Support for very weak rock associated with faults and shear zones

Evert Hoek

Distinguished lecture for the opening of the *International Symposium on Rock Support and Reinforcement Practice in Mining*, Kalgoorlie, Australia, 14-19 March, 1999

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SYNOPSIS: Controlling the stability of excavations in very weak rock associated with faults and shear zones requires the innovative use of combinations of support systems such as rockbolts, shotcrete, forepoles and, in some cases, yielding steel sets. Drainage and the sequence of excavation play critical roles in reducing the potential for failure. These approaches are illustrated by means of practical examples.

INTRODUCTION

Faults and shear zones present special challenges in tunnelling because they can lead to sudden and uncontrolled collapses unless appropriate action is taken as soon as they are encountered. The very weak and highly deformable nature of the materials and, in some cases, the presence of large volumes of high pressure water trapped behind the impermeable fault materials results in squeezing or flowing ground conditions. In order to control this behaviour, support must not only have sufficient capacity but it must be installed in a sequence that does not allow uncontrolled deformation of the tunnel.

Early detection of the presence of faults or shear zones is a very important component in the overall process of support design. When the presence of faults is suspected in the rock mass through which the tunnel is being excavated, a probe hole ahead of the advancing face should be made mandatory. This hole can be percussion drilled and the rate of penetration, colour and volume of return water and the character of the chippings should be monitored during the drilling. This will indicate significant differences in the rock mass character ahead of the face and, where very weak rock is indicated, a diamond-drilled probe hole can be used to explore the rock in more detail. In civil engineering tunnel driving these probe holes are typically drilled during weekend maintenance shifts and their length should be approximately the total advance during the week plus one tunnel diameter. This ensures that the rock has been explored for at least one tunnel diameter ahead of the advancing face.

When the presence and the approximate extent of a fault or shear zone has been confirmed, steps have to be taken to design a support system and a sequence of excavation and support installation to deal with the anticipated conditions. It is essential that all the required support elements should be available close at hand before the fault itself is exposed to any significant extent.

Depending upon the nature and extent of the fault and whether water is present, a variety of support systems and excavation sequences can be used. Before discussing these alternatives and considering some practical examples, it is necessary to consider some fundamental issues related to tunnelling through weak rock.

ROCK MASS STRENGTH / IN SITU STRESS

In the context of this discussion, a rock mass is considered to be weak when its in situ uniaxial compressive strength is less than about one third of the in situ stress acting upon the rock mass through which the tunnel is being excavated. This can be demonstrated by means of the plot of tunnel convergence versus the ratio of rock mass strength to in situ stress given in Figure 1. This plot shows a sudden increase in convergence for a strength/stress ratio of less than about one third. The plot was generated from a closed-form analysis of the development of rock mass failure surrounding an unsupported circular tunnel subjected to equal stresses in all directions. The analysis used follows that described by Duncan-Fama (1993) and by Hoek, Kaiser and Bawden (1995).

A Monte Carlo simulation was used to carry out this analysis for 2000 iterations for uniform distributions of the rock mass properties, tunnel radius and in situ stress level. The rock mass properties were varied from fair to extremely poor, corresponding to the properties of weak sandstones and mudstones down to material that can almost be classed as soil. The in situ stresses were varied from 2 to 20 MPa, corresponding to depths below surface from 75 to 750 m, and the tunnel diameters were varied from 4 to 16 metres.

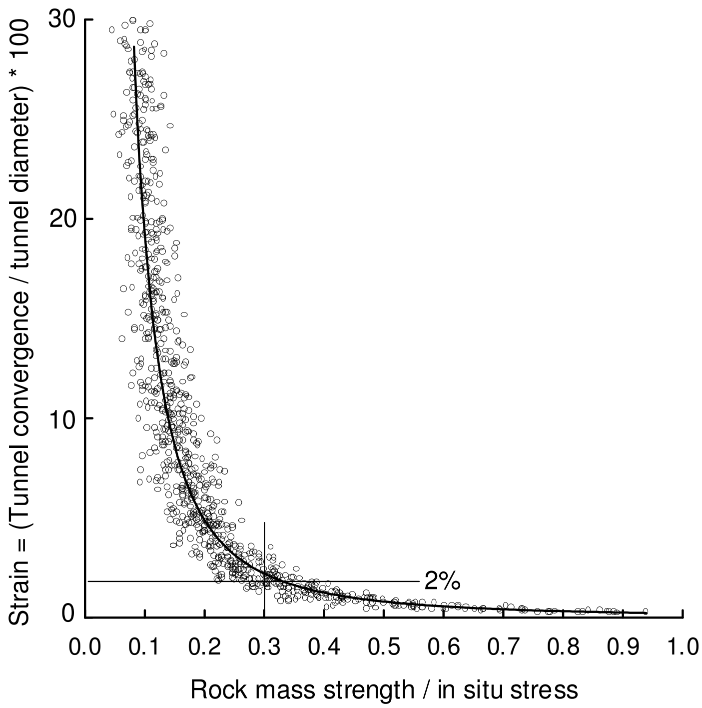


Figure 1: Plot of tunnel convergence against the ratio of rock mass strength to in situ stress.

CRITICAL STRAIN

Sakurai (1983) has suggested that the stability of tunnels can be assessed on the basis of the strain in the rock mass surrounding the tunnel. The strain is defined by the ratio of tunnel convergence to tunnel diameter. A critical strain of approximately 2% represents the boundary between 'stable' tunnels that require minimal support and 'unstable' tunnels that require special consideration in terms of support design.

The application of this concept to practical tunnel problems is illustrated in Figure 2 that shows the percentage strain observed during the construction of three tunnels in Taiwan[1]. It can be seen that those tunnels categorised as requiring special support consideration fall above a line that is well defined by Sakurai's critical strain concept.

Note that all of the tunnels included in Figure 2 were constructed successfully, including those that suffered strains of approximately 10%. In some of these cases the tunnels had to be re-mined since the profiles were no longer adequate to accommodate the service structures for which they were designed.

Sakurai's critical strain of 2% has been plotted in Figure 1. It can be seen that this corresponds well with the earlier conclusion that tunnels, excavated under conditions where the rock mass compressive strength is less that about one third of the in situ stress level, will suffer serious stability problems unless adequately supported.

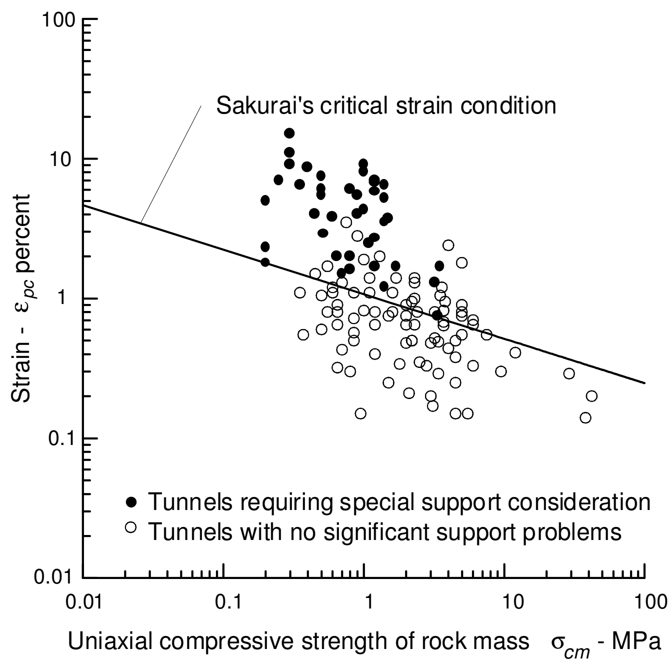


Figure 2: Percentage strain for different rock mass strengths. The points plotted are for the Second Freeway, the Pinglin and the New Tienlun Headrace tunnels in Taiwan.

