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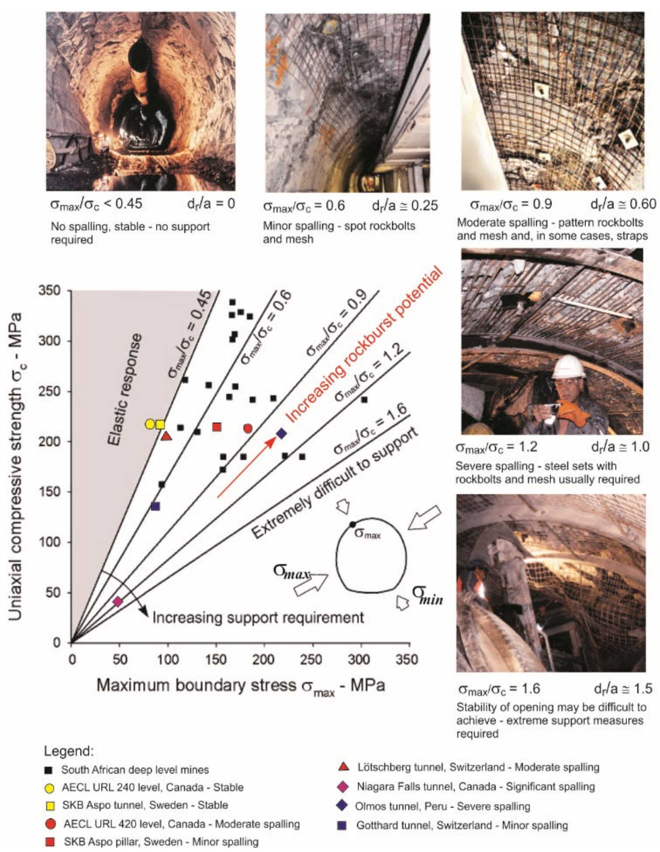
Africa Geotechnical Legends: Dr. Evert Hoek

Designs and Disasters in Rock Engineering Q&A

1. **Although having been investigated over multiple decades, rock bursts within underground coal and metal/non-metal mining operations continue to present safety and design issues especially as operations continue at a deeper depth and in more logically complex conditions. Based on your experience, do you have any comments on design or considerations when developing in burst-prone stress/geologic conditions?**

The answer to this question requires a complete chapter of its own since there are a multitude of questions within the question. However, to simplify the issue down to its basics it can be said that the threat of significant rockbursts tends to occur in massive strong intact rock subjected to very high stresses, as illustrated in the figure below. The increasing threat of spalling and rockbursting, with an increasing ratio of maximum boundary stress to uniaxial intact rock strength, is illustrated in this figure.



Hoek, E and Marinos P, G. Tunnelling in overstressed rock. (2010) *Rock Engineering in Difficult Ground Conditions - Soft Rocks and Karst.* Vrkljan, I (ed). London: Taylor and Francis Group, 49-60.

Plot of the maximum boundary stress to uniaxial compressive strength for a simple tunnel excavation in intact rock. The ratio dr/a for each tunnel is the approximate ratio of the diameter of the damaged rock zone to the diameter of the tunnel. Local spalling in the rock surrounding the tunnel is common when the maximum boundary stress exceeds the uniaxial strength. The depth and severity of this spalling increases with stress and, eventually, the failure takes the form of rockbursts. These occur when the deformation of the overstressed rock mass exceeds the capacity of the rock forming the excavation boundary, including support if installed. The excavation boundary shape has a significant influence on the damage caused by the rockburst, with planar surfaces in rectangular excavations suffering more damage than excavations with approximately circular profiles. In order to anticipate rockburst problems in mining and civil engineering projects, some form of numerical analysis, along the lines illustrated above, should be carried out. Fortunately, the development of sophisticated three-dimensional numerical tools makes it possible to analyze practically any excavation shape and to estimate the zones of potential failure. These analyses can include significant discontinuities, such as faults, which can impact the failure process. Having done this analysis of potential failure in the rock mass surrounding the excavation under consideration, what can then be done to improve the design? The in situ stresses, assuming that these have been measured or estimated, cannot be changed. The only option is to change the shape and orientation of the excavations in the hope that the extent of failure can be minimized. In very high stress situations, changing the shape and orientation of the excavations may not achieve the desired reduction of the extent of failure. What can then be done to minimize the potential for rockbursts? In one deep

TBM driven tunnel in Peru, blasting of the rock ahead of the advancing face was tried in an attempt to reduce rockbursts, but no significant reduction was achieved. In some other cases, heavy rock support has been installed in the tunnel in the hope that the resulting reduction in tunnel closure would minimize rockbursts. In general, this has been met with minimal success. Hence, there may be situtions in which the elimination of rockbursts is not possible and where their occurrence has to be accepted as an ever present threat.

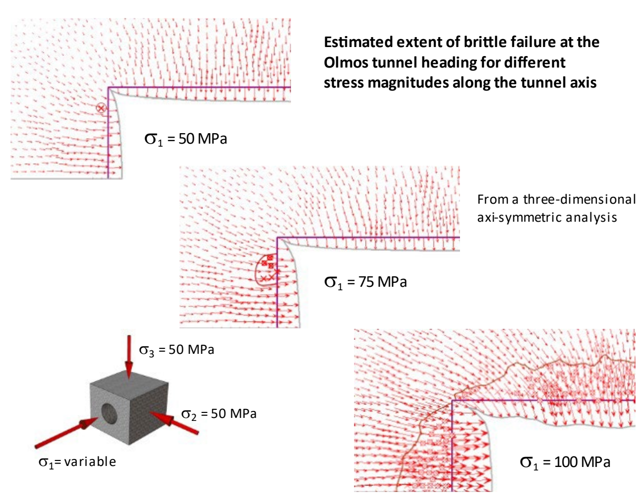
2. **Can you share some of your experience related to rock bursting conditions?**

My introduction to rock engineering occurred when I was employed by the South African Council for Scientific and Industrial Research in 1957 where I became involved in the study of brittle failure of hard rock under very high stresses in very deep level gold mines. As outlined in the answer to question 1, rockbursts are generally associated with hard strong rocks subjected to very high stresses, and this is the case in both photographs reproduced below. The failures are implosive and very violent. The resulting damage, as is evident in these photographs, can result in severe disruption to the mine operation as well as being very dangerous to workers in the mine.

a. Results of a rockburst in a South African gold mine at a depth of approximately 3000 m below surface, photographed in the early 1960s.

b. Rockburst resulting from the failure of a pillar in a hard rock mine in Canada. Photographed in about 1980.

c. Severe damage to the Olmos Tunnel TBM due to a rockburst.



d. Analysis of potential failure zone for different in situ stresses.

A severe rockburst in the Olmos Tunnel in Peru, excavated by a tunnel boring machine (TBM) is illustrated in photograph c above. This tunnel, which transfers water 20.1 km from the Huancabamba River to the arid Pacific Coastal Watershed, was driven by drill and blast and by a Tunnel Boring Machine at depths of up to 2 km below surface. Very high in situ stresses were encountered, with the horizontal stress associated with the mountain formation, being dominant. Figure d illustrates this issue which is discussed in detail in the following reference. Reference: M.S. Diederichs, E. Eberhardt, B. Fisher, Consideration of stress and structural influence on high stress response in deep tunnelling – the Olmos Tunnel, Peru. G. Anagnostou, H. Ehrbar (Eds.), *Proceedings of the World Tunnel Congress*, CRC Press (2013).

3. **Any suggestions for avoiding issues, given what you have said about difficulty with modifications to contract requirements and bringing in experts early? Are geotechnical baseline reports helpful? How do you recognize potential problems early?**

The two branches of engineering in which rock mechanics plays a significant role are mining and civil engineering. The role of geotechnical engineers is quite different in these disciplines and it is worth discussing them separately.

In mining projects, the roles of geologists and geotechnical engineers are clearly defined, with geologists involved in finding and defining the deposits and the surrounding rock masses and mining/geotechnical engineers designing the mining process are dealing with problems. Large mining companies have a well-defined approach to staffing, and it is not unusual for these companies to have geologists and geotechnical engineers in head office and on individual mines. These specialists identify problems and, when necessary, use consultants or government research organizations to assist in solving these problems. However, small mining companies seldom have these resources. Therefore, the approach to geotechnical engineering generally involves bringing in a specialist or consultant when problems of stability or production can no longer be ignored. With little background in the project, it is difficult for these specialists to operate effectively and there are no obvious solutions to this situation.

Civil engineering projects frequently involve the hiring of individual consultants and/or specialist companies to deal with geological and geotechnical problems. When the project "owner" is a national hydroelectric or transportation organization, the head office generally has good contacts with specialists, with whom they have been associated for many years. The process of assigning these specialists to new projects is simple. They are frequently brought into the project during the very early stages of project definition. However, this scenario does not apply to most civil engineering projects in which owners or project managers may have very little experience dealing with geological and geotechnical problems or with the process of finding suitably qualified consultants or staff members. In these situations, specialists tend to be appointed when a problem has developed and where the problem of finding an experienced and well-qualified specialist can be very difficult. Most of these projects are operated on a contractual basis. When a specialist is brought in and recommends significant changes which require modification of the contracts, this can be a very difficult process, sometimes requiring very expensive re-negotiation or even cancellation of the contracts.

Geotechnical baseline reports can be very useful when they are used on large projects where a formal process of contract and resource management is in place. In my experience of consulting in about 30 countries around the world, I have only seen geotechnical baseline reports used in a very small percentage of the projects in which I have been involved. They are a North American development which is unknown in most countries.

How do you recognize potential problems early? You engage someone who has hands-on experience dealing with the type of problem that you are facing and pay attention to their recommendations. The real problem is where do you find such an individual? The current international situation for the training of rock engineers is inadequate. Many Mining and Civil Engineering departments in universities offer geotechnical courses, but many of these courses, particularly in civil engineering, are limited to soil mechanics. Where rock mechanics courses are provided, it is unusual to find academics who have hands-on consulting experience which would enable them to deal with some of the problems that I have outlined in my presentation. It is difficult for me to write this statement but, having been an academic for 15 years at Imperial College in London and at the University of Toronto, I consider myself to be well qualified to offer an opinion. I have no simple solution to offer.

4. **Can we apply Barton Q-system classification to slopes?**

The Barton Q system was developed with an emphasis on tunnelling, but there is absolutely no reason why it should not be used for rock slope problems. Barton discussed the wider uses of his criterion in a 2013 paper, referenced at the end of this answer. I have no problems with this paper in which Barton sets out the assumptions, derivations and uses of his equations. As with the use of any classification, the user must be fully aware of the assumptions and the meaning of equations and their components in applying the classification to a specific problem. The user must accept full responsibility for the answers obtained and their application to specific problems.

Barton, N. Shear strength criteria for rock, rock joints, rockfill and rock masses: Problems and some solutions. Journal of Rock Mechanics and Geotechnical Engineering, 5 (2013) 249-261.

5. **Can we calibrate disturbance factor for different rock types?**

The disturbance factor D in the Hoek-Brown criterion is a very crude attempt to provide some adjustment of the influence of blasting on rock mass strength. It is particularly important when considering large rock mass volumes, such those involved in open pit mining, where massive blasts are used to mine the rock. Generally, hard brittle igneous rocks are more severely damaged by blasting than softer sedimentary rocks. However, this crude distinction is as far as I would go in trying to quantify D for different rock types. A more important consideration is the overall geometry of the excavation and the purpose and design of the blast itself.

I have attempted to discuss some of the issues of blast design and damage in a chapter entitled, Blasting Damage in Rock, in my notes on Practical Rock Engineering at: https://static.rocscience.cloud/assets/resources/learning/hoek/Practical-Rock-Engineering-Chapter-16-Blasting-Damage-in-Rock.pdf.

6. **Can you please comment on the significance of in-situ stress regime on pit wall stability?**

In general, in situ stresses are ignored in the design of open pit mine slopes, particularly for large mines. This is because the predominant stresses are vertical, due to the self-weight of the rock mass. Therefore, many analyses are carried out using two-dimensional limit-equilibrium methods. However, when the geometry of the pit is complex or when specific stress-related problems in the surrounding rock mass need to be considered, full three-dimensional finite element or finite different analyses have been used. I have discussed one such situation in my presentation, in which I described the Chuquicamata mine slope stability analysis and conveyor transfer chamber design in a paper entitled: Hoek-Brown failure criterion and GSI – 2018 Edition, Journal of Rock Mechanics and Geotechnical Engineering, 11(3) June 2019, Pages 445-463.

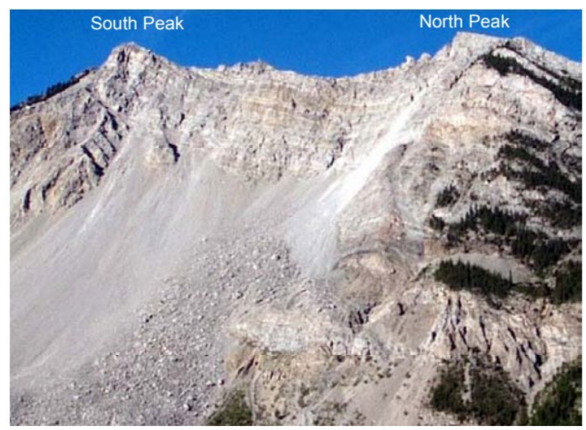
7. **Could the probability of large landslides like the Vajont Dam be predicted by geological studies? Is it possible to predict such large mass movements with field and laboratory studies?**

The answer is yes, but the problem is one of logistics.

The photograph on the right was taken in 2013 and illustrates the status of the Chuquicamata open pit mine in Chile. When this photograph was taken, the pit was approximately 1 km deep, 4 km long and 3 km wide and the slope angle at the bottom of the pit, on the right-hand side, is 53 degrees. The main ore-bearing fault can be seen running diagonally across the left-hand side of the pit.

The geotechnical program, associated with the design of the pit, was initiated in 1992. It included laboratory testing of all the rock types occurring in the pit, diamond core drilling for both ore body definition and geotechnical information and mapping of exposed rock on benches. The geotechnical data base included 185 km of core drilling and 196 km of bench mapping.

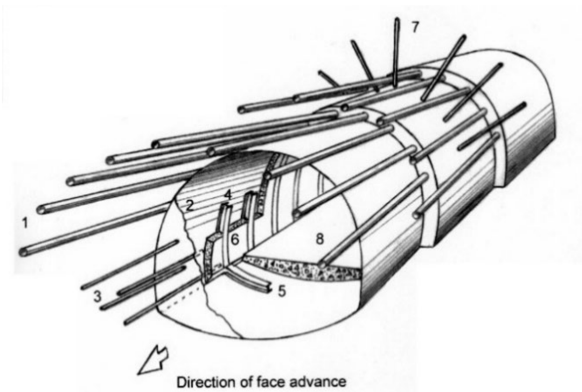
One of Canada's largest rockslides occurred on April 29, 1903, on Turtle Mountain in the Crowsnest Pass, which is approximately 250 km south of Calgary, in southwestern Alberta. Approximately 82 million tons of rock buried a portion of the town of Frank, killing more than 70 people. The photograph on the right shows this catastrophic failure of Turtle Mountain which is known as the Frank Slide. Since then, various government groups, universities, and geotechnical consulting companies have been studying the potential for a second slide.



Read, R.S. 2003. A framework for monitoring the South Peak of Turtle Mountain - the aftermath of the Frank Slide. In Proc. 3rd Canadian Conference on Geotechnique and Natural Hazards, Edmonton, Alberta, Canada June 9 - 10, 2003. pp 261-268.

Hence, while investigation, interpretation and possibly prediction of a land slide is possible, the scale and the cost of such an investigation would be very large. An interesting discussion on the possibility of a wide scale landslide identification program can be found at the following website: https://www.asc-csa.gc.ca/eng/blog/2016/11/24/detecting-landslide-risks-on-a-wide-scale.asp.

8. Could you please explain the effectiveness of forepoles and how you selected the diameter and spacing (intervals) of the forepoles?



This drawing shows the installation of support in a weak rock tunnel, with the use of forepoles (1) and face dowels (3) to provide support for the advancing face. The forepoles are typically 4 to 6 cm diameter steel pipes, usually with small holes drilled along the pipe to permit grout injection after placement. In the case illustrated, the tunnel span is 8.5 m and the forepoles are 12 m long. The lateral spacing of the forepoles is governed by the nature of the rock mass surrounding the tunnel. For interlocking rock blocks, the forepole spacing would be 3 to 5 times the average block size. For fine grained material, the spacing would be close enough to support the rock between them. This is best determined by trial and error spacing as the tunnel advances. When used, the fiberglass face dowels have a similar spacing to the forepoles and may be 8 to 12 m long. Both the forepoles and the dowels should have an overlap of about one-third of their length at each transition step. The other support elements illustrated are installed behind the face, as the tunnel advances. The support system illustrated has been used on many tunnels, particularly in Europe, and proved to be very effective in supporting the rock. When the correct equipment is used to install the forepoles and dowels, the support installation process can be very effective and good tunnel advance rates can be achieved. An example of forepole drilling is illustrated in the video included in https://www.youtube.com/watch?v=6hTBzyyj-DE.

