Predicting tunnel squeezing problems in weak heterogeneous rock masses

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Part 1: Estimating rock mass strength

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Introduction

In tunnelling through heterogeneous rock masses, such as the flysch, it is important to attempt to obtain reliable estimates of potential tunnelling problems as early as possible. This enables the tunnel designer to focus on the selection of optimum routes and to devote the appropriate resources to the investigation of those areas in which tunnelling problems are anticipated.

In the following text, a methodology is presented for estimating potential tunnel squeezing, such as that illustrated in Figure 1. This methodology does not provide the tunnel designer with a final design of the tunnel excavation sequence and support system to be used – these require additional analyses, which are not covered in this paper. However, the end product of the analysis presented gives a reliable first estimate of the severity of potential squeezing problems and an indication of the types of solutions that can be considered in overcoming these problems.

Part 1 of the paper deals with estimating the strength and deformation properties of weak heterogeneous rock masses while Part 2 deals with the prediction of tunnel squeezing problems.

Figure 1: Squeezing problems in a 10 m span tunnel top heading in a fault zone. Approximately 1 m of inward displacement is visible in the roof and sidewalls.

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The methodology presented here is based upon the use of the Hoek-Brown criterion for estimating the strength and deformation characteristics of rock masses (Hoek and Brown, 1997). However, the calculation of tunnel deformation is not dependent upon this criterion – any realistic criterion for estimating rock strength and deformation can be used, provided that the same process is used in deriving the final curves relating tunnel deformation to rock strength.

The geological model

Fookes (1997) gave an excellent description of the numerous steps required in the development of a Geological Model. This model, whether conceptual, hand-drawn or in the form of a computer generated three-dimensional solid model, is the basic building block upon which the design of any major construction project must be based. A good geological model will enable the geologists and engineers involved in the project to understand the interactions of the many components that make up the earth's crust and to make rational engineering decisions based on this understanding. On projects where an adequate geological model does not exist, decisions can only be made on an ad hoc basis and the risks of construction problems due to unforeseen geological conditions are very high.

In most developed countries, reliable regional geology maps exist and the geological libraries may well contain more detailed maps where investigations have been carried out for resource development or other purposes. Consequently, the starting point of any tunnel route assessment should be a thorough literature survey to determine what geological information is already available.

This should be followed by a walkover survey in which topographic forms, rock outcrops and any other significant geological features are noted and used in the construction of the first geological model. Such a model, although still crude, may be adequate for comparison of alternative routes and for avoiding obvious problem areas such as major landslides.

Once the route has been selected, the next step in the construction of an engineering geology model that will almost certainly involve a diamond drilling programme in which the rock mass is explored at the depths of the proposed tunnel. On the basis of a carefully planned drilling programme and the already constructed crude geological model, it should be possible to build an engineering geology model that is sufficiently detailed for final tunnel design.

Estimation of rock mass properties

A critical step in the methodology discussed in this paper is the selection of reliable rock mass properties that can be used, in conjunction with the depth of the tunnel, to estimate the response of the rock mass to the stresses induced by tunnel excavation. One of the most widely used criteria for estimating these rock mass properties is that proposed by Hoek and Brown (1997) and this criterion, with specific adaptations to weak heterogeneous rock masses, is briefly summarised in the following text.

Note that the Hoek and Brown criterion, and indeed any of the other published criteria that can be used for this purpose, assume that the rock mass behaves isotropically. In other words, while the behaviour of the rock mass is controlled by movement and rotation of rock elements separated by intersecting structural features such as bedding planes and joints, there are no preferred failure directions. The "isotropic" nature of heterogeneous rock masses such as flysch can be appreciated from the photograph reproduced in Figure 2.

Figure 2: Appearance of sheared siltstone flysch in an outcrop

These failure criteria should not be used when the rock mass consists of a strong blocky rock such as sandstone, separated by clay coated and slickensided bedding surfaces. The behaviour of such rock masses will be strongly anisotropic and will be controlled by the fact that the bedding planes are an order of magnitude weaker than any other features. In such rock masses the predominant failure mode will be gravitational falls of wedges or blocks of rock defined by the intersection of the weak bedding planes with other features which act as release surfaces. However, if the rock mass is heavily fractured, the continuity of the bedding surfaces will have been disrupted and the rock may behave as an isotropic mass.

In applying the Hoek and Brown criterion to "isotropic" rock masses, three parameters are required for estimating the strength and deformation properties. These are:

1. the uniaxial compressive strength σ_{ci} of the "intact" rock elements that make up the rock mass,
2. a constant m_i that defined the frictional characteristics of the component materials in these rock elements, and

3. the Geological Strength (GSI) that relates the properties of the intact rock elements to those of the overall rock mass.

Each of these parameters is dealt with in turn in the following sub-sections.

Uniaxial compressive strength σ_{ci} of intact rock

In dealing with heterogeneous rock masses it is extremely difficult to obtain a sample of "intact' core for testing in the laboratory. Practically every sample obtained from rock masses such as that illustrated in Figure 2 will contain discontinuities in the form of bedding and schistosity planes or joints. The samples will probably also contain several of the component rock types that make up this heterogeneous rock mass. Consequently, any laboratory tests carried out on core samples will be more representative of the rock mass than of the intact rock components. Using the results of such tests in the Hoek-Brown criterion will impose a double penalty on the strength (in addition to that imposed by GSI) and will give unrealistically low values for the rock mass strength.

In some special cases, where the rock mass is very closely jointed and where it has been possible to obtain undisturbed core samples, uniaxial compressive strength tests have been carried out directly on the "rock mass" (Jaeger, 1971). These tests require an extremely high level of skill on the part of the driller and the laboratory technician. The large-scale triaxial test facilities required for such testing are only available in a few laboratories in the world and it is generally not worth considering such tests for routine engineering projects.

One of the few courses of action that can be taken to resolve this dilemma is to use the Point Load Test on samples in which the load can be applied normal to bedding or schistosity in samples. The specimens used for such testing can be either irregular pieces or pieces broken from the core. The direction of loading should be as perpendicular to any weakness planes as possible and the fracture created by the test should not show any signs of having followed an existing discontinuity. It is strongly recommended that photographs of the specimens, both before and after testing, should accompany the laboratory report since these enable the user to judge the validity of the test results. The uniaxial compressive strength of the intact rock samples can be estimated, with a reasonable level of accuracy, by multiplying the point load index I_s by 24, where $I_s = P/D^2$. P is the load on the points and D is the distance between the points.

In the case of very weak and/or fissile rocks such as clayey shales or sheared siltstones, the indentation of the loading points may cause plastic deformation rather than fracture of the specimen. In such cases the Point Load Test does not give reliable results.

Where it is not possible to obtain samples for Point Load Testing, the only remaining alternative is to turn to a qualitative description of the rock material in order to estimate the uniaxial compressive strength of the intact rock. A table listing such a qualitative description is given in Table 1.

