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SELECTION OF ROCK MASS PROPERTIES FOR TUNNEL DESIGN

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ABSTRACT

In order to design tunnel in rock, some estimates of in situ stresses and rock mass properties are required. These are very difficult parameters to obtain and there are no simple laboratory tests or field measurements that can provide this information. In many cases, the best that can be hopes for is a set of numbers based on intelligent guesses.

This paper explores the processes used to make some of these guesses and describes how the results are then applied to engineering design. It is shown that, with care, rational engineering decisions can be made in spite of the limitations of the input data. In recent years the development of computer hardware and software has made it much easier to investigate the influence of ranges of values for each of the input parameters. However, care has to be taken that the design is driven by sound geological reasoning and rigorous engineering logic rather than by the very attractive images that appear on the computer screen.

1. INTRODUCTION

Many geologists are uncomfortable with the requirement to assign numbers to geology and many will con-tend that geological materials, not being man-made like steel or concrete, cannot be quantified. While I have some sympathy with these views, I have to face the reality that engineering design requires numbers in the form of in situ stress, pore water pressure, rock mass strength and deformation modulus. These numbers are required for the calculation of the stability of slopes, the bearing capacity of foundations, the support capacity for tunnels and the movement of groundwater contaminants. Without these numbers the process of engineering design is not possible.

Of course rock and soil are not man-made and their properties can vary greatly over short distances. The interactions of different components in a rock mass can be very complex and these interactions are difficult to quantify. These variations must be recognized and incorporated into the numbers themselves and the use to which the numbers are put in the engineering design process. Quoting a rock mass classification value to three decimal places betrays a complete lack of understanding of the process of quantifying rock mass properties. On the other hand, assigning excessively large ranges to each parameter can result in equally meaningless results.

A good engineering geologist and a good geotechnical engineer, working as a team, can usually make realistic educated guesses for each of the parameters required for a particular engineering analysis. It is the selection of reasonable values for the parameters and the choice of appropriate engineering design methods that I wish to explore in this paper.

[1] Adapted from 'Putting Numbers to Geology – an Engineer's Viewpoint', The Second Glossop Lecture,
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2. THE GEOTECHNICAL ENGINEERING DESIGN PROCESS

The end product of the work carried out by a geotechnical engineer is generally the complete design of a slope, a foundation or a tunnel. An example of a typical flow path for a geotechnical engineering design, adapted from Hoek and Brown (1980), is illustrated in Figure 1. In this case, the design is for an underground excavation but a similar diagram can be constructed for any other structure for which the geotechnical engineer is responsible. From this figure it will be obvious that the design process progresses from a largely qualitative preliminary assessment of potential problems to a highly quantitative analysis of support capacity and excavation performance for the situations that require such an analysis.

Note that the engineering design process need only be taken as far as necessary to satisfy the designer that the requirements of safety and stability have been met. It may be possible, on the basis of a very simple semi-quantitative analysis, to conclude that there are no conditions likely to lead to instability and to terminate the design at this point. On the other hand, in cases where the structural conditions are very unfavourable or where the rock mass strength is very low compared to the in situ stresses, a very detailed numerical analysis may be required.

In complex cases it may be necessary to run the numerical analysis concurrently with construction and adjust the excavation sequence and support systems to satisfy the design requirements established by back-analysis of the observed excavation behavior.

The geological model is a dynamic tool that changes as more information is exposed during the excavation process. It is only for very simple geological environments that the geological model can be established early in the site investigation and design process and left unaltered for the remainder of the project. The more usual condition is that the model is continually refined as the project progresses through the various stages of design and construction.

3. PRELIMINARY PROJECTS FEASIBILITY ASSESSMENT

During the very early stages of project evaluation and design, when practically no quantitative information is available and when the geological model is fairly crude, the design process relies heavily on precedent experience and very general rules of thumb. For example, in evaluating three alternative highway routes through mountainous terrain, the engineering geologist or geotechnical engineer would look for routes with the mini-mum number of unstable landforms, ancient landslides, difficult river crossings and the minimum number of tunnels. Simple common sense says that these factors represent problems and the potential for increased cost.

This may sound a trivial example but it is amazing how often a highway will be laid out by transportation engineers with more concern for lines of sight and radii of curves than for the geological conditions which happen to occur along the route. It is then up to the engineering geologists and geotechnical engineers to sort out the problems and, where necessary, to propose an alignment that is more appropriate for the geological conditions.

Precedent experience is also an important consideration at this stage of the design process. When evaluating the potential problems along a proposed tunnel route it is very useful to visit and to talk to engineers and contractors who have worked on tunnels in similar geological conditions within a few tens of kilometers of the site, if such tunnels exist.

During the preliminary design stage, the engineer is probably less important than the geologist. The engineer is there to convey the general requirements and constraints of the project and it is up to the geologist to construct a geological model and to provide the qualitative assessment of whether these conditions can easily be met or whether it would be better to look for another site.

4. PRELIMINARY ENGINEERING EVALUATION

Once the qualitative process described above has been exhausted and the options have been narrowed down to one or two, it may become necessary to move into a more quantitative process in which the engineer starts to assume the leading role in the design process. It is at this stage in the design process and, in my opinion, only at this stage, that classification schemes play an important role.