9. Should we take seismic effects into account in Deep Tunnels with large overburden?

Experience in tunnels in active seismic regions has shown that earthquakes have a minimal impact on the stability of the rock mass surrounding the tunnel. Overall movement of the rock mass will dislodge loose equipment in the tunnel but, because the rock mass moves as a body, there is little shearing movement in the rock mass itself. Note that this does not apply to very shallow tunnels, with depth less than 3 to 5 tunnel diameters, since surface damage can propagate downwards and interact with the tunnel structure.

10. Do you think that the Hoek-Brown criteria could get better (can be improved upon)?

The Hoek-Brown criterion was developed as one component of an industry-funded research project on underground excavation design, carried out at the Imperial College of Science and Technology in London. This project resulted in the publication of the book: Hoek E.; Brown E.T. (1980). *Underground Excavations in Rock*. London: Institution of Mining and Metallurgy. A comprehensive discussion on the criterion can be found in the following paper: Eberhardt, E. The Hoek–Brown Failure Criterion. *Rock Mech Rock Eng* **45,** 981–988 (2012). https://doi.org/10.1007/s00603-012-0276-4.

In the following chapter: https://www.rocscience.com/assets/resources/learning/hoek/Practical-Rock-Engineering-Chapter-11-Rock-Mass-Properties.pdf, I have stated that: “Given the inherent difficulty of assigning reliable numerical values to rock mass characteristics, it is unlikely that ‘accurate’ methods for estimating rock mass properties will be developed in the foreseeable future. Consequently, the user of the Hoek-Brown procedure or of any other equivalent procedure for estimating rock mass properties should not assume that the calculations produce unique reliable numbers. The simple techniques described in this section can be used to explore the possible range of values and the impact of these variations on engineering design."

This chapter gives several examples of how the Hoek-Brown criterion was used in assigning rock mass properties and how these properties were used in sensitivity studies on practical design problems. I have now retired completely from all professional activities and research and so I will not be contributing any further to this topic. However, I am certain that there are others who will accept the challenge of improving existing failure criteria or developing new ones.

11. Do you think InSAR technology is a good replacement for other mining monitoring systems in open-pit mines to reduce costs and increase measurement accuracy?

Interferometric Synthetic Aperture Radar (InSAR) is a technique used to measure displacements over time, based on the comparison of multiple radar images. These images are generally produced by satellites for displacements of the scale of rock engineering problems discussed in this document. I have no personal experience in the use or interpretation of InSAR images, but my impression is that excellent results have been obtained from their use and that the interpretation of these images is likely to improve with time. I have no doubt that this technique will be used increasingly as a supplement or replacement for conventional mine monitoring systems.

12. In the last 30 years, tunnel diameters have doubled. What are the differences in the challenges with small and large tunnels?

The principal difference between small and large tunnels in rock masses is that the volume of rock that can fail and result in deformations or fallouts is significantly larger for large tunnels. This means that simple traditional support techniques such as steel-sets, rockbolts, mesh and shotcrete may not be adequate for the control of stability in 20 m span tunnels or caverns as they were in 5 m diameter tunnels. Consequently, far more attention must be paid to the failure processes involved in the rock surrounding large tunnels and enhanced support techniques, such as the use of forepole as described in my answer to question 8, or long pre-stressed cables rather than rockbolts must be utilized. Fortunately, excellent results have been obtained in using these newer techniques and it is likely that improvements in techniques such as grouting, and the use of deformable support systems, will improve with time.

This brings me back to the answers that I provided for question 3 in which I have written: How do you recognize potential problems early? You bring in someone who has hands-on experience in dealing with the type of problem that you are facing and pay attention to their recommendations.

13. Is it possible for the geotechnical engineer to consider scale effects on rock masses and defects strengths more accurately? Can that be applied to large-scale rock slope designs?

Scale effects related to the strength and deformation characteristics of large-scale rock masses are extremely difficult to quantify accurately because calibration tests are practically impossible to execute. The basis for the estimates made in developing the scale effect methodology included in publications on the Hoek-Brown criterion is based upon back analysis of practical problems, including large scale rock slopes. I cannot visualize how the accuracy of the available values can be improved significantly at this time.

14. What method or equipment did you use to measure in situ stress? How reliable were they?

In situ rock stresses can only be measured with some degree of accuracy in intact rock in which underground excavations have been excavated. An excellent summary of the methods used was published in 2003 in the following paper:

Ljunggrena,C, Changa,Y, Janson,T and Christiansson, R. An overview of rock stress measurement methods *International Journal of Rock Mechanics & Mining Sciences* 40 (2003) 975–989.

A table from this paper, summarizing the methods of in situ stress measurement, with advantages, limitations and suitability is reproduced below:

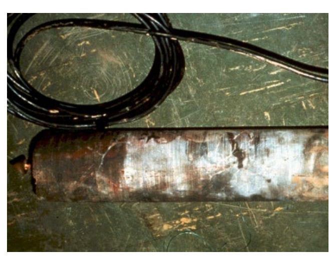
Stress measurement methods and key issues related to their applicability

Method	2D/3D	Advantages	Limitations	Suitable for
Overcoring	2D/3D	Most developed technique in both theory and practice	Scattering due to small rock volume. Requires drill rig	Measurements, depth down to 1000 m
Doorstopper	2D	Works in jointed and high stressed rocks	Only 2D. Requires drill rig	For weak or high stressed rocks
Hydraulic fracturing	2D	Measurements in existing hole. Low scattering in the results. Involves a fairly large rock volume. Quick	Only 2D. The theoretical limitations in the evaluation of σ_H. Disturbs water chemistry	Shallow to deep measurements. To obtain stress profiles
HTPF	2D/3D	Measurements in existing hole. Can be applied when high stresses exist and overcoring and hydraulic fracturing fail	Time-consuming. Requires existing fractures in the hole with varying strikes and dips	Since the method is time consuming, it is of most interest in situations where both overcoring and hydraulic fracturing fail
Core discing	2D	Existing information, which is obtained already at the drilling stage	Only qualitative estimation	Estimation of stress at early stage
Borehole breakouts	2D	Existing information obtained at an early stage. Relatively quick	Restricted to information on orientation. Theory needs to be further developed to infer the stress magnitude	Occurs mostly in deep holes
Focal mechanisms	2D	For great depths	Information only from great depths	
Kaiser effects	2D/3D	Simple measurements	Relatively low reliability	Rough estimations
ASR/DSCA/RACOS	2D/3D	Usable for great depths	Complicated measurements on the micro-scale, sensitive to several factors	Estimation of stress state at great depth
Back calculation	2D	Quick and simple. High certainty due to large rock volume	Theoretically not unique solution	Can only be used during construction of the rock cavern
Analysis of geological data	2D/3D	Low cost	Very rough estimation, low reliability	At early stage of project

I was first involved in rock stress measurement in 1957 when I was employed as a research engineer in the South African Council for Scientific and Industrial Research. One of my colleagues, Eric Leeman, had developed a three-dimensional stress measuring tool which could be glued into a borehole and then over-cored to de-stress the rock and determine the stresses to which it had been subjected. The process of installation and over-coring is illustrated in the following series of pictures:


Installation stress cell


Detail of stress cell


Over-cored rock sample


Stress cell bonded in rock

Leeman, E. R., Hayes, D. J. (1966): A Technique for Determining the Complete State of Stress in Rock Using a Single Borehole, Proc. 1st. Cong. ISRM (Lisbon), Vol. II, pp. 17–24.

An improved stress cell, described by Worotnicki and Walton in 1976, was developed by the Australian Commonwealth Scientific and Industrial Research Organisation:

Worotnicki, G., Walton, R. J. (1976): Triaxial Hollow Inclusion Gauges for the Determination of Rock Stress in situ. Proc. ISRM Symp. on Investigation of Stress in Rock and Advances in Stress Measurement. Supplement, pp. 1–8, Sydney.

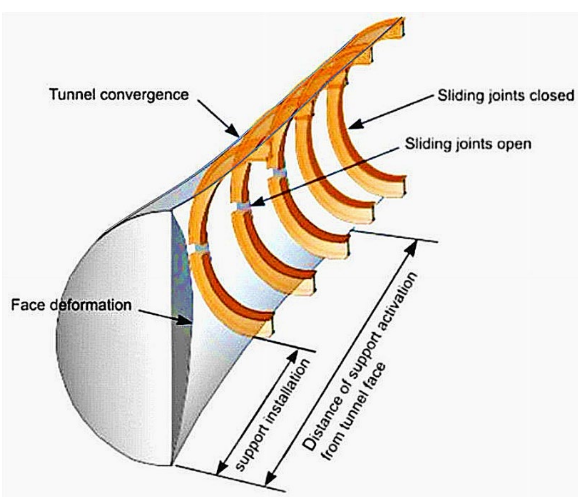
A new method has been developed to measure the induced stresses in the vicinity of an excavated surface and further to use these results to interpret the in-situ state of stress at the excavation-scale. The new linear variable differential transformer (LVDT) cell method, described in the following paper, produces a 3D stress state.

Siren, T, Hakala, M & Perras, MA 2017, Reliable in situ rock stress measurement from the excavation surface, in M Hudyma & Y Potvin (eds), *UMT 2017: Proceedings of the First International Conference on Underground Mining Technology*, Australian Centre for Geomechanics, Perth, pp. 477-486.

In-situ stress measurement is a necessary process, particularly in the design of large underground excavations, but it involves the use of specialized equipment by contractors who have the skills necessary to carry out the field work and interpretation of the results. It is important that provision should be made in design and construction contracts for this work to be done as early as possible during the site investigation and early excavation stages of a project.

15. Can you please provide examples of yielding support?

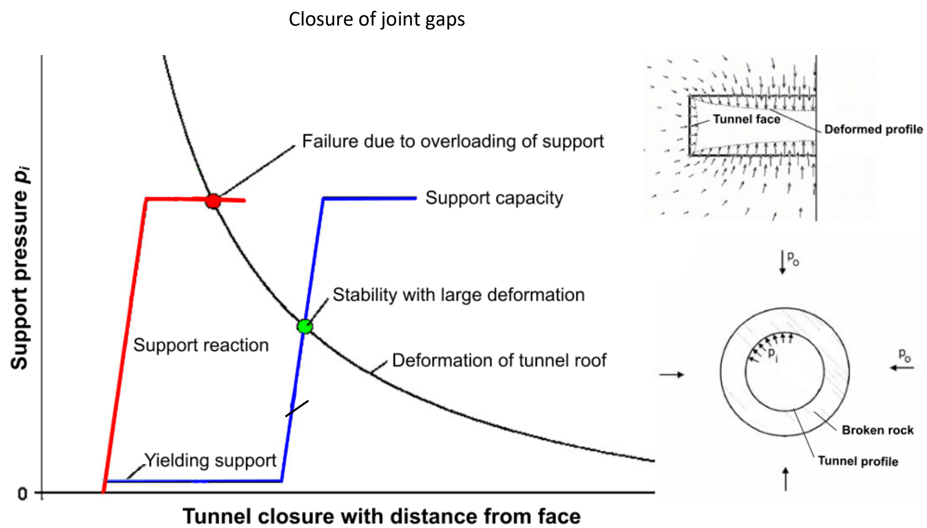
A typical yielding support installation is illustrated in the following figure. This was used in the Yacambu-Quibor project in Venezuela and the reasons for its use as well as the design and implementation are fully described in the following paper: Hoek, E & Guevara, R. 2009. Overcoming squeezing in the Yacambú-Quibor tunnel, Venezuela. *Rock Mechanics and Rock Engineering*, 42(2) 389 – 418.



In the case illustrated above, the yielding element is an empty space and there is very little resistance to movement except when the ends of the set segments come into contact. This design is useful when a set diameter has been chosen for the final tunnel. Many other gap filling devices have been used in yielding elements. These include hydraulic jacks, hydraulic cylinders with controlled pressure release fitting, wood blocks which fail under load and cans with different wall thicknesses. All these yielding elements play the same role and the choice of which one to use depends on local circumstances and the opinion of the tunnel designer.

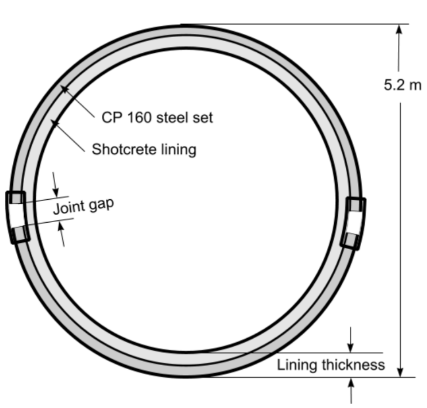
16. For yielding support, how is the appropriate gap estimated? Does this need to be done case by case?

The gap is estimated, case by case, based on an analysis such as that described in the paper: Hoek, E & Guevara, R. 2009. Overcoming squeezing in the Yacambú-Quibor tunnel, Venezuela. *Rock Mechanics and Rock Engineering*, 42(2) 389 – 418.



When a tunnel is excavated in a rock mass which is not strong enough to withstand the loss of support which occurs when the rock mass inside the tunnel is removed, the surrounding mass will fail progressively. This failure causes inward movement, or squeezing, of the rock mass surrounding the tunnel as shown in the illustration on the right-hand side of the figure.

The plot of tunnel closure with distance from face for different support pressures illustrates the role of yielding support delaying the activation of support in a tunnel in squeezing ground.



An example of a yielding support system installed in a 5.2 m diameter tunnel is illustrated here. Two joint gaps, each 30 cm wide will reduce the tunnel diameter to approximately 5m when the gaps are completely closed. The behavior of this support system is represented by the blue line in the plot of tunnel closure versus support pressure. Assuming that the steel set, with fully open yielding joint gaps, is installed within the first two metres from the tunnel face, progressive closure of these gaps would occur immediately and would continue until the gaps had closed. At this point, the set would react to the load imposed on it by the closure of the tunnel. Stability, shown by the green dot, occurs when the pressure required to prevent further closure of the tunnel is matched by the support provided by the steel sets.

Long term stability of the tunnel requires the installation of a shotcrete or concrete lining within the tunnel, as illustrated in the drawing.

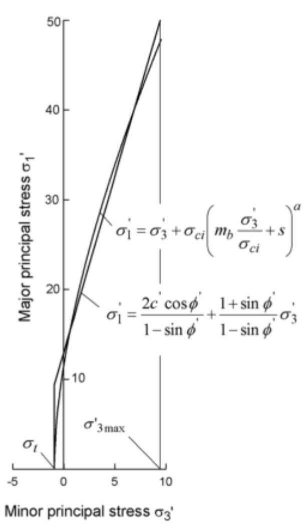
17. In what area of rock engineering do you see the most potential for innovation in the future?

Increasing population densities in cities around the world will require the construction to foundations, road and rail lines and cuts and tunnels to accommodate them. Mines and quarries will become larges and deeper in order to produce the minerals required by our civilization. Dams and hydroelectric facilities, both surface and underground, will increase in number and size. Hence, greater demands will be placed on the design and construction of foundations or excavations and on the processes used in the creation of these structures.

In my opinion, the techniques for the design and construction of most of these facilities are already available and the emphasis needs to be placed on increases in efficiency in the use of these tools and on the training of geologists, engineers and technicians to use these tools. This brings me to a question, already discussed in Question 3, which I will repeat here.

The real problem is where do you find such an individual? The current international situation for the training of rock engineers is inadequate. Many Mining and Civil Engineering departments in universities offer geotechnical courses, but many of these courses, particularly in civil engineering, are limited to soil mechanics. Where rock mechanics courses are provided, it is unusual to find academics who have hands-on consulting experience which would enable them to deal with some of the problems that I have outlined in my presentation. It is difficult for me to write this statement but, having been an academic for 15 years at Imperial College in London and at the University of Toronto, I consider myself to be well qualified to offer an opinion. I have no simple solutions to offer.

18. Can the Mohr failure envelop criteria from direct shear testing and from UCS and triaxial test still be used for the design of tunnels and slope stability?



The answer to this question is a very distinct yes, as illustrated in the figure on the left.

The critical issue is the minor principal stress σ'_3 range over which the two criteria are used in parallel. In the early days of the development of the Hoek-Brown criterion, two-dimensional slope stability analyses were frequently carried out by assuming a circular failure surface and calculating the minor principal stresses (σ'_3) at intervals spaced along the failure surface. The corresponding values of c and f were then calculated for each of these values and used to define the shear strength of the base of each slice. As can be imagined, this was a tedious process, and many slope stability programs now offer the alternatives of using either the Mohr Coulomb or Hoek Brown criterion for factor of safety calculations.

