such as free swell tests, Atterberg limit determinations, grain size analyses to determine the clay content (< 2 micron), and X-ray diffraction evaluations (to determine the type of clay minerals present). Terzaghi (1946) indicates that swelling ground will increase in volume by more than 2 percent when immersed in water and Heuer (1974) indicates that materials with a plasticity index exceeding 30 will exhibit a significant swelling potential. Figure 7 presents free swell data from some recent projects indicating the relationship between free swell (as a percentage) and plasticity index for sandstone, fault gouge, and claystone/shale/siltstone. The amount of montmorillinite (as a percentage), as determined from X-ray diffraction evaluations, is indicated on this figure for samples where this data is available. Kormornik and David (1969) conducted extensive testing and developed an equation between the water content, dry density, liquid limit, and the swelling pressure. Figure 8, based on their data, can be used to estimate the swelling pressure from the liquid limit and dry density. Howard et al. (1975) indicate that significant swelling problems are likely when the estimated swelling pressure is greater than about 250 kPa and very unlikely for swelling pressures less than 150 kPa. ISRM provides a test procedure for directly measuring the swelling pressure of a rock sample (ISRM, 1981).

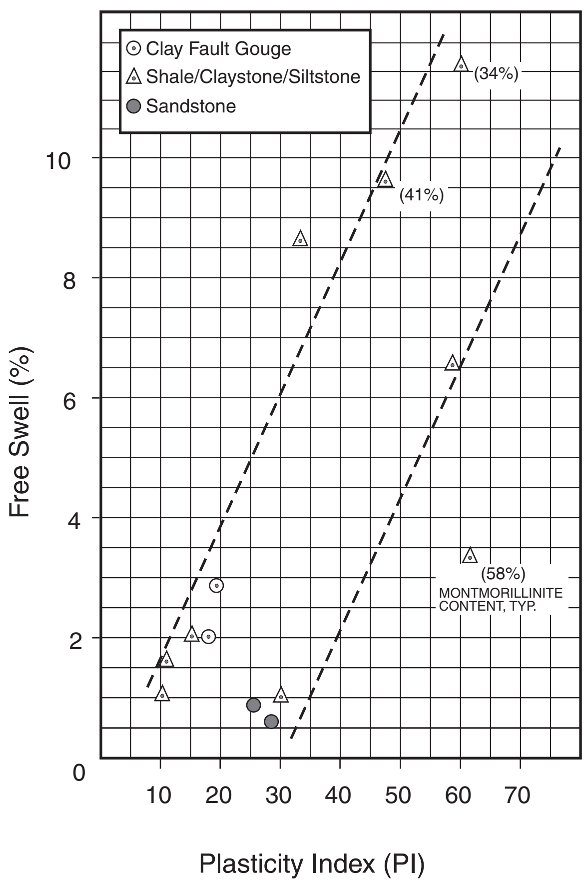

Figure 7. Relationship between free swell and plasticity index for various rocks

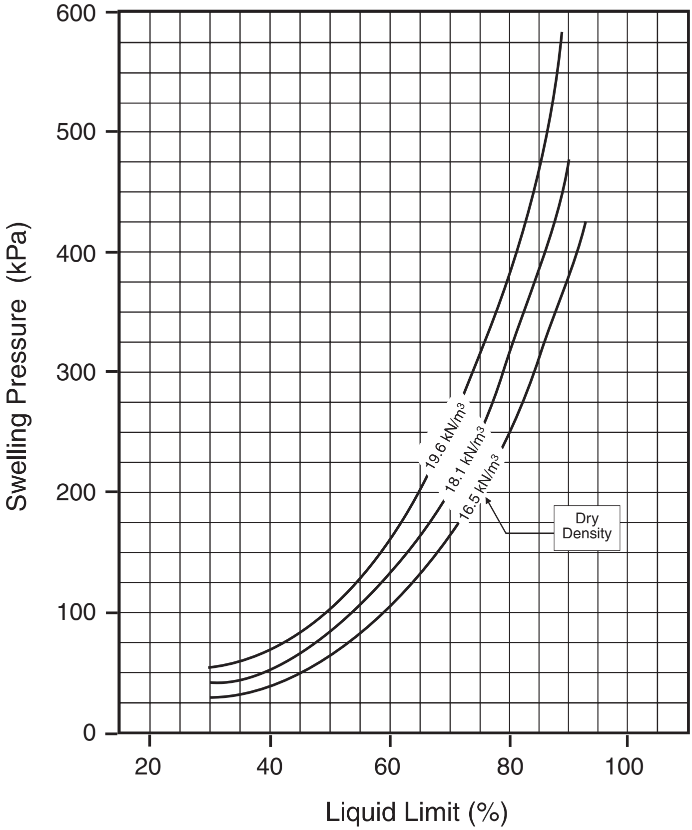

Figure 8. Relationship between swelling pressure and liquid limit

CONCLUSIONS

The behavior of weak rocks in tunnels can be anticipated prior to construction by a detailed investigation and testing program that focuses on determining the key rock properties needed to classify their performance in a tunnel. Key properties include the rock type and strength, mineralogy, stress-strain characteristics, in situ stress levels, and discontinuity conditions (type, spacing, and character). Figure 6 provides a classification approach that can be used to evaluate the immediate response of the rock mass. As discussed herein, the long-term performance also needs to be evaluated and this is particularly important for fine-grained rocks with high clay content.

REFERENCES

Barton, N., R. Lien, and P. Lunde. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics.* Vol 6. No. 4. 183–236.

Bieniawski, Z.T. 1984. *Rock Mechanics Design in Mining and Tunneling*. New York: John Wiley and Sons. 272 p.

Deere, D.U., R.B. Peck, J.E. Monsees, and B. Schmidt.1969. Design of tunnel liners and support systems. *Report prepared for U.S. Department of Transportation*. OHSGT Contract 3-0152. NTIS.

