Tunneling in Difficult Ground

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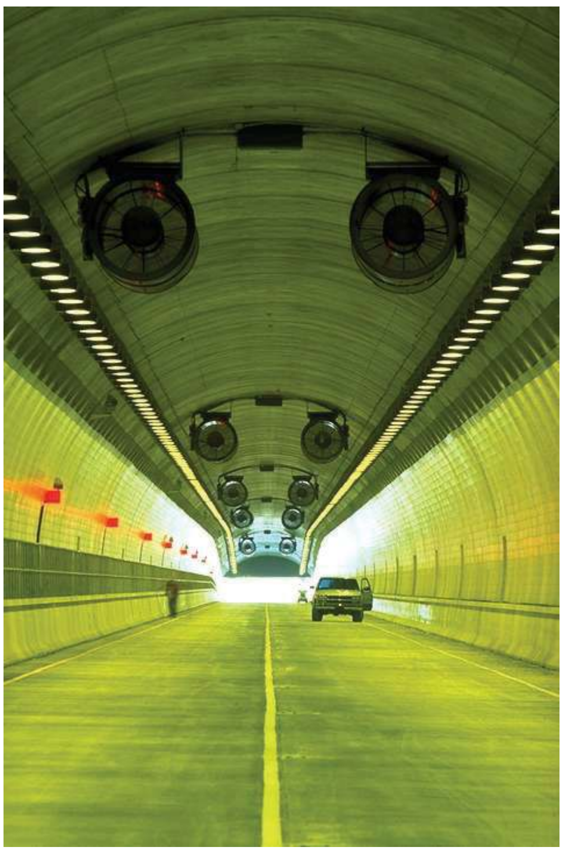
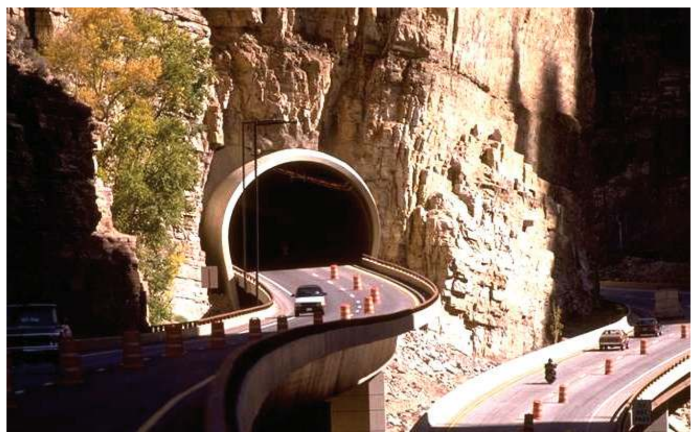
Technical Manual for Design and Construction of Road Tunnels — Civil Elements





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16. Abstract

The increased use of underground space for transportation systems and the increasing complexity and constraints of constructing and maintaining above ground transportation infrastructure have prompted the need to develop this technical manual. This FHWA manual is intended to be a single-source technical manual providing guidelines for planning, design, construction and rehabilitation of road tunnels, and encompasses various types of road tunnels including mined, bored, cut-and-cover, immersed, and jacked box tunnels. The scope of the manual is primarily limited to the civil elements of road tunnels.

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CONVERSION FACTORS

Approximate Conversions to SI Units			Approximate Conversions from SI Units		
When you know	Multiply by	To find	When you know	Multiply by	To find
(a) Length					
inch	25.4	millimeter	millimeter	0.039	inch
foot	0.305	meter	meter	3.28	foot
yard	0.914	meter	meter	1.09	yard
mile	1.61	kilometer	kilometer	0.621	mile
(b) Area					
square inches	645.2	square millimeters	square millimeters	0.0016	square inches
square feet	0.093	square meters	square meters	10.764	square feet
acres	0.405	hectares	hectares	2.47	acres
square miles	2.59	square kilometers	square kilometers	0.386	square miles
(c) Volume					
fluid ounces	29.57	milliliters	milliliters	0.034	fluid ounces
gallons	3.785	liters	liters	0.264	gallons
cubic feet	0.028	cubic meters	cubic meters	35.32	cubic feet
cubic yards	0.765	cubic meters	cubic meters	1.308	cubic yards
(d) Mass					
ounces	28.35	grams	grams	0.035	ounces
pounds	0.454	kilograms	kilograms	2.205	pounds
short tons (2000 lb)	0.907	megagrams (tonne)	megagrams (tonne)	1.102	short tons (2000 lb)
(e) Force					
pound	4.448	Newton	Newton	0.2248	pound
(f) Pressure, Stress, Modulus of Elasticity					
pounds per square foot	47.88	Pascals	Pascals	0.021	pounds per square foot
pounds per square inch	6.895	kiloPascals	kiloPascals	0.145	pounds per square inch
(g) Density					
pounds per cubic foot	16.019	kilograms per cubic meter	kilograms per cubic meter	0.0624	pounds per cubic feet
(h) Temperature					
Fahrenheit temperature(^{o}F)	5/9(^{o}F- 32)	Celsius temperature(^{o}C)	Celsius temperature(^{o}C)	9/5(^{o}C)+ 32	Fahrenheit temperature(^{o}F)

Notes: 1) The primary metric (SI) units used in civil engineering are meter (m), kilogram (kg), second(s), newton (N) and pascal (Pa=N/m^2).
2) In a "soft" conversion, an English measurement is mathematically converted to its exact metric equivalent.
3) In a "hard" conversion, a new rounded metric number is created that is convenient to work with and remember.

FOREWORD

The FHWA Technical Manual for Design and Construction of Road Tunnels – Civil Elements has been published to provide guidelines and recommendations for planning, design, construction and structural rehabilitation and repair of the civil elements of road tunnels, including cut-and-cover tunnels, mined and bored tunnels, immersed tunnels and jacked box tunnels. The latest edition of the AASHTO LRFD Bridge Design and Construction Specifications are used to the greatest extent applicable in the design examples. This manual focuses primarily on the civil elements of design and construction of road tunnels. It is the intent of FHWA to collaborate with AASHTO to further develop manuals for the design and construction of other key tunnel elements, such as, ventilation, lighting, fire life safety, mechanical, electrical and control systems.

FHWA intends to work with road tunnel owners in developing a manual on the maintenance, operation and inspection of road tunnels. This manual is expected to expand on the two currently available FHWA publications: (1) Highway and Rail Transit Tunnel Inspection Manual and (2) Highway and Rail Transit Tunnel Maintenance and Rehabilitation Manual.

Myint Lwin

M. Myint Lwin, Director
Office of Bridge Technology

CHAPTER 8
TUNNELING IN DIFFICULT GROUND

8.1 INTRODUCTION

Engineers like to work with materials having defined characteristics that do not change from one location or application to another. Unfortunately, geology seldom if ever cooperates with this natural desire but instead tends to present new and challenging conditions throughout the length of a tunnel. Some of these conditions approach the "ideal" closely enough that they can be approached as presented for rock and soft ground in Chapters 6 and 7. However, in many cases special approaches or arrangements must be made to safely and efficiently drive and stabilize the tunnel as it passes through this "Difficult Ground".

The factors that make tunneling difficult are generally related to instability, which inhibits timely placement or maintenance of adequate support at or behind the working face; heavy loading from the ground which creates problems of design as well as installation and maintenance of a suitable support system; natural and man-made obstacles or constraints; and physical conditions which make the work place untenable unless they can be modified.

This chapter is an update of the Chapter 8 "Tunneling in Difficult Ground" of the 2nd Edition Tunnel Engineering Handbook authored by Terrence G. McCusker (Brickel, et al., 1996) and emphasizes on creating and maintaining stable openings by mining or boring in difficult ground which actively resists such efforts. Chapters 6 through 10 presents design recommendations and requirements for mined and bored road tunnels. Mining sequentially based on the sequential excavation method (SEM) principles is discussed in Chapter 9. Chapter 10 addresses the design of various types of permanent lining applicable for rock tunnels.

8.1.1 Instability

Instability can arise from: lack of stand-up time, as in non-cohesive sands and gravels (especially below the water table) and weak cohesive soils with high water content or in blocky and seamy rock; adverse orientation of joint and fracture planes; or the effects of water. The major problems with mixed face tunneling can also be ascribed to the potential for instability and this class of tunneling will be discussed under this heading.

8.1.2 Heavy Loading

When a tunnel is driven at depth in relatively weak rock, a range of effects may be encountered, from squeezing through popping to explosive failure of the rock mass. Heavy loading may also result from the effects of tunneling in swelling clays or chemically active materials such as anhydrite. Adverse orientation of weak zones such as joints and shears can also result in heavy loading, but this is usually dealt with as a problem of instability rather than loading. Combinations of parallel and intersecting tunnels are a special case in which loadings have to be evaluated carefully.

