# 2. What is Rock Engineering?

## Introduction

Rock Engineering deals with the use of rock and rock masses as engineering materials. It utilizes all the techniques used in mechanical, civil, and mining engineering for designing structures in steel and concrete, but it incorporates the special strength and deformation characteristics of intact rock and fractured rock masses as well as the uncertainties associated with in situ and induced stresses in rock masses.

Structures have been built on, or in, rock for centuries as shown in the photograph reproduced in Figure 1. The basic design principles have been understood for a very long time. Rock engineering is merely a formal expression of some of these principles and it is only during the past few decades that the theory and practice in this subject have come together in the discipline that we know today as rock engineering.

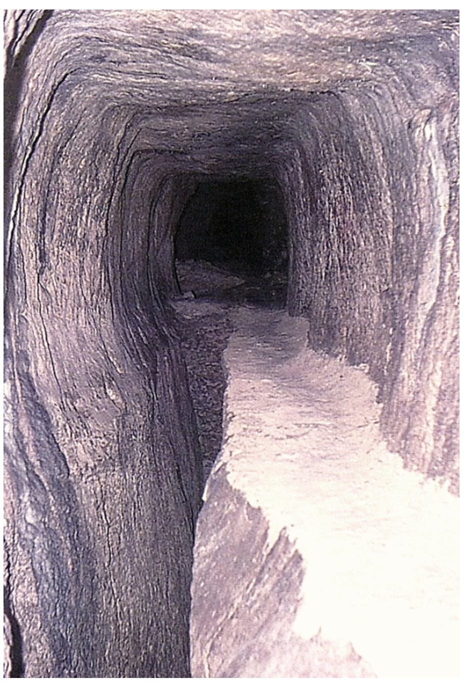


Figure 1: The 1036 m long Eupalinos water supply tunnel was built more than 2,500 years ago on the Greek island of Samos. This is the first known tunnel in rock to have been built from two portals with the two drives meeting in the middle.

This photograph was provided by Professor Paul Marinos of the National Technical University of Athens.

In December 1959, the foundation of the Malpasset concrete arch dam in France failed during first filling because of excessive water pressure in a fault upstream of the dam wall (Figure 2). The resulting flood killed about 450 people. In October 1963, about 2500 people in the Italian town of Longarone were killed because of a landslide-generated wave which overtopped the Vajont Dam.

Figure 2: Remains of the Malpasset Dam as seen today. Photograph provided by Dr Mark Diederichs, 2003.

These two disasters had a major impact on rock engineering. In 1960, a room and pillar coal mine at Coalbrook, South Africa, collapsed with the loss of 432 lives. This event was responsible for the initiation of an intensive research programme which resulted in significant advances in the methods used for designing coal pillars (Salamon and Munro, 1967).

These failures in the early 1960s confirmed the warning by Terzaghi (1936) who, in discussing slopes in the Panama Canal excavations wrote, 'The catastrophic descent of the slopes of the deepest cut of the Panama Canal issued a warning that we were overstepping the limits of our ability to predict the consequences of our actions ....'.

The development of rock engineering as a recognizable engineering discipline dates from about the period in which the major failures, discussed above, occurred. I consider myself extremely fortunate to have been involved in the subject since 1958. I have been in positions that required extensive travel, which have brought me into personal contact with most of the persons with whom the development of modern rock engineering is associated.

## Using rock as an engineering material

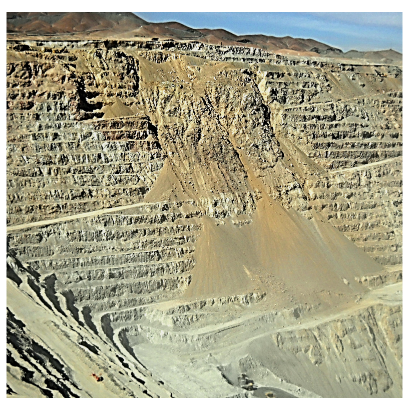
Many civil and mining structures are built in, or on, rock masses, which like steel and concrete, must be treated as engineering materials. Information on the properties of the rock masses and estimates of the stresses acting on the rock are required to design foundations, tunnels, large underground caverns, underground mines, highway slopes and open pit mines. This information is assembled from geological observations and measurements in the field, laboratory testing on diamond drilled core and specimens, measurements of in situ stresses and other properties which may be required in specific cases. Because of wide variations of these observations and measurements, the use of some form of reliability analysis is generally required by the designer to determine the support necessary to stabilize tunnels, calculate the permitted loading of foundations and estimate the safe angles for excavated slopes.

