Putting numbers to geology – an engineer's viewpoint

Evert Hoek

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The Second Glossop Lecture

Putting Numbers to Geology – an Engineer's Viewpoint

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Abstract

Assigning numbers to geology requires a delicate balance between the commonly held opinion that geology cannot be quantified and the over-optimistic view that every physical quantity can be described in precise mathematical terms. In reality, many geological characteristics cannot be quantified precisely and intelligent guesses based upon experience and logical arguments are the best that can be hoped for.

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.

Introduction

Professor Peter Fookes, in the First Glossop lecture (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 this, the Second Glossop lecture, I would like to take the process of design to the next step. I will attempt to describe how an engineer puts numbers to the largely qualitative model described by Fookes. Many geologists are uncomfortable with this requirement to assign numbers to geology and many will contend 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 underground excavations 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 recognised 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.

Help for artistically challenged geologists

The three-dimensional block drawings and sections included in the written version of the first Glossop Lecture, prepared by or with the assistance of Mr G. Pettifer, are miniature masterpieces of geological art. If only such drawings were available on all construction sites.

Unfortunately, I have to say that in my thirty-five odd years of consulting around the world I have seldom come across geological drawings that come close to the these in terms of clarity of presentation and transmission of useful engineering geology information. The converse is generally the case and I have spent many uncomfortable hours attempting to decipher geological plans and sections of less than adequate quality. Of course, it is not the artistic ability of the geologist that determines that accuracy of the geological interpretations being presented but it certainly helps when the drawings are well executed, clearly captioned and approximately to scale.

Help for artistically challenged geologists is on the way in the form of computer generated three-dimensional solid models. Such models are now relatively common in mechanical and structural engineering and even in the medical field. The models of greatest interest to geologists were developed to meet the needs of the mineral exploration geologists in their efforts to define the three dimensional shapes and ore grade distribution of sub-surface mineral deposits. For many years these geologists have used sophisticated statistical techniques and trend surface analysis to interpolate and extrapolate between borehole intersections. The evolution into three-dimensional computer modelling was a natural step.

The mining industry has embraced these computer modelling techniques and such models can now be found in mine planning and geotechnical departments as well as in the offices of the exploration and mining geologists.

One of the most spectacular examples of such a model has been constructed by the Geotechnical Group of the Chuquicamata open pit copper mine in northern Chile, illustrated in Figure 1. An example of a typical three-dimensional block model is illustrated in Figure 2. The 1998 shell of the Chuquicamata mine, showing the geological units exposed in the walls, is illustrated in Figure 3. In this case the computer operators are the geologists themselves and it is not unusual to see a geologist come in from the field and sit down immediately to enter the latest data into the model. This ensures that the model reflects the understanding and interpretation of the geologists and that it is not simply an illustration prepared by a computer technician who may not understand the on-going thinking that goes into building the geological model.

The advantages of these three-dimensional computer generated models are enormous. The model can be rotated and viewed from any direction, enlarged, sectioned and components can be removed or added at will. Trend surfaces representing interpolations or extrapolations between boreholes can be adjusted to fit the geologist's understanding of the tectonic processes involved in the formation of the rock mass. Work is now going on to take data from one of these models and to feed it directly into limit equilibrium slope stability analyses or numerical analyses of the stress and failure conditions around underground excavations.

The current cost of the hardware and software required for the generation of these three-dimensional models is approximately £50,000. This places it outside the range of all but the very largest civil engineering projects. However, with dramatic advances in computer software and the ever decreasing cost of computer hardware, it is conceivable that installations costing one tenth of the current system costs will be available within a few years. This would put these systems within reach of most agencies or consulting organisations with the need to interpret and present engineering geology data. I look forward with eager anticipation to the day when I see one of these models being used on a civil engineering project.

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 an underground excavation. An example of a typical flow path for a geotechnical engineering design, adapted from Hoek and Brown (1980), is illustrated in Figure 4. 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.