8.1.3 Obstacles and Constraints

Natural obstacles such as boulder beds in association with running silt and caverns in limestone are just two examples of natural obstacles that demand special consideration when tunneling is contemplated. In urban areas, abandoned foundations and piles present manmade obstructions to straightforward tunneling while support systems for existing buildings and for future developments present constraints which may limit the tunnel builder's options. In urban settings, interference conflicts, public convenience or the constraints imposed by the need or desire for connection to existing facilities will sometimes result in the need to construct shallow tunnels, which have a range of problems from working in confined spaces, avoiding subsidence and uneven ground loading and support.

8.1.4 Physical Conditions

In areas affected by relatively recent tectonic activity or by ongoing geothermal activity, both high temperatures and noxious, explosive or deadly gases may be encountered. Noxious gases are also commonly present in rock of organic origin; and elevated temperatures are commonly associated with tunneling at depth. In an urban setting, contaminated ground may be encountered and will be especially troublesome when found in association with other difficult conditions.

Where appropriate, some information is provided as to the reasons why the condition under discussion creates problems for construction. Some examples of each of the conditions referred to above are discussed briefly to yield insight into the problems and to define the range of solutions available.

8.2 INSTABILITY

8.2.1 Non-Cohesive Sand and Gravel

Cohesion in sands is more than a matter of grain size distribution. For instance, beach-derived sands normally contain salt (unless it has been leached out), which aids in making sand somewhat cohesive regardless of grain size. The moisture content then becomes a determining factor.

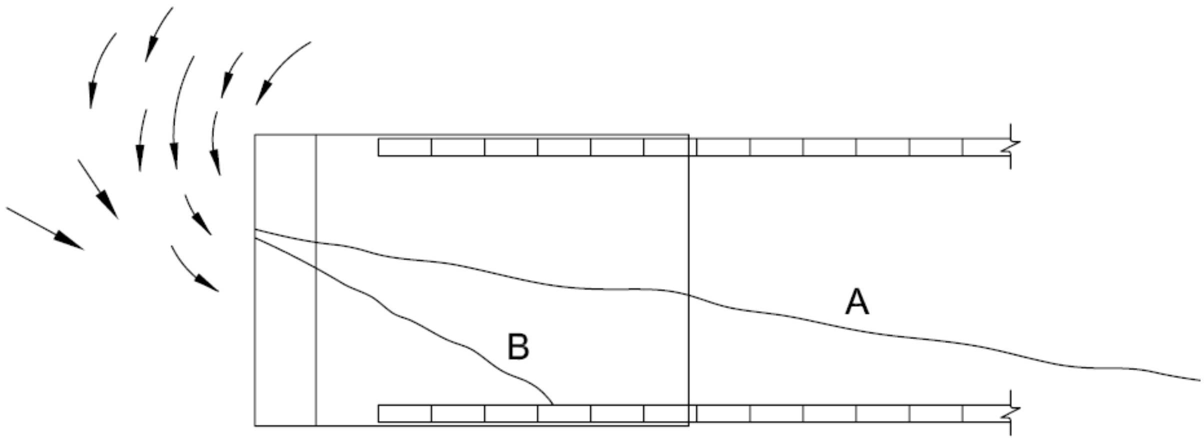
The age and geologic history of the deposit is also important since compacted dune sands with “frosted” grain surfaces may develop a purely mechanical bond; and leaching and redeposit of minerals from overlying strata may also provide weak to strong chemical bonding.

As discussed in Chapter 7, a very low water content amounting to less than complete saturation will provide temporary apparent cohesion as a fresh surface is exposed in tunnel excavation because of capillary forces or “negative pore pressure.” This disappears as the sand dries and raveling begins. Nevertheless, some unlooked-for stand up time may be available. In this case, it is important not to overrate the stability of the soil. As it dries out, the cohesion will disappear and it cannot be restored by rewetting the ground.

If groundwater is actually flowing through the working face, any amount may be sufficient to permit the start of a run which can develop into total collapse as shown in Figure 8-1.

There is no such thing as a predictably safe rate of flow in clean sands. Uncontrolled water flows affect more than the face of the excavation. If the initial support system of the tunnel is pervious, water flowing behind the working face will carry fines into the tunnel and may create substantial cavities--sometimes large enough to imperil the integrity of the structural supports. This phenomenon occurred in Los Angeles where a ruptured water main caused sufficient flow through a tunnel support system to cause a failure and resulting large sink hole in the street.

While factors such as compaction or chemical bonding may permit some flow without immediate loss of stability, this is not a reliable predictor. Soil deposits are hardly ever of a truly uniform nature. It has been observed in soft ground tunnels in recent deposits that all that is necessary to trigger collapse may be the presence of sufficient water to result in a film on the working face; i.e., there is no negative pore pressure to assist in stabilizing the working face. Of course, there is never a safety factor arising from surface tension (capillary action) in coarse sand or gravel.



Key:
A: Flowing Sand
B: Sand at Normal Angle of Repose

Figure 8-1 Flowing Sand in Tunnel

The cleaner the sand, the more liable it is to run or flow when exposed in an unsupported vertical face during tunnel construction. Single sized fine grained sands (UCS classification SP) are the most troublesome, closely followed by SP-SM sands containing less than about 7% of silt and clay binder. Saturated sands in these classes have been observed to flow freely through sheet piles and to settle into fans having an angle of repose of less than 5°. Unconfined SP sands will run freely, as in an hourglass, whether wet or dry, having some stability only when damp but less than saturated (no piezometric head). The large proportion of the sand particles of the same size allow the sand to move almost as freely over one another as would glass marbles.

Silt, intermediate in grain size between sand and clay, may behave as either a cohesive or non-cohesive material. In some areas it is common to find thin seams of saturated fine sandy silt trapped between clay beds in glacial deposits. In general, unless the seams are thicker than about 9-12 inches, when the silt layer is exposed in the wall of an excavation, the soil slumps out at intervals leaving a series of small shallow caves like entrances to burrows. The water appears to drain fast enough from the increased surface area exposed so that the remainder of the exposed material stabilizes.

The usual problem encountered with running sand is settlement and cratering at the surface with damage to structures or utilities in the area. If the ground is permeable, consolidation grouting of the entire sensitive area can be undertaken to stabilize the soil before tunneling. If dewatering is successful in depressing the water table below the tunnel invert, it may be found that the sand is just as unstable dry as wet. The alternative of using compressed air is attractive, provided the working pressure is very carefully controlled; but even so, the ground may be too dried out for stability.

If the face is a full face of sand and similarly weak materials, a slurry machine or an earth pressure balance machine, will be required. In general, rotary head tunneling machines for soft ground tunnels require very similar physical properties over the entire working face and the entire job. If these conditions do not prevail, then weaker ground, and running sands in particular, must be prevented from entering the shield more rapidly than is proper for the rate of advance. Slurry shields have the best opportunity of controlling variable conditions where running sands are present; but they will prove difficult to keep on line and grade in mixed face conditions if one of the beds present is even a strong clay. If the sand and clay beds are more or less evenly distributed (e.g., a varved clay), then this problem may not arise. Of the digger type shields, neither extensible poling plates nor orange peel breasting have proved to be generally successful, hence these machines are now rarely used.

A problem with all shield construction is the necessary difference in diameter between the shield and the lining. If the soil has no stand-up capability by the time it is exposed in the upper part of the tunnel before expansion of a primary lining or introduction of pea gravel or more commonly, grout into the annular space for non-expanded linings, then there will be loss of ground. If the unfilled annular space averages one inch in a 20 ft tunnel, the lost ground from this single cause is approximately 1.7% shown in Table 7.2 as “poor” practice. Even if only local ravelling takes place, it may choke off the flow of grout before the void can be filled with a continuous supporting fill material. This loss of ground results in a contribution to settlement.

8.2.2 Soft Clay

For the purposes of this discussion, soft clay includes any plastic material that will close around a tunnel excavation if free to do so. This will be the case if the overburden pressure at spring line exceeds the shear strength of the clay by a factor of about three or more. However, if the clay is sensitive and loses strength when remolded, the remolded strength will govern some of the clay behavior during tunnel construction. The phenomenon of sensitivity is mediated by several factors that cannot be fully discussed here but, in general, sensitivity may be suspected in clays with a high moisture content. Particularly at risk are marine clays from which the salt has been leached. The loss of strength may lie within a wide range, the ratio of undisturbed to remolded strength sensitivity being from 2 to 1,000. Moderate sensitivity of 2 to 4 is quite common. During remolding, the void ratio in the clay is reduced and free water is released. When this free water has access to a drainage path such as a sand bed or the tunnel itself, there will be a volume change in the soil mass which will result in surface settlement.