Table 1: Field estimates of uniaxial compressive strength of intact rock.

Grade*	Term	Uniaxial Comp. Strength (MPa)	Point Load Index (MPa)	Field estimate of strength	Examples
R6	Extremely Strong	> 250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100 - 250	4 - 10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, peridotite , rhyolite, tuff
R4	Strong	50 - 100	2 - 4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, sandstone, schist
R3	Medium strong	25 - 50	1 - 2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Concete, phyllite, schist, siltstone
R2	Weak	5 - 25	**	Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer	Chalk, claystone, potash, marl, siltstone, shale, rocksalt,
R1	Very weak	1 - 5	**	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife	Highly weathered or altered rock, shale
R0	Extremely weak	0.25 - 1	**	Indented by thumbnail	Stiff fault gouge

* Grade according to Brown (1981).
** Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results.

Constant m_i

The Hoek-Brown constant m_i can only be determined by triaxial testing on core samples or estimated from a qualitative description of the rock material as described by Hoek and Brown (1997). This parameter depends upon the frictional characteristics of the component minerals in the intact rock sample and it has a significant influence on the strength characteristics of rock.

When it is not possible to carry out triaxial tests, for the reasons discussed in the previous section, an estimate of m_i can be obtained from Table 2. Most of the values quoted have been derived from triaxial tests on intact core samples and the range of values shown is dependent upon the accuracy of the geological description of each rock type. For example, the term "granite" described a clearly defined rock type and all granites exhibit very similar mechanical characteristics. Hence the value of m_i is defined as 32 ± 3. On the other hand, the term "breccia" is not very precise in terms of mineral composition and hence the value of m_i is shown as 19 ± 5, denoting a higher level of uncertainty.

Influence of groundwater

The influence of groundwater on the behaviour of the rock mass surrounding a tunnel is very important and has to be taken into account in the estimation of potential tunnelling problems.

The most basic impact of groundwater is upon the mechanical properties of the intact rock components of the rock mass. This is particularly important when dealing with shales, siltstones and similar rocks that are susceptible to changes in moisture content. Many of these materials will disintegrate very quickly if they are allowed to dry out after removal from the core barrel. Hence, testing of the "intact" rock to determine the uniaxial compressive strength σ_{ci} and the constant m_i must be carried out under conditions that are as close to the in situ moisture conditions as possible. Ideally, a field laboratory should be set up very close to the drill rig and the core prepared and tested immediately after recovery.

In one example in which a siltstone was being investigated for the construction of a power tunnel for a hydroelectric project, cores were carefully sealed in aluminium foil and wax and then transported to a laboratory in which very high quality testing could be carried out. In spite of these precautions, the deterioration of the specimens was such that the test results were meaningless. Consequently, a second investigation program was carried out in which the specimens were transported to a small laboratory about 5 kilometres from the exploration site and the samples were tested within about an hour of having been removed from the core barrel. The results of this second series of tests gave very consistent results and values of uniaxial compressive strength σ_{ci} and constant m_i that were considered reliable.

When laboratory testing is not possible, point load tests should be carried out as soon after core recovery as possible in order to ensure that the moisture content of the sample is close to the in situ conditions.

The second impact of groundwater is that of water pressure and this manifests itself in a reduction in the strength of the rock mass due to the reduction in stress acting across discontinuities. This "effective stress" effect is taken into account in account in the analysis of stress induced progressive failure surrounding the tunnel. Many numerical programs incorporate the capability for effective stress analysis and one of these programs should be used for the final tunnel design.

In many cases, the effective stress effects are not significant during construction since the tunnel acts as a drain and the water pressures in the surrounding rock are reduced to negligible levels. However, if the groundwater conditions are re-established after completion of the final lining, the long-term effects of water pressure on rock mass strength should be investigated.

Table 2: Values of the constant m_i for intact rock, by rock group[3]. Note that values in parenthesis are estimates. The range of values quoted for each material depends upon the granularity and interlocking of the crystal structure – the higher values being associated with tightly interlocked and more frictional characteristics.

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerates (21 ± 3) Breccias (19 ± 5)	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
SEDIMENTARY	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
SEDIMENTARY	Non-Clastic	Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
SEDIMENTARY	Non-Clastic	Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
METAMORPHIC	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6	Gneiss 28 ± 5	
METAMORPHIC	Foliated*			Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
IGNEOUS	Plutonic	Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
IGNEOUS	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
IGNEOUS	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	
IGNEOUS	Volcanic	Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

* These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

[3] Note that this table contains several changes from previously published versions, These changes have been made to reflect data that has been accumulated from laboratory tests and the experience gained from discussions with geologists and engineering geologists.

A final effect of groundwater occurs when high water pressures or flows are encountered during construction. This gives rise to practical construction problems and facilities for dealing with these problems should be provided in the contract. The practical issues of water handling in wet tunnels are not dealt with in this paper.

Geological Strength Index GSI

The Geological Strength Index (GSI) was introduced by Hoek, Kaiser and Bawden (1995), Hoek and Brown (1997) and extended by Hoek, Marinos and Benissi (1998). This Index is based upon an assessment of the lithology, structure and condition of discontinuity surfaces in the rock mass and it is estimated from visual examination of the rock mass exposed in tunnel faces or surface excavations such as roadcuts and in borehole core.

The estimated GSI value of the rock mass is incorporated into calculations to determine the reduction in the strength of the rock mass compared with the strength of the intact rock components.

Table 3 can be used for estimating the GSI value for typical jointed rock masses while Table 4 was developed specifically for Flysch.

The term flysch is attributed to the geologist B. Studer and it comes from the German word “fliessen” meaning flow, probably denoting the frequent landslides in areas consisting of these formations.

Flysch consists of alternations of clastic sediments that are associated with orogenesis. It closes the cycle of sedimentation of a basin before the “arrival” of the poroxysme folding process. The clastic material derived from erosion of the previously formed neighbouring mountain ridge. Flysch is characterised by rhythmic alternations of sandstone and fine grained (pelitic) layers. The sandstone may also include conglomerate beds. The fine grained layers contain siltstones, silty shales and clayey shales. Rarely and close to its margins, limestone beds or ophiolitic masses may be found. The thickness of the sandstone beds range from centimetres to metres. The siltstones and schists form layers of the same order but bedding discontinuities may be more frequent, depending upon the fissility of the sediments.

The overall thickness of the flysch is often very large (hundreds to a few thousand metres). Different types of alternations occur in this thickness: e.g. persistence of sandstone or typical alternations or siltstone persistence. The overall thickness has often been reduced considerably by erosion or by thrusting. In fact, the formation is often affected by reverse faults and thrusts. This, together with consequent normal faulting, results in a degradation of the geotechnical quality of the flysch rock mass. Thus, sheared or even chaotic rock masses can be found at the scale of a typical engineering design.

Table 3: Geological strength index for blocky jointed rock masses.