These classifications, based upon experience and the back analysis of a large number of case histories, attempt to quantify the general rock mass conditions in terms of relatively simple numerical ratings. The final 'score' is then used to provide guidance on tunnel support, slope stability, the problems of excavating rock masses. Note that almost all of these classifications were developed to provide guidance on tunnel support and that their adaptation to slopes and foundations was not done by the original authors. The rock mass classification systems commonly used in the English language world have been summarized by Bieniawski (1989) and it is not my intention to discuss the details of these classifications here.

While classification systems can play a valuable role in the identification of potential problems and in suggesting typical solutions that have been used in similar categories of problems, there is a widespread tendency to use these classification schemes for more detailed engineering design. In some circumstances this can result in the generation of misleading and inappropriate design recommendations. For example, in tunneling through very weak rock masses in which squeezing can occur, the sequence of support installation is just as important that the details of the support systems used. None of the rock mass classification systems deal with excavation and support installation sequencing in anything like the detail required to design an appropriate tunneling method for these conditions. Consequently, when rock mass classifications are used in engineering projects, they should be used by someone with a full understanding of their origins and limitations. As a general rule they should always be used in conjunction with other design processes and never as the sole basis for the design of a rock engineering structure.

Returning to the question of the preliminary evaluation of a construction project, the aim should be to divide the problems into a series of approximate categories, depending upon the severity of each problem. Whatever numerical process is used, these categories should be treated as approximate guidelines rather than absolute design values. The whole purpose of the preliminary evaluation is to decide which components justify additional site investigations and analysis. The detailed design follows later.

5. DETAILED ENGINEERING DESIGN

Having identified those components of a construction project that require detailed analysis, the next step is to select the appropriate method of analysis and the input data required for this analysis. There are too many geotechnical problems and methods of analysis for me to cover in this paper so I will deal with only one - the design of tunnels in weak rocks.

In the context of this discussion I will define rock as weak when the in situ uniaxial compressive strength is less than the in situ stress level. Hence, a jointed rock mass with a uniaxial compressive strength of 3 Mpa will behave as a weak rock at depths of more than about 120 m. Under these conditions a tunnel would begin to show the first signs of stress induced failure.

In order to carry out a meaningful analysis of the stresses induced by the excavation of a tunnel or cavern it is necessary to estimate the in situ stresses in the rock mass and also the properties of the rock mass.

5.1 ESTIMATES OF IN SITU STRESS

Of all of the quantities that the geotechnical engineer is required to estimate or to measure, the in situ stress field in a rock mass is one of the most difficult. The vertical stress can be approximated, to an acceptable level of accuracy, by the product of the depth below surface and the unit weight of the rock mass. On the other hand, the horizontal stresses of interest to civil engineers are influenced by global factors such as plate tectonics and also by local topographic features.

Zoback (1992) described the World Stress Map project that was designed to create a global database of contemporary tectonic stress data. The data included in this map were derived mainly from geological observations on earthquake focal mechanisms, volcanic alignments and fault slip interpretations.

The results included in this map are very interesting to geologists but are of limited value to engineers concerned with the upper few hundred meters of the earth's crust. The local variations in the in situ stress field are simply too small to show up on the global scale.

A more useful basis for estimating horizontal in situ stresses was proposed by Sheorey (1994). He developed an elasto-static thermal stress model of the earth. This model considers curvature of the crust and variations of elastic constants, density and thermal expansion coefficients through the crust and mantle. A plot of the ratio of horizontal to vertical stress predicted by Sheorey's analysis, for a range of horizontal rock mass deformation moduli, is given in Figure 2. This plot is very similar in appearance to that derived by Hoek and Brown (1980) on the basis of measured in situ stresses around the world. While this similarity does not constitute a proof of the correctness of Sheorey's solution, it is at least comforting to find this correlation between theory and observations.

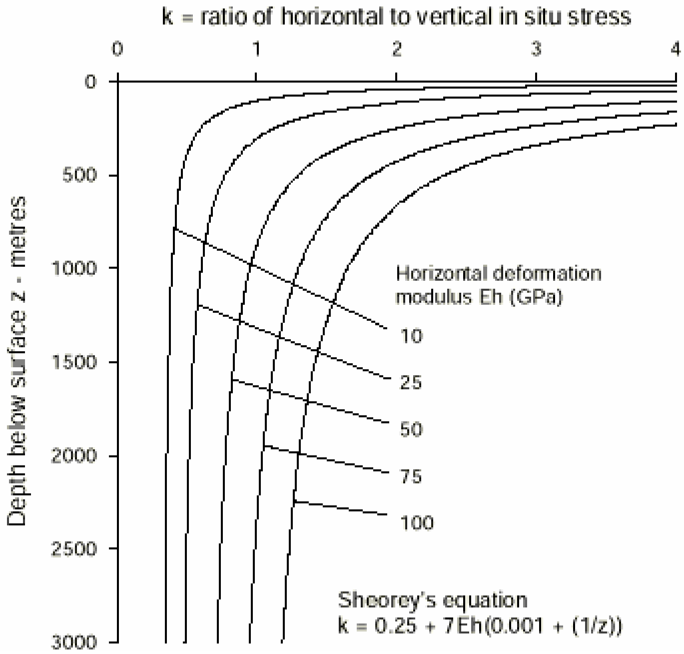


Figure 2: Ratio of horizontal to vertical in situ stress versus depth below surface. (Sheorey 1994)