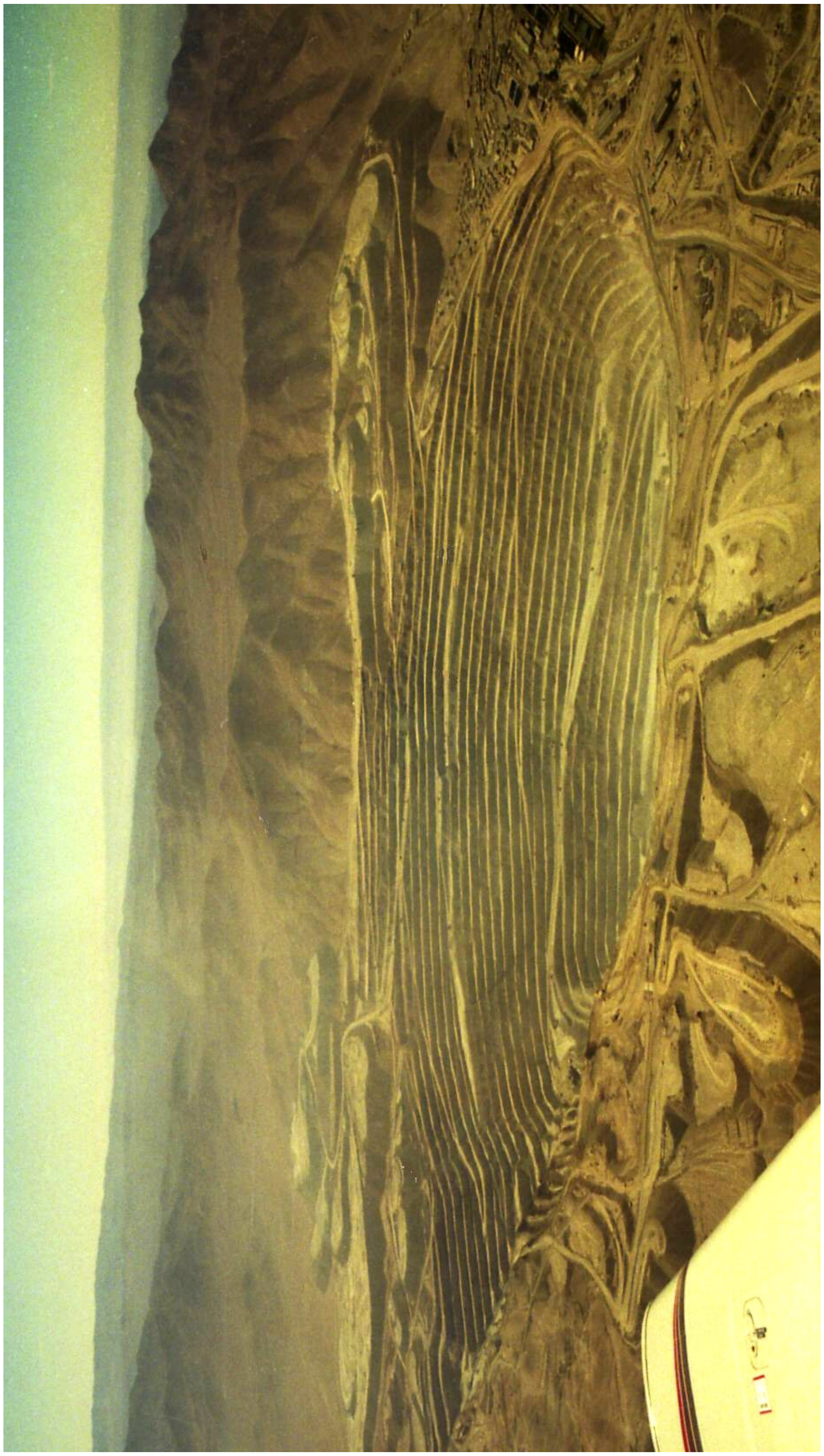

Fig **1**: Arial view of the Chuquicamata open pit copper mine in northern Chile.

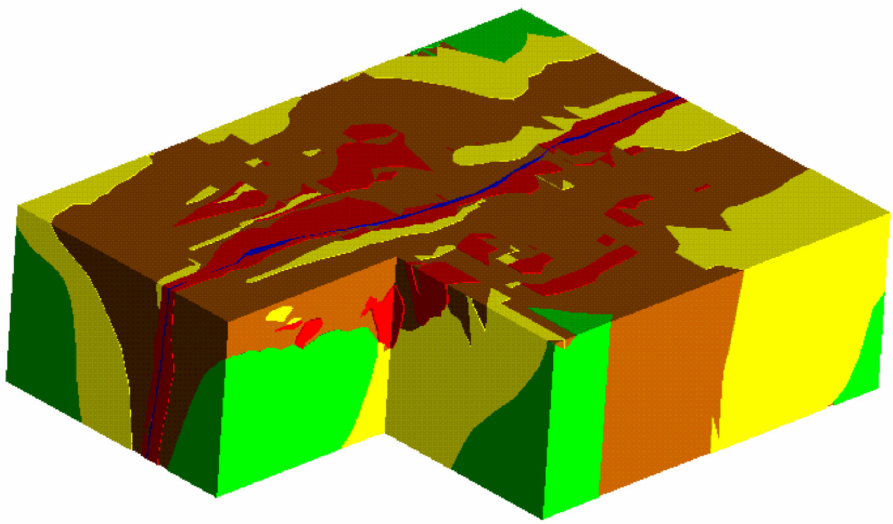

Fig. **2**: Example of a computer generated three-dimensional solid model of the rock mass in which the Chuquicamata open pit copper mine in northern Chile is being mined.

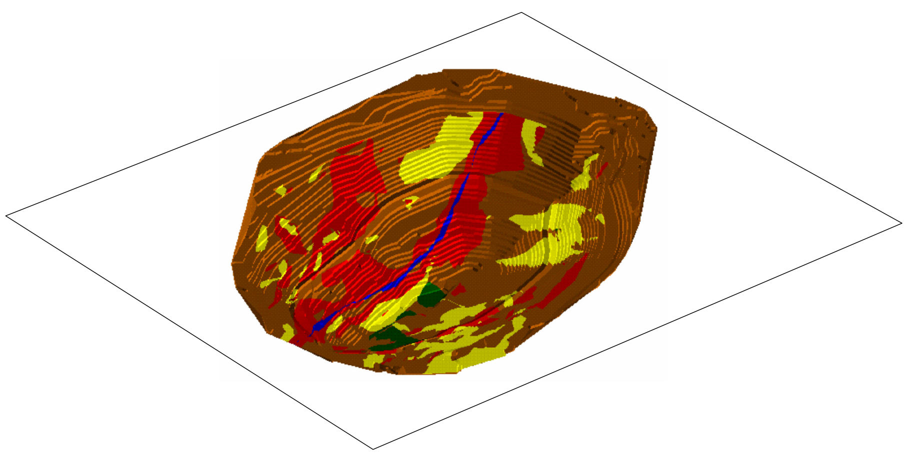

Fig. **3**: Chuquicamata open pit mine in 1998 showing the geological units exposed in the walls of the 750 m deep pit. Figures 2 and 3 were prepared by Mr Ricardo Torres of the Chuquicamata Geotechnical Group using the program Vulcan[1].