A detailed discussion on this issue can be found in a chapter on rock mass properties in Practical Rock Engineering on the Rocscience website at: https://www.rocscience.com/assets/resources/learning/hoek/Practical-Rock-Engineering-Chapter-11-Rock-Mass-Properties.pdf

19. During tunneling in extremely poor rock mass conditions (Q<0.01; unsupported rock stand up time is less than one hour), are the deformations or stresses of full-face excavation higher than those realized in heading and benching approaches?

In general, the stresses in the rock mass will be higher for a full-face excavation, but the distribution of these stresses is more favourable for overall support design and stability since there are no abrupt changes as in the case of top heading and bench. However, the difference in stress magnitudes is not as important as the timing of the excavation and support procedures. Top heading and bench excavation allows rapid installation of support and, where standup times are very short, this may be essential in order to protect the workers at the face.

In the case of the 5 m diameter Yacambu-Quibor tunnel in Venezuela, discussed in question 16, it was not possible to excavate a full-face tunnel in the worst rock conditions in which immediate installation of support, very close to the advancing face, was required to protect the miners. The time required to excavate a full-face tunnel was simply too long to allow adequate support to be installed. In this case, the difference in stresses was of secondary importance.

20. For a rock mass that has GSI >65, is the Hoek-Brown Criterion still applicable? If not, what are the tools/criterion recommended for such rock masses?

The Hoek-Brown criterion, originally developed to describe the failure of intact rock specimens, is certainly applicable to rock masses with GSI>65. An example of a 12m span top heading for a tailrace tunnel, excavated in massive gneiss with a GSI rating of 75, is illustrated in the photograph below. The depth of this tunnel is approximately 200 m below surface and only occasional rockbolts were used to support structurally defined slabs and wedges exposed in the excavation boundary. No stress analyses were required for these tunnels but, at greater depth, simple two-dimensional numerical models could be used to determine potential overstressing and support requirements.

Core in massive gneiss

Top heading for a 12 m span tailrace tunnel in the Rio Grande project in Argentina

Intact rock strength	σ_{ci}	110 MPa	Hoek-Brown constant	m_b	11.46
Hoek-Brown constant	m_i	28	Hoek-Brown constant	s	0.062
Geological Strength Index	GSI	75	Constant	a	0.501
			Deformation modulus	E_m	45000 MPa

21. From all the case studies that have been shown today, there is insufficient geological and geotechnical investigation during the early stage of the project. Often the challenge is the lack of commitment from the client/owner of the project to invest in geotechnical investigation. What are your tips to advise/persuade the project owner to do a proper investigation to avoid potential catastrophic failure in the later stage of the project?

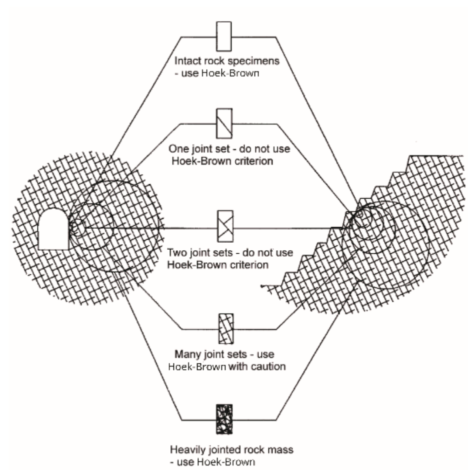
For this presentation I chose case studies which allowed me to demonstrate techniques that had been used to illustrate deficiencies in the site investigation and design process and how these deficiencies were dealt with during construction. I could have chosen an equal number of case studies that had been constructed without any of these problems because there had been individuals or teams in all the participating organizations, from the owner to the designers and contractors, who understood and implemented all the processes required for a successful project. Large, mature organizations, such as national power corporations, generally have these teams in place and it is not difficult to work with them since they understand the requirements or are receptive to recommendations on the need for adequate site investigation, design review and construction control. On the other hand, it is not unusual to have start-up or single project organizations who do not have such teams in place and who have problems understanding the need for adequate preliminary investigations of all kinds, adequate design reviews and effective control of construction processes.

When providing consulting services to organizations that do not have well established and experienced decision-making teams in place, it is important to try to establish contact with the highest decision-making individual or team in the organization, as early in the project as possible. If such a meeting can be set up, it is important that your organization should be represented by someone with the knowledge and experience to be able to persuade your client that geotechnical site investigations are essential and, if necessary, to present a detailed proposal and cost estimates for such investigations. For such meetings it is sometimes useful to bring in a senior consultant, with credible design and field experience in similar projects, to assist you in making your presentation, which should be well prepared but as informal as possible to allow discussion during the presentation.

I have been involved in many of these meetings during my long consulting career. Some of the meetings have been difficult and occasionally, completely unsuccessful. On the other hand, those that have worked out have set the path for realistic and successful geotechnical site investigation, design and contract supervision projects extending over many years and, more importantly, successful outcomes for the project owners.

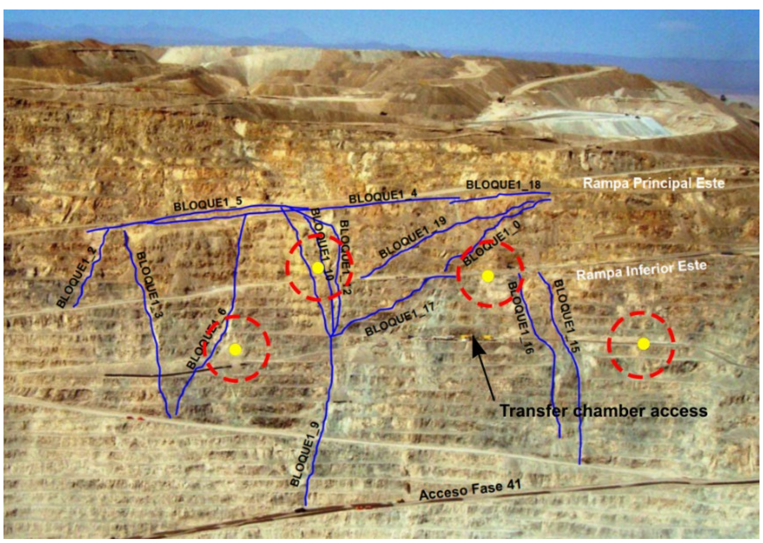
22. From your vast experience, how did you overcome numerical modeling challenges in complex geology with shear zones?

The Hoek-Brown criterion assumes that the rock to which it is applied is isotropic and homogeneous. It was derived from the results of triaxial tests on rock samples which were free from significant discontinuities. It should not be used for the analysis of anisotropic samples which contain through-going failure surfaces or families of parallel discontinuities.

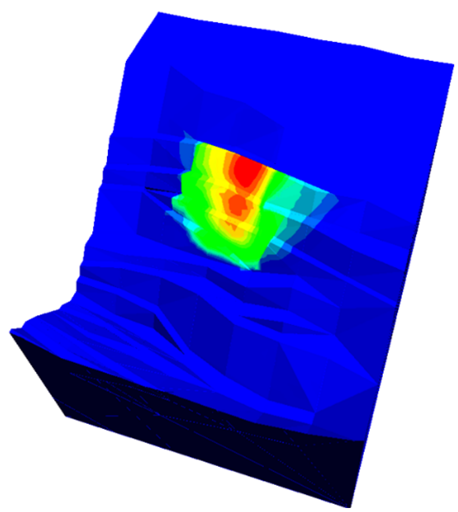


At the other end of the spectrum is a rock mass which contains numerous blocks of rock resulting from the intersection of several families of joints of similar discontinuities. As illustrated in the diagram opposite, the Hoek-Brown criterion can be used on this "rock-mass", with appropriate reductions in the strength and deformation characteristics of the individual rock blocks.

In some circumstances, it is appropriate to utilize the Hoek Brown criterion to define the properties of a large homogeneous rock mass and to superimpose individual through-going discontinuities such major joints, shear zones or faults. An example of this type of analysis is given below.



East face of the Chuquicamata open pit mine in Chile, showing major discontinuities in a homogeneous rock mass. Yellow dots mark locations of mirror targets for displacement monitoring.



Displacement contours in a 3DEC numerical model of this slope.

The example, illustrated on the previous page, is of a three-dimensional numerical analysis carried out on the East wall of the 980 m deep Chuquicamata open pit mine in northern Chile. A transfer chamber for a conveyor system, used to transport ore from an in-pit crusher at the base of the pit to the processing plant on surface, is located in the rock mass behind this face at a point marked on the photograph. The numerical analysis was carried out because of concerns for the stability of the transfer chamber when subjected to rock mass movements resulting from progressive mining at the base of the pit.

While the rock mass forming this face consists of interlocking blocks of rock, separated by joints with a spacing of 2 to 5 m, the scale of the slope justified the assumption that this rock mass could be treated as homogeneous and that it qualified for the application of the Hoek-Brown criterion. The major structural features, defined by the blue lines on the slope face, were treated is planar discontinuities, with their shear strength properties defined by the Barton shear failure criterion.

I was a member of a geotechnical and mining consulting board, reporting to mine management, and I supervised and participated in this analysis project. In 2012 the construction of the 3DEC model and the execution and interpretation were carried out by Pedro Varona of Itasca and Felipe Dur of the Chuquicamata geotechnical team. An example of the displacement contours calculated for the rock mass adjacent to the conveyor transfer station, with recently installed reinforcement, is given in the illustration on the bottom right of the previous page. The outcome of this analysis was that there would be no stability problems in the rock mass surrounding the cavern for the remaining years of operation, until 2018, when the open pit would cease operations and underground block caving mining would commence. This transition has now occurred successfully, and the conveyor transfer station is no longer in operation.

23. Regarding the example of a cavern where shale degradation was prevented by immediate shotcreting, how was the potential for swelling of shale and swell pressure on the support evaluated? Can you speak to excavation failures due to swelling and the best remedial measures (curved geometry, thick lining, anchors, etc.)?

The Drakensberg underground powerhouse today, 41 years since the project was designed and built between 1974 and 1981. (https://web.archive.org/web/20080509142138/http://www.eskom.co.za/about/providingelectricity/powerstations/drakensbergpumped_content.html)

The Drakensberg Pumped Storage Scheme cavern in which shotcreting was carried out, in the mid 1970s, to protect the relatively weak sedimentary rocks from deterioration due to moisture change. This decision was based on field observations of rock core specimens and the excavated surfaces of the cavern walls. It was concluded that the only remedial measures required involved protection of all exposed surface from drying out and that high quality shotcreting would suffice for this purpose. No studies on stress-induced rock deformation or on swelling pressures were carried out.

In designing underground excavations, deformations due to overstress or swelling must be analyzed in terms of their impact on the performance of the excavation. In some cases, such as highway tunnels, fixed interior dimensions are specified to accommodate vehicles and these dimensions must be adhered to. In other cases, such as the Yacambu-Quibor Tunnel in Venezuela, discussed in Question 16, large deformations were required to allow the support system to function properly. Fortunately, many excellent numerical tools are now available to permit these calculations to be performed.

24. Did you get the chance to work with Dennis Laubscher? What are his experiences working with MRMR?

I met Dennis Laubscher in about 1963 when he contacted the South African Council for Scientific and Industrial Research with a request for in situ stress measurements to be carried out in the Shabani Asbestos Mine in Southern Rhodesia (now Zimbabwe). I visited the mine, as the head of the CSIR rock mechanics division, to discuss this project with him. We met on a few occasions after this, but we did not work on many projects together.

One of my colleagues at the CSIR was Richard (Dick) Bieniawski and he took over my position as head of the rock mechanics division when I moved to Imperial College in London in 1966. We remained in contact for many years, and I followed his development of his Rock Mass Rating (RMR) over the years. The original classification was published in 1973 in the following paper: Z. T. Bieniawski, "Engineering classification of jointed rock masses," *Trans. South African Institute Civil Engineering*, **15** (1973). Similarly, one of the doctoral students at Imperial College was Nicholas (Nick) Barton and I have followed the development his Q system since it was published in the following paper in 1974: Barton, N., Lien, R. & Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. Rock Mechanics. 6: 4: 189-236.

Dennis Laubscher published his Mining Rock Mass Rating (RMR) in 1977 in the following paper: Laubscher, D.H. 1977. "Geomechanics classification of jointed rock masses - mining applications". *Transactions of the Institution of Mining and Metallurgy, Section A, Mining industry*. London. **86**: 1–8. This classification is an extended and modified version of Bieniawski's RMR.

I have used all these classifications in my own consulting work for many years. Each has advantages and disadvantages, and it is not unusual to see two of them used on the same project.

25. For the large cavern projects you presented, which of them had the highest vertical stresses? What support system did you use? At what depths would you be comfortable excavating large caverns in strong rock?

The photograph on the left illustrates the underground powerhouse cavern for the Nathpa-Jhakri hydroelectric project in northern India. This was one of the projects that I worked on as a member of the consulting board, reporting to the project management and the World Bank, who were responsible for some of the project funding. This project is typical of the numerous caverns that I have worked on, and it provides useful information in answer to this question.

The cavern lies within quartz-mica schist under a rock cover of about 300m. The 185.4 m long x 22.0 m wide x 48.4 m high excavation was mined from a central drift of 7 m diameter along the long axis at crown level and widened into full cavern span. Bench lowering was done in 9 stages. The rock mass for the major portion was found to be Fair with short reaches of Poor to Very Poor, based on Barton's Q classification data from the central exploration drift. The vertical principal stress was 7.6 MPa, with horizontal stresses of 10.7 and 5.12 MPa. The final support system comprised alternate rows of 6m and 8m long rock bolts at 1.5m centre to centre and 100mm of shotcrete with weld mesh for the roof. In addition, some 11 m long bolts were installed to take care of predicted deep wedges. The walls are by alternate rows of 7.5m and 9m bolts with the middle third supported by 9m and 11m long bolts, staggered at 3m x 1.5m spacing with 100mm thick shotcrete.

I was also marginally involved in the design of the 33m diameter x 30.8 m high cavern at a depth of 2070 m below surface in the Creighton nickel-copper mine in Sudbury, Ontario, Canada. The host rock is a blocky to massive granite-gabbro formation with an estimated GSI rating of 75 to 85, an unconfined uniaxial compressive strength of 240 MPa and a Young's modulus of 60 GPa. The in-situ stresses in this location are σ_1 = 95.2 MPa, σ_2 = 65.9 MPa and σ_3 = 51.4 MPa (vertical). Support consists of long tensioned and grouted steel cables in the surrounding rock mass with wire mesh and shotcrete to retain small surficial instability during construction. This cavern, which is concrete lined, houses the Sudbury Nutrino Observatory, which operated between 1999 and 2006. The underground laboratory was enlarged as a permanent facility and now operates multiple facilities as SNOLAB.

Reference: Castro, L.A.M.; McCreath, D.R. and Oliver, P. (1996). Rock Mass Damage Initiation Around the Sudbury Neutrino Observatory Cavern. *2nd North American Rock Mechanics Symposium,* Aubertin, Hassani & Mitri (eds), Balkema, Montreal, 2: 1589-1595.

26. Sometimes sites become complacent with updating initial numerical models as project progress. How do you think one should effectively account for the parameter variability in models with time?

Numerical analysis is involved in many levels in geotechnical engineering. At the highest level it is used for the design of major structures such as dam foundations, underground powerhouses or tunnels and these are basically one-off analyses which may take many months to complete. Generally, they start with the best assumptions available for material properties, loading and any other relevant factors and the output is a design or several designs which can then be presented to the project owner for discussions on the best solution for the particular needs of the project. Once a design is decided upon, the model will be used to investigate more detailed issues and to assist with practical construction decisions as the project progresses. There is no room for complacency at any time in this process and it is the responsibility of the designer or consultant to ensure that the process of modelling and input selection is kept up to date.

At the other end of the spectrum are analyses that are run on a routine basis in projects such as designing the benches and slopes for an active open pit mine or the lining design for a long tunnel. It is essential that these analyses should be reviewed and updated on a regular basis, particularly if the running of the analyses has been delegated to a junior engineer or technician. A senior member of staff should be allocated the duty to review the analyses and the choice of input information on a regular basis.

In my experience, the best way to account for parameter variability with time is to run a sensitivity analysis in which significant strength parameters, particularly cohesive strengths are reduced progressively. This will give the user an understanding of the significance of strength reduction in their analysis. In some cases, the changes may be minimal and can be ignored or the parameters reduced to a lower but credible level. Where changes in the analyses are significant, careful thought should be given to how and why and by how much input parameters can vary over the life of the project and detailed analyses should be carried out to determine what steps can be taken to deal with these changes.