Neither Sheorey's equation nor the trends established by Hoek and Brown account for local topographic influences on the in situ stress field. Hence, when making estimates of the in situ stress field in a mountainous area, adjustments must be made to account for these topographic factors. For example, the general relation-ships discussed above may indicate a horizontal stress of approximately twice the vertical stress for the rock mass at a depth of 300 m. In deciding upon the in situ stresses to be applied to the analysis of a tunnel to be located at this depth in the side of a steep valley, the horizontal stress at right angles to the valley axis could be reduced to a value equal to the vertical stress. This would account for the stress relief due to the down-cutting of the valley. No such stress relief would occur parallel to the valley axis and so the horizontal stress in this direction would be kept at twice the vertical stress.

In carrying out an analysis of the stresses induced by the creation of an underground excavation, it is prudent to consider a range of possible in situ stresses. In the example discussed above, the horizontal stress at right angles to the valley axis could be varied from one half the vertical stress to twice the vertical stress. The stress parallel to the valley could be varied from a minimum value equal to the vertical stress to a maximum value of three times the vertical stress. An exploration of the effects of all possible combinations of these stress values would give a good indication of whether or not these in situ stresses would be critical to the design of the underground excavations. In cases where a preliminary analysis indicates that the design is very sensitive to the in situ stresses, measurement of the in situ stresses has to be considered a priority in the on-going site investigation and design process.

5.2 ESTIMATES OF ROCK MASS PROPERTIES

Hoek and Brown (1980) proposed a methodology for estimating the strength of jointed rock masses. This technique has been refined and expanded over the years and the latest version is described in a recent paper and technical note. (Hoek and Brown 1997, Hoek 1998).

The basic input consists of estimates or measurements of the uniaxial compressive strength (σ_{ci}) and a material constant (*mi*) that is related to the frictional properties of the rock. Ideally, these basic properties should be determined by laboratory testing as described by Hoek and Brown (1997) but, in may cases, the information is required before laboratory tests have been completed.

To meet this need, tables that can be used to estimate values for these parameters are reproduced in Tables 1 and 2.

The most important component of the Hoek-Brown system is the process of reducing the material constants σ_{ci} and *mi* from their 'laboratory' values to appropriate in situ values. This is accomplished through the Geological Strength Index GSI that is defined in Figure 3.

In the context of this paper, the GSI is a real case of putting numbers to geology. It has been developed over many years of discussions with engineering geologists with whom I have worked around the world. Careful consideration has been given to the precise wording in each box and to the relative weights assigned to each combination of structural and surface conditions.

Table 1: Field estimates of uniaxial compressive strength.

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, limestone, marble, rhyolite, tuff
R4	Strong	50 - 100	2 - 4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, phyllite, sandstone, schist, shale
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	Claystone, coal, concrete, schist, shale, 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, rocksalt, potash
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
R0	Extremely Weak	0.25 - 1	**	Indented by thumbnail	Stiff fault gouge

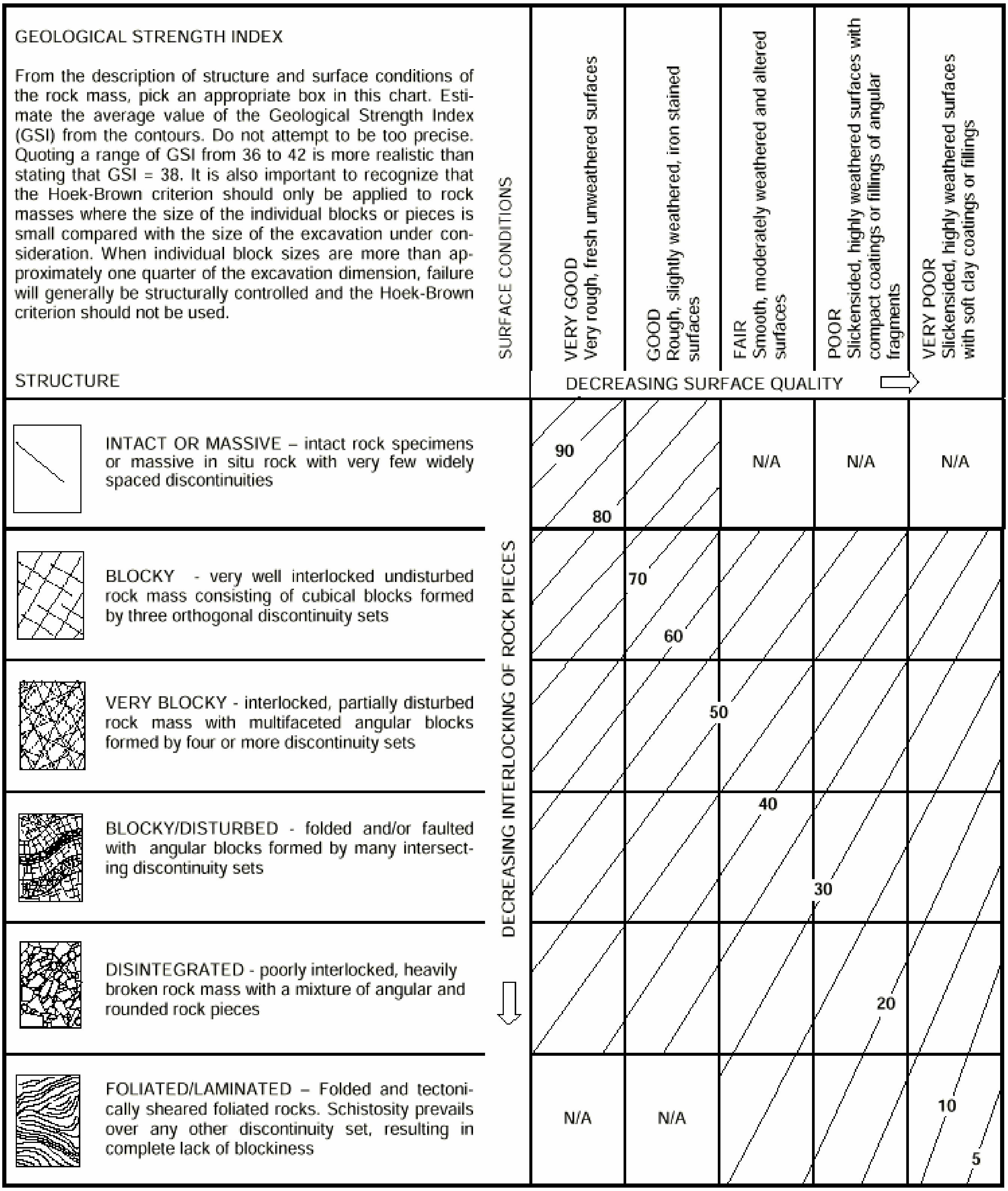
*Grade according to Brown (1981).
**Point load tests will give highly ambiguous results on rocks with a uniaxial compressive strength of less than 25 MPa.