Failure of a tunnel in a weak rock mass at a depth of 1000 m below surface.

Failure of the toe of a slope in jointed rock mass.

Rockfall danger in a steep slope in massive rock.

Brittle spalling failure in a mine excavation in very strong intact rock at 3000 m below surface.



Shallow slope failure in a jointed rock mass in a large open pit mine.

Stable bridge foundation on massive rock.

Figure 3: Examples of the types of problems that are encountered by rock engineers.

Typical problems encountered by rock engineers in the field are illustrated in Figure 3. These involve failure of rock or rock masses at different scales and under different loading conditions which may be simple vertical gravity loads due to the self-weight of loose blocks in slopes or foundations of dams. On the other hand, in the case of underground excavations, failure may be induced by in situ stresses in the rock mass surrounding tunnels, mining excavations or large underground caverns.

**Intact rock strength**

A starting point in understanding the engineering properties of rock masses is a discussion of the strength and deformation properties of intact rock. Figure 4 illustrates a tray of intact rock core obtained by diamond drilling into a rock mass in which a foundation, slope or tunnel is to be excavated. Also included in this figure is an illustration of a triaxial cell in which intact rock specimens can be subjected to a range of triaxial stresses, with deformation of the intact rock being measured by means of strain gauges. Core drilling, specimen preparation and testing procedures, and the interpretation of the results, are described in detail in the chapter on intact rock in these notes.