27. Can you comment on the derivation of Disturbance Factor, D? Would you equate a "D" value of 1.0 with post-peak strength?

The disturbance factor D is a very crude parameter that was originally introduced to provide a basis for evaluating blast damage in large open pit mines. It is very important to remember that it should only be applied to the rock mass in the immediate vicinity of a blast and not to the entire rock mass, when used for such problems.

Gradually, the factor D became known as a Disturbance Factor rather than a Blast Damage Factor. It is now used to reduce strength factors for deterioration of the rock mass due to weathering, long term creep displacements and any other factor that may result in property changes. In my own work, I tend to use it in sensitivity analyses when choosing rock strength and deformation parameters to use in a design. Generally, I assign a value of D = 1 to the free surface, which is adjacent to the disturbed zone, and I then scale the value of D to zero at the outer boundary of this zone. For simplicity's sake, I normally use a linear reduction of D with distance, but I have had many discussions on whether a curved reduction should be used. Considering the very crude assumptions that must be made in defining the extent of the disturbed zone, I doubt that the use of a non-linear scaling of the reduction in D would have a significant impact.

I have not associated D = 1 with post-peak strength, but this may be as good an estimate as any other assumption.

28. What do you think is the future for numerical modelling in rock and soil mechanics?

I started my career in rock mechanics in 1958 when all my calculations were carried out using a slide rule. Computers were only in their infancy at the time and the use of available computers was limited to a few specialists, which excluded ordinary people like me. It was only in the mid-1960s that computing facilities became more generally available, and this generally involved a walk to the computer centre, carrying a pile of cards which had been laboriously punched with input data on the problem being analyzed. In 1966 I was appointed as an academic in the Imperial College of Science and Technology in London and, over the next 9 years, I was able to work with a group of outstanding graduate students exploring the potential for numerical analysis in rock mechanics. In 1983 I was able to purchase my own personal computer and learn how to input my own data, using the "Basic" programming instructions. Simple circular failures on slopes or wedge failures in tunnels were now possible.

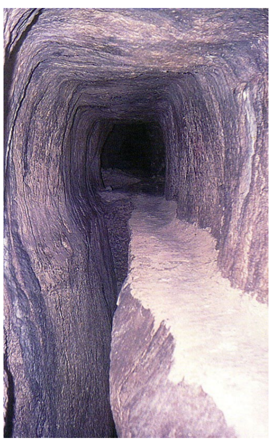
Jumping ahead to the present, we now have access to incredible computing facilities and to an array of sophisticated and very powerful programs that can be used in the analysis of almost any problem in rock or soil mechanics. The problem now is the choice of which program to use and how to avoid the chosen program running away with you. It is very easy to run incredibly fast programs and to observe very attractive and informative images that appear on your screen. However, these images are not necessarily correct, and you have to use all the knowledge at your disposal to decide whether the results can or should be used in the arriving at the solution to your problem.

While I have not yet used artificial intelligence personally, I believe that the next stage of numerical modelling in geotechnical engineering will have to involve this technology to assist users with the interpretation of sophisticated software. In the article listed below*, Xin-She Yang and co-authors have written the following introduction to an article entitled Artificial Intelligence in Geotechnical Engineering: Applications, Modeling Aspects, and Future Directions:

"Geotechnical engineering deals with materials (e.g., soil and rock) that, by their very nature, exhibit varied and uncertain behavior due to the imprecise physical processes associated with the formation of these materials. Modeling the behavior of such materials is complex and usually beyond the ability of most traditional forms of physically based engineering methods. Artificial intelligence (AI) is becoming more popular and particularly amenable to modeling the complex behavior of most geotechnical engineering materials because it has demonstrated superior predictive ability compared to traditional methods. Over the last decade, AI has been applied successfully to virtually every problem in geotechnical engineering. However, despite this success, AI techniques are still facing classical opposition due to some inherent reasons such as lack of transparency, knowledge extraction, and model uncertainty."

* Mohamed A. Shahin. Artificial Intelligence in Geotechnical Engineering: Applications, Modeling Aspects, and Future Directions, Chapter 8 of the book Metaheuristics in Water, Geotechnical and Transport Engineering, Edited by Xin-She Yang, Amir Hossein, Siamak Talatahari and Amir Hossein Alavi. 2013 Elsevier Inc. London, Waltham, MA, pp. 169-204.

29. Nowadays, most tunnelling or excavation projects use support elements. In comparison, many old projects did not use support but still remain intact or stable after hundreds of years. Is it because of their understanding of rock behaviour/features or the ground support technology they used?



Unsupported small tunnel in Greece dating from 500 BC.

Victoria project headrace tunnel on the in Sri Lanka excavated without support in high quality gneiss.

Nathpa Jhakri project headrace tunnel in northern India with support failure in a 400 m wide fault zone.

The tunnel on the left is about 2500 years old and it is located on the Greek island of Samos. It is about 1 km long and it was constructed in good quality limestone, at shallow depth without any support, to transport water to the city of Pythagoreio. In the case of this tunnel, the miners were simply using their experience in excavating a small tunnel in good quality rock. There are many tunnels of this kind all over the world and the key common factor is that they were mined in good rock at shallow depth.

The tunnel in the centre is a relatively shallow and short headrace tunnel on the Victoria hydroelectric project in Sri Lanka, constructed in good quality gneiss in 1980-1984. It is the only tunnel that I have worked on where, due to very high-quality blasting carried out by the contractor, I was prepared to agree to excavation of the tunnel without support. A concrete lining was placed in the excavated tunnel to optimize its hydraulic performance.

The tunnel on the right is the 27 km long, 10 m span headrace tunnel for the Nathpa Jhakri hydroelectric project in the foothills of the Himalayas in northern India. Conventional steel set support was used successfully, except in a major fault zone as shown in the photograph. Support by means of forepoles, as discussed in Question 8, had to be used to stabilize this section of the tunnel. Considering that additional forepole support was required for only about 400 m of faulted material, the steel set method of support, with a final concrete lining in the completed circular tunnel, is appropriate.

In answer to your question, I consider that the Greek miners certainly understood that they needed to find good rock so that they could mine the small sized shallow tunnel without support. Similarly, in the headrace tunnel in Sri Lanka, very good rock at shallow depth permitted the construction of an unsupported tunnel.

On the other hand, the use of support is justified in tunnels, such as the long Nathpa Jhakri Tunnel in India, because delays in construction due to unexpected failures are unacceptable in a major project of this kind. In fact, with the demand for tunnels to meet the needs of the increasing size and complexity of cities, the transportation routes between them and the facilities required to service them, I consider that the use of support is a necessary requirement for modern tunnelling. This support must be optimized to meet the specific needs of the rock mass in which the tunnel is being mined and also to facilitate efficient access to the construction faces in order to permit the highest possible advance rates.

30. What do you think of people who say they do not trust numerical modeling even though it is a very good method for stability analysis?

As discussed in Question 28, numerical models have evolved to a very high level of sophistication over the past 40 years. There are relatively few people who have lived and been heavily involved in this development and who fully understand the advantages and limitations of numerical modelling. An engineer or geologist with no background and experience in numerical analysis is bound to have questions and doubts about the apparent ease with which many of their younger colleagues accept the validity of answers provided by these analyses. Despite my long association with the development of numerical analysis, I sometimes have my own doubts about this heavy reliance and acceptance of the results of numerical modelling.

In my opinion, this is a problem of communication rather than of technical issues. Many technical people, who are heavily reliant upon computers for the services which they provide, have difficulty in communicating a description of what they are doing and the results they are producing to those with a different background and function. On the contrary, the individual who only use a computer to send emails and letters, is likely to question the results of a sophisticated analysis of any kind. Fortunately, I believe that this problem will disappear when the next generation takes over our roles since many of this new generation will have literally cut their teeth on computing devices of one form or another. Hopefully, they will find it easier to understand that a carefully programmed and operated computer will actually produce reliable and useful answers.

31. Extraction of stopes below 1390mL always encountered extreme ground control challenges to the point that sections had to be abandoned on several occasions as the only safe alternative. Between 1390mL and 1457mL, ore extraction has been extremely poor even with mass blasts, resulting in the mine leaving remnants between some of the blocks. The combined effect of the stoping-induced stresses and stress redistribution from the remnants has created complex loading conditions on some blocks leading to unpredictable failure mechanisms. The mine has had to modify ground support systems on several occasions to deal with changes in rock mass deformation behaviour. Destress blasting has been done on two occasions, but the intended result was not achieved. Further investigations intended to come up

with a more tactical approach of mining the blocks are still going on. In the meantime, reliable quick fixes requiring mostly ground support rehabilitation and modification of blast sequences continue to be applied. What could be the best approach to address this problem?

The scenario described in this question is unfortunately not unique to the mine on which you work. I have encountered similar problems in several mines around the world and I regret to say that there are no magic quick fixes. The only approach that I have seen to help in such situations is to bring in a very experienced practical mining engineer who has worked with similar problems and who can work with you in resolving yours. Finding such an individual is extremely difficult since very few consultants and even fewer academics will have the required hands-on experience in these specific problems and the individuals who can help are generally too busy fixing problems on their own mine.

The only advice that I can give is that you discuss this with colleagues and managers on your mine and try to ascertain whether there are mines known to them, or their friends in the mining world, where similar problems have been encountered and where some practical solutions have been found. You will not find such information in mining magazines or technical literature since the practical problem solvers are generally too busy to find time to write up their experiences. You have to find these individuals and attempt to persuade them, your management and your colleagues, that they may be able to help you.

Your statement that, "Investigations intended to come up with a more tactical approach of mining the blocks are still going on", also sounds very familiar since redesign of the blocks may be an appropriate solution, but it is also one that is extremely difficult to implement in an operating mine.

I regret that I am unable to provide a simpler solution and I wish you success in finding an individual who can help.

32. You indicated in your presentation that canopy tubes pre-support (forepoles) are effective in maintaining tunnel stability in poor ground. What is your opinion or experience in terms of the effectiveness of this support when groundwater inflows in the face are encountered?

This is not an unusual problem, and it is generally solved by drainage of the face and the rock mass above the forepoles. If you are lucky, you will have all the equipment required readily available at the heading. The adjacent photograph shows a forepole drilling machine for 12 m long 15 mm diameter steel pipe forepoles. When installed as forepoles, which typically have patterns of small holes drilled through the steel along the pipe, high pressure grout is injected into the pipe and the surrounding rock to anchor the forepole. Leaving out the grouting step turns the forepole into a drain and a few such drains can be very effective in reducing water pressures which can cause stability problems in the rock mass. In some cases, the rock mass may be stable enough that simple drillholes, without the steel pipes, will provide effective drainage.

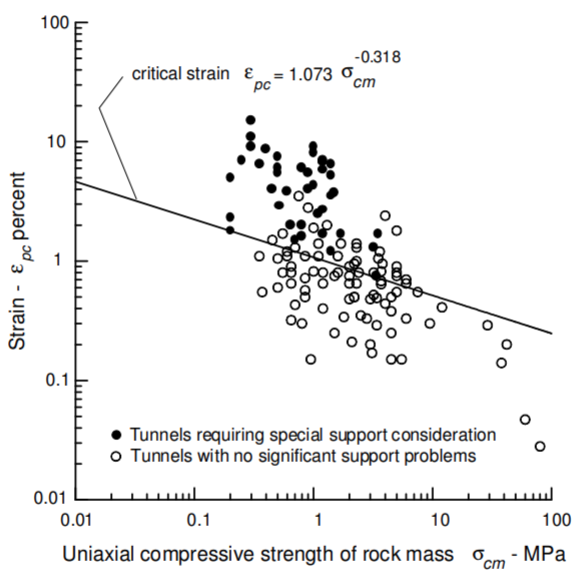
33. How reliable is it to verify analysis results from numerical models with empirical methods such as RMR or Q System? Is this good practice?

Classifications such as RMR and Q were developed to provide direct guidance on rock bolting and other support systems that can stabilize a tunnel in a rock mass from which the input parameters for the classification can be estimated. Where appropriate conditions exist to apply these methods, it is strongly recommended that they be used to give guidance on the support requirements. Numerical analyses of the support requirements and behavior patterns of these tunnels can also be constructed from the information provided by the RMR and Q classifications and these analyses may add useful information to the classification recommendations. Hence, to answer your question, checking the support requirements for tunnels can benefit from the use of both classification systems and numerical analyses, when these methods can be applied.

Classifications such as RMR and Q are less useful in dealing with complex geometries which occur at tunnel intersections, in pillars which are sometimes included in tunnels, or in more complex multiple excavation layouts. In these cases, the value of the classifications is their provision of information which can be used in estimating the rock mass properties for numerical analyses of these complex geometries. The classifications may not be directly applicable for support recommendations in these circumstances since the simple excavation dimensions included in the classification may not be compatible with the mined excavation shapes.

34. What are your views on the integration of GIS and survey technologies into tunneling and its future?

The use of GIS (Geographic Information System) and survey technologies in tunnelling is well outside my expertise. However, measuring tunnel geometries and deformations in tunnels is a critical part of tunnel design and stability analysis. Sakurai* has published an excellent book which discusses the use of measured tunnel deformations for the back analysis of rock mass strength and deformation properties and the rock mass. I have also discussed this issue in a paper** from which I have reproduced the following plot of tunnel strain (tunnel convergence/tunnel diameter) for different rock mass strengths. Hence, I believe that any measurement technologies that can be used to measure deformations in tunnels as they advance will be very valuable.



Plot of percentage strain versus rock mass strength showing the critical strain determined for support design for the Second Freeway, the Pinglin and the New Tienlun headrace tunnels in Taiwan, from information supplied by Dr J.C. Chern of Sinotech Engineering Consultants Inc., Taipei.

*Sakurai, S. 2017. *Back Analysis in Rock Engineering*. CRC Press, Taylor & Francis Group, London.

**Hoek, E. 1998, Tunnel support in weak rock, Keynote address, *Symposium of Sedimentary Rock Engineering*, Taipei, Taiwan, November 1998.

35. How do soluble rocks that deteriorate over time with seepage impact the Hoek-Brown failure criterion in tunneling and open cast quarry face?

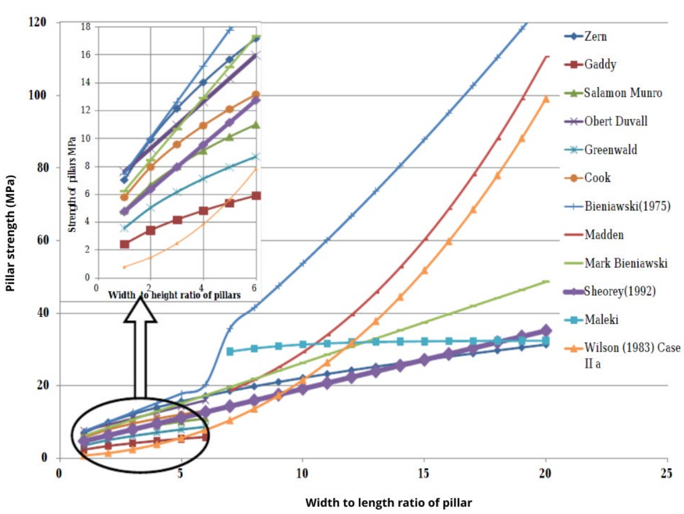
I believed that the deterioration of the strength of soluble rocks due to seepage in tunnels or slope faces has a significant impact on the long-term performance of the rock mass. In fact, I have made an allowance for this deterioration in the design of support for the Yacambú-Quibor Tunnel in Venezuela in the following paper: Hoek, E. and Guevara, R. 2009. Overcoming squeezing in the Yacambú-Quibor Tunnel, Venezuela. *Rock Mechanics and Rock Engineering*, 42(2) 389 – 418. However, I must admit that this analysis was based on a conservative estimate that was founded on very little factual evidence. I consider that this topic would benefit from some research in a rock mechanics laboratory with appropriate equipment.

36. I am wondering what methods were implemented to estimate pillar strength in the past and before the establishment of numerical methods. Also, how reliable are (empirical) pillar stability data compared to numerical simulations?