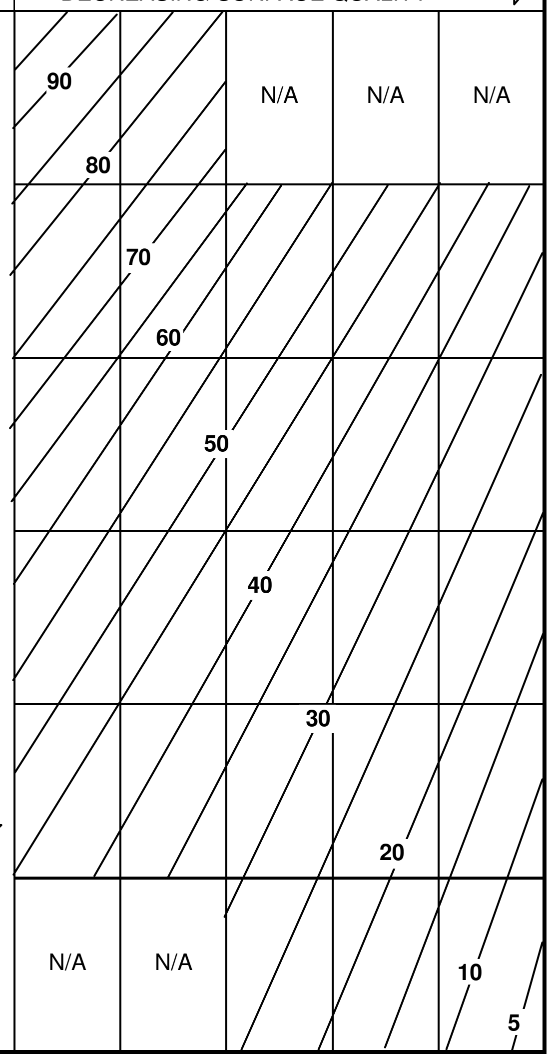
ROCK MASS STRENGTH ESTIMATES

As a first approximation, the in situ stress can be assumed to equal the product of the depth below surface and the unit weight of the rock mass. The uniaxial compressive strength of the rock mass can be estimated from the Geological Strength Index (GSI) proposed by Hoek and Brown (1997) and extended by Hoek, Marinos and Benissi (1998). This descriptive index, which depends upon the structural characteristics and the surface conditions of discontinuities in the rock mass, is defined in Table 1.

[1] Information in this plot was supplied by Dr J.C. Chern of Sinotech Engineering Consultants Inc., Taipei.

Table 1: Table for estimating *GSI* (Hoek and Brown 1997, Hoek, Marinos and Benissi 1998)

GEOLOGICAL STRENGTH INDEX From the description of structure and surface conditions of the rock mass, pick an appropriate box in this chart. Estimate the average value of the Geological Strength Index (GSI) from the contours. Do not attempt to be too precise. Quoting a range of GSI from 36 to 42 is more realistic than stating that GSI = 38. It is also important to recognize that the Hoek-Brown criterion should only be applied to rock masses where the size of the individual blocks or pieces is small compared with the size of the excavation under consideration. When individual block sizes are more than approximately one quarter of the excavation dimension, failure will generally be structurally controlled and the Hoek-Brown criterion should not be used. STRUCTURE	SURFACE CONDITIONS	VERY GOOD Very rough, fresh unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered and altered surfaces	POOR Slickensided, highly weathered surfaces with coatings or fillings of angular fragments	VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings
		DECREASING SURFACE QUALITY ⇨				
INTACT OR MASSIVE – intact rock specimens or massive in situ rock with very few widely spaced discontinuities	DECREASING INTERLOCKING OF ROCK PIECES ⇩			N/A	N/A	N/A
BLOCKY - very well interlocked undisturbed rock mass consisting of cubical blocks formed by three orthogonal discontinuity sets						
VERY BLOCKY - interlocked, partially disturbed rock mass with multifaceted angular blocks formed by four or more discontinuity sets						
BLOCKY/DISTURBED - folded and/or faulted with angular blocks formed by many intersecting discontinuity sets						
DISINTEGRATED - poorly interlocked, heavily broken rock mass with a mixture of angular and rounded rock pieces						
FOLIATED/LAMINATED – Folded and tectonically sheared foliated rocks. Schistosity prevails over any other discontinuity set, resulting in complete lack of blockiness		N/A	N/A			



An approximate relationship between the ratio of rock mass strength σ_{cm} to laboratory rock strength σ_{ci} and the value of GSI is given by:

$$\sigma_{cm} = 0.019\sigma_{ci}\, e^{0.05GSI} \tag{1}$$

As an example of the application of the information presented above, consider a rock mass which has been assigned a GSI = 23. The laboratory compressive strength of the rock mass is 5 MPa and hence, from equation 1, the rock mass compressive strength is estimated at 0.3 MPa. A tunnel being excavated at a depth of 150 m below surface in a rock mass with a unit weight of 0.025 MN/m^3 will be subjected to an in situ stress of approximately 3.8 MPa. Hence, the ratio of rock mass strength to in situ stress is approximately 0.08 and, from Figure 1, this is clearly in the category where serious stability problems can occur unless appropriate support is installed.

CHARACTERISTIC LINE CALCULATION

Fenner (1938) introduced the concept of calculating the convergence associated with the formation of a 'plastic' zone, or zone of damaged rock, surrounding an advancing tunnel. The basic elements of this concept are illustrated in Figure 3 which shows that the size of the plastic zone depends upon the equivalent support pressure p_i. In an unsupported tunnel the value of p_i reduces from the in situ stress value p_o to zero with distance from the face. The rock starts reacting to the oncoming tunnel about one-half tunnel diameter ahead of the face. At the face about one third of the total deformation has occurred and the final total deformation and complete formation of the plastic zone occurs about 1.5 tunnel diameters behind the face. The relationship between the support pressure p_i and the inward deformation δ of the tunnel walls is known as the characteristic line and a typical example is shown in Figure 4. Included in this are plots of the thickness of the plastic zone and of the reaction of a support system installed in the tunnel. This support reaction curve is discussed in the next section of this paper.

The equations used to calculate the curves presented in Figure 4 together with a sample spreadsheet are included in Appendix 1.