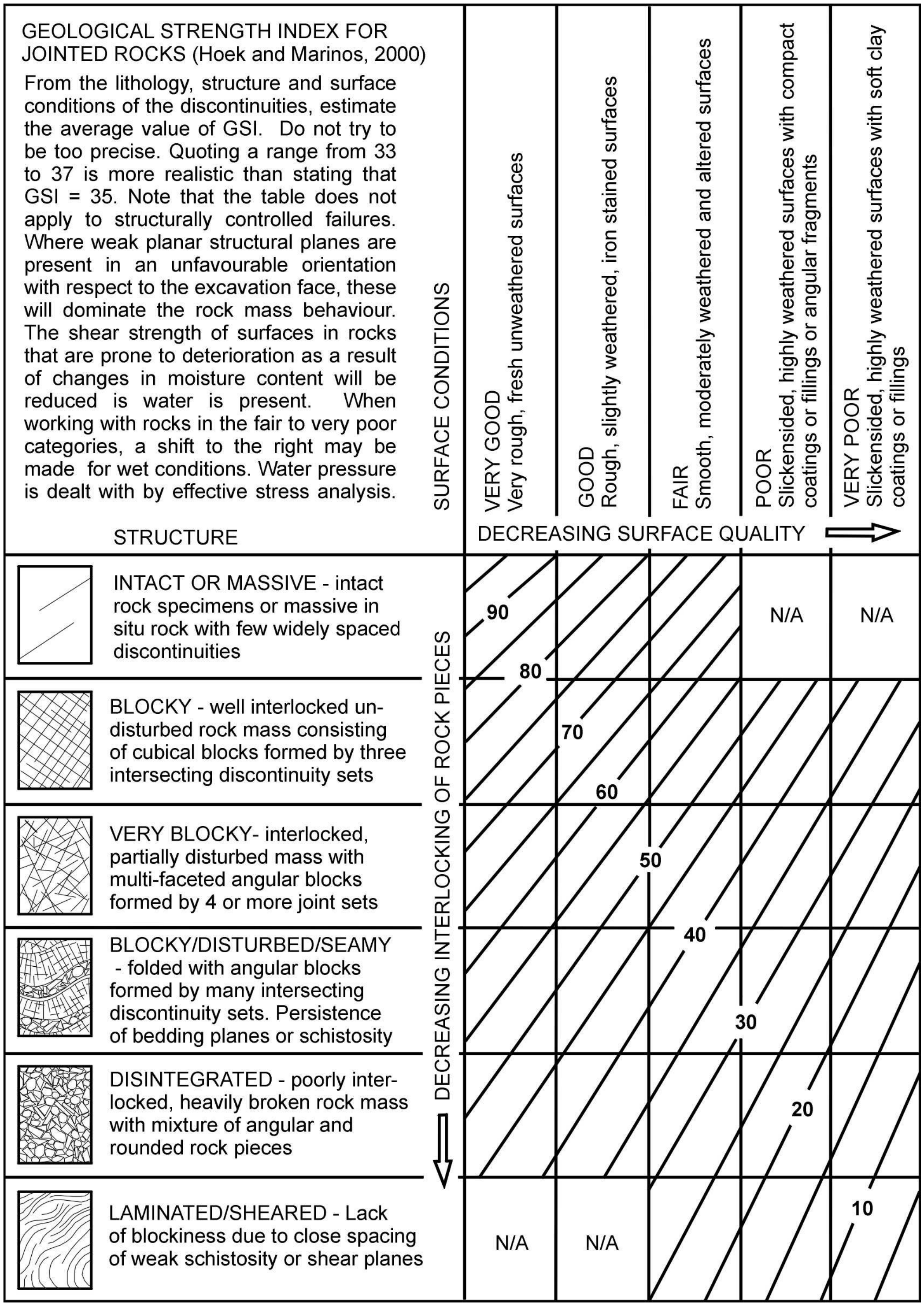
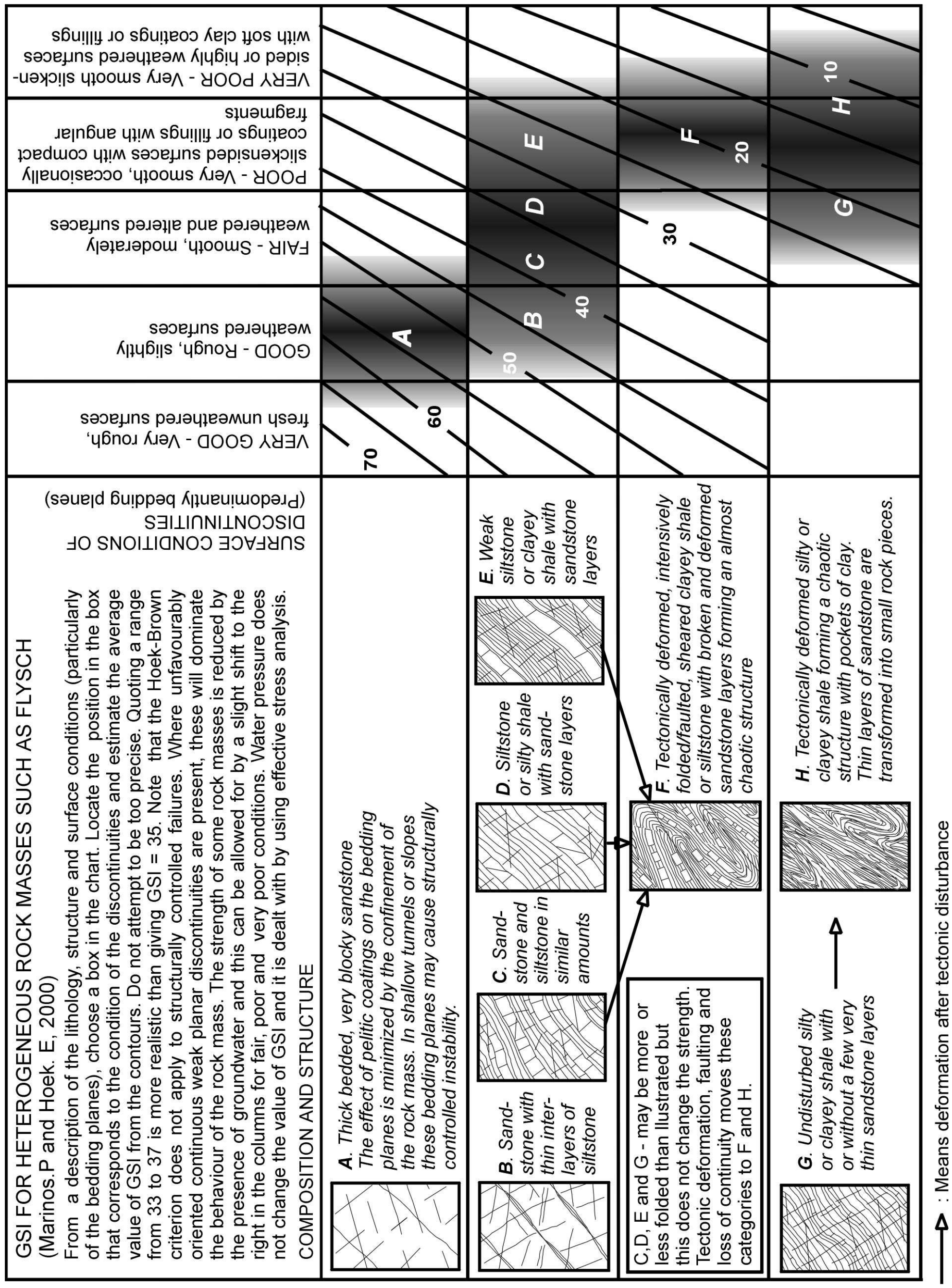