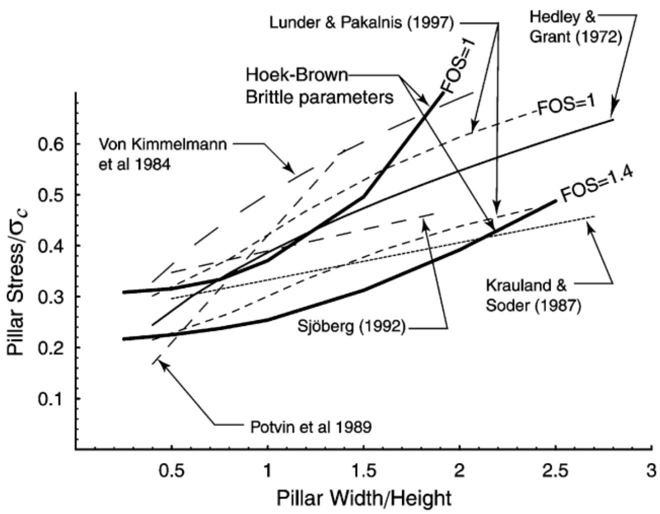
An extensive review of coal pillar design was published in the following paper: Jawed, M. M, Sinha, R. K, and Sengupta, S. Chronological development in coal pillar design for bord and pillar workings: A critical appraisal. Journal of Geology and Mining Research Vol. 5(1) pp. 1-11, January 2013. In this review the authors give a brief description of many coal pillar strength equations, starting with the following equation published by Zern in 1928 in the "Coal Miner's Pocketbook":

where C_p is the pillar strength, C_1 is the coal strength parameter, w_p is the pillar width and h_p is the pillar height.

Based on extensive testing of coal pillars in-situ and on some numerical simulations, numerous authors published more elaborate relationships which are all described in this review paper and illustrated in the figure, from the paper. As can be seen in this plot, there is a wide variation in the calculated coal pillar strengths, particularly for very wide pillars.



In their conclusion to the paper, the authors offer the following comment:
Different approaches for pillar design have been discussed in this paper. The suitability of the pillar design approaches, however, remain quite difficult as mentioned by Mark and Barton (1997) "…despite the fact that textbooks have considered laboratory testing as an integral part of pillar design for nearly 30 years, it has remained controversial. One reason is that coal remains notoriously difficult to test." The empirical methods are the best design approach towards the pillar design in particular conditions, provided they meet the performance criteria with respect to failed and stable cases of pillars. Any pillar design by numerical or other methods must be compared with the empirical relations of other workers as well as the locally developed empirical relations.



A detailed discussion of the strength of hard-rock pillars has been presented in Martin, C. D and Maybee, W,G. The strength of hard-rock pillars, International Journal of Rock Mechanics & Mining Sciences 37 (2000) 1239–1246. A figure from this paper, shown opposite, compares hard-rock pillar stability formulas with elastic two-dimensional modeling results, using the Hoek-Brown criterion with brittle rock parameters. As for the coal pillars, the pillar strength is directly related to the pillar width-to-height ratio, but failure is seldom observed in hard-rock pillars where the width-to-height ratio is greater than 2.

I have not been involved in detailed studies of pillar stability and so I will not comment further on this subject. However, the two papers referred to above give comprehensive discussions on this topic, including comments on the comparison between empirical methods and numerical calculations.

37. What are your thoughts on joint apparent cohesion in rock slope design?

Apparent cohesion, describing joint roughness, interlocking, and the conditions of the joint walls, is used in the analysis of rock slopes in both limit equilibrium and numerical analyses. Civil engineering standards are cautious about the use of apparent cohesion during design, and guidelines often recommend considering a null or low value. However, in mining engineering applications, apparent cohesion is an important component of stability analyses since it is sometimes necessary to work with slopes that are marginally stable.

Comprehensive discussions on this topic can be found in the following papers:

Rulliere, A, Rivard, P, Peyras, L and Breul, P. Influence of Roughness on the Apparent Cohesion of Rock Joints at Low Normal Stresses, Journal of Geotechnical and Geoenvironmental Engineering, Volume 146, Issue 3, March 2020.

Barton, N. Shear strength criteria for rock, rock joints, rockfill and rock masses: Problems and solutions. Journal of Rock Mechanics and Geotechnical Engineering, Volume 5, Issue 4, August 2013, Pages 249-261.

I have used apparent cohesion in many analyses involving shear failure and my experience is that caution and great care are required in estimating the cohesive and frictional properties assigned to critical failure surfaces. This is particularly important in zones in which confining pressures are low and when the consequences of failure are large.

38. In terms of (in situ) stress field, what is the best way to model a fault zone?

In general, whether using limit equilibrium or numerical analyses, I am inclined to model a fault zone by treating it as a zone with boundaries defined by the geometry of the better-quality rock on either side. This geometry of the fault should be clearly defined, in three-dimensions if necessary, and properties assigned to the fault material should be obtained, if possible, from laboratory shear tests. The analysis would then assume that failure of the zone is within the fault material and that there is no wall- to-wall contact. Failure of the fault zone subjected to in situ stresses would then be analyzed by assigning appropriate shear strength and deformation characteristics to the zone itself, as well as the surrounding rock masses.

Very narrow faults, in which contact of the walls is likely, should be treated by determining an apparent cohesive strength, as described in the previous question, and assuming a low friction angle for the fault material itself.

I am not sure what in situ stress field you are thinking of, and I am hoping that the answer provided above is relevant.

39. In the Himalayan topography of India, can TBMs be effectively used? In this environment, large variations in geological conditions occur and over relatively short distances.

I can see no reason why TBMs cannot be used effectively in the Himalayan topography of India, with which I am well familiar, having worked on several projects there over many years. It is essential that the correct TBM should be chosen to deal with different geological conditions, since hard rock tunnelling requires an open face machine while soft rock tunnelling requires the machine to generate a support pressure at the face. This is a topic that requires detailed discussion with the TBM manufacturer before decisions are made on the type of machine that will suit the anticipated geological conditions along the tunnel route. Unfortunately, this is a topic that exceeds the scope of this question-and-answer format since it is complex engineering discipline in its own right. However, an Internet search will show you that TBMs are being used successfully in many projects around the world in a wide range of geological environments.

I visited the Olmos Tunnel in the Andes in Peru, during construction of a 12.5 km long, 5.3m diameter tunnel excavated by a TBM. To complete the tunnel, the Robbins TBM had to pass under mountain cover of up to 2,000m. This held tremendous in-situ rock stresses that resulted in more than 16,000 rock bursting events. About 17% of these were classified as severe. A description of this project can be found on the following website: https://www.tunneltalk.com/TBM-Recorder-Jan12-TBM-conquers-Andes-at-Olmos.php.

40. To ensure stable rock masses, a tie rods technique is nowadays used, even in dams. Is it possible to use this technique in tunnels stabilization?

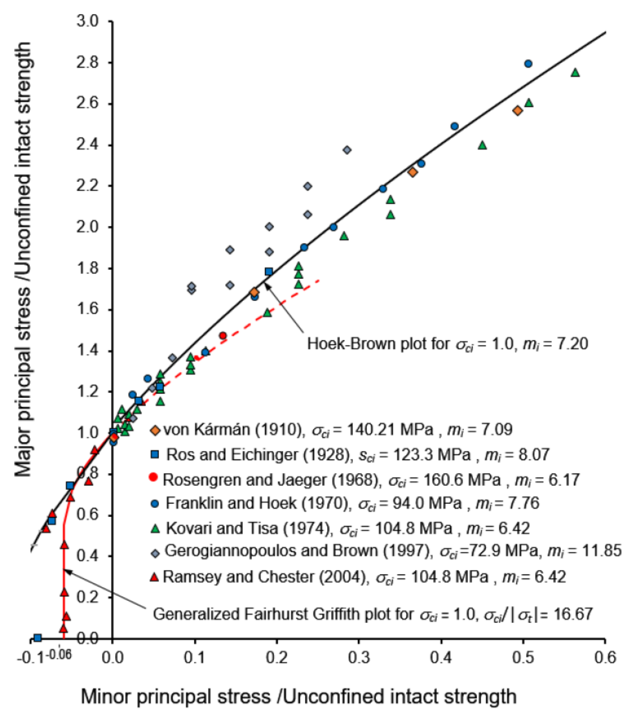
Grouted, tensioned steel cables are used frequently in tunnels and caverns in both civil and mining engineering projects around the world. To me, a "tie rod" implies that the cable passes through a rock body and has face plates and anchors at either end. I have personally recommended the use of a series of such tie rods to stabilize an overstressed pillar between two adjacent tunnels.

An excellent book, that includes this topic, entitled Cablebolting in Underground Mines by Hutchinson, D.J and Diederichs, M.S was published in 1996 by BiTech Publishers Ltd, 173 – 11860 Hammersmith Way, Richmond, British Columbia, Canada, V7A 5G1. https://www.scribd.com/doc/241737534/cablebolting-in-underground-mines-pdf.

41. In your opinion, what are the current challenges in rock mechanics that you suggest researchers and students should work on?

In my opinion, one of the deficiencies in current rock mechanics research is a lack of physical testing and field observations. With several notable exceptions, my impression is that many rock mechanics teaching and research establishments, mainly in universities, tend to concentrate on theoretical research and that the equipment in their laboratories is used very infrequently. I strongly believe that every student should have some experience of specimen preparation and of uniaxial, triaxial, point load testing and similar simple tests to see and understand how rock behaves and how it fails. Similarly, there is no substitute for spending time in the field mapping, collecting and writing up data of the type required to understand rock mass behavior. This type of information and experience is essential for a full understanding of the meaning of most of the theoretical studies which fill the literature in this field.

Tensile strength of intact rock mass been discussed in Hoek, E and Brown, E.T, The Hoek-Brown failure criterion and GSI, *Journal of Rock Mechanics and Geotechnical Engineering*, 2019, 11(3), 445-463. As illustrated in the plot on the right, only one complete set of triaxial tensile strength data was available at that time. In an appendix to this paper, a full description of the equipment required to prepare and test intact rock specimens, to obtain this data, is described. This type of testing, together with conventional triaxial compression testing, is an important addition to rock mechanics courses at universities, to give students the experience discussed above.



A topic in rock mechanics in which information is generally deficient is that of cutting and blasting rock and rock masses. These are the physical processes used in creating all tunnels, slopes, and foundations. Research in this topic is challenging and expensive, but the results can be very important in practical rock engineering applications.

An endless number of topics requiring theoretical research is available. However, before reading publications and turning on your computer to work on a research topic, it is worth asking yourself and your advisors whether the topic that you have chosen is of real interest and practical value to the growing field of rock engineering.

42. In your opinion, is the H-B criterion applicable for very deep mines? Samples taken from such grounds are highly confined and time passes between when the samples are taken and when they are tested in the lab, which can damage the samples and result in underestimated rock strength.

In my experience, the lower levels of very deep mines are generally located in hard and strong rock units which do not suffer significant deterioration when transported to surface laboratories for testing. Hence, the strength loss with time and with changes in the environment is generally insignificant in the assessment of the strength and deformation characteristics of the rock. In fact, most of my early work in this field, in which laboratory testing played a major role, was on quarzitic rock, with uniaxial compressive strengths from 300 to 300 MPa, from 2000 to 3000 m deep gold mines in South Africa (Janisch, P.R. Gold in South Africa, J. S, Afr. Inst. Min. Metall., 86(8), 1986, 273-316.) Mining was effectively in intact rock to which the Hoek-Brown criterion is applicable.

I have worked in several tunnels and caverns, with depths between 500 and 1500 m below surface, in a variety of rock masses in which deterioration and time-dependent changes have been a problem. For want of a better alternative, I have generally used the Hoek-Brown criterion in the analysis of problems in these projects, with estimated property changes applied to reduce the intact rock strength with time. This is a crude approximation of the process and sensitivity studies of the impact of the property changes on the rock mass behavior and support requirements are essential components of any design.

43. Are the geotechnical investigations required for slope stability analysis and remedial support for assumed rock mass parameters sufficient for design of support?

In the conclusion of a presentation on *Design Challenges, Disasters and Lessons in Rock Engineering*, I have written: "Rock and rock masses are effective engineering materials provided that their characteristics are recognized and incorporated into designs. These designs should minimize induced tensile stresses and ensure that confinement is provided, either by careful choice of the geometry of the structure or by the provision of reinforcement or support."

The phrase "…provided that their characteristics are recognized and incorporated into designs," is another way of asking this question and the key is the recognition of the rock mass characteristics. There is no simple answer to this question since the techniques for recognition of rock masses characteristics vary enormously, depending upon the location of the project, the process of site investigation and design and the budget allocated by the owner to this process, the experience of the geologists and engineers working on the project, and numerous other local factors. The geotechnical investigations may be a one-day site-visit by a geologist or engineer, hired by the owner to give an opinion on the properties of the rock mass on how it should be mined and, if necessary, supported. At the other end of the spectrum is a site investigation programs such as that adopted by the Chuquicamata mine in Chile, which lasted for the 25 years during the open pit mining process. It included detailed laboratory tests on all the rock types on the mine property as well as 185 km of borehole core logging and 195 km of bench mapping. Clearly, the information obtained in the one-day site visit would be totally inadequate to any design, including that of the required support.
The data base at Chuquicamata was used very effectively for every aspect of the design including, where necessary, the design of support.

44. Is it possible to briefly discuss the critical rock parameters needed for mining at extreme depth (>3000 m)?

As described in the answer to Question 42, the rock in which mining is carried out, at depths of this magnitude, tends to be strong intact rock. This rock can carry very high loads, but it is prone to rockbursting (implosive failure) when it fails. Critical rock mass parameters required for the design of mining excavations at this depth include:

- the intact rock strength and deformation characteristics,
- the presence of any significant contact planes, shear zones or faults
- the in-situ rock stresses
- the geometry of the excavations at each stage of the mining process.

The design process involves a numerical model, or models, (preferably three-dimensional) of each significant change of mining geometry, with regional in situ stresses imposed on the model boundaries. The properties of the rock mass are applied to the entire model and any significant contact planes, shear zones or faults are superimposed on the model. A failure criterion, such as the Hoek-Brown or Mohr-Coulomb criterion is applied to the rock mass and any failures in the rock mass surrounding the excavations are analyzed in detail.

An excellent discussion of the analysis of potential rockburst conditions are contained in the following paper: Diederichs M.S., Early assessment of dynamic rupture hazard for rockburst risk management in deep tunnel projects. J. S, Afr. Inst. Min. Metall., Volume 118, March2018, pages 193-204.

45. Regarding Chuquicamata, has seismic activity had significant effect on stability? As I understand, this mine is in a high seismicity area.

In the design of the Chuquicamata open pit, seismic loading has been discussed and has been included in some slope design calculations*. I am not aware of any records in which slope failures due to earthquakes have been mentioned.

*Tapia, A, Contreras, L.F, Jefferies, M and Steffen, O. Risk evaluation of slope failure at the Chuquicamata mine. Slope Stability 2007 – Y. Potvin (ed), Australian Centre for Geomechanics, Perth.

46. Have you ever dealt with expansive rocks in your work? If yes, what kind of mitigation solution did you use?

I have never dealt with expansive rocks in my work.

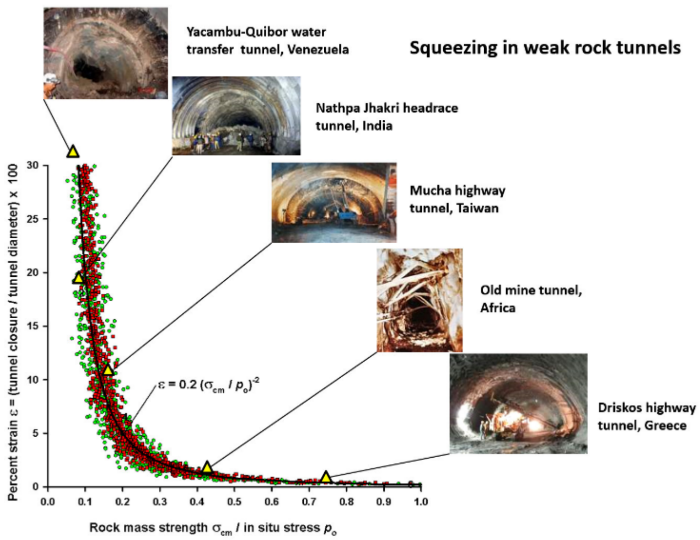
47. In your opinion, what is the best method to control the seismic effects regarding hydraulic fracturing in underground mines?

I am presuming that this question refers to earthquakes induced by underground hydraulic fracturing. I have no experience in this field, but I can refer you to the following paper on this topic that may be of value to you.

Ellsworth, W.L. Injection-induced earthquakes. Science, Vol 341, July 2013. (http://dx.doi.org/10.1126/science.1225942).

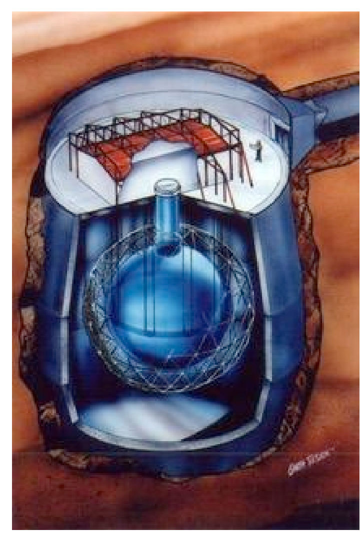
48. What was the displacement order of magnitude for the rock squeezing case you presented?