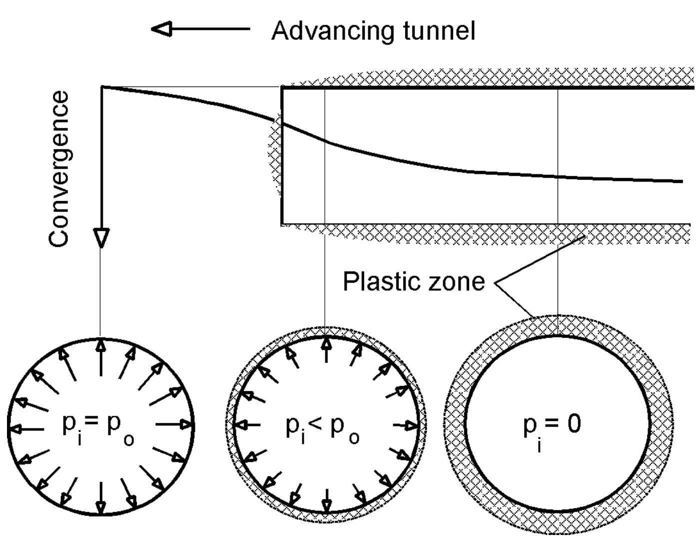


Figure 3: Assumed support pressure p_i at different positions relative to the advancing tunnel face. (Not to scale).

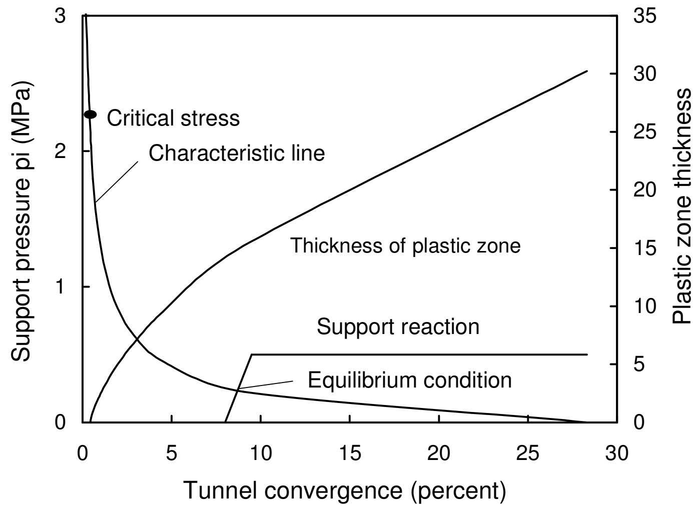


Figure 4: Example of characteristic line analysis including a plot of plastic zone thickness and of a support reaction curve for a 10 m diameter tunnel at 150 m depth.

The characteristic line calculations included in Appendix 1 are based upon the assumption that the rock mass surrounding the tunnel fails with no increase in volume. This is an appropriate assumption for very weak rock masses of the type associated with faults and shear zones in which the rock mass is likely to crush rather than to fail in a dilatant manner. A number of analyses in which dilation and also time-dependent failure of the rock mass have been published (Brown et al 1985, Panet 1993). These analyses can give a very useful insight into the behaviour of different types of rock masses. However, users should beware that the sophistication of the equations does not seduce them with an illusion of precision. In fact, all these analyses are only crude approximations of the actual tunnel behaviour, because the simplifications required to allow the equations to be solved seldom reflect the actual in situ conditions. For example, all of these analyses assume that the tunnel is circular and that it is subjected to a hydrostatic stress field in which stresses in all directions are equal.

In spite of its limitations, the characteristic line method has one very important attribute that makes it a very useful engineering design tool. The closed form of the equations used in these analyses makes it possible to assign probability distributions to each of the significant variables and to carry out Monte Carlo analyses that give distributions of the output variables. This is the process that was used to generate the points plotted in Figure 1. This type of probability analysis has also been used by Grasso et al (1997) to obtain an equivalent ‘factor of safety’ for tunnel support designs.

SUPPORT CHARACTERISTICS

When support is installed in a tunnel where plastic failure has occurred in the surrounding rock mass, the support pressure p_i provided by the support depends upon the stiffness of the support, its maximum load bearing capacity and the distance from the face when it was installed. The support acts very much like a system of springs and the support pressure increases with increasing deformation until the capacity of the system is exceeded. Hoek and Brown (1980) and Brady and Brown (1990) have published equations for calculating the stiffness and capacity of different support systems and these have been used to estimate the support characteristics given in Figure 5. Note that these characteristics are based upon the assumption that the support system is symmetrical around the tunnel. In other words, it is assumed that the steel sets and concrete or shotcrete linings are completely circular and that the rockbolts are installed in the roof, sidewalls and floor of the tunnel. These assumptions do not reflect typical support installations in the field and so, as in the case of the characteristic line calculations, they should be used to explore behaviour patterns rather than to calculate support characteristics to three decimal places.

As an example of the application of the information contained in Figure 5, consider the case of the tunnel behaviour illustrated in Figure 4. The total strain of the tunnel without support is approximately 28%, which means that the 10 m diameter tunnel will converge 2.8 m. From experience, I would suggest that the rock mass surrounding the tunnel would not be capable of sustaining this amount of convergence and that, without immediate support, this tunnel would collapse. Immediate support in this case means the installation of a suitable support system immediately behind the advancing face. As stated earlier, approximately one third of the final convergence has already occurred at the tunnel face. Consequently the earliest that conventional support can be installed is at a convergence of about 9%. It is probable that the face itself would also require support in the form of grouted fibreglass dowels or an umbrella of forepoles.

The plot in Figure 4 shows that the thickness of the plastic zone at a convergence of 5% is about 12 m. As a general rule, rockbolts or cables should have 1 to 2 m of anchorage in undisturbed rock outside the plastic zone. This means that rockbolts or cables of about 14 m would be required to provide support in this case and installation of these systems in a 10 m diameter tunnel is not a very efficient operation. In view of the uncertainty associated with the reliability of the anchorage in this poor quality rock mass, I suggest that rockbolts or cables are not an appropriate support system for this case and that steel sets or lattice girders should be considered.

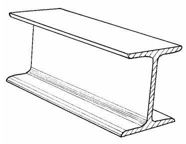
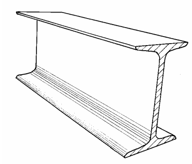
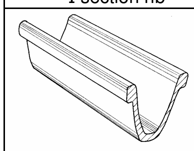
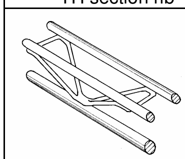
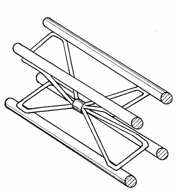
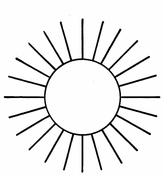
From the information given in Figure 5, the support capacity of full-circle 200 x 200 mm wide flange ribs, 150 x 200 mm I section ribs, 124 x 108 mm TH section ribs and three-bar lattice girders at a spacing of 1 m in a 10 m diameter tunnel have support capacities of approximately 0.5 MPa. The maximum elastic strain that can be sustained by these girders is about 1% and, since they are installed at a convergence of 9%, this gives a total convergence of 10%. This support behaviour is plotted in Figure 4 as the support reaction curve.

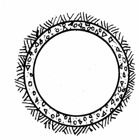
Equilibrium conditions are achieved when the convergence of the tunnel and the support system are equal and, as shown in Figure 4, these conditions are defined by the intersection of the characteristic line and the support reaction curve.