As discussed in Chapter 7, Equation 7-1 is used to calculate a Stability Number to estimate ground behavior in tunneling. Table 7-2 summarizes the behavior of cohesive soils during excavation. As shown in Table 7-2, if the cohesive soil is to be stabilized so that closure around the tunnel lining is minimized and stable control of line and grade are maintained, the critical number must be reduced below about 5; this will enable reasonable control of alignment and grade. Equation 7-1 can be writem to the following equation:

$$P_a = P_z - (N_{crit} \times S_u) \qquad \text{8-1}$$

where Ncrit is the critical number, Pz is the overburden pressure at tunnel spring line, Pa is the working pressure in a compressed air tunnel or the equivalent average pressure provided by the initial support system, and Su is the undrained shear strength of the soil in compatible units. As an example, if N is to be maintained at a value of 5, the overburden pressure is 40 psi and the unconfined shear strength of the soil is 1,000 psf = 7 psi, then from Equation 8-1, the required working pressure in the tunnel will be $(40 - 5 \times 7) = 5$ psi. From this same equation, it can be seen that if the shear strength of the soil is reduced by remolding caused by passage of the shield through the ground to a value of 250 psf, then the required air pressure for stability increases to over 30 psi, transforming the project from a relatively straightforward one to a difficult one.

Attempting to calculate the required volume of grout injection into the annular void between shield excavation and lining in clays often is not a fruitful exercise. It will certainly be possible to inject the requisite volume of grout, but it may be difficult to make it flow around the tunnel perimeter in an even layer. The best results are obtained by establishing multiple simultaneous injection points permanently fixed within the shield tail and passing through the tail seals. Grout is injected throughout the time the shield is in motion. For this system to work, the lining must be a bolted segmented lining with built-in gashets between segments. It must be expected that for simultaneous injection through multiple ports while the shield is in motion there will be a substantial learning curve before all elements of the system are functioning smoothly to achieve the desired result.

It is generally difficult to use any mechanical excavation equipment in this type of ground except for a slurry shield or earth pressure balance shield (EPB). These days, the two types of machine are approaching interchangeability with the continuing development of chemical additives (conditioners). The edge goes to slurry machines in coarse geology and/or where the rock crusher may be needed to reduce rock or boulders to a size that will pass the machine.

The EPB is preferred as being somewhat more flexible in varying conditions and somewhat less expensive than a slurry shield. In order to control pressure in the plenum chamber behind the cutterhead, a screw conveyor is required. The rotational speed of the screw is matched to the advance rate of the EPB and pressure in the plenum is monitored using multiple sensors. If boulders are likely to be encountered, especially if they will be larger than can pass through the screw conveyor, the cutterhead must be fitted with disk cutters in addition to the drag bits normally associated with this type of machine. This topic is covered in more detail below in Section 8.4.1 dealing with boulders.

8.2.3 Blocky Rock

As discussed in Chapter 6, rock is a basically strong material which requires little or no structural support when intact; although it may require protection from exposure to air, water or from fluids conveyed in the tunnel. However, when the rock joints and fractures are open sufficiently that the natural rugosity of the block surfaces will not prevent movement of rock blocks or substantial fragments, the rock is said to be "blocky." If the joints and fractures contain clay-like material resulting from weathering or light shearing, then the rock is described as "blocky and seamy." As can be seen from Table 6-7, this may raise the rock load by a factor of approximately three. In zones where the rock has small folds, but is open along the direction of the folds, it may be free to move in only one direction. Such rock is still blocky.

When rock is subjected to the action of explosives, high-pressure gases flow into any fissures in the rock before they have finished their explosive and rock-fracturing expansion. Even in hard granite, a result of blasting is the creation of micro-fissures extending well outside the blasted perimeter. In blocky rock, the effect may well extend more than a tunnel diameter outside the desired finished surface; a good deal of overbreak and potential loosening and movement of blocks is likely to result.

Another problem with this type of rock is that it is highly susceptible to the destabilizing effects of water flowing through the fracture system with sufficient energy to dislodge successively more rock. This action is dealt with more fully in a later section. Finally, it is quite likely when blocky and seamy rock is encountered in a tunnel excavation, especially in heavily folded strata, that there will be zones where the weathering has proceeded to a conclusion resulting in the presence of weak earth-like material with little capacity to sustain loads or to preserve the tunnel outline.

All of the rock conditions described require early and carefully placed primary support to preserve ground stability and to provide a safe workplace. Even before support installation, it is necessary to minimize surprises by scaling off any loose rock which will present a hazard to the crews installing the support system. Many still prefer to use steel ribs and wood lagging in this type of rock. It provides positive support and is quickly installed in tunnels less than about 5 meters in diameter. Unfortunately, crews still have to work under the unsupported rock to install the ribs and lagging; the material costs are high; the presence of timber results in the possibility of future uneven loading on the permanent tunnel lining as wood rots out and steel corrodes; and it becomes relatively difficult to ensure good contact between the lining concrete and the rock even after contact grouting.

For these reasons the use of shotcrete and rock bolts has become popular. In rock known to be blocky and therefore to need support, an initial layer of shotcrete about 5 cm thick should be applied as soon as possible in the tunnel crown. This is followed by the installation of pattern rock bolts whose length and diameter are governed principally by the tunnel diameter. (See Chapter 6 for more details)

8.2.4 Adverse Combinations of Joints and Shears

Jointing systems in rock arise from many causes, some of which are noted here. Sedimentary rocks, and particularly limestone, typically have three more or less orthogonal joint sets arising from the modes of deposition and induration which formed them. Not all joints are continuous, but those in any set are parallel. There may be many sets or, in weak, massive sandstone, for instance, only one or two. Joints and fracture systems combine to break up the rock mass into interlocking fragments of varying sizes and degrees of stability.

In the absence of direct evidence to the contrary, it should be assumed that shears and faults are continuous throughout their intersection with the tunnel excavation. In schistose materials, weathering usually follows a foliation plane to great depths, even in temperate climates when a weak zone has been formed by slippage along that plane. Other faulting may cause the development of extensive fracture systems in any direction. A section through the project area perpendicular to the strike of the exposed surfaces in schistose materials will generally reveal a saw-tooth profile with one of the surfaces parallel to the foliation. Continuation of the plane thus defined to tunnel elevation will be a preliminary indicator of the presence of sheared and weathered rock in the excavation.

Continuous joints and shears can define large blocks with little or nothing to hold them in place once the tunnel excavation has been completed. It is important to identify the locations of blocks with the potential for falling out in order to provide support during cautious excavation. For large diameter tunnels in particular, this requires an assessment of the potential before construction begins, mapping during construction, and control of drift size and round length to ensure against complete exposure of an unstable block in a single round. Readers are referred to Chapter 6 for details.

The difficulty of controlling the correct placement of steel sets in multiple drift headings works against the use of this kind of support. Initial rock bolting followed by reinforced shotcrete is a reasonable approach. In all cases where rock bolts have to be located to take direct and reasonably predictable loads, it is better that they be installed ahead of the shotcrete while the joint locations are still visible. If mechanical rock bolt installers cannot be used, then the crews must be protected by overhead cages.

8.2.5 Faults and Alteration Zones

Tectonic action, high pressure and high temperatures may metamorphose rock into different structures with unpredictable joint patterns. The uplift and folding of rocks by tectonic action will cause fracturing perpendicular to the fold axis along with faulting where the rock cannot accommodate the displacements involved, so that shears develop parallel to the fold axis. Other types of faults arise as the earth accommodates itself to shifting tectonic forces. Faults or shears may be thin with no more significance than a continuous joint or they may form shear zones over a kilometer wide in which the rock is completely pulverized but with inclusions of native rock, sometimes of large size.

All of the conditions briefly described above may be additionally complicated by the presence of locked-in stress, high overburden loads, or water.

Dealing with the conditions encountered in such fault zones and weathered intrusive zones depends on the excavation method in use, the depth below the ground surface, the strength of the fault gouge, the sheared material or the weathered or altered rock, and the water conditions. Water problems are discussed in general in the next section, including consideration of the difficult water conditions commonly found in association with faults; however, to the extent that they affect the selection of construction methods appropriate to fault crossings, they are referred to here.

Current technology provides other solutions, such as the use of precast concrete lining in the weak ground with supplementary jacking capability to enable the lining to provide the jacking reaction for the thrust of the TBM.