Table 2: Values of the constant mi for intact rock, by rock group. Note that values in parenthesis are estimates.

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerate (22)	Sandstone 19	Siltstone 9	Claystone 4
				Greywacke (18)		
	Non-Clastic	Organic		Chalk 7		
				Coal (8-21)		
		Carbonate	Breccia (20)	Sparitic Limestone (10)	Micritic Limestone 8	
		Chemical		Gypstone 16	Anhydrite 13	
METAMORPHIC	Non Foliated		Marble 9	Hornfels (19)	Quartzite 24	
	Slightly foliated		Migmatite (30)	Amphibolite 25 - 31	Mylonites (6)	
	Foliated*		Gneiss 33	Schists 4 - 8	Phyllites (10)	Slate 9
IGNEOUS	Light		Granite 33		Rhyolite (16)	Obsidian (19)
			Granodiorite (30)		Dacite (17)	
			Diorite (28)		Andesite 19	
	Dark		Gabbro 27	Dolerite (19)	Basalt (17)	
			Norite 22			
	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

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

Figure 3: Table for estimating the Geological Strength Index GSI of a rock mass (Hoek, Marinos and Benissi, 1998)



Based on intuition, experience and the back analysis of a number of case histories, relationships have been developed between GSI, σ_{ci} and mi and the various rock mass properties required for engineering analyses. These relationships, described in detail by Hoek and Brown (1997), have been used to generate the charts for cohesion, friction angle and modulus of deformation given in Figures 4, 5 and 6.