The 'factor of safety' of the support system can be defined as the ratio of the maximum support capacity to the support pressure required for equilibrium. In this case, this factor of safety is approximately 2. While this may seem to be excessive, it must be remembered that it has been assumed that the full-circle steel support systems function perfectly. This may be too optimistic an assumption. As will be shown in the practical examples discussed later, there are times during the excavation of the tunnel and the installation of support when the support elements are subjected to unfavourable loading conditions and when they can be over-stressed. Hence, some reserve capacity is appropriate.

Note that all of the support pressures in Figure 5 have been plotted for steel set or rockbolt spaced at 1 m and that, in order to determine the support pressures for other spacings, the equations given for each support type should be used.

When support types are combined, the total available support pressure can be estimated by summing the maximum allowable pressures for each system. However, in making this assumption it has to be realised that these support systems do not necessarily act at the same time and that it may be necessary to check the compatibility of the systems in terms of deformation. For example if lattice girders embedded in shotcrete are installed immediately behind the tunnel face, they will accept load immediately while the shotcrete will accept an increasing amount of load as it hardens (compare curves 24, 25 and 26 in Figure 5). Depending on the rate of advance of the tunnel, it is necessary to check that the capacity of the lattice girders is not exceeded before the shotcrete has hardened to the extent that it can carry its full share of the load.

Support type	Flange width - mm	Section depth - mm	Weight – kg/m	Curve number	Maximum support pressure p_{imax} (MPa) for a tunnel of diameter D (metres) and a set spacing of s (metres)
 Wide flange rib	305	305	97	1	$p_{i\max} = 19.9D^{-1.23}/s$
	203	203	67	2	$p_{i\max} = 13.2D^{-1.3}/s$
	150	150	32	3	$p_{i\max} = 7.0D^{-1.4}/s$
 I section rib	203	254	82	4	$p_{i\max} = 17.6D^{-1.29}/s$
	152	203	52	5	$p_{i\max} = 11.1D^{-1.33}/s$
 TH section rib	171	138	38	6	$p_{i\max} = 15.5D^{-1.24}/s$
	124	108	21	7	$p_{i\max} = 8.8D^{-1.27}/s$
 3 bar lattice girder	220	190	19	8	$p_{i\max} = 8.6D^{-1.03}/s$
	140	130	18		
 4 bar lattice girder	220	280	29	9	$p_{i\max} = 18.3D^{-1.02}/s$
	140	200	26		
 Rockbolts or cables spaced on a grid of s x s metres	34 mm rockbolt			10	$p_{i\max} = 0.354/s^2$
	25 mm rockbolt			11	$p_{i\max} = 0.267/s^2$
	19 mm rockbolt			12	$p_{i\max} = 0.184/s^2$
	17 mm rockbolt			13	$p_{i\max} = 0.10/s^2$
	SS39 Split set			14	$p_{i\max} = 0.05/s^2$
	EXX Swellex			15	$p_{i\max} = 0.11/s^2$
	20mm rebar			16	$p_{i\max} = 0.17/s^2$
	22mm fibreglass			17	$p_{i\max} = 0.26/s^2$
	Plain cable			18	$p_{i\max} = 0.15/s^2$
	Birdcage cable			19	$p_{i\max} = 0.30/s^2$

Support type	Thickness - mm	Age - days	UCS - MPa	Curve number	Maximum support pressure p_{imax} (MPa) for a tunnel of diameter D (metres)
 Concrete or shotcrete lining	1m	28	35	20	$p_{i\max} = 57.8D^{-0.92}$
	300	28	35	21	$p_{i\max} = 19.1D^{-0.92}$
	150	28	35	22	$p_{i\max} = 10.6D^{-0.97}$
	100	28	35	23	$p_{i\max} = 7.3D^{-0.98}$
	50	28	35	24	$p_{i\max} = 3.8D^{-0.99}$
	50	3	11	25	$p_{i\max} = 1.1D^{-0.97}$
	50	0.5	6	26	$p_{i\max} = 0.6D^{-1.0}$

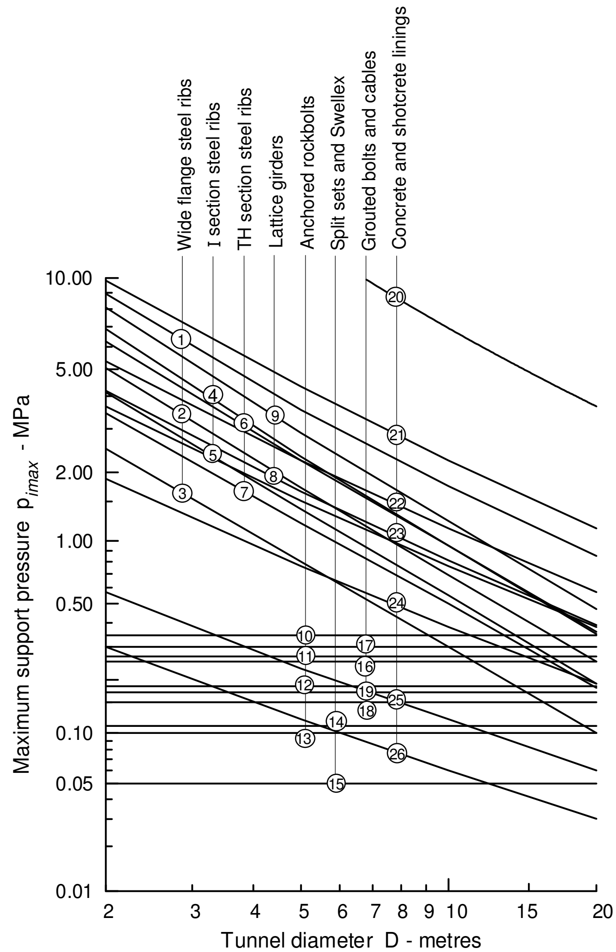


Figure 5: Approximate maximum capacities for different support systems installed in circular tunnels. Note that steel sets and rockbolts are all spaced at 1 m.

NUMERICAL ANALYSIS OF SUPPORT

The characteristic line calculation presented above is a very useful tool that is adequate for many practical support design problems. However, since it only gives an estimate of the final support capacity, it cannot be used to investigate the details of the excavation and support installation sequence required to deal with very difficult tunnelling problems such as those under consideration in this paper. In such cases, a numerical analysis will provide a more complete analysis.

Fortunately, there are several excellent programs available commercially that make it possible to carry out these analyses quickly and efficiently. Two of the best-known programs are FLAC[2], a very powerful finite difference program, and PHASE2W[3] a simpler and more user-friendly finite element program. The program PHASE2W (Windows 95 version) was used to carry out a more detailed analysis of the problem discussed in the first part of this paper.

The tunnel to be analysed has a modified horse-shoe shape of 10 m span and it is being excavated, as part of a drill-and-blast tunnel driving operation, through very weak rock at a depth of 150 m below surface. The properties of the rock mass, from the spreadsheet given in Appendix 1, are as follows:

Geological Strength Index GSI	23
Rock mass compressive strength σ_{cm}	0.28 MPa
Friction angle ϕ	22.15°
Cohesive strength c	0.1 MPa
Rock mass deformation modbulus E	473 MPa
Rock mass Poisson's ratio ν	0.3

It is assumed that the in situ stress field is hydrostatic, in other words the stresses are the same in all directions. This is a reasonable assumption for very weak rock such as that in a fault or shear zone, since this type of rock has already undergone failure and is incapable of sustaining significant stress differences. Hence, even if the far field stresses are asymmetrical, the stresses within the fault zone are likely to be approximately hydrostatic.