In general, fault crossings offer conditions akin to those of mixed face tunneling and the same methods are available to deal with them. Different circumstances come into play with deeper tunnels, especially if these are of large diameter. Such tunnels are usually long and logistics are important. The comparative lengths of fault zone and normal tunnel dictate that the construction method be efficient for the normal tunnel. Nevertheless, sufficient flexibility is required to permit safe and reasonably expeditious construction through the worst conditions likely to be encountered. Drill and blast excavation is still commonly used in such tunnels. Rock bolts and shotcrete then become the preferred support system, although steel ribs and lagging or steel ribs with shotcrete are also still used. TBM successes in these conditions have been few. There are two principal problems: the loose material in the fault runs into the buckets and around the cutters and stalls the cutterhead; and if the fault contains cohesive material, it squeezes and binds the cutterhead and shield with similar results.

One solution to the problem of loose or loosened ravelling and running material is to establish a grout curtain ahead of the TBM and then to maintain it by continuing a grout and excavation cycle throughout the fault-affected portion of the drive. Even if imperfect--as consolidation grouting tends to be, especially when placed from within the tunnel in conditions providing limited access--it is likely that a properly designed and executed program will add sufficient stability to the ground to permit progress. It should be noted that any such program will be expensive and time-consuming. It is therefore unlikely that any contractor will willingly do the necessary work unless it has already been envisaged in the contract as a priced bid item. It is also important to recognize that if water is running into the tunnel through the working face, a bulkhead will be required to stop the flow while the initial grouting is in progress. Grouting into running water is a slow and expensive way to establish a grout seal.

Within limits, the squeezing problem can be dealt with in part in TBM tunneling by tapering the shield and making its diameter adjustable within limits; and by bevelling the cutterhead itself to the extent that this is possible without interfering with the efficiency of the buckets. Expandable gauge cutters are also used, but this is still a developing technology. One of the problems is that there is a tendency for local shearing of the cutter supports to result in an inability to withdraw the cutter once it has been extended. Also, since such cutters are acting well outside the radius of the buckets, muck which falls to the invert is not collected but provides an obstruction the cutters must pass through repeatedly. This grinds the debris finer and finer and abrades the cutter mounts as well as the cutter disk. This makes it necessary to provide means for eccentric cutterhead rotation so that the invert is properly swept. Unfortunately, squeezing is commonly, if not most often, manifested preferentially in the tunnel invert.

8.2.6 Water

It was Terzaghi's view that the worst problems of tunneling could be traced to the presence of water. Among other things, he considered that (except for circular tunnels) it was prudent to double the design rock load on the tunnel lining when the tunnel was below the water table. This in itself would not be a serious problem, since most tunnel linings are already limited as to their minimum dimensions by problems of placement rather than by design considerations. However, there are many other problems that are associated with the presence of water. Several are discussed below, working in sequence from clay to rock and, within rock, from weak and fractured to strong and intact.

8.2.6.1 Clay

Most clays are at least slightly sensitive. This arises from the microstructure of clay soils which are composed largely of platy minerals. As with a heap of coins, the packing is not perfect, even though the clay is relatively impermeable. Each fragment is held in place by some combination of free body equilibrium forces, ionic interaction and chemical or mechanical bonds at the contact points. The pores of the clay are generally filled with water, which may contain salts in solution. Disturbance of the clay results in disruption of the bonding, migration of water and at least temporary weakening of the clay structure. The free water will be released at any temporary boundaries formed by shearing. As the clay reconsolidates, it is likely to gain strength over the initial condition, but this will be a protracted process.
The immediate effect, and the one that affects tunnel construction, is loss of shear strength throughout the disturbed mass. In organic silty clays, the sensitivity is commonly about 4, indicating a fourfold loss of strength upon remolding. This is associated with an initial water content of about 60%. As shown on Page 8-4, a four fold loss of strength can result in more than a sixfold increase in the required support. In any one material, the sensitivity may vary greatly, depending on the water content. Sensitivities as high as 500 to 1,000 may be found in some clays, such as the Leda clay commonly encountered in previously glaciated areas. Marine clays such those found in Boston lose salt by diffusion when situated below the water table. Such clays are typically highly sensitive.

Tunneling is already sufficiently challenging in moderately sensitive clays as the critical number (Section 8.2.2) suffers a local fourfold or more increase. For shielded tunneling, it is very important to avoid excessive efforts to correct line and grade as it is easily possible to create a situation in which control is lost.

A further effect of disturbance of sensitive clays is directly dependent on the loss of pore water expressed from the clay. The volume change results directly in rapid subterranean and surface settlement. In addition, the clay closes rapidly on to the tunnel lining, resulting in even greater settlement unless sufficient compensation and/or contact grout can be injected promptly.

8.2.7 Mixed Face Tunneling

Tunneling in mixed face conditions is a perennial problem and fraught with the possibility of serious ground loss and consequent damage to utilities and structures as well as the prospect of hazard to traffic. The term "mixed face" usually refers to a situation in which the lower part of the working face is in rock while the upper part is in soil. The reverse is possible, as in basalt flows overlying alluvium encountered in construction of the Melbourne subway system. Also found are hard rock ledges in a generally soft matrix bed of hard rock alternating with soft, decomposed and weathered rock; and non-cohesive granular soil above hard clay (as in Washington D.C.) or above saprolite (as in Baltimore). The definition can also be extended to include boulders in a soft matrix (discussed elsewhere in this chapter) and hard, nodular inclusions distributed in soft rock (e.g., flints beds in chalk or garnet in schist).

The primary problem situation is the presence of a weak stratum above a hard one as clearly illustrated in Figure 8-2 for the construction of the 2.3 km long C line and the 4 km long S line of the Oporto Metro project as a part of the mass transit public transport system of Porto, Portugal (Babendererde et al., 2004). The highly variable nature of the deeply weathered Oporto granite overlying the sound granite posed significant challenges to two 8.7 m diameter EPB Tunnel Boring Machines.

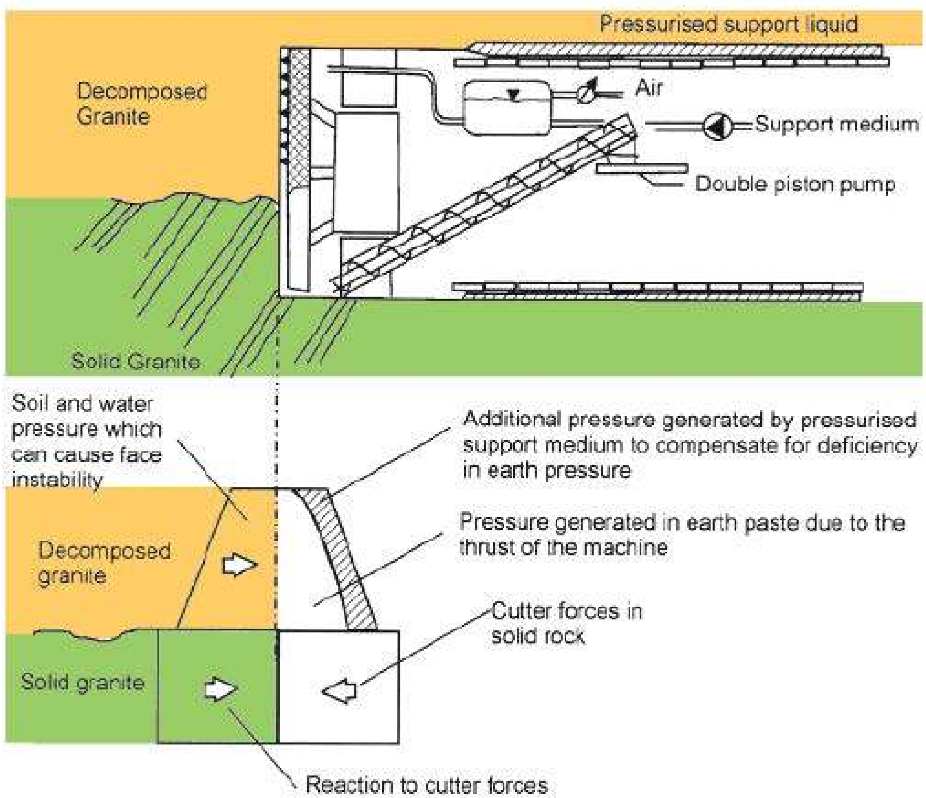


Figure 8-2 Mixed Face Tunneling Example (Babenderede et al., 2004)

There will always be water at the interface which will flow into the tunnel once the mixed face condition is exposed. This increases the hazard because of the destabilization of material already having a short stand-up time. Stabilization therefore calls for groundwater control as well as adequate and continuous support of the weak material. Moreover, this support must be provided where energetic methods, such as drill-and-blast excavation, are required to remove the harder material.