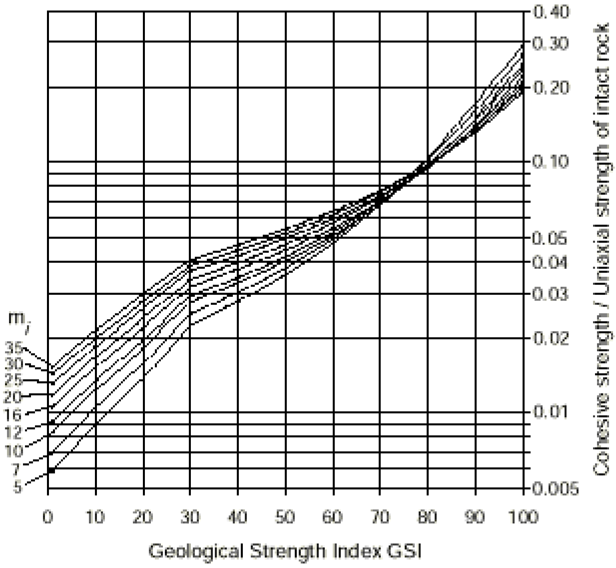


Figure 4: Cohesive strength versus GSI

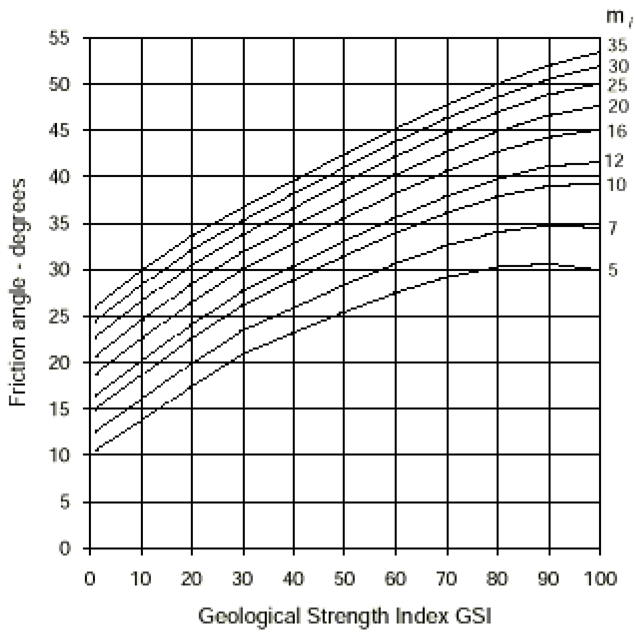


Figure 5: Friction angle versus GSI

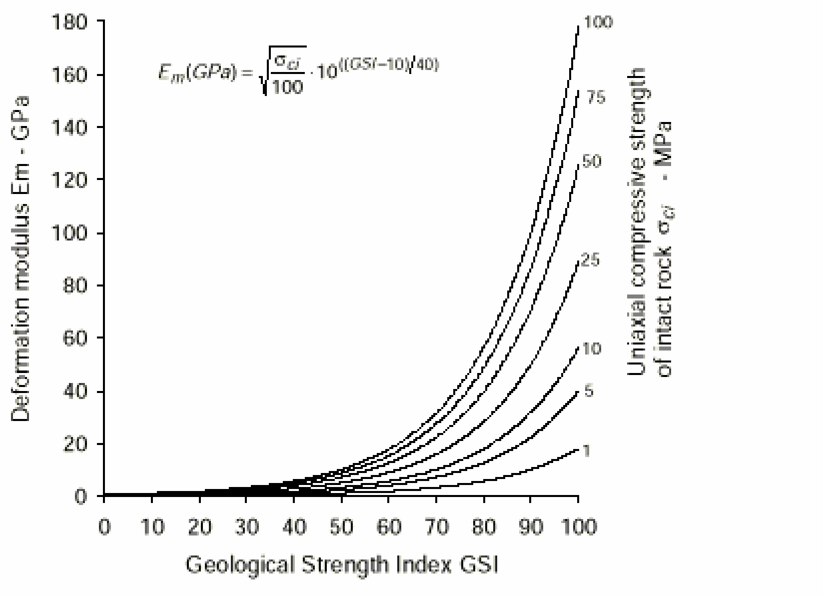


Figure 6: Deformation modulus versus GSI.

6. PRACTICAL EXAMPLE

A 27 km long, 10 m internal diameter concrete-lined headrace tunnel is currently under construction as part of the 1500 MW Nathpa Jhakri hydroelectric project on the Satluj river in Himachel Pradesh, India. The rock masses through which the tunnel passes are either metamorphic, consisting of gneisses, schists, quartzites and amphibolites or igneous consisting of granites and pegmatites. The engineering geological conditions associated with the project have been evaluated by the Geological Survey of India (Geological Survey of India 1988, Jalote et al 1996) on the basis of surface mapping, exploration boreholes and a few exploration adits. Excellent maps and sections were available before the commencement of underground excavation. In addition to conventional descriptive and structural maps, the rock mass has been classified in terms of Bieniawski's RMR system (Bieniawski 1989), Barton, Lien and Lunde's Q system (Barton et al 1974) and the GSI system described above.

At the time of writing (April 1999), the bulk of the tunnel excavation has been completed and the predic-tion of tunnelling conditions provided by the Geological Survey of India has proved to be accurate and a useful guide to the steps to be taken in excavation and support. One of the sections still to be completed is a 360 m long stretch through the Daj Khad shear zone. It is this part of the tunnel that I wish to discuss. The dramatic impact of the Daj Khad shear zone on the stability of the tunnel top heading is illustrated in Figure 7. This shows a closure in excess of one meter due to the heavy loads being imposed on the support system.

The rock mass in the vicinity of the Daj Khad shear zone is predominantly quartz mica schist with some serecite schist and a few gneiss bands and one amphibolite zone. The shear zone itself comprises a number of steeply dipping seams of fractured blocky rock with kaolinised and sericitised material. The uniaxial compressive strength of the schist that makes up the bulk of the rock mass is approximately 10 MPa under the saturated conditions that occur at the tunnel depth of between 200 and 300 m through this zone. The value of the rock mass constant *mi* has been assumed equal to 10 for the entire zone (see Table 2). The variation of the Geological Strength Index GSI through the rock mass associated with the Daj Khad can be represented by a truncated normal distribution defined by a mean value of 27, a standard deviation of 7, a minimum value of 6 and a maximum value of 45. This distribution is based on studies carried out by Geodata S.p.A. of Turin, consultants to the Nathpa Jhakri Joint Venture, the contractors on this stretch of headrace tunnel. The methodology employed by Geodata in arriving at this distribution has been described in a recent paper by Russo et al (1998).

Figure 7: Large convergence in the Nathpa Jhakri headrace tunnel top heading due to the influence of the Daj Khad shear zone.