The first issue to be checked is the stability of the tunnel face. It has already been established, by means of the characteristic line calculation, that the 'plastic zone' surrounding the tunnel is very large and that substantial support is required to maintain its stability. What about the stability of the face and what support measures need to be taken to keep this stable?

Figure 6 shows the results of a three-dimensional analysis carried out using the axi-symmetric option in PHASE2W. This shows that the displacements of the face are of a similar order to those of the tunnel walls and this suggests that special measures will be required in order to prevent collapse of the face during excavation. These issues are discussed in detail in the following section.

TUNNEL FACE STABILITY

Water pressure and drainage

The first practical question to be considered is whether there is likely to be water pressure behind the face. Fault zones are generally less permeable than the surrounding rock mass and tend

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[2] Available from the ITASCA Consulting Group Inc., Thresher Square East, 708 South Third Street, Suite 310, Minneapolis, Minnesota 55415, USA, Fax + 1 612 371 4717. Internet: http://www.itascacg.com.

[3] Available from Rocscience Inc., 31 Balsam Avenue, Toronto, Ontario, Canada M4E 3B5, Fax + 1 416 698 0908. Internet: http://www.rocscience.com.

to act as dams. Consequently, there is a reasonable chance that a large volume of high-pressure water may be trapped behind the face. Driving the full face of a 10 m span tunnel into a fault zone with water trapped behind it is an invitation to disaster.

It is essential, when working in such ground, that a percussion-drilled probe hole be advanced well ahead of the face at all times. This will give warning of the presence of high pressure water and allow time for drainage measures to be set up before the fault material is exposed in the face. In general, drainage of the water is the most satisfactory solution and it may be necessary to install additional pumping capacity to deal with the water volume. In some cases, for example when mining or tunnelling under the sea or under a lake, drainage may not be the best option and grouting of the rock ahead of the face may have to be considered. The purpose of this grouting is to create a zone of impermeable rock in which the water pressure and flow can be controlled during tunnel driving.

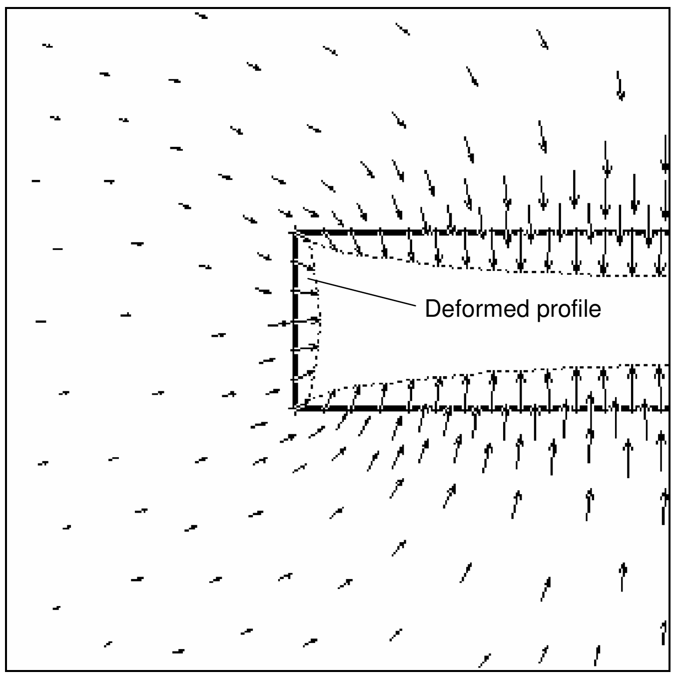


Figure 6: Vertical section through a three-dimensional finite element model of the failure and deformation of the rock mass surrounding the face of an advancing circular tunnel.

Face support

Once the issue of water has been taken care of, the next question is how to prevent collapse of the face. Depending upon the severity of the problem, several options are available and these are reviewed below.

Grouted fibreglass dowels

The simplest option for face support is to reinforce the face with grouted fibreglass dowels which provide excellent support but are easy to mine through as the face is excavated. Experience suggests that these work best when the rock mass has a low clay mineral content and where friction rather than cohesion control its shear strength. This is because efficient operation of the dowels requires a high strength bond between the grout annulus and the rock mass and this is difficult to achieve in clay-rich cohesive materials. In the fault zone under consideration here, the cohesive strength is low (0.1 MPa) and the friction angle reasonably high (22°). Hence there is a good chance that fibreglass dowels would work well. However, given the magnitude of the 'plastic' zone in this case, it would be unwise to rely on such dowels as the only method of face support and I recommend that they be used to supplement other support systems rather than to replace them.

Partial face excavation

In tunnelling through weak ground it is generally accepted that the stability of the face depends upon the area exposed. Consequently, one commonly used technique for maintaining stability is partial face excavation in which the tunnel is driven in stages such that the area of each face is small enough to control. One of the many alternative ways of doing this is illustrated in Figure 7 where a technique favoured by the German and Austrian tunnel engineers is used.

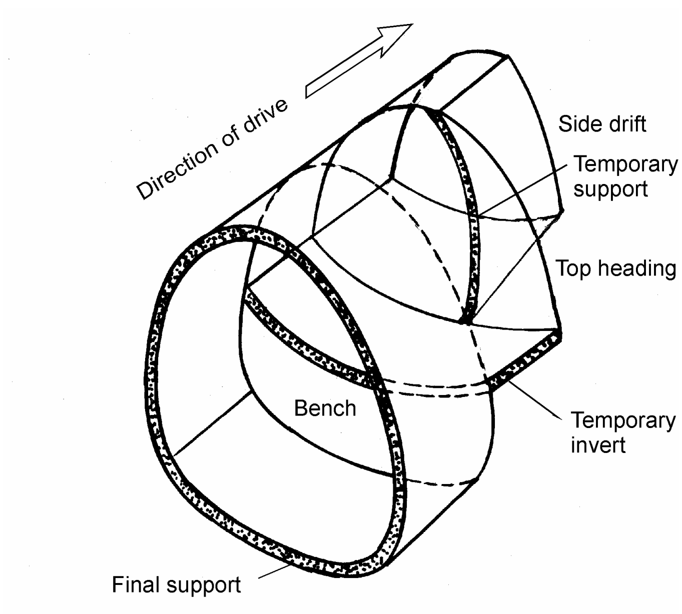


Figure 7: Partial face excavation method using a side drift followed by top heading and bench.

In this method, a side drift is excavated first and this is supported by installing the final support, such as steel sets embedded in shotcrete, against the outside wall and temporary support on the vertical wall and on the floor. Where possible, this temporary support should consist of fibre-reinforced shotcrete since this is easy to excavate when the tunnel is enlarged. However, in very heavy squeezing conditions, heavy weld mesh or steel ribs may have to be embedded in the shotcrete to provide sufficient support capacity. In many cases it is advantageous to drive the side drift all the way through the fault or shear zone, since it can be used to establish efficient drainage and ventilation arrangements before the main drive is attempted.

The excavation of the side drift is followed by opening the top heading to full span and this involves destruction of the temporary wall and the extension of the temporary invert to full span. This process should be carried out in steps of a few metres since one side of the top heading is effectively unsupported during the excavation process.