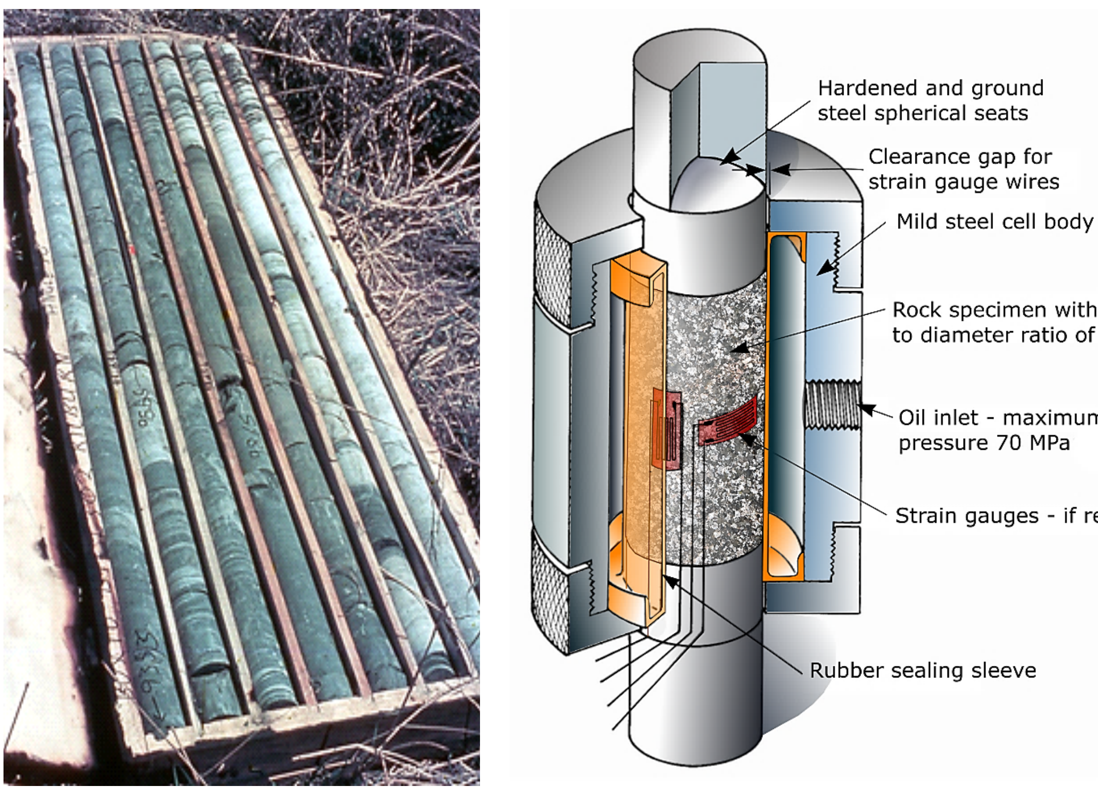



Figure 4: Intact rock specimens recovered by diamond drilling in an in-situ rock mass and a triaxial cell for testing core samples to determine their strength and deformation characteristics.