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[1] Available from Maptek Perth, 92 Roe Street, Northbridge, Western Australia 6003, Phone: + 61 8 9328 4111, Fax: + 61 8 9328 4422, email: info@perth.maptek.com.au

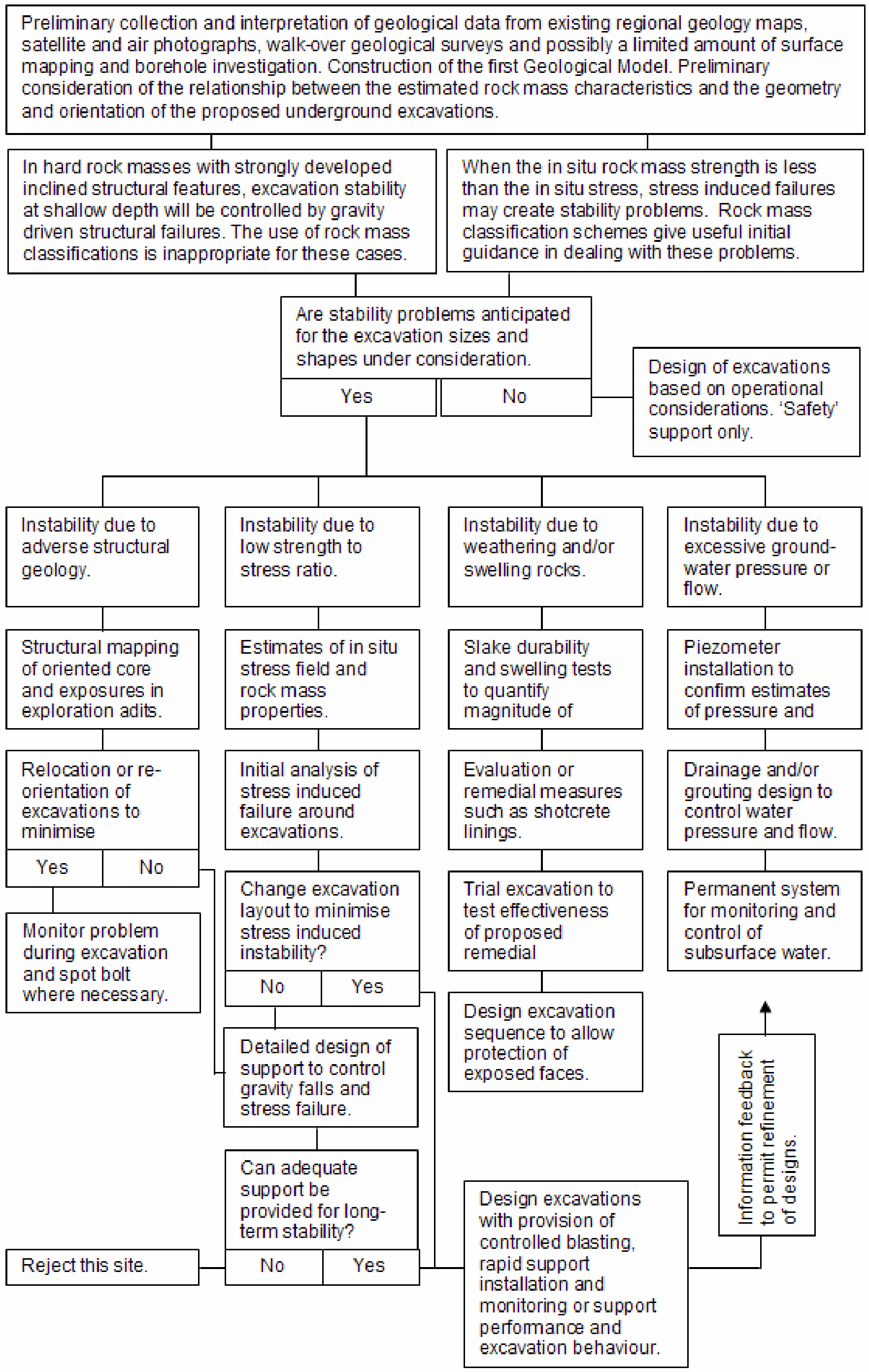

Fig. **4**: Flow path for the geotechnical design of underground excavations in rock. (Hoek and Brown 1980).

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 behaviour.

Note that 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.

Preliminary project 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 minimum number of unstable landforms, ancient landslides, difficult river crossings and the minimum number of tunnels. Simple common sense says that all of 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 kilometres of the site, if such tunnels exist.

Care has to be exercised in how this precedent experience is interpreted and applied. I remember visiting an open pit mine in the United Kingdom many years ago and asking why the slopes had been designed at the unusual angle of 53 degrees. The answer I received was that the company's mines in the United States seemed to operate successfully at this angle – hardly an appropriate extrapolation by any stretch of the imagination.

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, based on the geological model, to provide the qualitative assessment of whether these conditions can easily be met or whether it would be better to look for another site.

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 or the ease with which a rock mass will cave in a block caving mining operation. The rock mass classification systems commonly used in the English language world have been summarised by Bieniawski (1989) and it is not my intention to discuss these classifications further here. Incidentally, there are at least seven different rock mass classification systems in use in Japan and probably similar numbers in other non-English speaking countries.

Table 1: Rockfall Hazard Rating System. After Pierson and van Vickle (1993).