DMJM/Woodward-Clyde. 1997. Geotechnical Data Report, Riverside Badlands Tunnel. *Report prepared for the Metropolitan District of Southern California.* Specification No. 1365. Vol. I-III.

Dobereiner, L., and M.H. de Freitas. 1986. Geotechnical properties of weak sandstones. *Geotechnique*. 36. No.1. 79–84.

Dolcini, G., R. Grandori, M. Marconi. 1990. Water supply revamp for Bogota. *Tunnels & Tunnelling*, September. 33–38.

Franklin, J.A. and R. Chandra. 1972. The slake-durability test. *International Journal of Rock Mechanics and Mining Science*, Vol. 9, 325–341.

Heuer, R.E., 1974. Important Ground Parameters in Soft Ground Tunneling. *Proceedings of Subsurface Exploration for Underground Excavation and Heavy Construction.* ASCE. New York. 41–55.

Hoshino, K. 1981. Consolidation and strength of the soft sedimentary rocks. *Proceedings of the International Symposium on Weak Rock*. Tokyo. Japan. 155–160.

Howard, T.R., T.L. Brekke, and W.N. Houston. 1975. Laboratory Testing of Fault Gouge Materials. *Bulletin of the Association of Engineering Geology.* Vol. XII. No. 4. 303–315.

International Society for Rock Mechanics (ISRM). 1981. *Rock Characterization Testing and Monitoring*. New York: Pergamon Press. 211 p.

Jethwa, J.L., B. Singh, and B. Singh. 1984. Estimation of ultimate rock pressure for tunnel linings under squeezing rock conditions - A New Approach. *Proceedings of Design and Performance of Underground Excavations.* ISRM. Cambridge. England. *231–238.*

Komornik, A., and D. David. 1969. Prediction of swelling pressure of clays. *Journal of Soil Mechanics and Foundations Division.* ASCE. Vol. 95. SM 1. January. 209–225.

Morgenstern, N.R. and K.D. Eigenbrod. 1974. Classification of argrillaceous soils and rocks. *Journal of the Geotechnical Division.* ASCE. Vol. 100. No. GT10. October. 1137–1156.

Phien-wej, N. and E.J. Cording. 1991. Sheared shale response to deep TBM excavation. *Engineering Geology.* Elsevier Scientific Publishing Co. Amsterdam. Vol 30. 371–391.

Richards, J.A., F. Remmer, and J.C. Sharp. 1992. Design and construction of a segmental lining for a machine bored tunnel – Delivery Tunnel North, Lesotho Highlands Water Project. *TUNCON ’92, Design and Construction of Tunnels.* Maseru. Lesotho.

Sperry, P.E., and R.E. Heuer. 1972. Excavation and support of Navajo Tunnel No. 3. *RETC Proceedings,* Vol. 1. SME/AIME. New York. 539–571.

Terzaghi, K. 1946. Introduction to Tunnel Geology. *Rock Tunneling with Steel Supports*, R.V. Proctor and T.L. White eds. Youngstown: Commercial Shearing. 19–99.

Wallis, S. 2000. Sydney's Northside Storage Tunnel alliance holding up under pressure. *Tunnels and Tunnelling International.* May 37–40.