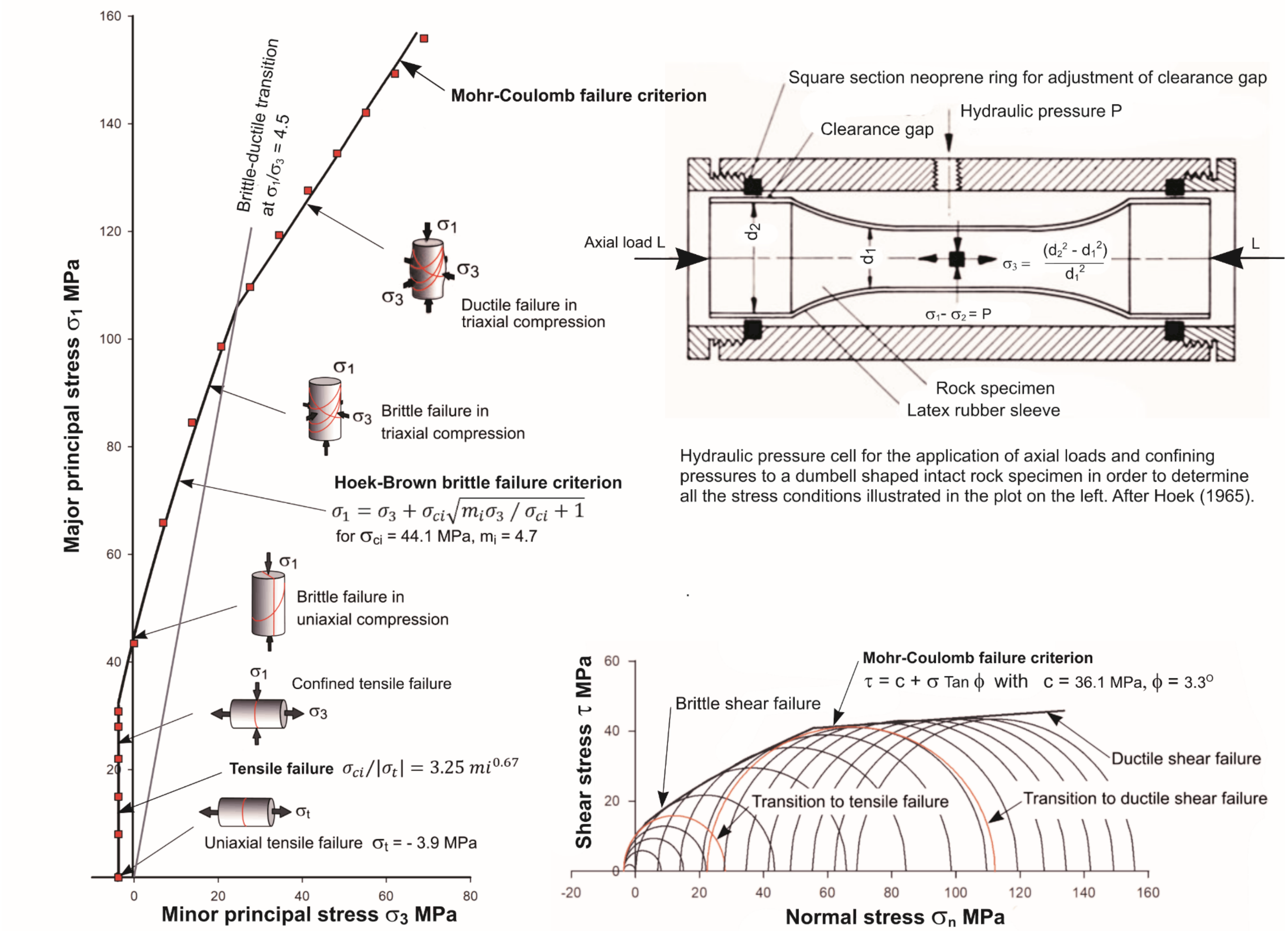



Figure 5: An interpretation of the results of triaxial tests on intact rock specimens.

Figure 5 presents the results of triaxial compression tests on samples of Wombeyan Marble by Paterson (1958) and tensile tests on Carrara marble by Ramsey and Chester (2004). The plot indicates that the tensile tests fall along a well-defined tension cut-off. Brittle failure of the intact rock is represented by the Hoek-Brown failure criterion which defines brittle shear failure in the stress regime between tensile failure and ductile failure at