Dewatering can reduce the head of water, but it cannot remove the groundwater completely; nor can it be realistically expected to offer control on an undulating interface with pockets and channels lower than the general elevations established by borehole exploration. Compressed air working will not deal with water in confined lenticular pockets and it is usually inappropriate when the length of the mixed face and soft ground conditions amount to only a few percent of what is otherwise a rock tunnel. Also, recent experience where extensive beds of clean (SP and SP-SM) sands have been major components of the weak ground shows that compressed air alone will not stabilize the ground which becomes free-flowing as soon as it has dried out. Therefore, on the whole, consolidation grouting is to be preferred in this situation.

It is emphasized that the best time to seal off groundwater is before it has started to flow into the tunnel. Once the water is flowing, it is extremely difficult to stop it from within the tunnel except by establishing a bulkhead.

8.3 HEAVING LOADING

8.3.1 Squeezing Rock

When a tunnel opening is formed, the local stress regime is changed. The radial stress falls to zero and the tangential stresses increase to three times the in situ overburden load (neglecting the effects of any locked-in stress resulting from past tectonic action that has not been relieved). If the unconfined compressive strength of the rock is less than the increased tangential stress, a mode of failure will be initiated which is described as "squeezing rock". As elastic failure occurs, with consequent reduced load-bearing capacity of the ground, the load is transferred by internal shear to adjacent ground until an equilibrium condition is reached. If the ground develops brittle failure and is shed from the tunnel walls, then there will be no residual strength of the failed ground to share in the load redistribution. If the ground is sufficiently weak or the overburden load too great, the unrestrained tunnel may close completely.

8.3.2 The Squeezing Process

The detailed mechanism of ground movement is complex and depends on the presence or absence of water and swelling minerals as well as on the physical properties of the ground. For the purposes of this discussion, however, the squeezing process may be described as follows.

8.3.2.1 Initial Elastic Movement

As the tunnel is excavated, stress relief allows elastic rebound of ground previously in compression to relieve stress. This stress relief occurs beyond the working face as well as around the tunnel excavation. In thinly laminated rocks such as schist and phyllite, the modulus of elasticity parallel to the foliation is likely to be much higher than that in the perpendicular direction. Therefore, the elastic movement immediately distorts the shape of the excavation as the rock moves a greater distance perpendicular to the foliation than parallel to it. Moreover, since the rock can move more easily along regular foliation planes than perpendicular to them, more than one factor is at work determining the actual distortion of the tunnel shape. The elastic rebound takes place in all tunnel excavations and is not properly a part of squeezing, which is associated with changes in the rock structure. However, the associated increase in tangential stress in the rock initiates the next phase of movement (squeezing) as the rock fails. As the rock moves toward the tunnel opening, the circumference of the tunnel shortens. There is a limit imposed by the modulus and strength of the rock on how far this process can continue before elastic failure is initiated. Consider a rock of compressive strength 35 Mpa and an elastic modulus of 17,500 Mpa. The circumferential strain per unit length at failure will be 35/17,500 cm/cm or 2 mm/m. For a tunnel of 2 m radius therefore, a shortening of this radius by about 4 mm implies the initiation of impending elastic failure at the exposed rock surface. This does not mean that the rock suddenly loses all strength (unless it is brittle enough to flake off the wall) but rather that its residual strength is greatly reduced. As the tangential shear stress builds up there will come a time when the differential stress is sufficient to cause internal shear failure. This is manifested by the development of new parting surfaces where the overstressed rock separates from the neighboring rock.

8.3.2.2 Strength Reduction

When the rock remaining is insufficiently strong to carry the increased load passed to it as shearing progresses it will fail in turn. In strong and brittle rocks, this failure can result in explosive release of rock fragments from the surface in a phenomenon known as "rock bursting." A somewhat gentler expression of the same phenomenon is known as "popping rock," which is still a dangerous phenomenon. Because these occurrences actually remove rock from the surface there is obviously no residual load-carrying capability of the failed rock. In weaker and less brittle rock the failed material stays in place and enters a plastic or elasto-plastic regime. Its modulus of elasticity and its unconfined compressive strength (which represent its load-carrying capacity) may be reduced by two orders of magnitude, but it can still support some load. In the meantime, the load shed by the failed rock at the perimeter of the opening is transferred deeper into the rock mass where the degree of confinement is higher and the ultimate load-bearing capacity is therefore also higher. The phenomenon may be modelled step-wise, but it is truly a continuous process and will cease only when the total load has been redistributed. Depending on the amount of excess load-carrying capacity available in the partially confined rock around the tunnel perimeter, the stress regime may be affected up to several tunnel diameters away from the opening.

Compounding the stress increase, which leads to failure, is the similar regime in the dome ahead of the working face. The abutment of this dome is the already overstressed rock behind the working face. The problem is therefore three-dimensional in the region affected. The initial movements associated with strength reduction take place quite fast, so that as much as 30% of the final loss of tunnel size may be completed within one to one and a half tunnel diameters behind the working face.

8.3.2.3 Creep

As a consequence of the reduced elastic modulus and the reduced strength of the rock additional radial movement of the tunnel walls occurs. In the zone outside the tunnel, the rock properties are substantially changed. In particular, both the elastic modulus and the unconfined compressive strength decrease continuously (but not in a linear fashion) from their original values still existing in undisturbed rock toward the tunnel wall. The tunnel decreases in diameter as the weakened material creeps toward the tunnel boundary. The rate of movement is roughly proportional to the applied load. The movement is therefore time-dependent (after the initial elastic stress relief, which may be regarded as essentially instantaneous). As the ground is allowed to strain, so the strength of the support required to restrain further movement is reduced. However, depending on the amount of squeezing, shear failures and dilatation accompanying failure may result in unstable conditions in the tunnel walls and crown. Since the timing, location and amount of such failures are not subject to precise definition, support is usually introduced well before the full amount of potential movement has occurred.

8.3.2.4 Modeling Rock Behavior

Because of the nature of the failure mode, elasto-plastic and visco-elasto-plastic mathematical models have been developed to describe the resulting movements and to evaluate the stress regimes for tunnels in rock. These models are not exact but correspond sufficiently well with experience to be useful. Unfortunately, for any given tunnel they depend on the use of information which can only be derived from experience in the specific tunnel involved. This is the origin of the observational approach to tunnel support exemplified by the Sequential Excavation Method (SEM) discussed in Chapter 9.

It has been noted from experimental work that the net load appearing at the tunnel surface varies with the tunnel diameter as a power function. The loading is also dependent on the rate of tunnel advance. It is therefore clear that when such conditions are encountered, the smallest tunnel diameter adequate for the purpose should be selected. Experience also shows that circular tunnels are easier to support than any other shape.

8.3.2.5 Other Factors

If the rock contains porewater, negative pore pressures are set up as the rock moves toward the tunnel. This provides limited initial support until the negative pore pressure is dissipated. In addition, the new pressure gradient set up by the release of confining pressure results in seepage pressures toward the tunnel boundary. In regions of high hydrostatic head, significant increases in rock loading can occur. It is also thought that even small proportions of swelling clay minerals in the rock can contribute significantly to rock loads when water is present. This water need not be flowing--only present in the pores. When all factors contributing to rock mass behavior have been identified and quantified, it may be possible to develop more exact predictive models and to devise new means for controlling and improving ground behavior. In the meantime, we must make do with approximations based on experience.

8.3.2.6 Monitoring

Rate of squeeze and rock loads are somewhat dependent on tunnel size and rate of advance. It is essential in squeezing (or swelling) conditions--or even in blocky and seamy rock where joint closure may create problems--to establish a program of convergence point installations which will be routinely used to monitor the amount and rate of movement of the tunnel walls. This information collected over time and collated with the behavior of the tunnel support system will provide the information needed both to predict and to install the appropriate amount of support as tunneling progresses. This technique lies at the heart of SEM tunneling in rock (Chapter 9). Geotechnical instrumentation is discussed in Chapter 15.

8.3.3 Yielding Supports

One approach to squeezing rock is to go to a simple and workable system of yielding supports as illustrated in Figure 8-3. The number of yielding joints can be modified to provide the needs of the rock currently being excavated since all components are manufactured on site. Each joint permits up to 22 cm of closure. (See
Figure 8-4) It has been found essential to shotcrete the gaps once the closure nears the limit allowed without the steel sections actually butting together. Failures have been common when this butting has been allowed to happen. It has also been found that allowing the invert to heave freely for twenty to thirty days before making an invert closure allows the total support system to resist all remaining loads with some reserve capacity for long term load increases. Other, more complicated yielding systems have been designed and used.