Excavation of the bench to form the full tunnel profile should also be carried out in steps of a few metres. The removal of the temporary invert leaves the top heading support 'suspended' until the lower legs of the steel sets can be installed and the final invert placed. In heavy squeezing conditions, a vertical bench parallel to the face is a good method of excavating the full profile since it allows systematic installation of the set and closure of the invert to be carried out with the benefit of support from the bench.

Placing steel sets and the application of shotcrete in a large tunnel requires heavy equipment to operate close to the face. This means that placing the final invert can present significant practical problems. Many tunnellers will attempt to advance the tunnel as far as possible and to leave the invert to be completed as an off-line activity tens of metres behind the face. In squeezing ground such as that under consideration here, this is always a serious mistake since it will allow floor heave and severe inward deformation of the installed roof and sidewall support. Failure to close the invert in time is probably the most common cause for the failure of support systems in squeezing ground.

The complete process of side drift, top heading and bench excavation has been simulated by numerical analysis and the results are presented in Figure 8. In spite of the approximations required in carrying out this analysis, the results provide a very useful guide to the adequacy of the excavation sequence and support systems.

Comparison of the results of the characteristic line calculation (Figure 4) and the finite element analysis (Figure 8) shows that the stepwise excavation and support installation sequence provides a significant improvement on the full-face excavation process depicted in Figure 4. Convergence is limited to about 5% and the maximum depth of the plastic zone in the roof is about 15 m. Note that this finite element model has been gravity loaded and the self-weight of the broken rock results in more failure in the rock above the roof than in that below the floor.

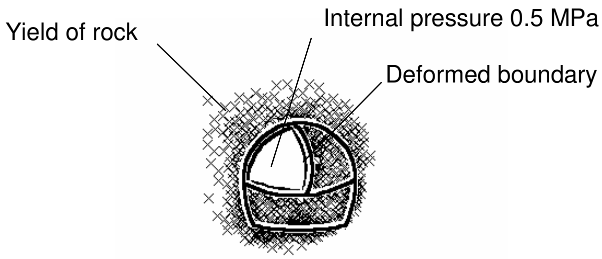
In the finite element analysis it was assumed that the final support consists of steel sets, typically 200 x 200 mm wide flange sections, embedded in shotcrete. While it is difficult to incorporate the properties of this composite system into currently available numerical models, it is possible to make a reasonable approximation based upon a consideration of when different elements of the support system are activated.

On the basis of such assumptions it was found that some minor yield of the support occurred, particularly at the connections between the upper and lower portions of the sidewall support and at the junctions of the sidewall support and the invert. This yield, which is associated with the sequence of excavation and support installation, is relatively common in this type of support system and has no major practical significance. It normally shows up as minor spalling in the shotcrete and this can easily be repaired by chipping out the damaged material and applying fresh shotcrete. Unless there is on-going time-dependent deformation, it is unlikely that this damage will recur once the damage has been repaired.

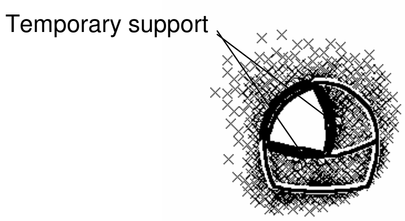
Advancing under a forepole umbrella

As an alternative to the partial face method described above, the umbrella arch method is sometimes used, particularly by Italian tunnellers, for advancing through difficult ground (Carrieri et al 1991). In a 10 m span tunnel of the type being considered here, the method would typically involve installing 12 m long 75 mm diameter grouted pipe forepoles at a spacing of 300 to 600 mm. These forepoles would be installed every 8 m to provide a minimum of 4 m of overlap between successive umbrellas. A sketch of a typical forepole umbrella is given in Figure 9.

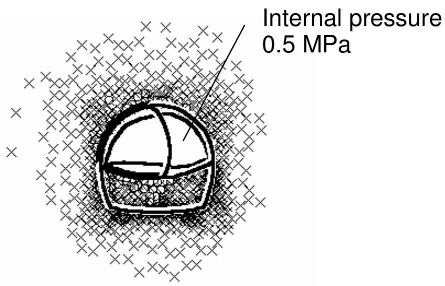
a. Excavation boundaries defined but no rock removed, model allowed to consolidate.



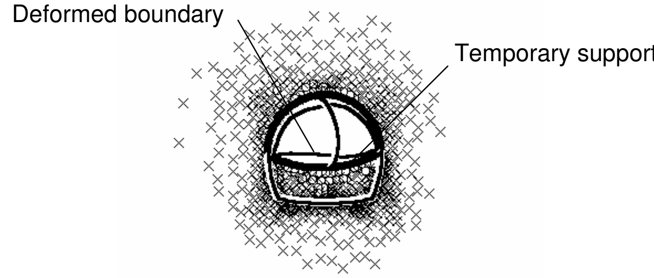
b. Excavation of side drift and application of an internal pressure of 0.5 MPa to simulate support provided by the advancing face.



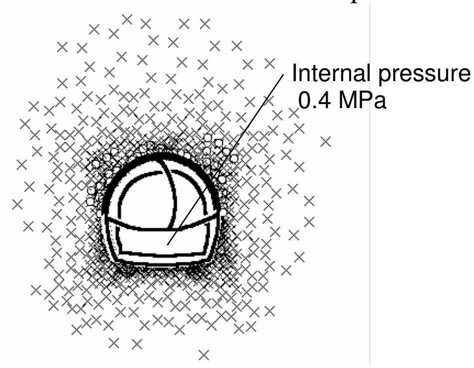
c. Installation of support in the form of steel sets embedded in shotcrete against the tunnel wall and temporary shotcrete invert and vertical partition. Removal of internal pressure.



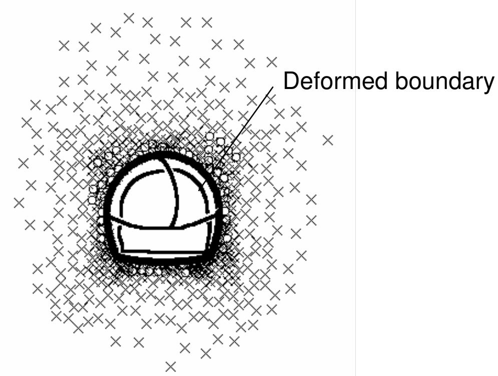
d. Excavation of top heading and application of internal pressure of 0.5 MPa to newly excavated right hand side to simulate support from face.



e. Installation of support in the form of steel sets embedded in shotcrete on tunnel walls and temporary shotcrete invert. Removal of internal pressure.



f. Excavation of lower bench and application of internal pressure of 0.4 MPa to simulate support provided by face.



g. Completion of sidewall support and casting of concrete invert. Removal of internal pressure to allow full load on the completed support.

Figure 8 : Finite element analysis simulating the excavation sequence for mining a 10 m span tunnel through a fault at a depth of 150 m below surface. Final displacements for the roof are 250 mm, sidewalls 275 mm and floor heave is 190 mm.

A first step in this method usually involves drilling holes, up to 30 m ahead of the face, for drainage. This is followed by the drilling of the 12 m long holes and installation of the pipe forepoles to form the umbrella arch. In some cases, depending upon the nature of the rock mass being tunnelled through, jet-grouted columns are used rather than the grouted pipe forepoles. The tunnel is then advanced 8 m before the cycle is repeated to create another protective umbrella.