Table 4. GSI estimates for heterogeneous rock masses such as Flysch



Selection of σ_{ci} and m_i for Flysch

In addition to the GSI values presented in Table 4, it is necessary to consider the selection of the other "intact" rock properties σ_{ci} and m_i for heterogeneous rock masses such as Flysch. Because the sandstone layers or usually separated from each other by weaker layers of siltstone or shales, rock-to-rock contact between blocks of sandstone may be limited. Consequently, it is not appropriate to use the properties of the sandstone to determine the overall strength of the rock mass. On the other hand, using the "intact" properties of the siltstone or shale only is too conservative since the sandstone skeleton certainly contributes to the rock mass strength. Therefore, it is proposed that a 'weighted average' of the intact strength properties of the strong and weak layers should be used. Suggested values for the components of this weighted average are given in Table 5.

Table 5: Suggested proportions of parameters σ_{ci} and m_i for estimating rock mass properties for Flysch.

Flysch type see Table 4.	Proportions of values of σ_{ci} and m_i for each rock type to be included in rock mass property determination
A and B	Use values for sandstone beds
C	Reduce sandstone values by 20% and use full values for siltstone
D	Reduce sandstone values by 40% and use full values for siltstone
E	Reduce sandstone values by 40% and use full values for siltstone
F	Reduce sandstone values by 60% and use full values for siltstone
G	Use values for siltstone or shale
H	Use values for siltstone or shale

Estimating rock mass properties

Having defined the parameters σ_{ci}, m_i and GSI as described above, the next step is to estimate the mechanical properties of the rock mass. The procedure making these estimates has been described in detail by Hoek and Brown (1997) it will not be repeated here. However, a spreadsheet for calculating the mechanical properties is reproduced in Table 6. This spreadsheet can be used for shallow tunnels and slopes (less than 30 m depth) but, in the context of this paper, the values for "deep" tunnels will be used and a value of greater than 30 should be inserted for the depth below surface in the input data section.

The uniaxial compressive strength of the rock mass σ_{cm} is a particularly useful parameter for evaluating potential tunnel squeezing problems. It can be calculated directly from the spreadsheet (as was done in plotting the curves in Figure 3) or it can be estimated by means of the following equation which gives an estimate of σ_{cm} for selected values of the intact rock strength σ_{ci}, constant m_i and the Geological Strength Index *GSI* :

$$\sigma_{cm} = (0.0034 m_i^{0.8})\sigma_{ci}\{1.029 + 0.025 e^{(-0.1 m_i)}\}^{GSI} \qquad (1)$$

In order to estimate the deformation of a tunnel subjected to squeezing, an estimate of the deformation modulus of the rock mass is required. This can be obtained from equation 2, originally published by Serafim and Pereira (1983) and modified by Hoek and Brown (1997).

$$E(GPa) = \sqrt{\frac{\sigma_{ci}}{100}} \cdot 10^{((GSI-10)/40)} \qquad (2)$$

The values obtained from equation 2 are plotted in Figure 4.

Table 6: Spreadsheet for the calculation of rock mass properties

Input:

sigci =	10	MPa	mi =	10		GSI =	30	
Depth of failure surface or tunnel below slope =				25	m	Unit wt. =	0.027	MN/n3

Output:

stress =	0.68	MPa	mb =	0.82		s =	0.0004	
a =	0.5		sigtm =	-0.0051	MPa	A =	0.4516	
B =	0.7104		k =	3.95		phi =	36.58	degrees
coh =	0.136	MPa	sigcm =	0.54	MPa	E =	1000.0	MPa

Calculation:

									Sums
sig3	1E-10	0.10	0.19	0.29	0.39	0.48	0.58	0.68	2.70
sig1	0.20	1.01	1.47	1.84	2.18	2.48	2.77	3.04	14.99
ds1ds3	21.05	5.50	4.22	3.64	3.29	3.05	2.88	2.74	46.36
sign	0.01	0.24	0.44	0.62	0.80	0.98	1.14	1.31	5.54
tau	0.04	0.33	0.50	0.64	0.76	0.86	0.96	1.05	5.14
x	-2.84	-1.62	-1.35	-1.20	-1.09	-1.01	-0.94	-0.88	-10.94
y	-2.37	-1.48	-1.30	-1.19	-1.12	-1.06	-1.02	-0.98	-10.53
xy	6.74	2.40	1.76	1.43	1.22	1.07	0.96	0.86	16.45
xsq	8.08	2.61	1.83	1.44	1.19	1.02	0.88	0.78	17.84
sig3sig1	0.00	0.10	0.28	0.53	0.84	1.20	1.60	2.05	7
sig3sq	0.00	0.01	0.04	0.08	0.15	0.23	0.33	0.46	1
taucalc	0.04	0.32	0.49	0.63	0.76	0.87	0.97	1.07	
sig1sig3fit	0.54	0.92	1.30	1.68	2.06	2.45	2.83	3.21	
signtaufit	0.14	0.31	0.46	0.60	0.73	0.86	0.98	1.11	

Cell formulae:

```
 stress = if(depth>30, sigci*0.25,depth*unitwt)
     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)
  sigtm = 0.5*sigci*(mb-SQRT(mb^2+4*s))
   sig3 = Start at 1E-10 (to avoid zero errors) and increment in 7 steps of  stress/28 to stress/4
   sig1 = sig3+sigci*(((mb*sig3)/sigci)+s)^a
 ds1ds3 = IF(GSI>25,(1+(mb*sigci)/(2*(sig1-sig3))),1+(a*mb^a)*(sig3/sigci)^(a-1))
   sign = sig3+(sig1-sig3)/(1+ds1ds3)
    tau = (sign-sig3)*SQRT(ds1ds3)
      x = LOG((sign-sigtm)/sigci)
      y = LOG(tau/sigci)
     xy = x*y              x sq = x^2
      A =    acalc =    10^(sumy/8 - bcalc*sumx/8)
      B =    bcalc =    (sumxy - (sumx*sumy)/8)/(sumxsq - (sumx^2)/8)
      k = (sumsig3sig1 - (sumsig3*sumsig1)/8)/(sumsig3sq-(sumsig3^2)/8)
    phi = ASIN((k-1)/(k+1))*180/PI()
    coh = sigcm/(2*SQRT(k))
  sigcm = sumsig1/8 - k*sumsig3/8
      E = IF(sigci>100,1000*10^((GSI-10)/40),SQRT(sigci/100)*1000*10^((GSI-10)/40))
```

Notes:

For the "deep" tunnels discussed in this paper, enter a number greater than 30 for the depth below surface. This value is used to calculate the stress range over which the calculation is performed.