<table>
<tr><td colspan="3" rowspan="2">CATEGORY</td><td colspan="4">RATING CRITERIA AND SCORE</td></tr>
<tr><td>POINTS 3</td><td>POINTS 9</td><td>POINTS 27</td><td>POINTS 81</td></tr>
<tr><td colspan="3">SLOPE HEIGHT</td><td>25 FT</td><td>50 FT</td><td>75 FT</td><td>100 FT</td></tr>
<tr><td colspan="3">DITCH EFFECTIVENESS</td><td>Good catchment</td><td>Moderate catchment</td><td>Limited catchment</td><td>No catchment</td></tr>
<tr><td colspan="3">AVERAGE VEHICLE RISK</td><td>25% of the time</td><td>50% of the time</td><td>75% of the time</td><td>100% of the time</td></tr>
<tr><td colspan="3">PERCENT OF DECISION SIGHT DISTANCE</td><td>Adequate site distance, 100% of low design value</td><td>Moderate sight distance, 80% of low design value</td><td>Limited site distance, 60% of low design value</td><td>Very limited sight distance, 40% of low design value</td></tr>
<tr><td colspan="3">ROADWAY WIDTH INCLUDING PAVED SHOULDERS</td><td>44 feet</td><td>36 feet</td><td>28 feet</td><td>20 feet</td></tr>
<tr><td rowspan="4">GEOLOGIC CHARACTER</td><td rowspan="2">CASE 1</td><td>STRUCTURAL CONDITION</td><td>Discontinuous joints, favorable orientation</td><td>Discontinuous joints, random orientation</td><td>Discontinuous joints, adverse orientation</td><td>Continuous joints, adverse orientation</td></tr>
<tr><td>ROCK FRICTION</td><td>Rough, irregular</td><td>Undulating</td><td>Planar</td><td>Clay infilling or slickensided</td></tr>
<tr><td rowspan="2">CASE 2</td><td>STRUCTURAL CONDITION</td><td>Few differential erosion features</td><td>Occasional erosion features</td><td>Many erosion features</td><td>Major erosion features</td></tr>
<tr><td>DIFFERENCE IN EROSION RATES</td><td>Small difference</td><td>Moderate difference</td><td>Large difference</td><td>Extreme difference</td></tr>
<tr><td colspan="3">BLOCK SIZE
QUANTITY OF ROCKFALL/EVENT</td><td>1 FT
3 cubic yards</td><td>2 FT
6 cubic yards</td><td>3 FT
9 cubic yards</td><td>4 FT
12 cubic yards</td></tr>
<tr><td colspan="3">CLIMATE AND PRESENCE OF WATER ON SLOPE</td><td>Low to moderate precipitation; no freezing periods, no water on slope</td><td>Moderate precipitation or short freezing periods or intermittent water on slope</td><td>High precipitation or long freezing periods or continual water on slope</td><td>High precipitation and long freezing periods or continual water on slope and long freezing periods</td></tr>
<tr><td colspan="3">ROCKFALL HISTORY</td><td>Few falls</td><td>Occasional falls</td><td>Many falls</td><td>Constant falls</td></tr>
</table>

Table 2: Example of the application of the Rockfall Hazard Rating System

Category	*Description*	*Points*
Slope height	30 m	81
Ditch effectiveness	Limited catchment	27
Average vehicle risk	50% of the time	9
Percentage of decision sight distance	Very limited sight distance, 40% of low design value	81
Roadway width, including paved shoulders	28 feet / 8.5 m	27
Geologic character – Case 1	Discontinuous joints, adverse orientation, Planar	27
Block size / quantity of rockfall	3 ft (1.3 m) / 12 cu. yards or cu. metres	81
Climate and presence of water on slope	Moderate precipitation or short freezing periods or intermittent water on slope	9
Rockfall history	Many falls	27
	Total score	369

A classification system that is probably almost completely unknown in the United Kingdom but which, for me, embodies the essential elements of a good classification system for preliminary engineering design is the 'Rockfall Hazard Rating System'. This system was developed by the Federal Highway Administration in the United States for the preliminary evaluation of rockfall hazards and the allocation of priorities for remedial work (Pierson and van Vickle 1993). The key elements of this rating system are contained in the table reproduced as Table 1. Detailed instructions and examples on the evaluation of each of the nine components of the system are given in the FHWA manual.

I like this classification because it is based on a set of simple visual observations, most of which can be carried out from a slow moving vehicle as would be required for the preliminary evaluation of miles of mountain highway. The system also contains all the components required for a complete engineering evaluation of the risks to the public. These include highway design factors as well as geometrical and geotechnical factors, all presented in clear and unambiguous terms.

An example of a typical rockfall hazard evaluation, based on this system, is given in Table 2. The authors of the FHWA manual give no direct instructions on how the total score obtained from this rating system should be used. It is intended for use as a tool to assist management in the allocation of resources and these decisions will vary from state to state. From personal discussions with one of the authors I learned that, in the State of Oregon, slopes with a rating of less than 300 are assigned a very low priority while slopes with a rating of more than 500 are identified for urgent remedial action.

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.

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 underground excavations 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.

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 (199) 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 metres 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 5. 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.

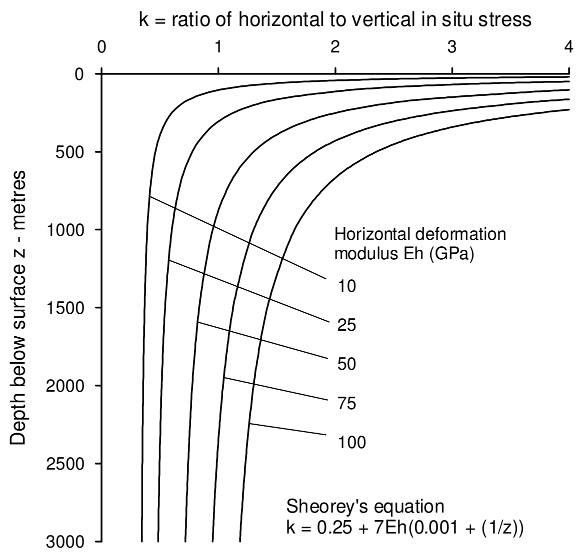

Fig. **5**. Ratio of horizontal to vertical in situ stress versus depth below surface. (Sheorey 1994)

Note that 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 relationships 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 an underground powerhouse 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 ongoing site investigation and design process.