Based upon this GSI distribution and assuming that the vertical in situ stress is uniformly distributed with a minimum of 5.4 MPa and a maximum of 8.1 MPa, corresponding to depths below surface of 200 and 300 m, a Monte Carlo simulation has been carried out to determine the convergence of the rock mass surrounding the 10 m diameter tunnel. This calculation is too detailed for inclusion in this publication but the equations used to set up the spreadsheet for the simulation are described in Hoek and Brown (1997) and Hoek (1998). The results of the simulation are plotted, in dimensionless form, in Figure 8. Note that this plot is for an unsupported tunnel.

It is evident, from this plot, that the convergence of the tunnel show a dramatic increase when the uniaxial compressive strength of the rock mass falls below about one tenth of the in situ stress. Unless adequate support is provided, the tunnel will almost certainly collapse for the lowest quality rock conditions under the highest in situ stresses. These findings are consistent with the results of recent research on tunneling in weak rocks. I have found that the very unstable conditions develop in unsupported tunnels of almost any shape for rock mass strengths less than 0.1 to 0.2 of the maximum in situ stress.

In passing, it is worth mentioning that trends such as this are of great value to geotechnical engineers. If a trend is found to be consistent over a wide range of conditions, this usually indicates that some basic law is at work and, if this law can be isolated, it may be possible to describe it in mathematical terms. This is an important part of the process of putting numbers to geology.

Taking the study of the Natha Jhakri tunnel to the next stage involves a more refined numerical analysis and, in order to demonstrate this process, I have used the finite element program PHASE2 developed at the University of Toronto[2] .

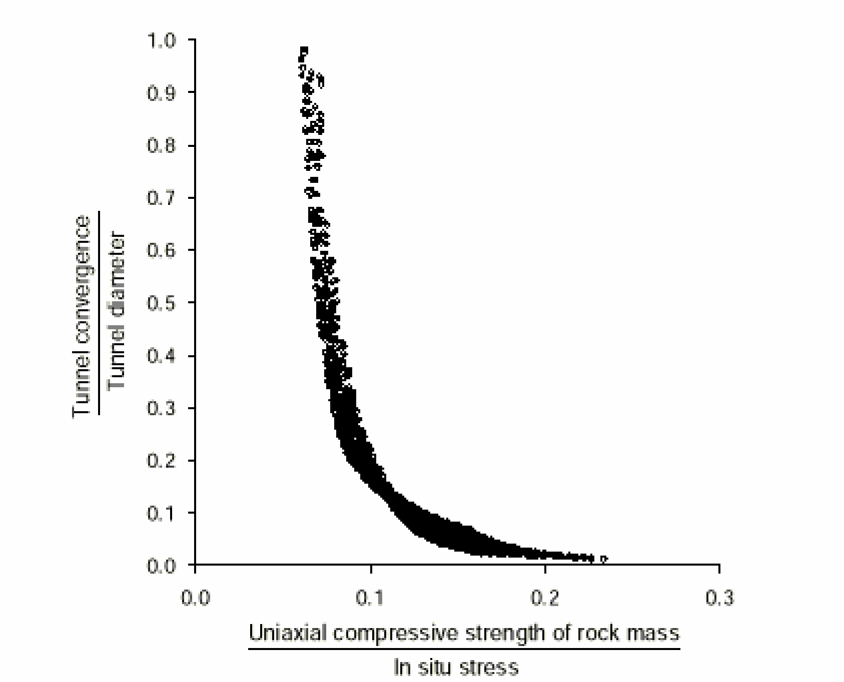


Figure 8: Convergence of an unsupported tunnel versus ratio of uniaxial compressive strength of the rock mass to in situ stress.

I have considered two cases, one defined by a GSI of 45, representing the better rock mass conditions in this zone, and the other defined by a GSI of 20 that is typical of the shear zone itself. As discussed earlier, the uniaxial compressive strength of the intact schist is taken as $\sigma_{ci} = 10$ MPa and the value of the material constant *mi* is 10. The corresponding values of cohesion, angle of friction and deformation modulus, estimated from Figures 4, 5 and 6, are given in Table 3. The uniaxial compressive strength (UCS) of the rock mass is calculated from the equation UCS=2*c* cos φ / $(1-\sigma\iota\nu\ \phi)$ and the values for the two cases are included in this table.

[2] Details available from Rocscience Inc., 31 Balsam Avenue, Toronto, Ontario, Canada M4E 3B5, Fax 1 416 698 0908, Email: software @ rocscience.com   Internet: http://www.rocscience.com

Table 3: Rock mass properties for two examples analyzed.

Property	Case 1	Case 2
Intact rock strength σ_{ci} MPa	10	10
Material constant m_i	10	10
Geological Strength Index	45	20
Cohesive strength c MPa	0.4	0.2
Friction angle ϕ degrees	30	23
Deformation modulus MPa	2500	550
Rock mass UCS, MPa	1.4	0.6
In situ stress MPa	6.75	6.75
UCS/in situ stress	0.21	0.09

In situ stresses along the tunnel route have been measured by hydraulic fracturing and by overcoring techniques (Bhasin et al 1996). However, because of the general weakness of the rock mass in the region of the Daj Khad shear zone, it has been assumed that the rock mass cannot tolerate significant stress differences and that all three principal in situ stresses are equal. An average tunnel depth of 250 m has been used to derive the in situ stress value of 6.75 MPa used in these analyses.