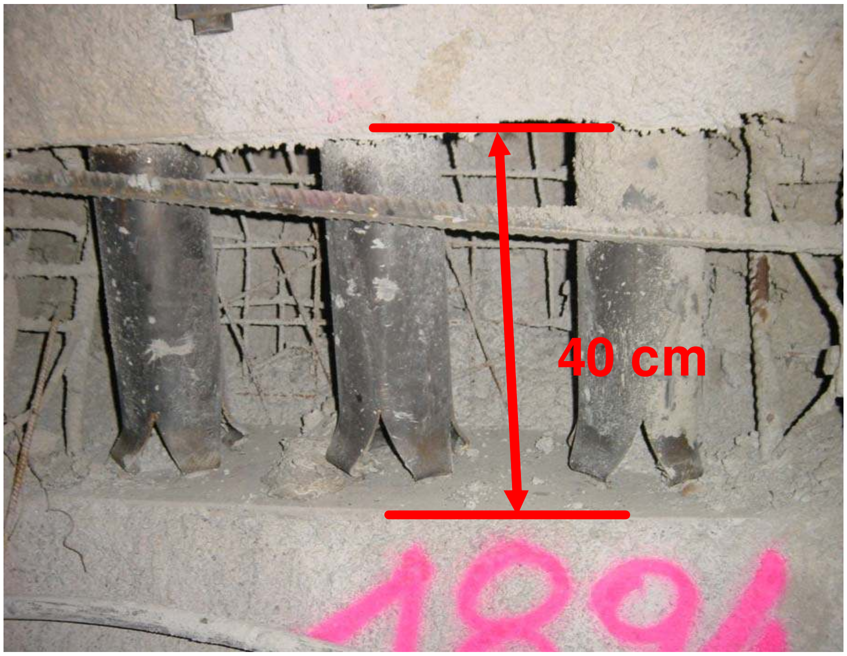


Figure 8-3 Yielding Support in Squeezing Ground

Figure 8-4 Yielding Support Crushed to 20 cm (One-half) (ILF, 2008)

In summary, the support system provides a relatively low initial support pressure and permits almost uniform stress relief for the rock in a controlled manner around the entire circumference of the tunnel while preventing the rock from ravelling. The shotcrete is not damaged by the convergence because of the yielding joints and so maintains its integrity, provided that timely closures are made. After allowing practically all of the stress relief required by the elasto-plastic stage of rock deformation, the support system is made rigid whence it can support a pressure of 3.8 MPa which is available to deal with long term creep pressure.

8.3.3.1 Timber Wedges and Blocking

The use of blocking tightened against the ground by pairs of folding wedges introduces a structural element that can be allowed to fail by crushing. Observation of progressive failure coupled with experience provides warning that the behavior of the steel supports should be closely monitored in case a decrease in spacing or an increase in section becomes necessary.

8.3.3.2 Precast Invert

When squeezing is sufficiently severe to be troublesome. it will be seen that the prime tendency is for the lower sidewalls to move in and for the invert to heave. The loss of strength of the rock in the invert leads to the rapid development of muddy and unstable conditions under the tunnel haulage operations. In this case it may be desirable to use precast concrete invert slabs kept up close to the working face in place of invert struts. In one location where high squeezing occurred, such slabs were heaved up and required maintenance to keep the track on grade. However, they did provide a good and trouble-free surface otherwise.

8.3.4 TBM Tunneling

Because of the number of large tunnels now under consideration where the use of TBMs is contemplated and where squeezing conditions may become important, the following discussion is extended, even though not based on a great deal of current experience.

The majority of examples of tunnels in squeezing ground are related to the crossing of faults. TBMs have been troubled in this situation by inrushes of water carrying sand and finely divided rock or by blocks of rock jamming between the cutters. The second of these problems has been dealt with in many tunnels in otherwise normal conditions. The primary solution is the use of a machine design which allows only a limited projection of the cutters forward of the cutterhead by means of a face shield ahead of the structural support element. The second development is a design which permits worn cutters to be changed from within the tunnel, so that no access is required in front of the cutterhead. There has, as yet, been no easy solution for the problem of the cutterhead and its buckets being choked with sand and rock fragments while unrelenting water flows are in progress. It becomes a difficult and slow process of cleaning out and gaining progress slowly until the affected area has been cleared. Also in such conditions, the presence of a shield is important to protect the machine and to provide temporary support to material with no stand-up time. In some circumstances, if the condition is known to exist or to be likely to exist, probing ahead to identify the precise location can give an opportunity to stabilize the ground with grout injections, keeping a bulkhead thickness ahead of the excavation at all times. It is sometimes possible to allow most of the water to drain out of the ground, but this is not a reliable approach to prediction of construction methods. Shielded TBMs have been used successfully in such conditions, but unfortunately the use of a long shield militates against successful use in squeezing ground.

The other major problem, whether or not in a fault or shear zone, is the closure of the ground around the cutterhead shield and any protective shield behind the cutterhead. Many TBMs, have been immobilized because the load on the shield system was too high to permit the machine to advance. One way to approach this problem is by the use of a short shrinkable shield on the machine.

It is not anticipated that tunneling in squeezing ground or fault zones will ever become a simple and routine operation because of the erratic and unpredictable variability of conditions. However, the current climate of opinion is that virtually all tunnels can be attacked by TBM methods whenever there is an economic advantage in doing so.

As previously noted, the difficulty of predicting rock behavior in squeezing ground has played a major role in the development of observational methods for determination of rock support requirements. However, if tunneling by TBM is selected, some of the flexibility of the observational method is removed because it is difficult to see the face or to measure movements Hence, decisions must be made at the time the TBM is designed as to the amount of ground movement to be anticipated or permitted and the design of the support system to accept the loadings implied at different stages of the tunneling operation.

Since squeezing of soft rock does not usually lead to immediate instability, it should be possible and practical to delay major support installation until a high percentage of the total strain has taken place and ground loading has been reduced. Sixty to seventy percent of the potential ground movement has usually taken place within about three diameters of the working face. If the total amount of squeezing is not great, it may not be necessary, or even desirable, to delay support installation so long.

Ideally, final support is not installed until convergence is less than one millimeter per month. The loading associated with a given amount of convergence is dependent on the parameters of the project. It is also important to take into consideration any long-term requirement for the tunnel to carry water. Finally, it must be realized that if groundwater is to be totally excluded from the tunnel, the final lining must be designed to carry the full hydrostatic head unless the aquifer is fully sealed off by consolidation grouting. If groundwater is admitted, whether in a controlled manner or by allowing local cracking of the lining, then only seepage pressures need be accounted for. In the case of weak squeezing ground or faulted rock with an unknown potential for swelling behavior, the latter alternative appears undesirable.

8.3.5 Steel Rib Support System

Steel ribs set close to the tunnel surface and blocked from it are often used as the initial support system for rock tunnels especially those constructed by conventional drill-and-blast methods. Wood, concrete or steel lagging may be placed between the ribs and the rock to secure blocky or ravelling ground or welded wire fabric may also be used. The same system can also be used in TBM tunnels, but it is necessary to allow an initial small distance between rib and ground so that the last rib segment can be positioned conveniently. In normal tunneling, this space is later closed by expanding the rib against the ground. Especially in squeezing ground, the rib must be blocked to the rock all around its perimeter. As the ground movement occurs and continues, it will squeeze past the ribs and stress relief will occur. In this type of installation, it is necessary for the ribs to be as stiff as possible to prevent displacement and buckling. The chief safeguard is to install steel ties and collar braces at intervals around the rib. The collar braces are typically steel pipe sections set between the ribs. The ties then pass through holes in the web of the steel section and through the pipe forming the collar brace. These members are also subject to deformation by the invading ground. If this creates any substantial problem, angle irons welded to the inner face of the ribs can be substituted.

It is significant that in tunnels where the ribs have buckled under squeezing load but have been left in place, they commonly retain enough structural strength to provide support. The problem is that the squeezing usually intrudes on the required final profile of the tunnel.