higher stress levels. For these marbles, a transition to ductile failure occurs at the ratio of $\sigma_1/\sigma_3 = 4.5$ and this ductile failure is defined by the linear Mohr-Coulomb criterion which is generally used to define failure in soils.

Figure 6 shows that the relationships presented in Figure 5 apply to most rock types of interest in rock engineering.

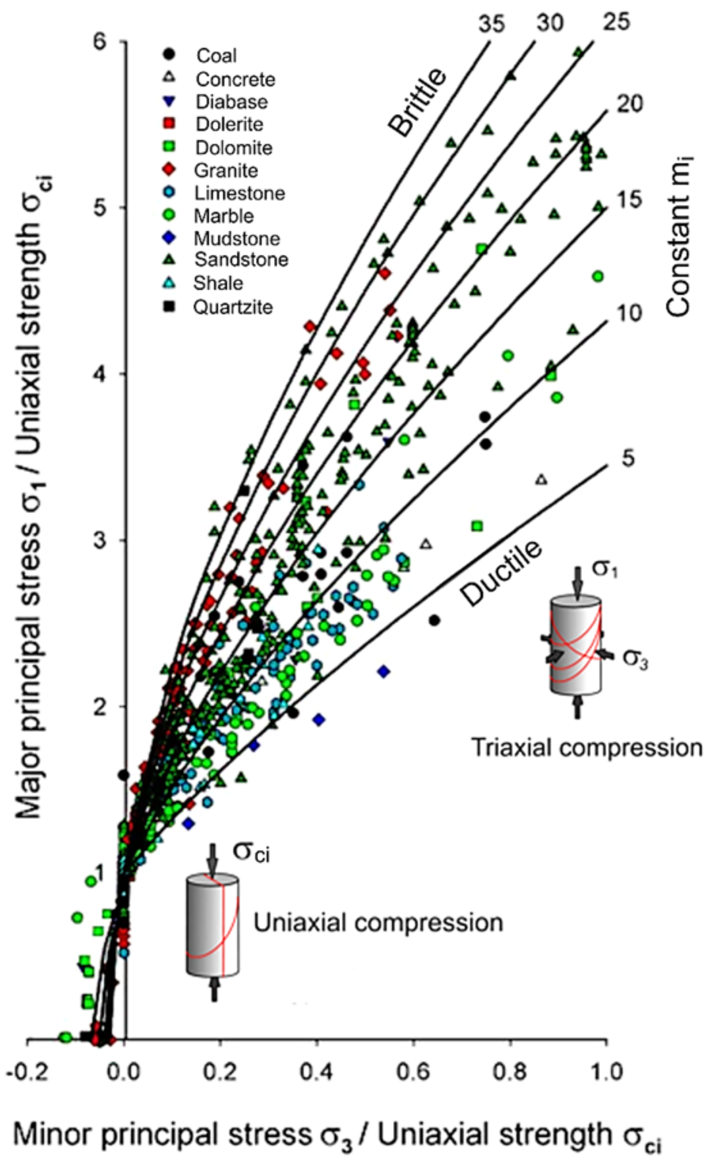



Figure 6: Plot of triaxial and tensile test data for a wide range of intact rock samples, illustrating that relationships presented in Figure 4 apply to most rock types.