The 5-m diameter Yacambu-Quibor Tunnel in Venezuela, when inadequately supported, suffered complete closure. In other words, the ratio of tunnel closure to tunnel diameter was 100% in some cases. The successful final tunnel design was based on a closure of the tunnel from an excavated tunnel of 5.2 m diameter to a final diameter of 5.0 m, in other words, a closure of 4%.



49. What is the greatest depth in hard rock that you would suitably be confident in design (for a large civil engineering cavern associated with nuclear storage/hydropower etc.) with all the support technology we have? Are we able to reach mining depths under the difficult constraints of civil engineering?

In answer to question 25, I discussed the 33m diameter x 30.8 m high cavern at a depth of 2070 m below surface in the Creighton nickel-copper mine in Sudbury, Ontario, Canada. This cavern, which is concrete lined, houses the Sudbury Neutrino Observatory, which operated between 1999 and 2006. The underground laboratory was enlarged as a permanent facility and now operates multiple facilities as SNOLAB.



In fact, although it is adjacent to, and accessed from, the operating Creighton mine, this was a civil engineering structure and, in my opinion, it met all the civil engineering constraints which a nuclear storage or hydropower plant would have to meet.

Here are some deep structures:*

1. Deepest Laboratory: The China Jinping Underground Laboratory is the deepest laboratory, at 2400 m underground in Sichuan, China.
2. Deepest Tunnel: The Gotthard Base Tunnel, located in the Alps in Switzerland, is the world's deepest railway tunnel, 2,450 m under a mountain.
3. Deepest Mine: The Mponeng Gold Mine in South Africa is 4,000 m deep.

*https://alansfactoryoutlet.com/the-deepest-underground-structures-in-the-world/#:~:text=What%20Are%20the%20Deepest%20Underground,a%20depth%20of%2012%2C262%20meters

50. During your presentation, you mentioned that significant displacement had been observed over an extended monitoring period for a rock slope failure. Per your experience, which displacement was predominant? Was it vertical (along slope surface) or horizontal (bulging out)? If the bulging type displacement is considered as a precursor of rock slope failure, can LIDAR detect this lateral displacement accurately?

Slope displacement monitoring on an open pit mine slope was generally carried out on the inclined slope face by means of optical distance measurement using mirror targets and distance measuring theodolites. In general, the vertical movements in the rock masses in which slopes have been mined are significantly greater in the horizontal direction than they are in the vertical direction. While this method is still used on mines, such as the Chuquicamata mine in Chile which has a very extensive optical measurement setup, the use of LIDAR to carry out these measurements is increasing. There are many papers, in the technical literature, describing LIDAR techniques for slope monitoring*. I have no doubt that this method will become dominant over the next decade.

*https://www.researchgate.net/publication/241792019_The_use_of_LiDAR_to_overcome_rock_slope_hazard_data_collection_challenges_at_Afternoon_Creek_Washington

51. Regarding the plot of strain vs sigC/in-situ stress, a Monte Carlo analysis was completed to generate the distribution of points, based on the available data. With advances in Machine Learning and AI, do you have any thoughts on updating some of these earlier methods using more recent advances?

The use of Machine Learning and AI will undoubtably play a large role in future developments in rock engineering and I have a great deal of optimism on this topic. However, at the age of 88, I have now retired from participation in such advances, and I will leave it in the capable hands of a younger generation of rock mechanics engineers.

52. Stacked rock retaining walls are common in BC. Is it reasonable to assign these the properties of very poorly blasted rock and apply the Hoek-Brown failure criterion to analyze them?

This would be a simple analysis but a crude one – I would only use it if the answer was not critical and if the consequences of a potential failure could be managed. My preferred solution would be to use a discrete element model* in which the individual blocks and their surface characteristics can be modelled explicitly, and the failure mechanism of the structure analyzed in detail.

As always in rock engineering, the method adopted for solving a problem is not always a simple out-of-the-box analysis, but a solution that has been chosen on the basis that it can incorporate the essential components and issues that dictate the failure process being analyzed.

*Vyazmensky, A, Stead, D, Elmo, D and Moss, A. Role of Rock Mass Fabric and Faulting in the Development of Block Caving Induced Surface Subsidence, Rock Mechanics and Rock Engineering, 2009, 43(5):533-556.

53. In your opinion, what is the most important and challenging problem in rock engineering?

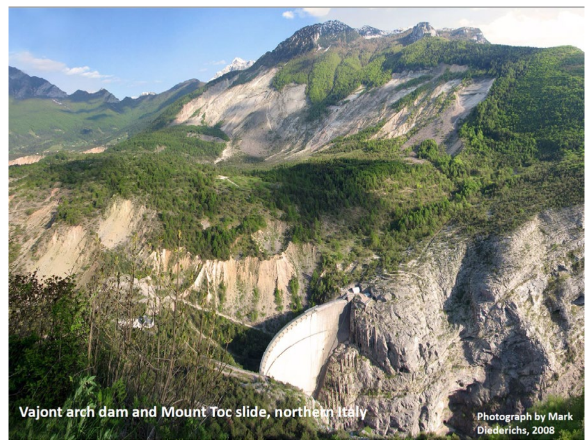
The behaviour of jointed rock masses is one of the most common problems encountered in the field and it is one of the most difficult to analyse and interpret the results. Excellent programs have been developed for the study of these problems but, as in the example discussed in the previous question, the analysis of a jointed rock mass problem is not as simple as turning on a computer to run an analysis. Very careful thought must be given to what the issues are that are being analyzed and how the results are to be processed in terms of available practical solutions. Hopefully, with time, more rock engineers will become familiar with these programs and will be able to apply them with confidence to the many problems in which jointed rock masses are the principal component.

54. The post failure photo of Mt. Toc leading to overtopping of the Vajont dam looks like it contains a very continuous slip plane. Is this a fault or foliation plane, and is it the theory that as the reservoir was filled, increased pore pressures along this surface where it daylighted into the reservoir face led to the failure?

I visited the Vaiont Dam site in about 1970, but I did not visit the site behind the dam, which is shown in the photograph opposite. Hence, I am not qualified to comment on the details of the slip surface and the failure mechanism.

A very detailed study of the Vaiont Slide was carried out, for the US Army Corps of Engineers, by Hendron and Patton in 1985*. The following summary is presented in the Abstract of this report:

"This report describes the efforts to confirm the existence and nature of clay seams in the slide mass and to confirm the possible existence of an "old" slide at the site. These efforts were made by (a) firsthand field observations of the geology, (b) an examination of preslide and postslide air photographs, (c) laboratory testing of samples of failure plane materials, and (d) an examination and translation of geologic and other documents related to preslide and postslide conditions. Stability analyses of the Vaiont Slide are presented in the report which are relatively consistent with all the observed facts."



Reproduced with permission from Professor Mark Diederichs, Queen's University, Kingston, Ontario, Canada.

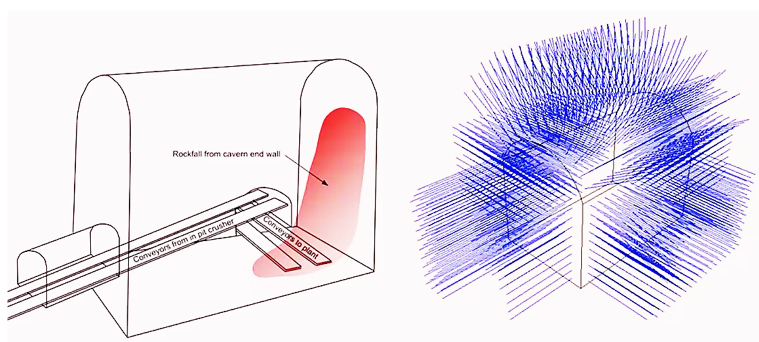
The study confirmed that the Vaiont Slide was a reactivation of an old slide. The slide moved upon one or more clay layers which were continuous over large areas of the surface of sliding. Three-dimensional stability analyses were required due to the magnitude of the upstream inclination of the clay layers forming the base of the slide. The angle of shearing resistance of the clay layers was determined to be about 12 degrees. The fluid pressure distributions used were consistent with the only piezometric data available before the 1963 slide and with an interpretation of the local groundwater flow system including the presence of karstic terrain above the slide. Results of the analyses completed for key periods in the history of the slide agree with the known slide behaviour during these periods. The results also indicate that the reduction in the factor of safety caused by reservoir filling alone was approximately 12 percent, while the reduction caused by rainfall or snowmelt ranged from 10 to 18 percent. Correlations made between cumulative precipitation, reservoir levels, and slide movement records provide a well-defined "failure" envelope. "

*Hendron, A.J and Patton, F.D. The Vaiont Slide, A Geotechnical Analysis Based on New Geological Observations of the Failure Surface. US Army Corps of Engineers, Technical Report GL-85-5 under Contract No. DACW39-79-C-0063, June 1985, 324 pages.

55. This question regards Quantification of the GSI and D values in the Hoek–Brown criterion during initial studies that take place before the actual slope/ tunnel cutting. My question is that RQD can be based on rock cores results obtained the subsurface exploration, how about discontinuity surface condition rating (SCR)? Can it be determined based on rock cores before the actual openings of the slope?

The photograph opposite illustrates core recovered from a copper deposit located about 400 below a horizontal ground surface. Hence, there were no outcrops which could be used to gain an overall impression of the surface condition rating. Despite this, full classifications were carried out on these core samples and the geologists had to use their experience and best judgement to arrive at reasonable values for all the classification parameters, including the surface condition rating. I believe that, with experienced geologists, the errors were not significant in the overall rock mass evaluation.

56. At Chuquicamata, was there any warning before the transfer cavern failure, i.e. any monitoring data being collected that indicated the failure might occur? If not, how did this failure impact the future monitoring of that area?



As far as I know, there was relatively little monitoring of the transfer cavern before the failure. The failure itself was not very large but, unfortunately, it fell on the conveyor belt driver end, as shown in the drawing opposite, and put it temporarily out of commission. The impact of this was serious since much of the ore was carried to the mills by this conveyor. Since the transfer cavern would be subjected to further deformation because of ongoing mining, it was decided that the cavern would be repaired and the additional reinforcement, in the form of 10 to 15 m long stressed cables, would be installed. Several extensometers were installed in the rock mass surrounding the cavern and these were monitored for the remaining successful operating life of the cavern. The mining method has now been changed from open pit to block caving and this conveyor system and transfer cavern are no longer in operation.

57. What advise do you have for young, up and coming geo professionals?

In my view it is essential that you should spend at least one, or preferably two, years working as a geologist or engineer in a construction or mining project between degrees if you decide to specialize with MSc and/or PhD degree studies. This will give you hands-on experience of the problems that you are likely to encounter later in life but, more importantly, it will give you the opportunity of dealing with these problems in a non-academic environment.

There are many challenging opportunities for well qualified geo-professionals, including teaching, but you have to think very carefully about which of these opportunities will be best suited to your lifestyle and ambitions, particularly at the start of your career. Staying close to home is seldom the best option when you start out so you should be prepared to move to any part of the world which offers the best opportunities for your next step.

58. Other than your family, what is your proudest achievement over the course of your career?

In 1966, at the age of 33, I was appointed a Reader and, in 1970, a Professor in the Royal School of Mines, one of the colleges of the Imperial College of Science and Technology in London. Not only was this an honour but it was the gateway into an international career that gave me access to the individuals and the institutions responsible for most of the important development of the emerging discipline of rock mechanics or, as I prefer to call it, rock engineering.

59. What are the best practices in monitoring and predicting ground failure?

By far the most effective information gathering tool in rock engineering is the monitoring and analysis of the deformations which occur when structures constructed from, or in, rock are subjected to load changes. These measurements, if they are well planned, carefully executed and interpreted, and clearly reported, can provide very detailed information of the changes which occur when failure initiates and propagates in a rock mass. Much of our knowledge of rock mass behavior comes from this type of information which has generally been derived from major construction projects in which monitoring is a requirement in the overall construction control process. Note that the displacements do not have to be large and catastrophic in order to be of value in this type of analysis.

60. What is the next big challenge for rock mechanics?

More of the same. Most of the significant problems in rock engineering have been identified and analyzed. There tends to be an over-emphasis on laboratory-scale problems, such as the interpretation of small-scale tests, and the challenges in setting up a monitoring program for large scale structure are difficult, but necessary. These larger scale projects are difficult for universities to accommodate and are probably best undertaken by larger government-sponsored research organizations.

61. What challenges do you see for rock mechanics practice in future? Where do you believe more research is warranted?

Many of the physical tools, such as laboratory testing or monitoring displacements in rock structures, are already in place as discussed in the previous two questions. The area in which most recent developments have taken place is in numerical modelling where there are ongoing developments and new programs being introduced on a regular basis. These programs are very powerful and, when used correctly, represent a means for pulling together all available information on a project and carrying out detailed analyses and sensitivity studies. I do not see any major gaps in either physical or numerical practices and applications in rock mechanics that require major investments in research projects. There is always a requirement for some research, but this tends to become evident to practitioners who will initiate or purchase the necessary research activities.

Unlike laboratory tests, in which standard methods have been in place for many years, software originally developed by university research groups have become commercial products, generally available from companies set up to produce and sell them. The problem facing users is how to make the correct choice of which programs to use and, having decided and purchased the programs, how to become proficient in understanding and using these programs. This situation is not unique to rock engineering and there are no simple answers, other than trial and error and discussions with colleagues and organizations who have used the programs.

The main problem that we face in rock mechanics is the availability of adequately trained geologists and engineers who can use all the currently available tools and programs. In many countries around the world there are no universities which teach or carry out research in rock mechanics. Technical specialists in major projects in such countries, funded by the World Bank or similar institutions, are generally expatriates with limited knowledge of the geology or traditions of the country, which is not an ideal situation, but it is unlikely that it will change in the foreseeable future.

62. Do you think improvements still have to be made in Rock Mass classification, because rock mass geology varies from place to place?

The principal benefits of rock mass classifications are that their use demands a methodical collection of a variety of geological data and that they can generally be used in any geological environment. The recommendations on rock support or other issues dealt with in these classifications can only be considered as useful overall approximations. It is essential that the user should recognize this and should not treat the classification recommendations as specifications, which dictate the length and spacing of rockbolts or thickness of shotcrete linings, and which cannot be varied. When used correctly, these classifications provide overall guidance to designers, contractors and construction supervisors who make practical decisions with these guidelines in mind, but with detail changes which depend upon their observations and experience.
If improvements can be made in the use of classifications, it is in the training of the users in the field in understanding the characteristics, advantages, and limitations of classification systems as an aid to, but not a replacement for, traditional excavation and rock support procedures.

63. What do you consider to be the most important challenges remaining in the analysis of underground excavations and support mechanisms? For example, defining joint shear strengths at the scale of openings/excavations?

The design of underground excavations requires a full understanding of rock mass properties, in situ stresses and the interaction of these components, as well as of support options, in designing the excavation and in setting out the construction methods and alternatives for the creation of the required excavations. I believe that current technology, if fully understood and correctly applied, is adequate for the analysis of underground excavations and support systems. I have been involved in several projects in many parts of the world where underground excavations of all sizes and functions have been successfully designed and constructed. It is essential that, for large projects, there should be someone permanently on site who has knowledge of, and experience in, excavation design and construction, including support choice and installation. On smaller projects, such individuals can usually be brought in on an as required basis.

64. What are some of the methods used to preserve the integrity of mudstone during sampling before sending samples to the lab?

Swelling and surface deterioration of mudstone, siltstone and shale samples is a common problem when these samples are left unprotected after being recovered by diamond drilling. This is due to moisture change in the near surface rock. It is essential that this process should be controlled if the specimens are to be tested for strength and deformation properties. The most common and probably the most effective method of protection consists of wrapping the samples in aluminum foil or a plastic film and sealing them with molten wax.

65. What is the best approach to assess the stability of a rock pillar, considering strength to stress ratio or magnitude of shear strains?

In my answer to question number 36, I discussed this topic in some detail, and I referred to two discussion papers. For pillars of excavations constructed in soft rock, such as coal, I recommend Jawed, M. M, Sinha, R. K, and Sengupta S. Chronological development in coal pillar design for bord and pillar workings: A critical appraisal. Journal of Geology and Mining Research Vol. 5(1) pp. 1-11, January 2013. For pillars in hard rock mines, I recommend Martin, C. D and Maybee, W,G. The strength of hard-rock pillars, International Journal of Rock Mechanics & Mining Sciences 37 (2000) 1239–1246.

I have not been involved in detailed studies of pillar stability, therefore, I will not comment further on this subject. However, the two papers referred to above give comprehensive discussions on this topic, including comments on the comparison between empirical methods and numerical calculations.