The equivalent Mohr Coulomb strength parameters are given by the values of "phi" (the friction angle ϕ) and "coh" (the cohesive strength c).

The two parameters of particular interest in this paper are the values of "sigcm" (the uniaxial compressive strength σ_{cm} of the rock mass) and "E" (the deformation modulus of the rock mass)

The program RocLab, which includes these calculations, can be obtained free from www.rocscience.com.

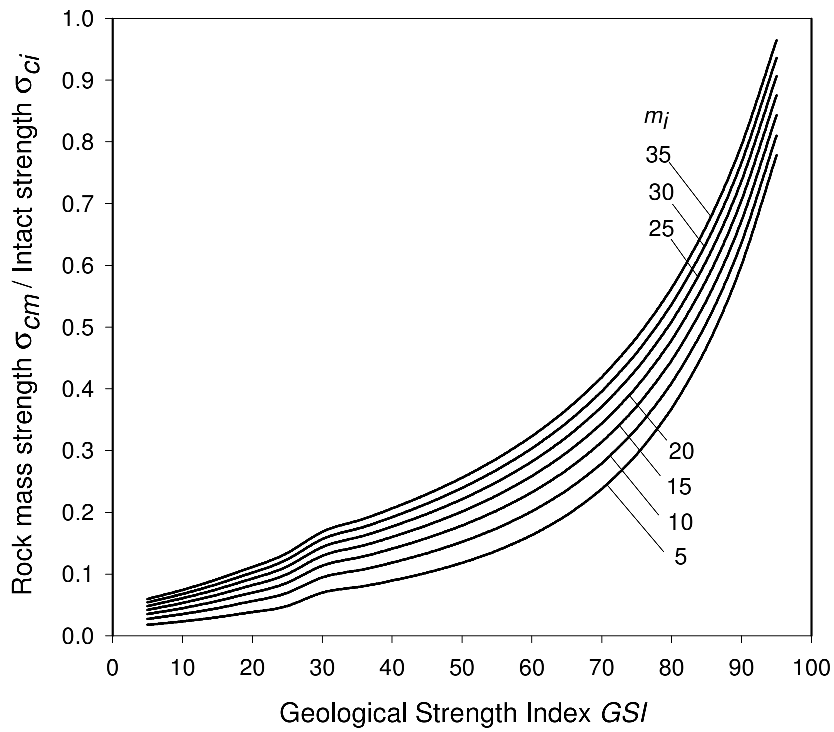


Figure 3: Plot of the ratio of rock mass strength / intact rock strength versus GSI for a range of m_i values.

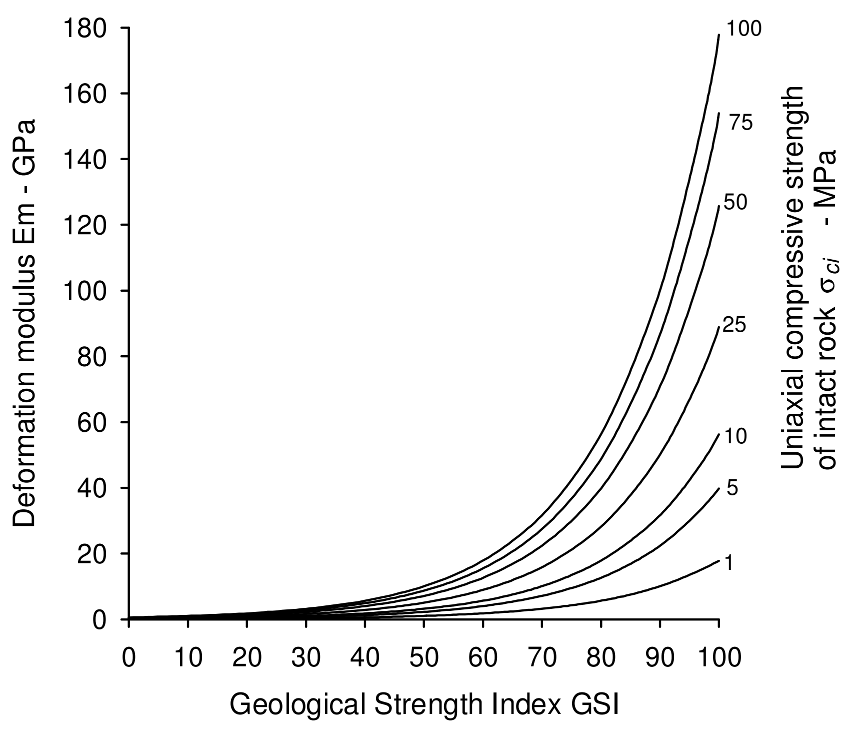


Figure 4: Plot of rock mass deformation modulus against GSI for a range of intact rock strength values.

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Potential squeezing problems in deep tunnels

Hoek (1999) published details of an analysis that showed that the ratio of the uniaxial compressive strength σ_{cm} of the rock mass to the in situ stress p_o can be used as an indicator of potential tunnel squeezing problems. Following the suggestions of Sakurai (1983), an analysis was carried out to determine the relationship between σ_{cm}/ p_o and the percentage "strain" of the tunnel. The percentage strain ε is defined as 100 x the ratio of tunnel closure to tunnel diameter.

Figure 3 gives the results of a study based on closed form analytical solutions for a circular tunnel in a hydrostatic stress field published by Duncan Fama (1993) and Carranza-Torres and Fairhurst (1999). Monte Carlo[4] simulations were carried out to determine the strain in tunnels for a wide range of conditions[5]. It can be seen that the behaviour of all of these tunnels follows a clearly defined pattern, which is well predicted by means of the equation included in the figure.