8.3.6 Concrete Segments

Segmental concrete linings take two quite different forms. The traditional bolted and gasketted lining is meant to be a final lining erected in one pass. Until recently, the more common application has been to use unbolted, ungasketted segments, with light reinforcement to allow handling, as a "sacrificial: primary lining. This latter type of lining is sacrificial only in the sense that it is allowed to sustain fractures resulting from jacking loads or redistribution of stress; it retains most of its initial load-bearing capacity. A final lining is always placed within this type of lining; it sometimes is an unreinforced concrete lining of nominal thickness, say 10 inches. The combination lining may be less expensive than the one-pass system and has the merit of flexibility. Problems arose with the precast concrete tunnel lining when there was insufficient erection space to allow for deviations normal to tunneling.

In electing to use a precast concrete lining decisions are necessary as to the amount of ground movement to be allowed and the backfill material to be used between the lining and the rock. In allowing for a large amount of potential ground movement, certain problems of erection stability arise. The lining will require support clear of the invert and a horizontal tie or blocking to keep it in shape during and after erection until backfill grouting is complete. There is time and skill involved in executing the work, but no significant difficulty.

Current technology is now trending towards the use of a one pass system of concrete segments. These segments are of high quality concrete and are usually bolted and gasketed at all joints. However, specially doweled circumferential joints are being used. It is necessary that such rings be cast and cured in a controlled factory environment and that they be of high strength concrete for high resistance and high elastic modulus. Steel fibers may be used in lieu of reinforcing steel in some applications.

It is important that the moving ground should not come into contact with the completed ring at a point. Distortion would necessarily result with a possible consequence of reducing load-bearing capacity. It is also possible to use compressible backfill in the annular void provided the material offers sufficient resistance to mobilize passive reactions sufficient to withstand distortion of the lining. At the least, careful consideration would be needed in specifying the strength and deformability of any compressible material to be used.

8.3.7 TBM Tunneling System

The principal components of a TBM affected by the difference between tunneling in squeezing and non-squeezing ground are discussed below. Chapter 6 presents major components and back up system for a tunnel boring machine.

8.3.7.1 Cutterhead

Many different cutterhead designs have been used over the years from the earliest flat heads with multiple disc cutters through domed heads, rounded edge flat heads and conical designs. These days the cutterhead geometry is selected on the basis of the ground it is expected to penetrate. It has been found preferable to arrange that at least the gauge cutters be designed to be changed from behind and it is now common to arrange this system for all cutters. A spoke design allows ready access to the working face and simplifies design in some respects. However, such machines offer little support if weak ground is encountered and it is generally considered prudent to use a closed face machine. Also, to protect the cutters and cutter mounts, a lighter false face is provided so that the cutter disks protrude only a short distance.

In conventional designs, the cutterhead is provided with its own shield as part of the cutterhead bucket system. The conventional design creates a drum about 4 feet (1.2 m) long almost in contact with the ground. In squeezing ground this shield is vulnerable to the pressure exerted by rock movement. It is therefore better that the shield be smaller in diameter than the excavation and that it be tapered toward the rear. The gauge cutters should be arranged to protrude beyond the main body of the cutterhead.

If the cutterhead is not in close contact with the ground, provision must be made to provide stable support in its place. This will be the equivalent of a sole plate as used for overcutter compensation in earth pressure balance machines. However, in order to provide for varying amounts of overcut, the support will need to be hydraulically actuated. Since it will be subjected to substantial shear loading, the design will have to be very stiff.

8.3.7.2 Propulsion

A TBM requires a reaction against which to propel itself forward. This reaction can be obtained by shoving directly against the tunnel support system with jacks spaced around the perimeter of the machine or by developing frictional resistance against the tunnel sidewalls.

The thrust needed to keep the cutterhead moving forward is about 25,000 kg per cutter. When the ground is weak, it is desirable to limit the bearing pressure on the tunnel walls because the weak rock would fail under even light loads, especially perpendicular to the direction of foliation. This would accelerate the rate of squeezing and might increase the total strain. At the same time it would be desirable to limit the length occupied by the grippers so as to minimize the necessary distance between the working face and any support system. This would probably require that there be multiple grippers covering most of the circumference but of limited length to minimize uneven bearing on the squeezing rock surface.

8.3.7.3 Shield

If any shield is felt to be desirable or necessary, it should be short and shrinkable. Many TBMs have been stuck because the ground has moved on to the shield and exerted sufficient load to stall the machine.

8.3.7.4 Erector

It is desirable to have complete flexibility in selecting the point at which ring erection is to take place. Therefore the erector should be free to move along the tunnel, mounted on the conveyor truss. A ring former should also be used to maintain the shape of the last erected ring until it has been grouted if concrete segmental lining is used.

8.3.7.5 Spoil Removal

Conventional conveyor to rail car systems or single conveyor systems designed for the tunnel size selected are appropriate.

8.3.7.6 Back-Up System

In order to keep the area between the grippers and the ring erection area as clear as possible, any ancillary equipment such as transformers, hydraulic pumps etc. should be kept clear of this space at track level.

8.3.8 Operational Flexibility

It is envisaged that the system outlined above would be capable of handling either steel ribs or precast concrete supports. If shoving off the supports were to be selected for TBM propulsion, the degree of flexibility would be less than with the use of a gripper system. It would also be more vulnerable to problems in any circumstance where the convergence rate was markedly higher than expected.

8.3.9 Swelling

Swelling phenomena are generally associated with argillaceous soils or rocks derived from such soils. In the field, it is difficult to distinguish between squeezing and swelling ground, especially since both conditions are often present at the same time. However, except in extreme conditions, squeezing is almost always self-limiting and will not recur vigorously, or at all, once the intruding material has been removed; while swelling may continue as long as free water and swelling minerals are present especially when the intruding material has been removed, thereby exposing fresh, unhydrated rock. Many European rail and highway tunnels are constructed in formations noted for their susceptibility to swelling. Most construction involves a more or less circular wall and roof section with an invert slab having a greater radius of curvature. Some of them are still being periodically repaired a century after construction. It has been noted in this connection that as the invert arches are excavated and replaced to more nearly circular configurations, the greater the time that elapses before the next repair is necessary.

Expansive clays are more common in younger argillaceous rocks, the proportions ranging from 65% in Pliocene and Miocene age material to only 5% in Cambrian and Precambrian. Montmorillonite is found in rocks of all ages as thin partings or thicker beds. Sodium montmorillonite is much more expansive than calcium montmorillonite.

8.3.10 Swelling Mechanism

Most swelling is due to the simultaneous presence of unhydrated swelling clay minerals and free water. Tunnel construction commonly creates these conditions. Minerals such as montmorillonite form layered platy crystals; water may be taken up in the crystal lattice with a resultant increase in volume of up to ten times the volume of the unhydrated crystal. The displacements resulting from this increase in volume give rise to the observed swelling pressures, whether in soil or in rock.

If possible water should be kept away from rock or soil containing swelling clay minerals; however, it must be realized that water from fresh concrete, water vapor from a humid atmosphere or pore water released from confinement within the rock will initiate the swelling process. Since the swelling will not passivate in the same way as squeezing generally will in rock, tunnel support must be designed to resist the swelling pressure (which can be measured in the laboratory), even if it proves possible to let some swelling take place without creating problems.

8.3.11 Other Rock Problems

Schists commonly contain clay minerals such as biotite, mica and chlorite. All of these are platy minerals and are found aligned with the foliation. If present as continuous layers, they have to be considered planes of weakness when assessing questions of rock stability. Similarly, weathered material in shears and mylonite not yet weathered indicate planes of weakness.

Anhydrite converts to gypsum in the presence of water with a volume increase of up to 60% accompanying the conversion. However, beds of anhydrite are not affected in the same way as finely divided rock since the reaction does not penetrate below the surface. However, if the anhydrite is fractured, the conversion will proceed faster and faster as more fracturing is developed by the expansive reaction. The actual amount of expansion will depend upon the void ratio of the anhydrite. As with other water-sensitive minerals, every effort should be made to keep water away from anhydrite. This may be a particular problem when fluid transport tunnels are being constructed since any leakage will result in major damage to the tunnel.

8.4 OBSTACLES AND CONSTRAINTS

8.4.1 Boulders

Practical experience of the value of cutterhead disks in such a situation was first developed in Warrington, England. A slurry shield was to be used for a tunnel originally expected to be in soils. A late decision to change the alignment because of local constraints forced the tunnel into an area where boulders and sandstone bedrock would be encountered in the invert. Since the equipment was already built, disk cutters were added to the head in the hope that they would solve the unexpected problem. These hopes were fulfilled. More recent investigation in Japan has indicated from experimental models that even very soft clay will provide sufficient support to hold boulders in place so that they are broken up by the action of disk cutters. On the other hand, rotary head excavators of various general designs have failed to deal successfully with boulders when drag picks were relied on.