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 (m_i) 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 3 and 4.

Table 3: 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 4: Values for the constant m_i for intact rock,. Note that the 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 m_i will be significantly different if failure occurs along a weakness plane.

The most important component of the Hoek-Brown system is the process of reducing the material constants σ_{ci} and m_i from their 'laboratory' values to appropriate in situ values. This is accomplished through the Geological Strength Index GSI that is defined in Figure 6.

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.

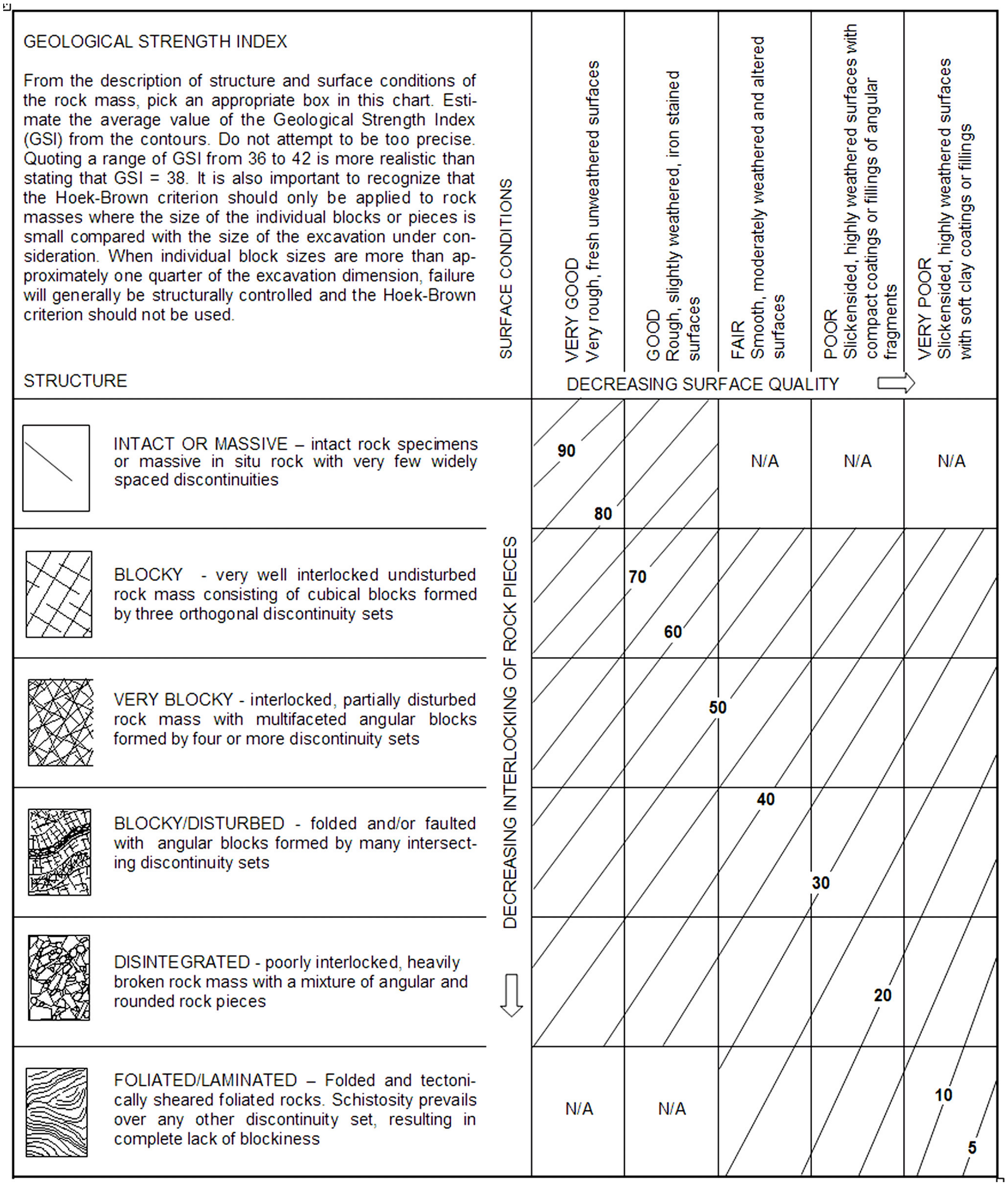

Fig. **6**: Geological Strength Index GSI on the basis of geological observations.

The version of the GSI chart presented in Figure 6 contains two new rows that have not yet been published elsewhere. The top row on 'intact or massive' rock is the result of work in Chile on cemented breccias that behave very much like weak concrete (personal communication from Dr Antonio Karzulovic). The bottom row on 'foliated/laminated/sheared' rock has been inserted to deal with very poor quality phyllites encountered in Venezuela (personal communications from Professors Rudolpho Sancio and Daniel Salcedo) and the weak schists being tunnelled through for the Athens Metro (Hoek, Marinos and Benissi 1998). It is probable that this figure will continue to evolve as experience is gained in the use of GSI for estimating rock mass properties in the wide range of geological environments to which it is being applied.

Based on intuition, experience and the back analysis of a number of case histories, relationships have been developed between GSI, σ_{ci} and m_i 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 7, 8 and 9.

These charts can be used to obtain approximate values for in situ properties. It is an absolute requirement that the engineer making these estimates should check their appropriateness by back analysis of the measured or observed excavation behaviour, once construction commences.