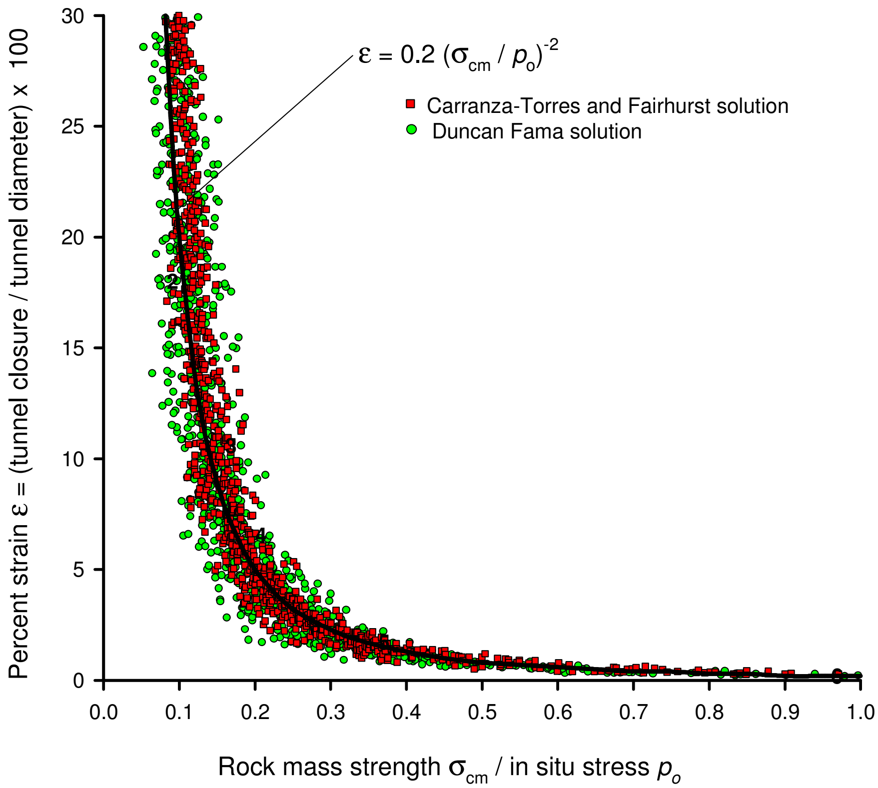


Figure 3: Plot of tunnel convergence against the ratio of rock mass strength to in situ stress. Note that this plot is for unsupported tunnels.

[4] These analyses were carried out by means of a commercially available add-in for a Microsoft Excel spreadsheet. This program, called @RISK, is available from the Palisade Corporation, 31 Decker Road, Newfield, New York 14867, Fax + 1 607 277 8001, http://www.palisade.com.

[5] For this study 2000 iterations were used with assumed uniform distributions for the following ranges of parameters : In situ stress 2 to 20 MPa (80 to 800 m depth), tunnel diameter 4 to 16 m, uniaxial strength of intact rock 1 to 30 MPa, Hoek-Brown constant m_i of 5 to 12, Geological Strength Index GSI of 10 to 35 and, for the Carranza-Torres solution, a dilation angle of 0 to 10.

The size of the plastic zone surrounding the tunnel follows a very similar trend to that illustrated in Figure 3. Chern et al (1998a, 1998b) found almost identical trends from a wide range of numerical analyses of different tunnel shapes and different stress fields. These studies were supported by a number of case histories of tunnels constructed in Taiwan.

The analysis presented above can be extended to cover tunnels in which an internal pressure is used to simulate the effects of support. Using a curve fitting process, the following equations were determined for the size of the plastic zone and the deformation of a tunnel in squeezing ground.

$$\frac{d_p}{d_o} = \left(1.25 - 0.625\frac{p_i}{p_o}\right)\frac{\sigma_{cm}}{p_o}^{\left(\frac{p_i}{p_o} - 0.57\right)} \tag{3}$$

$$\frac{\delta_i}{d_o} = \left(0.002 - 0.0025\frac{p_i}{p_o}\right)\frac{\sigma_{cm}}{p_o}^{\left(2.4\frac{p_i}{p_o} - 2\right)} \tag{4}$$

d_p = Plastic zone diamter
d_o = Original tunnel diameter in metres
δ_i = Tunnel sidewall deformation
p_i = Internal support pressure
p_o = In situ stress = depth × unit weight
σ_{cm}= Rock mass strength

Practical example

In order to demonstrate the application of the methods described in the previous text, an example of a 4.7 km long tunnel, with cover depths of up to 220 m, is presented. This tunnel passes through a series of typical flysch rocks and a portion of the engineering geology model created for this tunnel is illustrated in Figure 4a. The depth below surface of the tunnel is shown in Figure 4b.

A first crude check on potential tunnelling problems has been carried out by taking the lowest estimates of the Geological Strength Index GSI, the uniaxial compressive strength of the intact rock σ_{ci} and the constant m_i and calculating the uniaxial compressive strength of the rock mass σ_{cm} by means of equation 1. This value is then substituted into equation 3, with the support pressure p_i = 0, to obtain an estimate of the strain of the tunnel. An example of this calculation sequence is illustrated below.

$$\sigma_{cm} = (0.0034 m_i^{0.8})\sigma_{ci}\{1.029 + 0.025e^{(-0.1m_i)}\}^{GSI}$$

Chainage m	Tunnel depth m	Vertical stress p_o MPa	Intact strength σ_{ci} MPa	Constant m_i	Geological Strength Index GSI	Rock mass strength σ_{cm} MPa	Strength/stress σ_{cm}/p_o	Percentage strain ε
6865	100	2.6	25	10	20	1.01	0.39	1.33

$$\varepsilon = 100 \times \left(0.002 - 0.0025\frac{p_i}{p_o}\right)\frac{\sigma_{cm}}{p_o}^{\left(2.4\frac{p_i}{p_o} - 2\right)} \text{ for } p_i = 0$$

The results of these calculations are plotted in Figure 4c. This shows that the strains are less than 2% for most of the length of the tunnel and this suggests that most of the tunnel can be driven with relatively simple rockbolt and shotcrete support. However, there are stretches of the tunnel where there is a potential for large deformations. In particular, between chainages 8400 and 9000, there is a potential for strains of up to about 40%. Obviously, this stretch of the tunnel requires further analysis.

To carry out this more detailed analysis it is necessary to consider not only the lowest values but also the ranges of the Geological Strength Index GSI, the uniaxial compressive strength of the intact rock σ_{ci} and the constant m_i. It may even be necessary to go back to the raw field and laboratory data and to examine whether the values chosen are realistic. Where very severe tunnelling problems are indicated, one or more additional boreholes may be required in order to obtain fresh core samples for detailed evaluation in accordance with the guidelines presented earlier in this document.

In the case of this particular tunnel, the rock between chainages 8400 and 9000 is predominantly siltstone flysch and its low strength, combined with the relatively high cover, results in the tunnel squeezing problems predicted by the analysis discussed above. A careful review of all available data, including laboratory test data on borehole core, borehole photographs and surface outcrops of the siltstone flysch permitted the following estimates of rockmass parameters.