66. What is the best way to deal with water ingress problems in tunnels, especially if a large quantity of water is encountered (maybe due to some subsurface stream or water body /reserve) within the rock mass?

Water is always present in the rock masses in which tunnels are mined and, in some cases, the quantity of water can be significant, as illustrated in the adjacent photograph, showing water draining through boreholes for rockbolts. Drainage is the only effective way to control this water and it is essential that well thought out and adequately funded drainage systems should be part of the construction plan. In some cases, such as in the installation of forepoles illustrated in questions 8 and 32, drainage can be achieved by drilling a few extra holes and leaving them un-grouted. The tunnel should then have sumps and drainage ditches, or pumps provided to drain the water out of the tunnel. In other cases, an additional array of drainage holes may need to be planned and provision should be made for drilling these holes well in advance of the tunnel heading to ensure that the best construction conditions are available at the face. When passing under a river, it may be necessary to drill or tunnel into the rock mass above the tunnel route to control the water by drainage and grouting, if necessary, before the arrival of the tunnel.

67. What is the impact of new technologies based on BIM and non-destructive techniques in the analysis of rock mechanics problems?

Block-in-matrix rocks (bimrocks) is a description of rock masses introduced by Dr Edmund Medley*. Bimrocks are complex formations characterized by competent rock inclusions floating in a weaker matrix. Breccias, coarse pyroclastic rocks, and mixtures of rock in waste dumps are typical bimrocks. Many years ago, I had the pleasure of meeting Dr Medley when he visited Vancouver and we had a long discussion on the origins and applications of the methodology which he had introduced in his work on Bimrocks. Unfortunately, I was at the end of my career at that time. Therefore, I did not follow up on these fascinating discussions. However, I had been associated with the behavior of rock masses modelled as accumulations of independent blocks of rock through the work of Dr Peter Cundall and the programs that he developed for the software company, Itasca Consulting Group, in Minneapolis. The programs that were developed by Itasca have had a major impact on applied rock engineering around the world and, while I have not used the programs personally, I have acted as an independent consultant, or as a member of consulting boards on many projects in which these programs have been used for the analysis of very complex rock structures. One of these projects involved the analysis of the behavior of the East wall of the Chuquicamata open pit mine in Chile, described in question 22. I have little doubt that the analysis of the behavior of assemblies of independent rock blocks or elements will play an increasingly important role in the future of rock engineering.

*Medley, E.W. and Goodman, R.E., 1994. Estimating the block volumetric proportions of melanges and similar block-in-matrix rocks (bimrocks)., Proceedings, 1st North American Rock Mechanics Symposium, Austin, Texas, May 1994.

68. What is the most oblique angle to the total station that a prism can still be reasonably accurate?

I do not have a precise answer to this question but, from having observed many applications of precise displacement measurements using theodolites to measure distances from mirror targets, some of these installations involve measuring distances of up to a kilometer and I presume that the accuracy of the prism must be very high, almost certainly within 1 degree. Since the installation of the prisms involves one person adjusting the prism and another observing through a theodolite, the orientation of the prism is set by the adequacy of the signal received the observer.

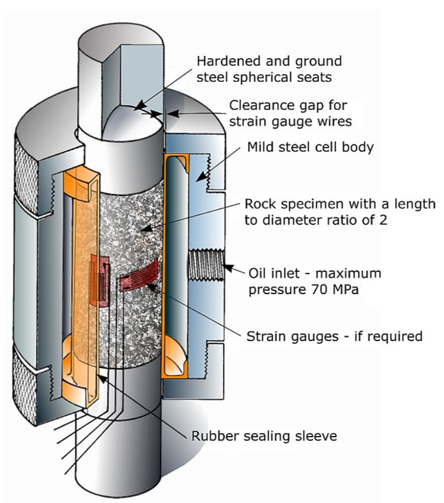
69. What is your guidance for building great working teams on major mining/civil projects?

This is a very difficult question to answer since the situation would be different for most every project. However, the common requirement would be for an understanding project owner and management and a very competent leader for the team. Strong support at the top is essential since many of the workers on the project would be inclined to view any rock mechanics projects as academic and of minimal value to the work in which they are engaged. In my experience, it generally requires almost year for a team set up to investigate the stability of slopes and tunnels or the adequacy of foundations, to establish credibility and a good working arrangement with the other participants in the project. Certainly, the leader of the team needs to come to the project with good experience with not only the technical aspects of the project but also in working with people in the field. The other members of the team need to have knowledge and experience in the topic for which they are responsible. It is probably useful to bring in a consultant, with experience setting up and working with such teams, for a day of discussions with management and the team leader, during the setting up of the team.

70. What methods can be used to predict failure in brittle rocks?

Failure in brittle rocks initiates at the boundaries of grains of hard, strong rock and propagates by tensile cracking. There are many publications on this subject because of its importance in mine and tunnel construction at great depths. A summary of some of the available information on this topic is given in the following paper by Hoek E, Martin CD, Fracture initiation and propagation in intact rock. A review, *Journal of Rock Mechanics and Geotechnical Engineering Journal of Rock Mechanics and Geotechnical Engineering*, Volume 6, Issue 4, August 2014, Pages 287-300.

The figures below illustrate equipment for typical laboratory tests, the appearance of partially developed brittle fractures in a hard rock specimen and the appearance of fully developed brittle failures in an underground mine excavation.



In the laboratory, brittle failure is studied by carrying out high quality loading tests in equipment such as the triaxial compression cell illustrated above.

The appearance of brittle failure in a triaxial specimen, loaded to 80% of the failure stress.

The appearance of brittle failure in a very deep level hard rock mine excavation in Africa.

71. What type of laboratory test would you recommend to determine the intact shear strength of very weak rock, like coal? In particular, what type of test can be done when the preparation of samples that meet the ISRM specifications is impossible, because the rock crumbles when cut it, but it is not weak enough (0.1 < UCS < 1 MPa) to be able to carry out a soil test?

I have carried out tests on very weak rocks such as coal, mudstone, siltstone and shale in equipment such as the triaxial cell illustrated above in Question 70. Great care, generally using a lathe, is required in specimen preparation, but it is certainly possible. For weaker materials, I would generally use a shear test with the broken specimen tamped in to form a uniform sample. This would not be an intact specimen test, but the friction angle would probably be equivalent to that of the intact material.

72. What type of rock will deteriorate mostly due to the wetting from dam reservoir or rain and snow? And what type will deteriorate most while being exposed to air?

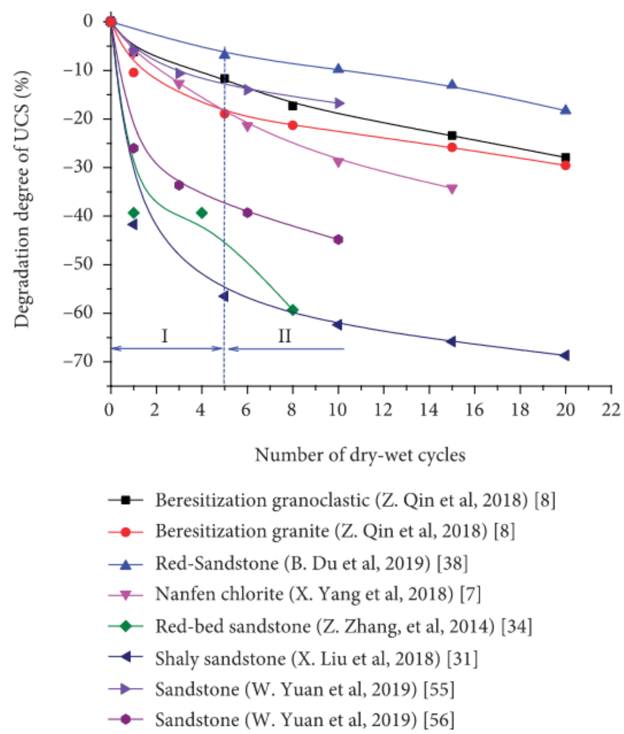
I graduated as a mechanical engineer and my knowledge of the chemistry of rocks is minimal. From observations, I understand some weak rocks are prone to weathering when exposed to moisture change, such as direct wetting due to rain or snow.

A Google search for a description of weathering on Wikipedia (https://en.wikipedia.org/wiki/Weathering) gave me the following classification of weathering of rocks:

- 1 Physical weathering
  - 1.1 Frost weathering
  - 1.2 Thermal stress
  - 1.3 Pressure release
  - 1.4 Salt-crystal growth
  - 1.5 Biological effects on mechanical weathering
- 2 Chemical weathering
  - 2.1 Dissolution
  - 2.2 Hydrolysis and carbonation
  - 2.3 Oxidation
  - 2.4 Hydration
  - 2.5 Biological weathering

In the figure opposite, significant deterioration takes place in some of the rocks tested and serious attention would have to be paid to this level of deterioration in designing a structure on or in these rock types.

Further discussion of this topic is beyond my level of expertise, and I recommend that you could obtain much more useful and reliable information from a discussion with a geologist.



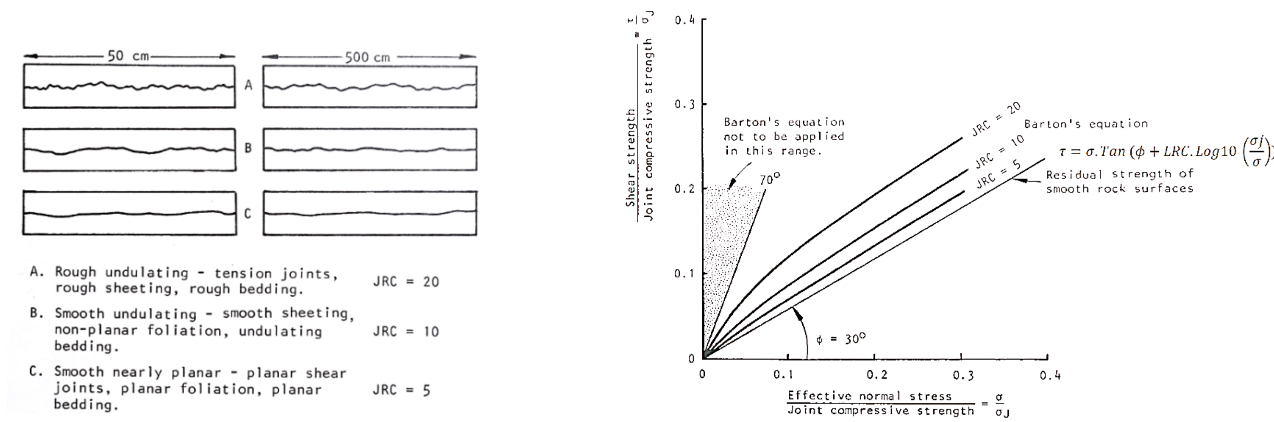
Plot of degradation of uniaxial compressive strength of various rocks due to wetting and drying, from: Zhang, Z, Niu, Y, Shang, X, Ye, P, Zhou, R and Feng, G. Deterioration of physical and mechanical properties of rock by cyclic drying and wetting. Geotechnical Engineering, Volume 2021, May 2021, Article ID 6661107.

73. What will be your advice on tackling the cyclic operational water pressure in jointed rock mass of aqueduct tunnel with intermittent pumping? The fluctuation of water pressure can possibly loosen the joint infilled materials with gaps as a result of washing out.

This is a very difficult question to answer without any knowledge of the site and the water pressure variations. However, a first approach would be the separation of the fluctuating water pressure of the water in the tunnel from the surrounding rock mass by a physical barrier, such as a concrete or steel lining. If a concrete lining is already present, an inspection to ensure that defects and cracks in the concrete have been repaired would be advisable.

74. When dealing with clean rock joints in structurally controlled rock masses, cohesion is usually assumed to be zero. The shear strength of clean rock joints is governed by roughness and interlocking and the conditions of joint walls. Apparent cohesion is obtained by performing a linear regression for the experimental shear strengths obtained in the laboratory for a given normal load interval. Therefore, cohesion is not really zero. How should we consider it in rock slope design?

This question was discussed in Hoek, E. and Bray, J.W. *Rock Slope Engineering*, Institution of Mining and Metallurgy, London, 1974, in which several alternative approaches were provides. One of these solutions, based on an interpretation of surface roughness by Barton, N.R., A relationship between joint roughness and joint shear strength, *Proc. International Symposium on Rock Fracture*, Nancy, France, 1971, Paper 1-8, is illustrated in the following figure.



75. When do you think that we should use some significant tensile strength in our rock constitutive models?

Obviously, when dealing with intact rock problems, the incorporation of tensile strength is important. This arises when a rock mass is tightly confined in pillars or in deep tunnels, where tensile strength plays an important role in spalling of the unsupported boundaries. It also occurs in the discrete element modelling of jointed rock masses in which large blocks may be incorporated in the analysis of a foundation, slope or underground cavern and where intact properties are assigned to the large blocks.

A useful review of tensile strength concepts and testing methods is presented in the following paper: Perras M.A. and Diederichs M.S. 2014. A Review of the Tensile Strength of Rock. Concepts and Testing, *Geotechnical and geological engineering*. 32(2), 525-546.

76. When doing a probabilistic analysis, which set of parameters makes more physical sense to be defined as random variables? The "GSI, mi, and D" or the "mb, s, a"?

GSI, mi and D are independent variables, measured in the field, while mb, s and a are dependent variables calculated from the values of GSI, mi and D. Consequently, GSI, mi and D are the random variables that should be used in a probabilistic analysis.

77. When percent strain becomes very significant (say, >>10-15%), should working with the strain and considering a support type that is much more "mobile" be considered? What types of mobile support have you used with good efficacy?

Strains of more than about 3 to 5% in poor quality rock masses surrounding tunnels require special consideration since most conventional steel sets, rockbolts and shotcrete or concrete linings cannot accommodate large deformations without some form of distress or failure. When it is necessary to maintain the profile of the tunnel or cavern to accommodate specific equipment of roadway clearances, control of deformations by increasing the capacity of rockbolts, steel sets or concrete linings is generally the approach adopted. In one case of a highway tunnel that I worked on, long tensioned cables were used to enhance the rockbolt support in an area of isolated poor rock to maintain the correct final profile. In other cases, such as water conveyance tunnels, where the profile is not critical, deformable support can be used to stabilize the tunnel. An example of such support is given in answers to questions 15, 16 and 48 which deal with the case of the Yacambú-Quibor tunnel in Venezuela where complete closure of the tunnel occurred in locations where inappropriate support had been installed and where steel sets with sliding joints, with a limit of 4% strain in the tunnel, were used to stabilize the tunnel during final construction.

A description of several types of yielding support that can be used in squeezing ground is presented in the following paper: Anagnostou, G and Cantieni, L, Design and analysis of yielding support in squeezing ground, Proc. 11th ISRM Congress, The Second Half-Century of Rock Mechanics, July 2007, Portugal.

78. Why is it that, even with damage cases throughout history, monitoring programs with instrumentation that could alert us are not the principal concern when we are developing underground constructions?

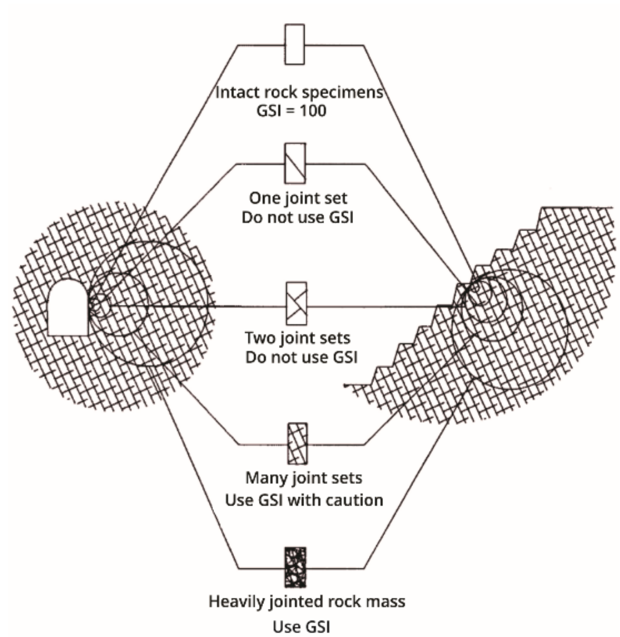
This problem is more related to logistics than to technical decisions. Designers of these projects tend to concentrate on information gathered during the site investigation and, while they will almost certainly include a recommendation for monitoring during and after construction, their brief does not include doing this work. At the other end, the Owner and the Contractor would generally not have made provision for monitoring programs in their overall plan and budget, unless they had been strongly advised to do so.