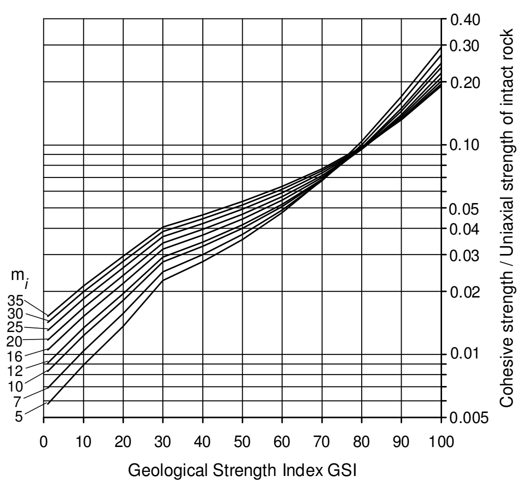

Fig. **7**: Cohesive strength versus GSI.

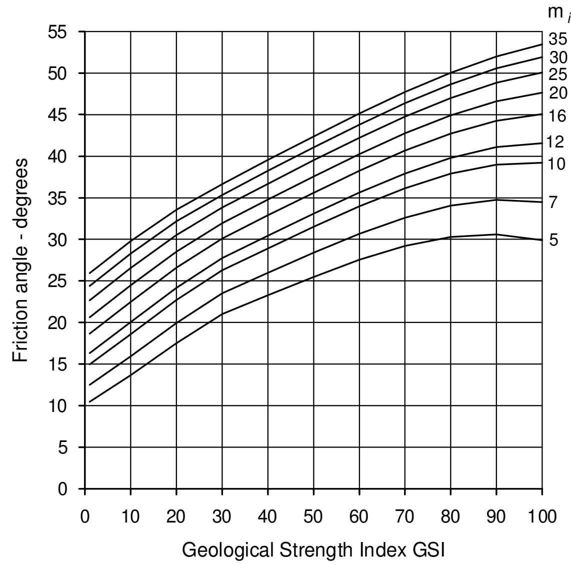

Fig. **8**: Friction angle versus GSI.

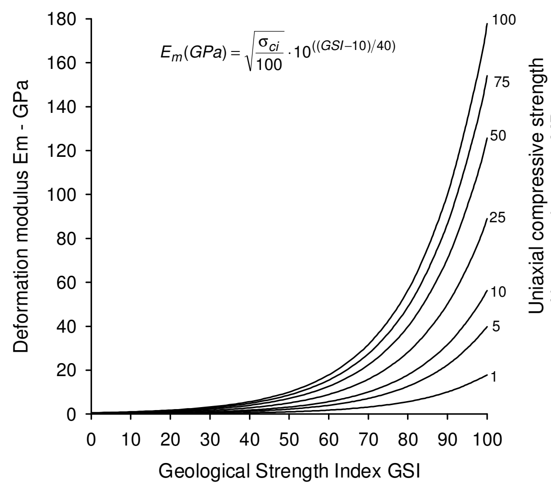

Fig **9**: Deformation modulus versus GSI.

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 (May 1998), the bulk of the tunnel excavation has been completed and the prediction 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 10. This shows a closure in excess of one metre 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 m_i has been assumed equal to 10 for the entire zone (see Table 4). 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).

Fig. **10**: 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 extent of the plastic zone and 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 Figures 11 and 12. Note that these plots are for an unsupported tunnel.

It is evident, from the plots given in these figures, that the size of the plastic zone and the convergence of the tunnel both show dramatic increases 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 as yet unpublished research on tunnelling 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.

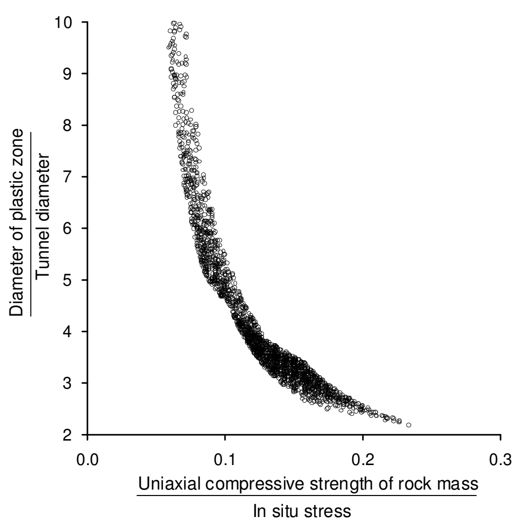

Fig. **11**: Size of plastic zone versus ratio of uniaxial compressive strength of rock mass to in situ stress.

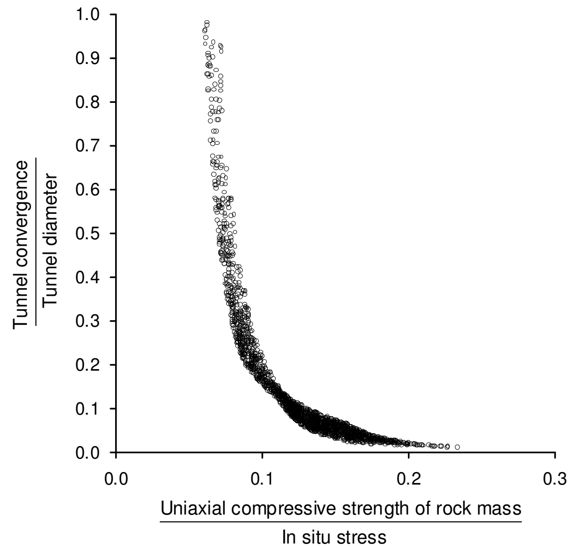

Fig. **12**: Tunnel convergence versus ratio of uniaxial compressive strength of the rock mass to in situ stress.

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. This software is one of a family of user-friendly but powerful programs developed with financial assistance from the Canadian mining industry. Development and distribution of these programs has now been taken over by a spin-off company called Rocscience Inc.[2].