Parameter	Mean	Std dev	Min	Max
Intact strength of siltstone flysch σ_{ci}	20	12	5	40
Constant m_i	8	2	5	11
Geological Strength Index GSI	20	5	13	28

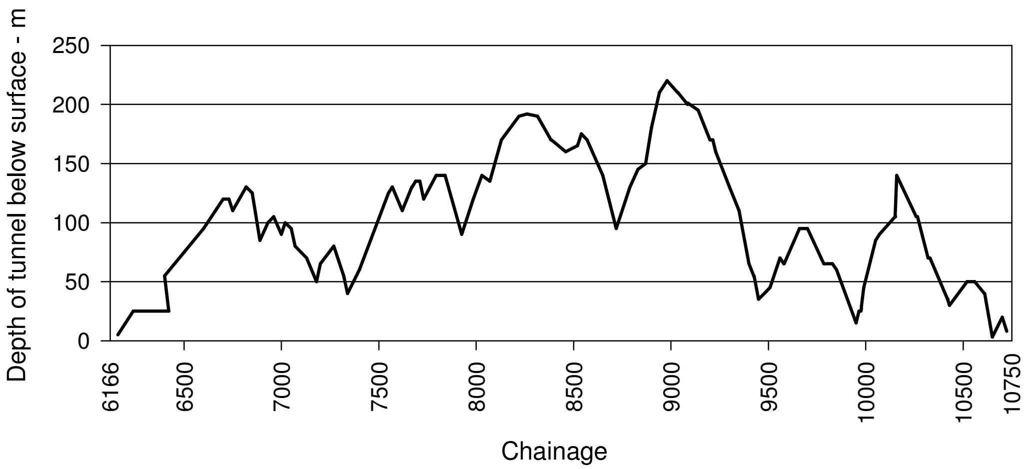
Note that the intact strength σ_{ci} has been assigned a large standard deviation of 12. This is to reflect the low level of reliability that can be placed on the laboratory test data. The maximum and minimum values assigned to each parameter are based upon a consideration of what would be typical for rock masses of this type.

An estimate of the range of in situ stresses acting on the tunnel between chainages 8400 and 9000 can be made from the depths shown in Figure 5b. An average depth below surface of 160 m is assumed, with a standard deviation of 60 m, a minimum of 100 m and a maximum of 220 m. Using an average unit weight of 0.027 MN/m^3, the average in situ stress is 4.3 MPa with a standard deviation of 1.6 MPa, a minimum of 2.7 MPa and a maximum of 5.94 MPa.

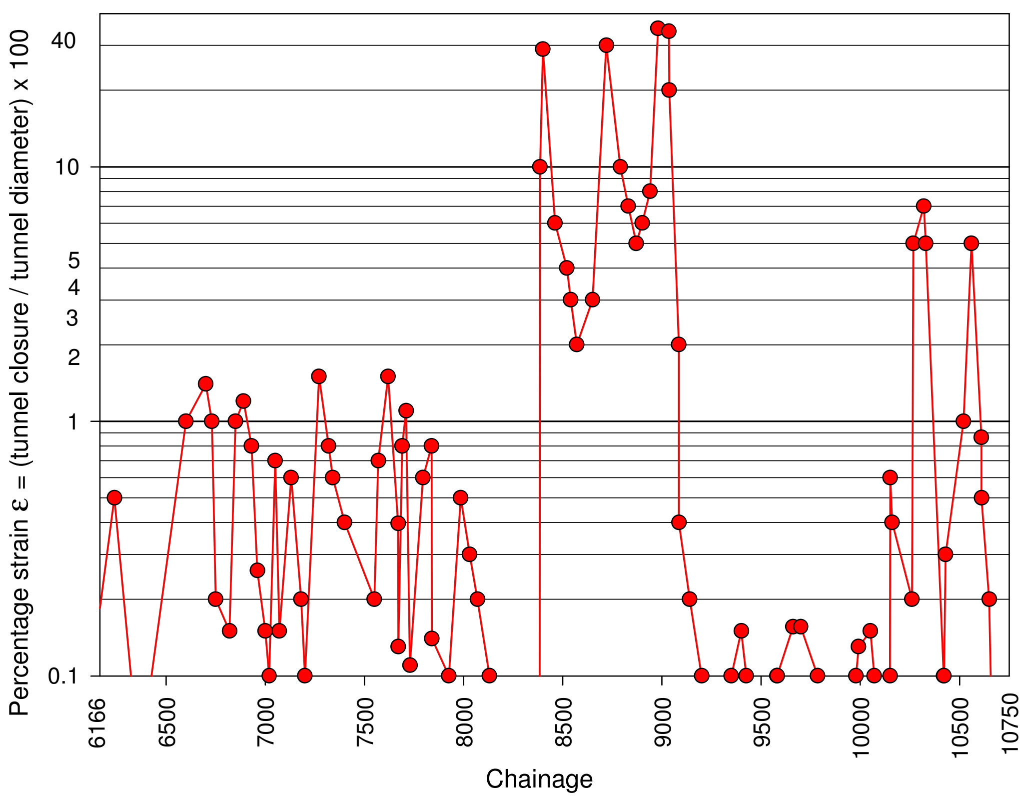
These parameters were substituted into an Excel spreadsheet and the add-in program @RISK was used to generate truncated normal distributions for the intact rock strength σ_{ci}, the constant m_i, the Geological Strength Index GSI and the in situ stress p_o. A Monte Carlo analysis was then carried out, using 5000 iterations, to generate the probability distribution of percentage strain. The results of this analysis are plotted in Figure 5.

Engineering geology model

a: Engineering geology model of tunnel



b: Depth of tunnel versus chainage along tunnel.



c: Maximum percentage strain – assuming lowest rock mass properties

Figure 4: Example of a preliminary analysis of a tunnel through Flysch.