The zone of failure for Case 1 is illustrated in Figure 9. The PHASE2 model simulates progressive failure as the tunnel is excavated. The process used to achieve this simulation involves transferring loads that cannot be carried by failed elements onto adjacent elements. A check is then performed to determine whether the loads imposed on these adjacent elements causes them to fail. The process is continued until no more elements are loaded to failure.

For Case 1, as shown in Figure 9, the failure zone extends about 3 m into the rock mass surrounding the 10 m span top heading. The convergence of the roof and haunches is approximately 40 mm and, in this example, the floor heave is also approximately 40 mm.

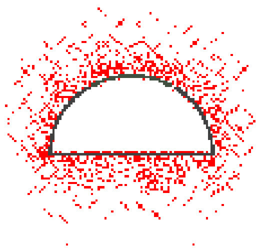


Figure 9: Extent of failure zone surrounding the tunnel top heading in a rock mass defined by GSI = 45. Shear failure is represented by the · symbol while tensile failure is denoted by the o symbol.

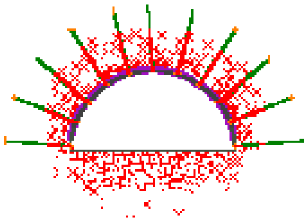


Figure 10: Reduced failure zone in the top heading roof due to the installation of untensioned fully-grouted rockbolts and steel fibre reinforced shotcrete.

Figure 10 shows that the top heading in the better quality rock mass (GSI = 45) can be stabilised by a combination of untensioned fully-grouted rockbolts and steel-fibre reinforced shotcrete. The rockbolts are 4 m long, 25 mm diameter and are installed on a grid pattern of 1.5 m x 1.5 m. The shotcrete layer is 100 mm thick. Typically a 25 mm thick layer of shotcrete is placed immediately after the excavation of a tunnel length of two to three metres. This is followed by the installation of the grouted rockbolts to within about 1 m of the face. A second layer of shotcrete is then applied to bring the total thickness up to 100 mm. In this case, no support of the floor is required since this is relatively stable and it will be excavated during the subsequent benching operation.

In deciding upon the adequacy of the support system, the extent of the failure zone in the reinforced rock mass is checked. Rockbolts passing through this failure zone will generally suffer yield of the grout/steel interface. This is not a problem provided that an unyielded anchor length of 1 to 2 m remains outside the zone of failed rock, as shown in Figure 10. The deformations in the rock mass must also be checked to determine whether there are any sections of the excavation perimeter that require additional support.

Note that other support systems, such as steel sets or lattice girders embedded in shotcrete, could also be used to stabilise this particular tunnel. The final choice of the support system depends upon overall cost and scheduling considerations.

The Daj Khad shear zone itself is characterised by a Geological Strength Index of approximately 20.

Mining through this poor quality rock mass results in a failure zone that extends about 15 m into the roof and floor, as illustrated in Figure 11. The size of this zone, together with the presence of kaolin, means that rock-bolt support will not be effective in this case. Steel set support is also difficult to design because of the large span of the top heading and the heavy squeezing pressures.

The support system chosen for mining through this difficult stretch of tunnel is similar to that used by Geodata on a number of previous projects (Carrieri et al 1991, Grasso et al 1993). This consists of a series of sub-horizontal holes, up to 24 m long, for geological exploration as well as pre-drainage and grouting of the rock mass ahead of the tunnel.. These are followed by a 12 m long umbrella of grouted pipe forepoles, forming a protective umbrella under which the tunnel can be excavated. Cemented fiberglass bars are used to stabilise the face and steel sets, radial rockbolts and a shotcrete or concrete invert are also used if required. Figure 12 shows the equipment used to drill the sub-horizontal holes and to install the forepoles in the Daj Khad stretch of the Nathpa Jhakri headrace tunnel.

The three-dimensional geometry of the tunnel heading and protective umbrella makes it very difficult to analyse this support system. Two-dimensional analyses, such as those described above, are not adequate. Grasso et al (1993) used an axisymmetric two-dimensional model to study the support provided by the forepole umbrella. However, I feel that a full three-dimensional analysis of this support system would be justified. Three-dimensional models capable of a full progressive failure analysis for this type of support system are becoming available but are not for the numerically timid. This type of analysis is best left to the numerical model specialist at this stage but they should be available as general design tool within a few years. However, as an interim step, a simplified two-dimensional solution is presented below.