A good example of a planned and adequately funded investigation and monitoring program is that which was installed and operated during the mining of the Chuquicamata open pit in Chile, described in detail in the presentation and in answers to questions 7 and 22 in this document. This program, which has been in operation for about 25 years, involves a geotechnical data base which included 185 km of core drilling and 196 km of bench mapping as well as over 1,000 prisms used for electro-optical displacement monitoring in the open pit. It was based on recommendations which I made, in association with Dr John Read, in 1992, when we were both involved as consultants to the mine. While the cost of this monitoring project was high, I consider it to be appropriate for one of the world's largest open mines with an annual production of about 300,000 tons of refined copper, 18,000 tons of molybdenum, and smaller quantities gold and other minerals.

In a typical civil engineering project involving underground construction, contracts for design and construction components are generally assigned to companies that specialize in this work. It is very important to ensure that appropriate instrumentation and monitoring of ongoing construction of underground components should be included in one of these contracts or in a separate smaller contract, to ensure that monitoring of support programs and excavation deformations is carried out. The results obtained from such monitoring programs are important for day-to-day construction decisions and for longer term resolution of contractual issues, should these arise.

79. With respect to the anisotropy of rock mass due to discontinuities, what is your recommendation for addressing the assignment of Geological Strength Index (GSI) values?

The Geological Strength Index (GSI) is intended for use in rock masses in which the number of joints is high enough and the orientation of the joints is random enough that the rock mass can be considered as isotropic. In other words, its strength and deformation properties will not vary significantly with direction.



The Generalized Hoek Brown criterion is represented by the equation:

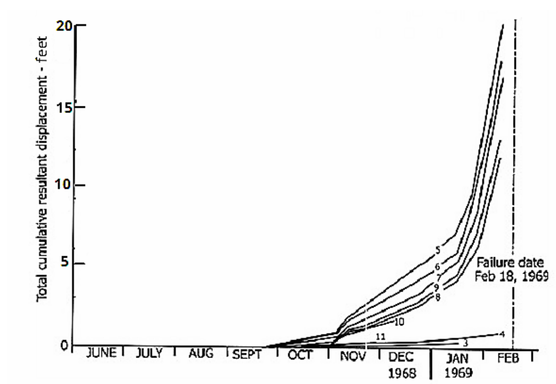
$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a$$

where σ_1' and σ_3' are the maximum and minimum effective principal stresses at failure.

GSI is used to modify the Generalized Hoek-Brown parameters as follows:

80. With the present monitoring technology of slope surface movements or surface deformation, would it be possible to predict the Time-to-Failure (ToF) with practical accuracy for early warning purposes?

Yes, the plot on the right shows measured displacements in the East wall of the Chuquicamata mine in 1968 and 1969 which failed on February 18, 1969. Movements in the slope had been observed about 6 months before the failure and an electro-optical monitoring system had been set up to measure the displacements. As shown by the plot on the right, an asymptotic increase in displacements in January and February 1969 was a clear indication that failure was imminent, and the equipment and personnel were evacuated from the mine. When the failure occurred on February 18, there was no damage to equipment or personnel.



81. Is there potential for rockburst in underground mines around 600m deep? Do we need to consider this mechanism at all? Also, would a seismic monitoring system be useful for such mines?

Rockbursts occur when the strength of highly stressed brittle rock is exceeded by the stresses which are concentrated around the excavation boundaries or in pillars in an underground mine or tunnel. The photograph on the right illustrates a hard rock pillar burst at a relatively shallow depth in a mine. In this case the pillar size was inadequate, and the rock failed in a manner like that which occurs in a uniaxial compression test in a laboratory.

Seismic monitoring is a useful tool in such a situation since an increase in the rate of microseismic events in the rock mass can give an advanced warning of impending failure, much like the displacement monitoring described in the answer to question 80.

82. You mentioned that shotcrete was used in a hydro power project to protect the rock from water, but won't the shotcrete eventually degrade and expose the rock?

Well designed and correctly placed shotcrete is very durable and it behaves very much like concrete. It would certainly protect the rock surfaces in tunnels and powerhouse caverns from the moisture in the underground environment. For tunnels in which water is transmitted, shotcrete can be used to improve the stability of small areas of minor spalling in hard strong rock masses. It is frequently used, in association with rock bolts, in unlined tunnels. However, when significant tunnel lengths have been mined through very poor-quality rock, such as that which can occur in a fault zone, the use of a full concrete lining is recommended.

83. Did you ever think that your work, including the Hoek-Brown criterion, would become such an inspiration (or enjoy such popularity) in rock mechanics?

The Hoek-Brown failure criterion was developed in the late 1970s and first published in Hoek, E and Brown, E.T. *Underground Excavations in Rock*. The Institution of Mining and Metallurgy, London, 1980. In effect, this book was the final report of a mining industry-funded research project carried out by a team of graduate students, working under my direction from 1972 to 1976 , in the Royal School of Mines at the Imperial College of Science and Technology in London.

In discussing a rock and rock mass failure criterion which would be of value to underground excavation designers, Hoek and Brown listed the following requirements:

a. It should adequately describe the response of an intact rock sample for the full range of stress conditions likely to be encountered underground. These conditions range from uniaxial tensile stress to triaxial compressive stress.
b. It should be capable of predicting the influence of one or more sets of discontinuities upon the behaviour of a rock sample. This behavior might be highly anisotropic, i.e. it will depend upon the inclination of the discontinuities to the applied stress direction.
c. It should provide some form of projection, even if approximate, for the behavior of the full-scale rock mass containing several sets of discontinuities.

Since no such criterion existed at that time, the authors took on the responsibility of developing one. Over the years this criterion has gained wide acceptance and has been used in many projects.

84. My question revolves around the statement, "rock engineering often involves a fair amount of extrapolation and guesswork." In your experience (i.e., from the start of your career to now) how applicable is this statement? Is it still true with all our technological advancements?

Unlike steel or concrete, which are man-made materials with tightly specified mechanical properties, rock masses have a wide range of strength and deformation properties which require definition in each individual project. Extrapolation and guesswork are required to fit the characteristics of individual rock masses into a general behavior pattern which can be accommodated by numerical analysis programs and construction manuals. Technological advances are unlikely to change these rock mass characteristics, although new methods of enhancing rock mass properties may be developed. I consider that the original statement continues to be applicable.

85. What is the best choice for Plastic Potential or Dilation Angle for the Generalized Hoek Brown constitutive equations? Would a Dilation Angle like that for the Mohr Coulomb be appropriate? How about a Plastic Potential function that has the same form as the Yield Criteria that uses a dilation parameter instead of "mb"?

The Hoek-Brown criterion was originally developed to deal with failure in strong, brittle rocks and no provision was made for pre- or post-failure ductile behavior. I do not have a simple answer to your question, but I would like to quote the following sentence: "However, it is a very challenging and difficult task to develop a constitutive model that can adequately represent the complete stress–strain behavior of rocks, especially for the nonlinear response such as dilation." from the following paper: Zhao, X.G and Cai, M. A mobilized dilation angle model for rocks. *Intnl. J. Rock Mech. Min. Sci.* Vol. 4, Issue 3, April 368-384.

Walton, G. and Diederichs, M.S. 2014a. Dilation and post-peak behaviour inputs for practical engineering analysis. Geotechnical and Geological Engineering. DOI 10.1007/s10706- 014-9816-x, present a symmetrical solution for displacements around a circular excavation. This solution has then been used to investigate the influence of mobilized dilation on displacements in the plastic zone around an excavation.

86. Will an intact rock always have better strength than fractured rock? Could there be a case where some rocks can give more strength when they are fractured?

I cannot visualize any case in which the intact rock would not be stronger than the fractured rock. Generally, rock fractures by shear failure of the grain boundaries and/or propagation of tensile cracks through the fractured rock. In every case that I have ever seen, this process results in a reduction of the strength of the specimen.

87. Apart from the monitoring tools/device data and warns, what are the physical signs of a slope failure?

In many cases of slope failure in excavated slopes, the first visible sign is the formation of a tension crack, parallel to the slope crest, as shown in the photograph opposite. Since rock and rock masses are very weak in tension, this is a very sensitive sign of the onset of displacement along the shear surface and it is not unusual to see tension cracks long before there is any other sign of slope failure.

In natural unexcavated slopes, the first sign of slope failure is generally the leaning of trees, as illustrated in the photograph. These trees illustrate that the slopes moved when the trees were young, probably when the roadway was excavated. Once the slopes stabilized, the trees continued to grow vertically.

Unstable slopes also exhibit spalling or slabbing of the slope face in response to the movement of the rock mass in which the slopes was excavated.

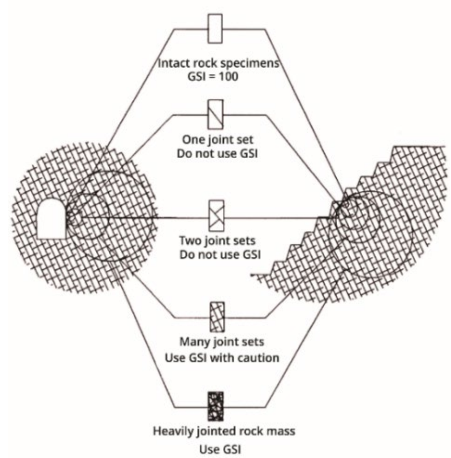
88. This is a question about Data uncertainty and Risk modelling during slope design process. Since it's difficult to avoid data uncertainty 100% during the slope design process, is it possible to prevent its impact during slope implementation process? If the risk model is well developed, will that help to reduce slope failure?

My interpretation of this question is that the writer is asking whether a well-designed rock slope model can result in a better interpretation of rock slope failure which, in some cases, may indicate a safer slope. My answer is that a slope model which correctly incorporates all the relevant information on shear strength parameters, the location and shape of the most critical failure surface, the impact or ground water pressures and the impact of tensile failure in the slope crest will produce more realistic and reliable factor of safety values than those obtained from a very simple model. This does not imply that the risk of slope failure is reduced; only that the result is more reliable.

89. You are my Guru. I am attending this lecture after my boss Mr. Dimitrios Katsaris, who has worked sometime under you, insisted me to attend (in his words it will be a lifetime pleasure to hear him) Many times we consider GSI to give an essence of rock strength, rock weathering grade or rock type. What does the scale of project have on GSI?

The Geological Strength Index (GSI) is a dimensionless parameter which is independent of the scale of the problem. As stated in my answer to Question 79, it is intended for use in rock masses in which the number of joints is high enough and the orientation of the joints is random enough that the rock mass can be considered as isotropic. In other words, its strength and deformation properties will not vary significantly with direction.

In a typical open pit mine in which a hard rock mass has many joints with an average spacing of 2m, the joint pattern in a 15-m bench would not be sufficiently uniform to apply GSI. On the other hand, the joint pattern in a 200-m high overall slope would qualify as a heavily jointed rock mass and as shown in the illustration opposite, GSI could be applied to this rock mass.



90. Any challenges and measures in tunneling through karstic limestone?

I have very little experience of tunnelling in karstic limestone, but I can recommend the following paper, by the late Paul Marinos, which deals with many problems of tunnelling in karst: Marinos, P.G., Tunnelling and mining in karstic terrane; an engineering challenge. *Geotechnical and Environmental Applications of Karst Geology and Hydrogeology*, Beck and Herring (eds),2001, Balkema publishers.

It must be recognized that tunnel in karstic rock can have significant environmental issues due to significant drainage of the overlying rock mass. Many of these issues are described in the following paper: Lva,X , Jianga,Y , Hub, Y, Caoa, M and Mao, Y. A review of the effects of tunnel excavation on the hydrology, ecology, and environment in karst areas: Current status, challenges, and perspectives. *Journal of Hydrology*, Volume 586, July 2020, 124891.

91. How deep (relative elevation) was the pillar in the underground zinc mine in NW Spain?

I do not have a precise depth for the pillars illustrated in my presentation but, from my recollection of the visit, I estimate the depth to be approximately 500 m below surface.

92. How do you characterize a rock mass with extreme mineralization?

I have no knowledge of this topic, but I can recommend the following paper for your consideration: Brzovic, A. and Villaescusa, E. Rock mass characterization and assessment of block-forming discontinuities during caving of primary copper ore at the El Teniente mine, Chile. Int. J Rock Mech. Min. Sci, Vol 44, Issue 4, 2007, pages 565-583.

93. Approximately how long were the cable anchors at Chuquicamata's transfer cavern?

The cables in the rock mass surrounding the Chuquicamata conveyor transfer cavern are approximately 15 m long.

94. If someone wanted to do more research on the Hoek-Brown criterion, what would be your recommendations?

A comprehensive review of the Hoek-Brown failure criterion derivation, advantages and disadvantages has been published in the following paper: Eberhardt, E., The Hoek-Brown failure criterion. Rock Mech. and Rock Eng. Vol 45, 2012, pages 981-988.

One of the most significant problems with the criterion is that it does not include the influence of the intermediate principal stress σ_2 and it has been shown that this can have a significant impact in using the criterion in some applications. This is discussed in the following paper: Priest SD (2005) Determination of shear strength and three-dimensional yield strength for the Hoek–Brown criterion. Rock Mech Rock Eng 38(4):299–327.

Research in this area would be an important contribution to the utilization of the Hoek-Brown criterion in practical rock engineering applications.

95. For slightly metamorphosed shale interbedded with sandstone (slate and metasandstone), can you offer thoughts on determining rock mass deformation modulus, using approaches such as Hoek and Diedrichs 2006? We can say that the rock mass is 25% sandstone, 75% slate and is Terzaghi-Deere Class IV rock, with RQD of 75%.

This question came in at the last minute and I have found it difficult to go back to the Hoek and Diederichs 2006 paper to try to understand our reasoning at that time. The equation proposed to estimate the deformation modulus of an in-situ rock mass is:

$$E_{\mathrm{rm}} = E_{\mathrm{i}}\left(0.02 + \frac{1 - D/2}{1 + \mathrm{e}^{((60+15D-\mathrm{GSI})/11)}}\right)$$

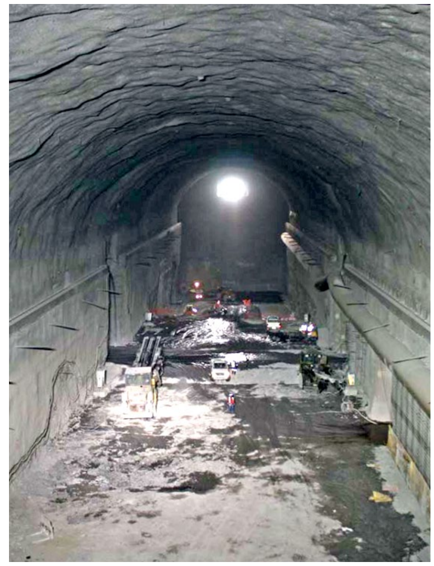
Where E_{rm} is the overall rock mass modulus, E_i is the estimated intact strength of the component rock blocks, D is the blast damage or disturbance factor and GSI is the Geological Strength Index. All these factors are open to a wide range of interpretations and, in using this equation in the field, my intention has always been to obtain a rough estimate of the overall deformation modulus which can be used as input for very crude sensitivity studies of the behaviour of the structure during construction and operation. This behaviour will vary greatly, depending upon the loading applied and whether you are talking about a dam, a mine slope, a tunnel or a large underground excavation.

From Table 3 in the Hoek and Diederichs paper you will see that the multiplication factor *MR* used to estimate the intact rock modulus from the equation $E_i = MR\ \sigma_{ci}$ is only marginally different for sandstone and slate. I would say that this difference is not significant, unless the orientation of the schistosity is significantly different for the two components, which is unlikely.

Hence, I suggest that the overall accuracy of your estimate of the rock mass deformation modulus would be in the range of 20 to 30% and that you would not be able to estimate the impact of the sandstone/slate combination from this analysis.

It may be that an analysis carried out for this extremely crude estimate of rock mass deformation modulus would indicate that your anticipated problems would be minimal, and you can then proceed with your design on the basis of these estimates. On the other hand, depending upon the project and the loads imposed on the rock mass, the deformation issues could be significant.

The photograph opposite shows an in-situ deformation test in an underground hydroelectric project in Taiwan. In this case, preliminary estimates of rock mass properties indicated that there could be significant problems with the stability of some of the underground tunnels and caverns. It was decided that in-situ tests, of the type illustrated were necessary and a number of detailed tests and analyses were performed. The design and construction of the underground complex proceeded without difficulty and the project has now been in operation for a number of years.
The photograph on the right shows the partially completed Ingula underground powerhouse complex in South Africa. Detailed geotechnical investigations were carried out for this project, described in the paper by Keyter and Varley referenced below:



Keyter, G.J. and Varley, P.M. *Design of the Ingula powerhouse caverns: General design considerations*, SANCOT Seminar, SAIMM, Ladysmith, South Africa, 2008.