## Jointed rock masses

While the behaviour of intact rock is of fundamental importance in rock engineering, an even more important topic, in practical applications in the field, is the behaviour of jointed rock masses, such as those illustrated in Figures 7 and 8. The discontinuities in these illustrations are bedding planes, joints, shear zones and faults resulting from deformations in the rock mass during formation and subsequent movements of the overall rock mass due to movements in the earth's crust. In hard rock masses these discontinuities tend to occur in families of planes, some of which extend for significant distances through the rock mass, as can be seen in Figure 7.

As shown in Figure 7, the spacing of the discontinuities, relative to the size of the structure under construction, is important in determining the mode of overall rock mass failure. The tunnel on the left has widely spaced bedding planes and joints and the failures in the rock mass surrounding the tunnel involve large blocks falling due to their own weight. The highway cutting on the right, created by careful blasting, has a stable vertical slope of interlocking blocks formed by predominantly horizontal and vertical discontinuities.  This type of rock mass can be dealt with as a homogeneous material when the spacing between the discontinuities is small compared to the dimensions of the rock mass under consideration.

Figure 7: A tunnel excavated through rock with widely spaced intersecting discontinuities and a slope cut into a blocky rock mass by careful blasting during the construction of a highway.

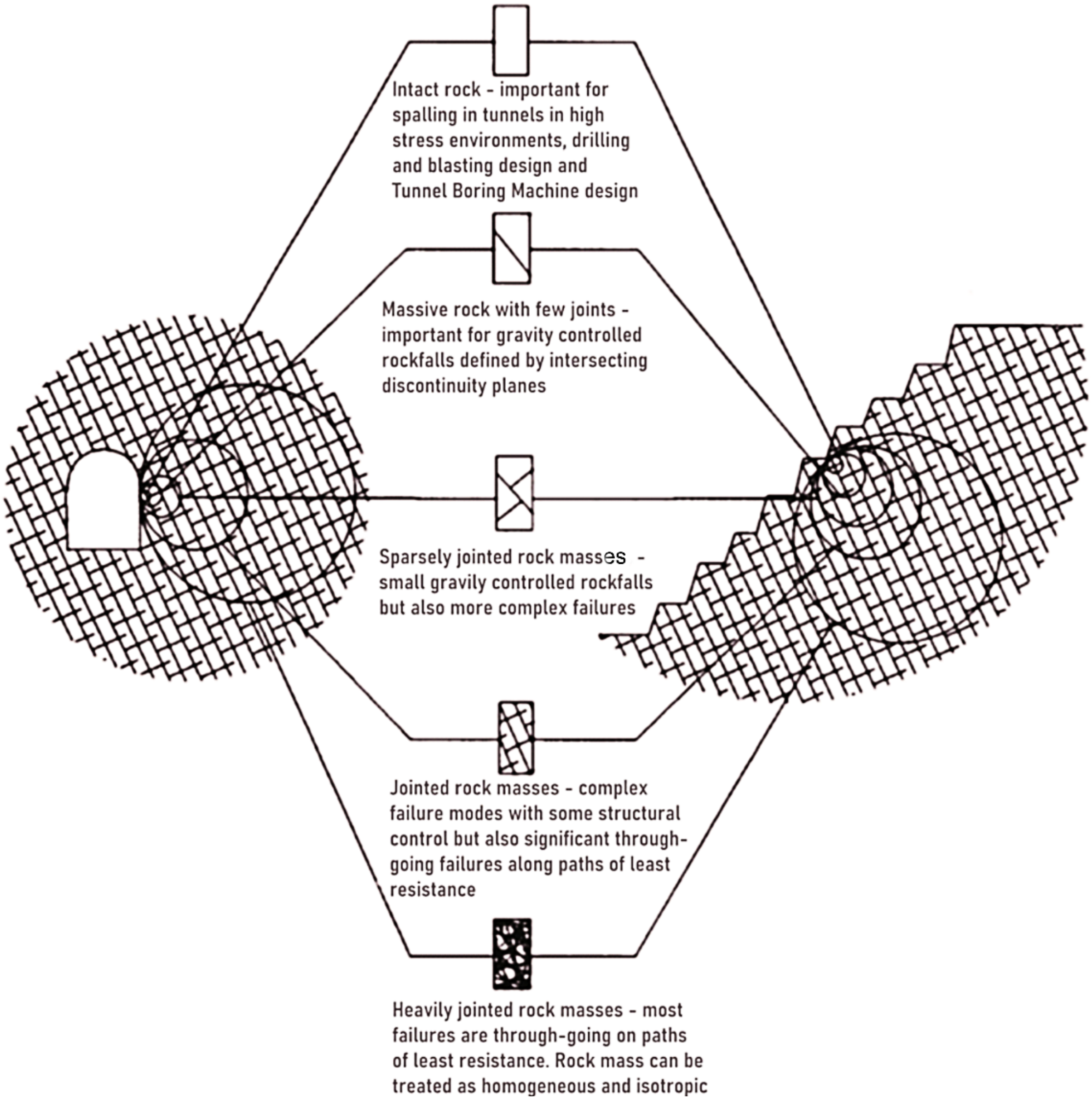



Figure 8: The influence of scale on jointed rock masses.

In addition to the location and orientation of the discontinuities in a rock mass, the shear strength of the discontinuity surfaces is important in any analysis of the failure process in a jointed rock mass, discussed in detail in a later chapter in these notes.

Figure 9 illustrates the basic principles of a shear test and a photograph of small shear test machine that can be used in the field, if necessary, for shear tests on joint surfaces in diamond drill core samples.