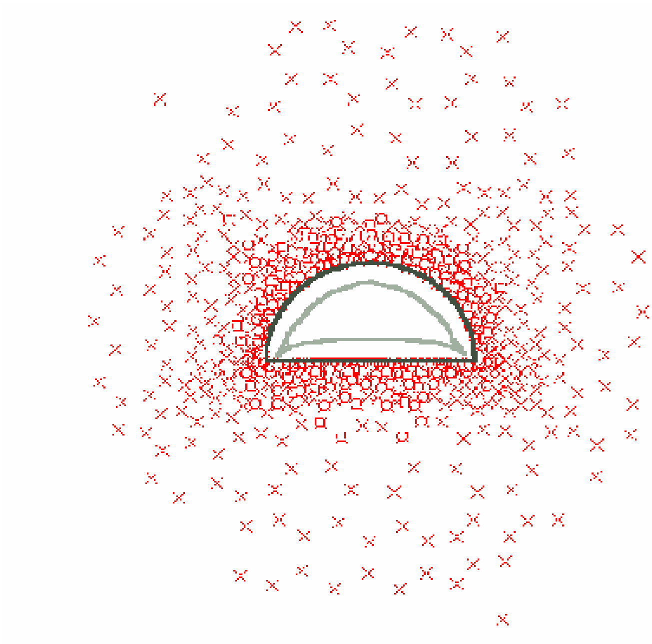


Figure 11: Failure zone surrounding the tunnel top heading in the Daj Khad shear zone, defined by a Geological Strength Index of 20. The tunnel convergence, shown by the deformed excavation boundary, is approximately 400 mm.

The 12 m long pipe forepole umbrella consists of 75 mm diameter pipes with a 6 mm wall thickness and they are installed at a spacing of 600 mm centre to centre. They are placed using an Odex type drill with the casing left in place and with grout injection after removal of the drill. Successive umbrellas are installed at 8 m intervals, giving a 4 m overlap to ensure continuity of protection. There are no general rules currently available for the support provided by forepoles and, in the absence of such rules, a crude equivalent model has been used in this analysis. This assumes that a process of weighted averages can be used to estimate the strength and deformation of the zone of 'reinforced rock'. For example, the strength is estimated by multiplying the strength of each component (rock, steel and grout) by the cross-sectional area of each component and then dividing the sum of these products by the total area. This equivalent strength value is then used to estimate the mechanical properties of the reinforced rock arch.

The results of this analysis are illustrated in Figure 12. This shows that the size of the failure zone and the magnitude of the displacements have been significantly reduced. In fact, a length of about 100 m of tunnel has already been excavated using this technique and observations of the tunnel behaviour suggest that the 'reinforced rock' arch actually behaves better than suggested by this simple analysis. However, since there is no theoretical justification for strengthening the reinforced rock arch any further, it is recommended that the 'weighted average' model be used until more rigorous models, derived from full three-dimensional analyses, become available.

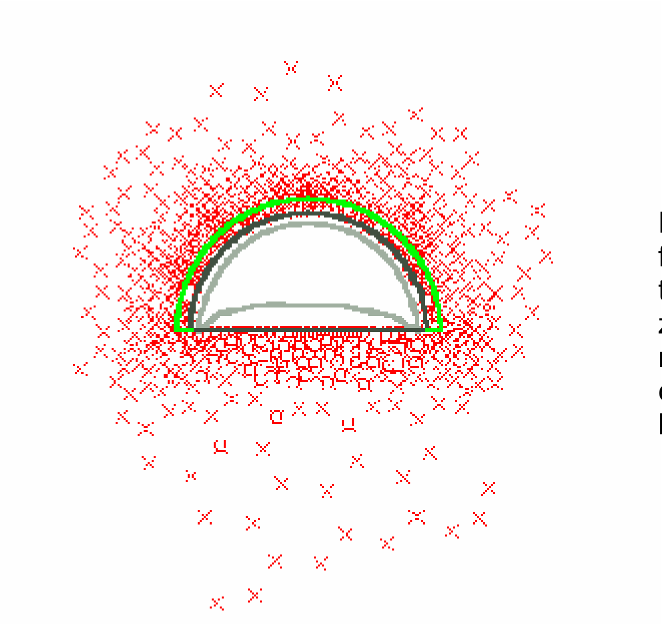


Figure 12 : Failure zone surrounding the forepole umbrella used to support the tunnel top heading in the Daj Khad shear zone. The size of the failure zone and the magnitude of the roof displacements, compared to those shown in Figure 11, have been significantly reduced.

7. CONCLUSION

Engineering design requires numbers. This is true whether the design utilises man-made materials such as steel or concrete or naturally occurring rocks and soils. One of the principal characteristics of natural materials is their variability and this makes it extremely difficult to assign reliable values to the properties required by engineering designers.

This paper has explored some of the methods that can be used by engineering geologists and geotechnical engineers to assess the geological factors that have an impact on engineering design. These start from the very crude estimates that are made during the early stages of a project on the basis of walk-over surveys and studies of available regional geology maps. At the other end of the spectrum are the input requirements of the very sophisticated numerical analyses used to assess the stability and support requirements for complex three-dimensional excavations in rock.

Figure 12: Installation of 12 m long grouted pipe forepoles to form a protective reinforced rock umbrella under which excavation of the top heading can proceed.

It is easy to conclude that there is never enough information and that, what there is, is unreliable because of the uncertainty associated with the methods of assigning numbers to geology. While these conclusions may be true they are not helpful to the design engineers who have to produce safe and economical designs, whether or not the information is adequate.

I have tried to demonstrate that it is possible to arrive at useable estimates of the properties required for an engineering design. This requires close co-operation between engineering geologists and geotechnical engineers and a good measure of common sense and practical judgment.

I would like to conclude with a statement contained in a general report presented almost 25 years ago:

"The responsibility of the design engineer is not to compute accurately but to judge soundly" (Hoek and Londe 1974). I consider that this statement is still true today.

8. ACKNOWLEDGMENTS

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