This method is frequently used in combination with other support systems such as steel sets embedded in shotcrete, face stabilisation by grouted fibreglass dowels and the use of a temporary invert to control floor heave.

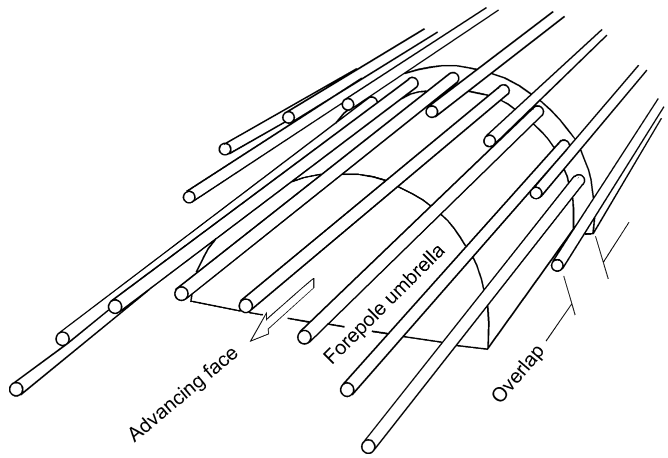


Figure 9: Sketch of tunnelling under the protection of a forepole umbrella.

Analysis of the performance of this system, particularly when used in combination with other support systems, is extremely difficult. Simplified finite element analyses, using axi-symmetric models, have been described by Grasso et al (1993) and similar studies, using FLAC3D have been described by Itasca (1997). However, these analyses are certainly outside the type of studies that could be considered part of routine tunnel engineering design and a great deal of reliance has to be placed on judgement and experience. It is unlikely that this situation will change any time soon.

In spite of the lack of design guidelines and analytical tools, the umbrella arch is a very powerful tunnelling tool which, in the hands of an experienced operator, can be used to excavate through very difficult ground conditions.

Yielding steel sets

A method that has been used successfully in many tunnels involves the installation of steel sets fitted with sliding joints such as that shown in Figure 10.

Sánchez and Terán (1994) describe the use of yielding elements in steel ribs for the support of the Yacambú-Quibor tunnel in Venezuela – regarded by many as one of the most difficult tunnels in the world. This 5.5 m diameter water supply tunnel through the Andes is being excavated though weak rock masses, including graphitic phyllites, at a maximum depth below surface of 1200 m.

In the weakest rock sections, the support consists of WF6x20 steel sets, at 1 m spacing, with two sliding joints. These joints are set to lock when an additional tunnel closure of about 300 mm has been achieved. The sets are installed immediately behind the tunnel face and they are embedded in shotcrete, except for a 1 m wide 'window' that is left for each of the sliding joints. Once the joints have moved and locked, usually between 5 and 10 m behind the face, the 'windows' are closed to complete the shotcrete lining. This support system has proved to be very effective and measurements of tunnel convergence, carried out over several years, have shown that the tunnel is completely stable.

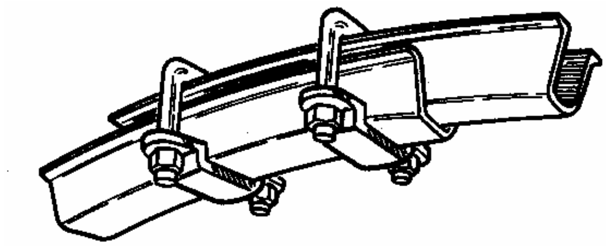


Figure 10: Assembly of a sliding joint in a Toussaint-Heintzmann or Top Hat section steel rib.

The installation of these sets immediately behind the face provides security for the men working at the face, in spite of the fact that the sets apply relatively little active support pressure to the rock mass during the sliding stage. The settings of the amount of sliding allowed in the joints is judged on the basis of the amount of tunnel convergence that can be tolerated before the full support reaction is activated (see Figure 4). Correct setting of the joints will achieve equilibrium between the tunnel convergence and the support reaction at much lower support pressures than for rigid steel sets. Consequently, provided that large tunnel convergence is acceptable, much lighter section support can be used than would otherwise be required.

Multiple tunnels

In one hydroelectric project in India, tunnelling through a large fault at depth proved to be extremely difficult and several collapses of the tunnel occurred as attempts were made to drive the tunnel at full size. Eventually a decision was made to split the tunnel into three smaller tunnels that would provide the same overall cross-section area for transmission of the water. Driving these three smaller tunnels, while still difficult was ultimately successful because, as shown in the case of the partial face excavation, dealing with smaller cross-sections has many practical advantages when tunnelling in difficult ground.

CONCLUSION

Tunnelling through very weak rock associated with faults and shear zones is a difficult problem, particularly when carried out under high in situ stress conditions. Conventional tunnel support such as rockbolts and shotcrete are seldom adequate to deal with heavy squeezing conditions that can occur when the rock mass surrounding the tunnel fails to a depth of several tunnel diameters.

Several alternative methods for maintaining stable face and tunnel stability have been explored in this paper. In some cases, limited theoretical analyses of the support systems are possible while, in other cases, reliance has to be placed on judgement and experience.

Tunnel driving costs, under the conditions described, are typically about three time average tunnel driving costs and advance rates seldom exceed about 1 m per day. Attempting to save money and time by adopting short-cuts or inadequate solutions invariably lead to even costlier failures.

Faults and shear zones exist in almost every rock mass and so it is inevitable that a tunnel engineer will be faced with the situations described in this paper at least once in his or her career. It is as well to attempt to learn from the experience of others rather than to attempt to sort out the problems when faced with a tunnel collapse. Reading about these problems is not a substitute for visiting a tunnel in which mining through a fault is in progress. Tunnel engineers should take every available opportunity to visit such tunnels.

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APPENDIX 1: ANALYSIS OF PLASTIC FAILURE

It is assumed that the onset of plastic failure, for different values of the effective confining stress σ_3', is defined by the Mohr-Coulomb criterion and expressed as:

$$\sigma_1' = \sigma_{cm} + k\sigma_3' \tag{A.1}$$

The uniaxial compressive strength of the rock mass σ_{cm} is defined by:

$$\sigma_{cm} = \frac{2c' \cos\phi'}{(1-\sin\phi')} \tag{A.2}$$

and the slope k of the σ_1' versus σ_3' line as:

$$k = \frac{(1+\sin\phi')}{(1-\sin\phi')} \tag{A.3}$$

where σ_1' is the axial stress at which failure occurs
σ_3' is the confining stress
c' is the cohesive strength and
ϕ' is the angle of friction of the rock mass

In order to estimate the cohesive strength c' and the friction angle ϕ' for an actual rock mass, the Hoek-Brown criterion (Hoek and Brown 1997) can be utilised. Having estimated the parameters for failure criterion, values for c' and ϕ' can be calculated.