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. 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 m_i

[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.

is 10. The corresponding values of cohesion, angle of friction and deformation modulus, estimated from Figures 7, 8 and 9, are given in Table 5. The uniaxial compressive strength (UCS) of the rock mass is calculated from the equation $UCS = 2c\cos\phi/(1-\sin\phi)$ and the values for the two cases are included in this table.

Table 5: Rock mass properties for two examples analysed.

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). The following values were found for the principal stresses:

σ_1 = 7.1 MPa, approximately parallel to valley,
σ_2 = 5.9 MPa, vertical stress,
σ_3 = 3.9 MPa, approximately normal to valley.

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.

As shown in Table 5, the ratio of the uniaxial compressive strength of the rock mass to the in situ stress is 0.21 for Case 1 and 0.09 for Case 2. These values fall on either side of the critical ratio of about 0.1 shown in Figures 11 and 12.

The zone of failure for Case 1 is illustrated in Figure 13. 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 13, 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. In many cases of weak rock tunnelling, floor heave is significantly larger than roof and wall convergence. This leads to the need for reinforcement of the floor, by rockbolting or by the placement of a concrete invert, in order to stabilise the tunnel.

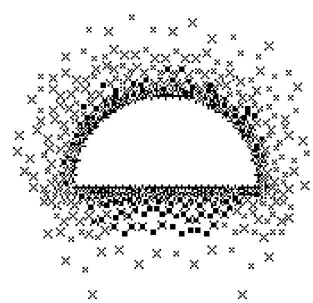

Fig. **13**: Extent of failure zone surrounding the tunnel top heading in a rock mass defined by GSI = 45. Shear failure is represented by the × symbol while tensile failure is denoted by the • symbol.

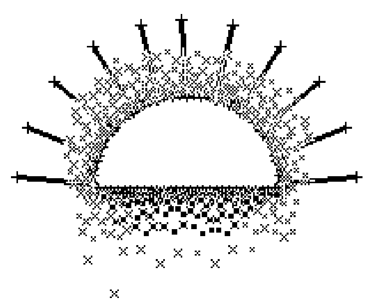

Fig. **14**: Reduced failure zone in the top heading roof due to the installation of untensioned fully-grouted rockbolts and steel fibre reinforced shotcrete.

Figure 14 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 14. 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 15. The size of this zone, together with the presence of kaolin, means that rockbolt 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 fibreglass bars are used to stabilise the face and steel sets, radial rockbolts and a shotcrete or concrete invert are also used if required.

Figure 16 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.

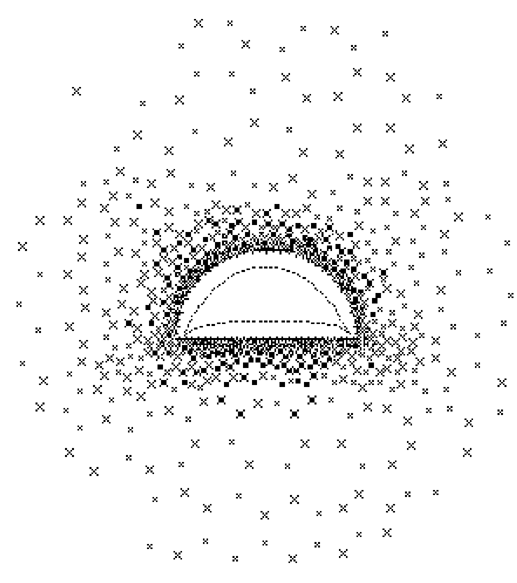

Fig. **15**: Failure zone surrounding the tunnel top heading in the Daj Khad shear zone, defined by a Geological Strength Index of 20. The tunnel convergence, shown by the deformed excavation boundary, is approximately 400 mm.

Fig. **16**: Installation of 12 m long grouted pipe forepoles to form a protective reinforced rock umbrella under which excavation of the top heading can proceed.

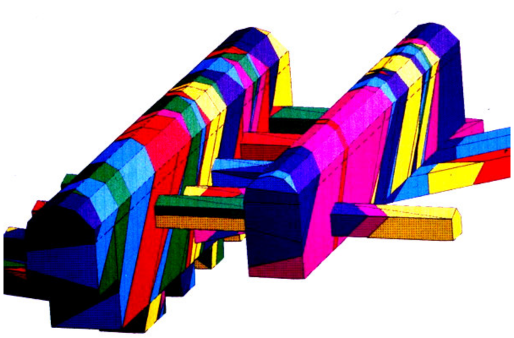

Fig. **17**: Isometric view of the three-dimensional numerical model of the underground powerhouse cavern and transformer gallery of the Nathpa Jhakri Hydroelectric Project.

One example of the type of three-dimensional model that can be used for these studies is illustrated in Figure 17. This 3DEC[3] model has been used in studies of the Nathpa Jhakri underground powerhouse complex, carried out by Dr B. Dasgupta of Advanced Technology and Engineering Services, Delhi. India.

Engineering risk assessment

The inherent variability of geological materials means that each material property should be defined by a range of values rather than by a single number. Hence, the end product of any analysis based on these numbers has to be assessed in terms of probability of occurrence or of engineering risk.

A detailed discussion on techniques for engineering risk assessment is beyond the scope of this paper and the reader is referred to the excellent book by Harr (1987) on this subject. However, the general concepts of this

[3] Available from ITASCA Consulting Group Inc., Thresher Square East, 708 South Third Street, Suite 310, Minneapolis, Minnesota 55415, USA. Fax 1 612 371 4717

form of analysis are illustrated in the following simple example.