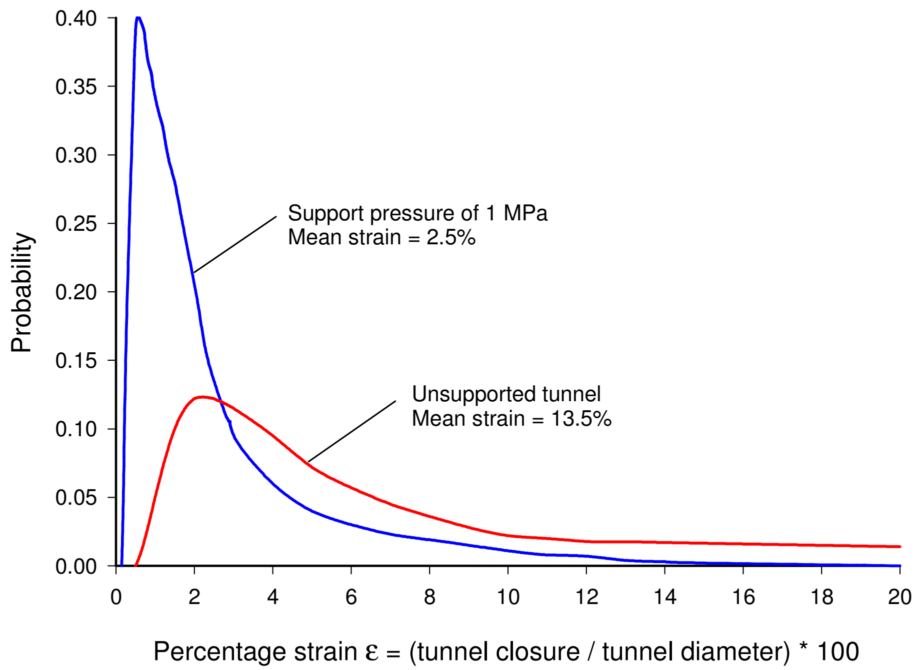


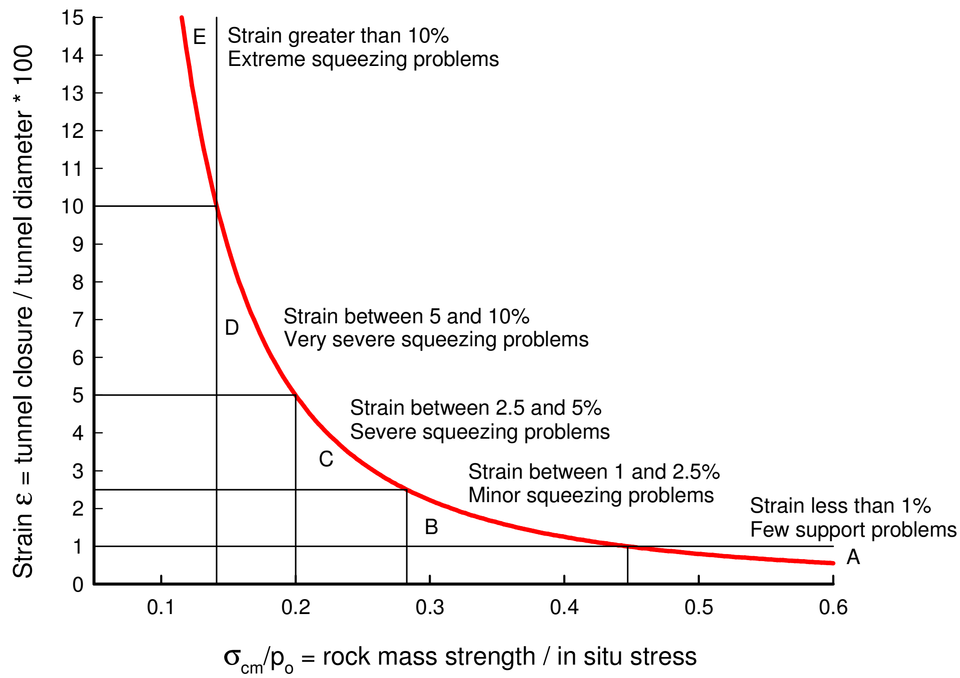
Figure 5: Probability distribution of percentage strain for an unsupported tunnel and a tunnel with an internal support pressure of 1 MPa in siltstone flysch between chainages 8400 and 9000.

The probability distribution curve for the unsupported tunnel shows that, for the between chainages 8400 and 9000, the mean strain is 13.5% and there is more than a 30% probability that the strain could exceed 10%. This confirms the analysis presented in Figure 4 showing that there is indeed some cause for concern about the stability of this section of tunnel.

In order to check on the adequacy of conventional support systems to stabilise this tunnel, the Monte Carlo analysis was re-run with a uniform support pressure of $p_i = 1$ MPa. This is the typical of the support pressures that can be generated in a 12 m span tunnel with rockbolts, lattice girders or steel sets used in combination with shotcrete lining (Hoek, 1999). The resulting probability distribution, plotted in Figure 5, shows that this level of support pressure reduces the mean strain to 2.5% and there is a 90% probability that the strain will be less than 5%. This suggests that stabilisation of the tunnel by means of relatively conventional support measures will be entirely feasible.

Note that, for strain levels in excess of about 5%, face stability problems can dominate the behaviour of the tunnel and it may be necessary to pre-support the face by forepoles and/or grouted fibreglass dowels or to reduce the cross-sectional area of the face by using multiple drift excavation methods (e.g. Lunardi, 2000).

This analysis, although very crude, gives a good first estimate of potential tunnelling problems due to squeezing conditions in weak rock at significant depth below surface. Where the engineering geology model is considered to be reliable, the type of analysis presented above can be used to divide the tunnel into sections according to the categories suggested in Figure 7.



	Strain ε %	Geotechnical issues	Support types
A	Less than 1	Few stability problems and very simple tunnel support design methods can be used. Tunnel support recommendations based upon rock mass classifications provide an adequate basis for design.	Very simple tunnelling conditions, with rockbolts and shotcrete typically used for support.
B	1 to 2.5	Convergence confinement methods are used to predict the formation of a 'plastic' zone in the rock mass surrounding a tunnel and of the interaction between the progressive development of this zone and different types of support.	Minor squeezing problems which are generally dealt with by rockbolts and shotcrete; sometimes with light steel sets or lattice girders are added for additional security.
C	2.5 to 5	Two-dimensional finite element analysis, incorporating support elements and excavation sequence, are normally used for this type of problem. Face stability is generally not a major problem.	Severe squeezing problems requiring rapid installation of support and careful control of construction quality. Heavy steel sets embedded in shotcrete are generally required.
D	5 to 10	The design of the tunnel is dominated by face stability issues and, while two-dimensional finite analyses are generally carried out, some estimates of the effects of forepoling and face reinforcement are required.	Very severe squeezing and face stability problems. Forepoling and face reinforcement with steel sets embedded in shotcrete are usually necessary.
E	More than 10	Severe face instability as well as squeezing of the tunnel make this an extremely difficult three-dimensional problem for which no effective design methods are currently available. Most solutions are based on experience.	Extreme squeezing problems. Forepoling and face reinforcement are usually applied and yielding support may be required in extreme cases.

Figure 7: Approximate relationship between strain and the degree of difficulty associated with tunnelling through squeezing rock. Note that this curve is for tunnels with no support.

The next steps

While the methods described above can give a useful indication of potential squeezing and support requirements for tunnels in weak ground, they cannot be considered adequate for final design purposes. The reader is reminded that the analysis is based upon a simple closed-form solution for a circular tunnel in a hydrostatic stress field and the support is assumed to act uniformly on the entire perimeter of the tunnel. These conditions are seldom met in the field since most large tunnels are excavated by top heading and benching and the tunnel shape and in situ stress conditions are seldom as simple as those assumed. While Chern et al (1998b) have shown that these predictions are acceptably accurate for application to actual tunnels, there remains a need to use a more sophisticated method of analysis for final design.

It is strongly recommended that, where significant potential squeezing problems have been identified, the tunnel should be subjected to numerical analyses. Several excellent two- and three-dimensional finite element and finite difference programs, written specifically for tunnel design, are now available commercially. These programs allow the user to model the sequential excavation and support systems for any tunnel shape, in situ stress field and rock mass conditions. It is suggested that each of the support categories proposed for the tunnelling operation should be subjected to detailed analysis by means of one of these programs.

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