Analysis of tunnel behaviour

Assume that a circular tunnel of radius r_o is subjected to hydrostatic stresses p_o and a uniform internal support pressure p_i as illustrated in Figure A.1. Failure of the rock mass surrounding the tunnel occurs when the internal pressure provided by the tunnel lining is less than a critical support pressure p_{cr}, which is defined by:

$$p_{cr} = \frac{2p_o - \sigma_{cm}}{1+k} \tag{A.4}$$

If the internal support pressure p_i is greater than the critical support pressure p_{cr}, no failure occurs, the behaviour of the rock mass surrounding the tunnel is elastic and the inward radial elastic displacement of the tunnel wall is given by:

$$u_{ie} = \frac{r_o(1+\nu)}{E_m}(p_o - p_i) \tag{A.5}$$

where E_m is the Young's modulus or deformation modulus and ν is the Poisson's ratio.

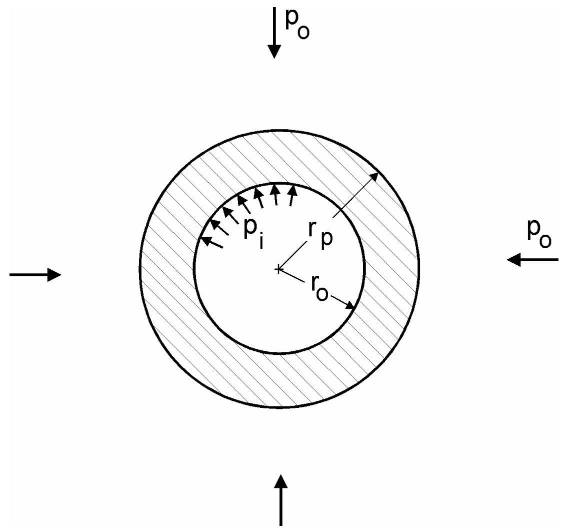


Figure A.1: Plastic zone surrounding a circular tunnel.

When the internal support pressure p_i is less than the critical support pressure p_{cr}, failure occurs and the radius r_p of the plastic zone around the tunnel is given by:

$$r_p = r_o\left[\frac{2(p_o(k-1)+\sigma_{cm})}{(1+k)((k-1)p_i+\sigma_{cm})}\right]^{\frac{1}{(k-1)}} \tag{A.6}$$

For plastic failure, the total inward radial displacement of the walls of the tunnel is:

$$u_{ip} = \frac{r_o(1+\nu)}{E}\left[2(1-\nu)(p_o - p_{cr})\left(\frac{r_p}{r_o}\right)^2 - (1-2\nu)(p_o - p_i)\right] \tag{A.7}$$

Appendix 1 (contd) - Characteristic line calculation for weak rock

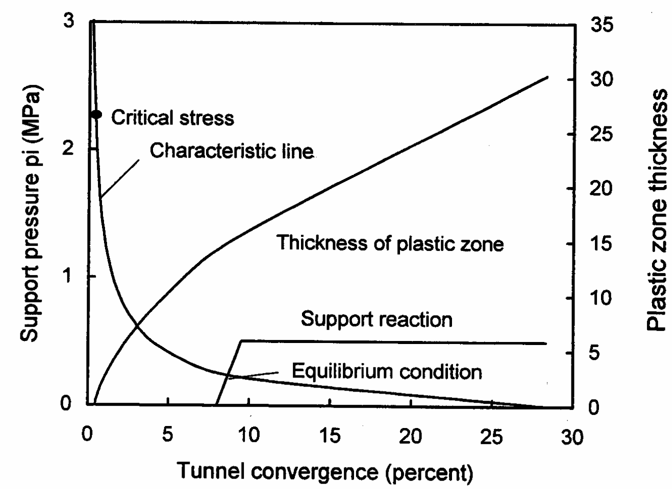
Input:	Intact strength sigci =	5	MPa	Intact rock constant mi =	8		Geological Strength Index GSI =	23	
	Poisson's ratio mu =	0.3		Tunnel diameter D =	10	m	In situ stress po =	3.8	MPa
	Max. support pressure =	0.82	MPa	Initial convergence =	5	percent	Average maximum strain ($s_{max.av}$)=	1.35	percent
Output:	Rock mass constant mb =	0.51		Rock mass constant s =	0.0000		Rock mass constant a =	0.535	
	Rock mass constant k =	2.21		Friction angle phi =	22.15	degrees	Rock mass cohesion coh =	0.10	MPa
	Rock mass sigcm =	0.28	MPa	Rock mass modulus E =	472.6	MPa	Critical support pressure pcr =	2.25	MPa

Plot:

													Support reaction calculation		
Tunnel convergence ui (m) =	2.830	0.903	0.451	0.271	0.180	0.128	0.095	0.074	0.059	0.049	0.041	0.000			
Tunnel convergence (%) =	28.30	9.03	4.51	2.71	1.80	1.28	0.95	0.74	0.59	0.49	0.41	0.00	8	9.5	28.30
Support pressure pi (MPa) =	0.00	0.22	0.45	0.67	0.90	1.12	1.35	1.57	1.80	2.02	2.25	3.75			
Plastic zone radius rp (m) =	35.2	20.2	14.5	11.5	9.6	8.2	7.3	6.5	5.9	5.4	5.0	5.0			
Plastic zone thickness (m) =	30.2	15.2	9.5	6.5	4.6	3.2	2.3	1.5	0.9	0.4	0.0	0.0			
Avabailable support (MPa) =													0.00	0.5	0.5

Calculation :

									Sums
sig3	1E-10	0.18	0.36	0.5	0.71	0.89	1.07	1.25	5.00
sig1	0.00	0.77	1.21	1.59	1.95	2.28	2.60	2.91	13.31
sig3sig1	0.00	0.14	0.43	0.85	1.39	2.04	2.79	3.64	11
sig3sq	0.00	0.03	0.13	0.29	0.51	0.80	1.15	1.56	4



Cell formulae:

mb = mi*EXP((GSI-100)/28)
s = IF(GSI>25,EXP((GSI-100)/9),0)
a = IF(GSI>25,0.5,0.65-GSI/200)
sig3 = Start at 1E-10 (to avoid zero errors) and increment in 7 steps of sigci/28 to 0.25*sigci
sig1 = sig3+sigci*(((mb*sig3)/sigci)+s)^a
k = (sumsig3sig1 - (sumsig3*sumsig1)/8)/(sumsig3sq-(sumsig3^2)/8)
phi = ASIN((k-1)/(k+1))*180/PI()
coh = (sigcm*(1-SIN(phi*PI()/180)))/(2*COS(phi*PI()/180))
sigcm = sumsig1/8 - k*sumsig3/8
E = IF(sigci>100,1000*10^((GSI-10)/40),SQRT(sigci/100)*1000*10^((GSI-10)/40))
pcr = (2*po-sigcm)/(k+1)
rp = IF(pi<pcr,0.5*D*(2*(po*(k-1)+sigcm)/((1+k)*((k-1)*pi+sigcm)))^(1/(k-1)),D/2)
ui = IF(rp>D/2,D*((1+mu)/E)*(2*(1-mu)*(po-pcr)*((rp/(0.5*D))^2)-(1-2*mu)*(po-pi)),D*(1+mu)*(po-pi)/E)

Support reaction calculation :

1. Pressure provided by support is zero at initial convergence which is the deformation at which the support is installed.
2. Maximum support pressure is achieved when the tunnel convergence equals the initial convergence plus the average maximum strain of the support system.
3. It is assumed that the support fails plastically and that the maximum support pressure remains constant after the support has reached its maximum capacity.