The problem is to determine the risk of failure of a slope excavated in a heavily jointed rock mass. The shear strength properties of this rock mass are defined by the normal distributions of cohesion and angle of friction given in Figure 18. These distributions were calculated by means of a Monte Carlo simulation, using assumed normal distributions defined by the following values (Hoek 1998):

Parameter	Mean	Standard deviation
UCS of intact rock, MPa	10	2.5
Intact rock constant m_i	8	1
Geological Strength Index	25	2.5

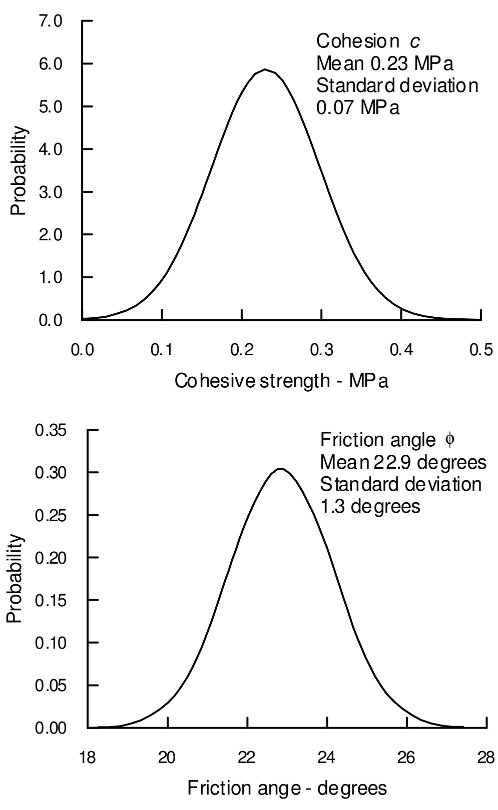

Fig. **18**: Normal distributions of cohesive strength and angle of friction for a heavily jointed rock mass.

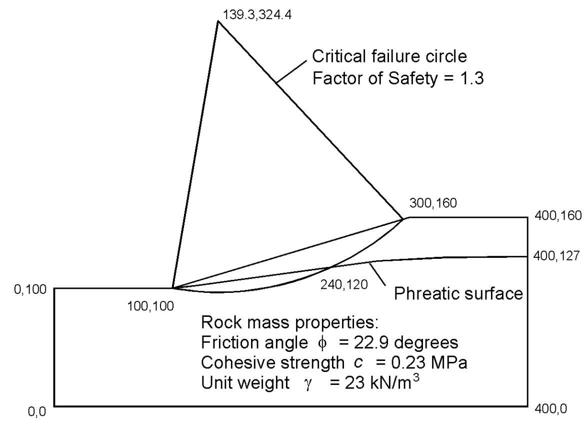

Fig. **19**: Slope and phreatic surface geometry, rock mass properties and critical failure surface for a homogeneous slope.

The geometry of the slope, with a height of 60 m and a slope face angle of 16.7 degrees, is defined in Figure 19. The program SLIDE[4] was used to carry out a critical failure surface search, using Bishop's circular failure analysis. Rosenbleuth's point estimate method (Hoek 1998, Harr 1987) was used to determine the mean and standard deviation of the normal distribution for the factor of the slope. This distribution is plotted in Figure 20.

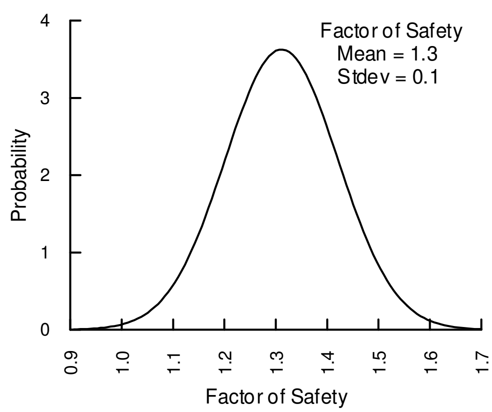

Fig. **20**: Normal distribution of the factor of safety of the slope defined in Figure 19.

[4] 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.

This plot shows that, for a mean factor of safety of 1.3 with a standard deviation of 0.1, the normal distribution curve extends from 0.9 to 1.7. This range is determined by the high quality of the input data. It was assumed that the uniaxial compressive strength of the intact rock as well as the material constant m_i were determined by laboratory testing and that the Geological Strength Index has been obtained by careful field observations by an experienced engineering geologist. Where poor quality input data is used for such an analysis, the mean value may be the same but the standard deviation and the range of factors of safety contained in the distribution curve will be much higher.

The probability of failure is defined by the ratio of the area under the curve for factors of safety of less than 1.0 divided by the total area under the normal distribution curve. As can be seen from Figure 20, this ratio is very small for the case considered. This suggests that, for this particular slope and for the quality of the input data used, a factor of safety of 1.3 will ensure that the risk of slope failure is negligible.

Finite failure risks are acceptable provided that they are considered in terms of the cost and consequences of failure. For example, a probability of failure of 10% may be acceptable in the case of an open pit bench or a logging road where traffic is restricted to trained personnel and where equipment is available to clear up the failure. On the other hand, this level of risk would be completely unacceptable for the abutment of a dam or the foundation of a high rise building.

Current technology for calculating the probability of failure, as described above, can only be used for relatively simple problems for which a deterministic solution can be obtained. As computer processing speeds increase, the application of these methods to more complex problems, such as the stability of underground excavations, will become feasible.

Note that other techniques are available for making an engineering risk assessment. These include the use of fault and decision tree analysis and some of these techniques are being applied to subjects such as the assessment of dam safety (Nielsen *et al* 1994). The huge societal and economic consequences of dam failures have attracted the attention of researchers in this field for many years and we can expect to see significant advances in risk analysis in the years to come (Anon. 1998).

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.

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 judgement.

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.

Acknowledgements

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Khad stretch of the Nathpa Jhakri headrace tunnel is acknowledged.

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