A particular difficulty sometimes occurs when boulder beds are encountered which have saturated fine silt in the void spaces between the boulders. This problem seems to be most often encountered in regions which have been subjected to glaciation. The loss of ground associated with flow of the saturated fines into the tunnel does not normally result in ground settlement, because the movement of any other material replacing the lost fines will generally be choked off. If this is not the case, or if it is felt undesirable to leave such voids unfilled, various courses of action are available. Compressed air working will drive water out of the silt and thereby stabilize it, provided that the boulder bed is not confined within impervious material. In such a case, compressed air working will not be very effective.

The use of an EPB fitted with disk cutters will be effective provided that the pressure in the plenum chamber is kept at a level high enough to balance the hydrostatic head in the silt. Slurry shield operation with the same restrictions would be even more effective, but at a higher cost. As a last resort, consolidation or replacement grouting may be employed behind the shield. The choice of method will depend on economics, as is often the case when selecting a construction method. If the condition exists in only a small part of a long tunnel, less efficient means may be selected for dealing with the boulder bed--even including local cut-and-cover work, if the tunnel is not too deep or the water table too high. In any case, full breasting of the face is required if the boulders are not in intimate contact with one another. It is conceivable that grout could be injected into the working face at a distance behind it so as to force out the flowing material. For such a program to be effective, it would be necessary to grout multiple points simultaneously so as to avoid development of a preferred path for escaping fines. The grout would also have to extend outside the tunnel perimeter for a sufficient distance to establish a plug which could be excavated without developing problems behind the shield. However, it must be said that in small tunnels, access for implementation of such a program is unlikely to be available.

8.4.2 Karstic Limestone

Karstic limestone is often riddled with solution cavities of various sizes. Depending on the geologic history of the locale in which it is found, cavities ahead of the excavation may be filled with water, mud or gravel or a combination of these. Flowing water may be present in large quantities. There may be an insufficient thickness of sound rock at tunnel elevation to provide safe support for tunneling equipment. All of these possibilities point out the need for thorough exploration before undertaking tunnel construction in limestone, particularly in an area where there is no prior history of underground construction or mining.

8.4.3 Abandoned Foundations

Abandoned foundations or other facilities, are to be expected in urban tunnels. To illustrate, on one project 898 piles were encountered during construction of rapid transit tunnels in an urban area. This was more than double the highest estimate. These piles were mostly unrecorded relics of earlier construction abandoned after fires which regularly ravaged the area during the late 19th century as well as piles left behind by successive reclamation operations, which moved the waterfront several hundred meters into the bay over a few decades.

All but two of the piles were timber; they were removed by cutting them into short lengths as they were exposed in the face of the shield using a hydraulically powered beaver-tail chain saw purpose-made for the job. The other two posed a different problem. One was concrete and the other steel. Since the tunnel was being constructed in compressed air, both burning the steel and breaking the concrete were non-trivial problems. Removal of these piles took about 10 times as long as removal of the timber piles. With a TBM, similarly, the wood piles can be cut by the disk cutters but a similar increase in time would be expected for the steel and concrete piles. As an additional problem, the lengths of pile left in place above the tunnel eventually crept downward as they sought to carry the weight of soil adhering to them as well as the artificial fill above. In a number of places it was necessary to reinforce the skin of the fabricated steel liner plates which were dimpled by the point loads exerted.

A second illustrative project involved construction of a storm drainage tunnel. For a short distance at the downstream portal, the tunnel was in soil; because of its short length it was driven without a shield. Since the soil was largely, if not entirely, fill, it proved difficult to maintain the tunnel shape until steel ribs were introduced between alternate rings of liner plate. It was known that old mill foundations lay ahead, but their location was uncertain. It was therefore deemed prudent to continue this tunneling method into the sandstone ahead for at least a short distance. During the drive, some of the foundations were found in the soil tunnel. Careful breasting to isolate the concrete was successful in controlling soil movement while the concrete was broken out. With the next advance of normal tunneling, the voids were promptly and completely filled and there was no encroachment on the tunnel profile.

It would be possible to multiply examples endlessly, but the key to all such problems is to gather the maximum available information, project the worst scenario and be prepared to deal with it as an engineering rather than an economic problem.

8.4.4 Shallow Tunnels

The problem with shallow tunnels is that side support is not reliable and loading on the support system is far from the usual comfortable assumption of essentially uniform radial load. It is quite common in urban situations to be restricted by the presence of significant structures--whether on the surface or underground. Consolidation grouting has been used where ground conditions are favorable and compaction grouting has also been used successfully to avoid the need for underpinning. In other cases; jet grouting, pipe canopies or other spiling may be appropriate. It is not practical to define the range of conditions leading to selection of any particular solution, since all such projects tend to have unique features. Sequential excavation method (Chapter 9), cut and cover method (Chapter 5) and/or jacked box tunneling method (Chapter 12) can be considered as well.

8.5 PHYSICAL CONDITIONS

8.5.1 Methane

Methane is commonly found where organic matter has been trapped below or within sedimentary deposits, whether or not they have yet been lithified. It is particularly common in the shaley limestones around the Great Lakes; in hydrocarbon--whether coal or oil-- deposits in Pennsylvania, West Virginia, Colorado or California and in many other localities. It should be noted here that "methane" is commonly used as a denotation of all of the ethane series that may be present, although methane is distinguished as the major component usually present. It is also the only member substantially lighter than air. Methane forms an explosive mixture when mixed with air and between about 5 and 15% of the total volume is methane. It is readily diluted and flushed from a tunnel by ventilation when encountered in the quantities that are normally expected.

Safety rules require that action be taken when methane is present in concentrations of 20% of the lower explosive limit. For practical purposes, this means a concentration of 1% by volume. It is necessary to use routine testing to determine whether or not explosive gasses are present. This testing is carried out in all tunnels identified as being gassy or potentially gassy. A positive rating of non-gassy is required to relieve the contractor of the duty to test, although in many localities it is deemed prudent to continue testing on a reduced schedule even though no gas has been identified in the tunnel excavation.

8.5.2 Hydrogen Sulfide

Hydrogen sulfide is present in association with methane often enough that its presence should always be suspected in gassy conditions. Its presence is easily identified in low concentrations by its typical rotten egg smell. It is a cumulative poison and deadly in low concentrations; a whiff at 100 percent concentration is generally instantly fatal. It should also be noted that, when hydrogen sulfide is present in low concentrations, the nose becomes desensitized to its presence. Apart from testing and maintenance of high volume ventilation, signs of its presence follow a sequence of headaches, coughing, nausea and unconsciousness. Concentrations should be limited to 10 ppm or less (depending on local regulations) of eight hour exposures.

Meticulous attention to ventilation, especially in work areas, is required when hydrogen sulfide is present or suspected. As with methane, ventilation must be maintained at high volumes for dilution 24 hours a day, 7 days a week, regardless of whether work is in progress or not. Even so, no shaft, pit or tunnel or other opening below grade should be entered without first testing the air. This is especially important if the purpose of entering is to repair a defective fan. Where possible, the gas should be extracted directly and discharged into the ventilation system without ever entering the tunnel atmosphere.

8.5.3 High Temperatures

The geothermal gradient is different in different localities within a range of about 2:1. As a rule of thumb, one degree Celsius per 100 meters of depth will be a reasonable guide. Where the tunnel is comparatively shallow--say less than about 150 m--there will be little effect. In fact, it will be found that the tunnel temperature is the average year-round temperature at that location.

Nevertheless, especially in areas of volcanism or geothermal activity or tropical temperatures, the temperature in deep tunnels can rise to body heat or higher. If hot water flows are present or if the tunnel is very humid (which is more common than not), conditions can be actively dangerous as sweating and evaporation are inhibited; heat stroke can be induced in such conditions. The only factor that can be directly controlled is the tunnel ventilation. By supplying air at a lower temperature, the local conditions can be kept bearable, especially if the incoming air is dry enough to accept evaporating moisture.

8.5.4 Observations

The significant effects and the construction problems resulting from the various difficult tunneling conditions discussed above make it clear that all of the possibilities associated with the geology and occupational history of the region in which new tunneling is contemplated need to be borne in mind from the construction as well as the design standpoint when the preliminary and final geotechnical exploration and testing programs are designed.

It is important that engineers designing a tunnel project develop a full understanding of the nature of the ground conditions affecting the construction; so that not only the field investigation but also the design development, specifications and geotechnical reports reflect a full understanding of the problems and the variety of potential approaches to their solution. In the end, the project owner's interests will be best served by thoughtful analysis and full disclosure of conditions and of the solutions foreseen, and of the underlying design approach rather than by avoiding the recognition of problems and their potential impact.