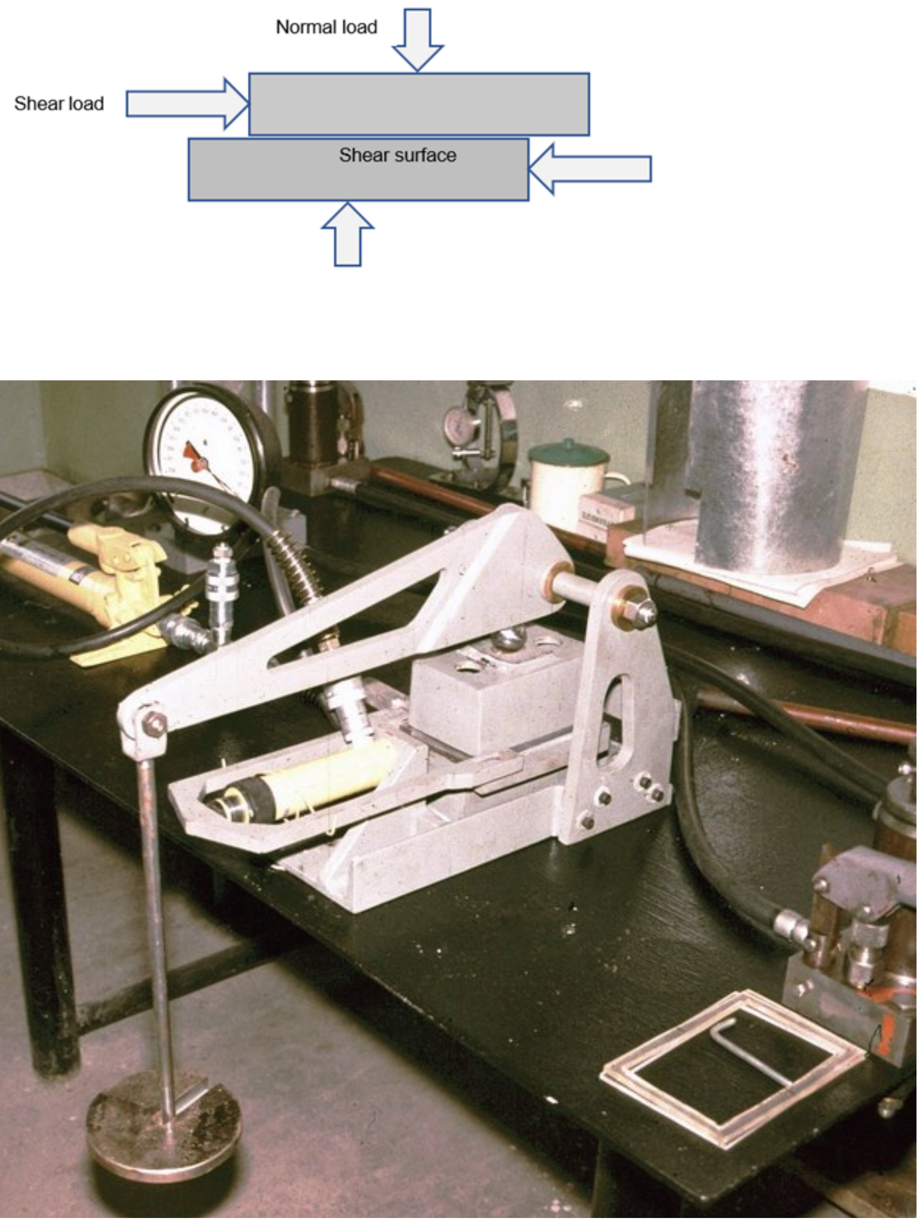



Figure 9: Diagrammatic illustration of a shear test and a simple shear machine for testing discontinuities in core samples.

The strength of a jointed rock mass can be described by the Hoek-Brown failure criterion defined by $\sigma_1 = \sigma_3 + \sigma_{ci}(m_i\sigma_3/\sigma_{ci} + s)^{0.5}$ (Hoek and Brown, 2019). This is the same equation used for calculating the intact rock strength but, for rock masses, the values of the constants $m_i$ and $s$ both decrease to account for the strength reduction due to the presence of discontinuities.

**Relationship between rock mass strength and in situ stress in tunnels**

Figure 10 is a plot of tunnel strain, defined by the ratio of tunnel closure to diameter, for a range of rocks with an increasing ratio of rock mass strength to the in-situ stress in the surrounding rock mass. This plot was published by Hoek and Marinos (2000). It was derived by running analyses to calculate tunnel strain for several thousand cases of tunnels of different diameters, different rock mass properties and different depths below surface. Theoretical solutions by Duncan Fama (1993) and Carranza-Torres and Fairhurst (1999) were used to calculate the strains for these cases. The behaviour of all the tunnel analysed follows a clearly defined pattern, which is adequately predicted by means of the equation $\varepsilon = 0.2\ (\sigma_{cm}/p_0)^{-2}$ included in the figure. Several illustrations of tunnel squeezing are also included in the figure.

Examples of the conditions that can be encountered in mining tunnels in rock masses, with a range of properties and in situ stresses, are given in Figure 10. Depending upon the stress distribution in the rock mass, shallow spalling of the rock in the walls of the tunnel can occur at a relatively shallow depth. This spalling propagates a limited distance into the rock and stabilizes relatively quickly. With an increase in depth, the spalling can become more severe and significant support, in the form of wire mesh and rockbolts, is usually required to stabilize the tunnel walls. At greater depths, violent shear failure or rockbursting of the rock mass surrounding the tunnel, can occur. This can be very dangerous and is difficult to control by the installation of support. The only mitigating factor is that, once the energy of the burst has been released, the rock mass stabilizes. This means that, in some cases it may be possible to install flexible support, such as steel ribs, which will keep the broken material in place until repairs can be made.

The presence of geological discontinuities, such as faults, shear zones and joints, results in a deterioration of the rock mass strength and deformation characteristics. This causes a decrease in the stability of the rock mass surrounding the tunnel. Combining the influence of both depth below surface and rock mass quality results in a severe reduction of tunnel stability. The photograph at the top of Figure 10 illustrates severe squeezing in the Yacambú-Quibor Tunnel in Venezuela, mined in weak graphitic phyllite at a depth of up to 1270 m below surface (Hoek and Guevara, 2009).

A long tunnel may pass through a variety of rock types and in situ stress conditions, with the result that more than one type of instability may be encountered. Recognition of which types of instability are likely to occur and what types of mining methods and support systems will be most effective is critical in the very early stages of route selection and planning. It is probably at this stage of a tunnelling project that an experienced consultant, with many years of practical experience, can provide the most useful input to a project.

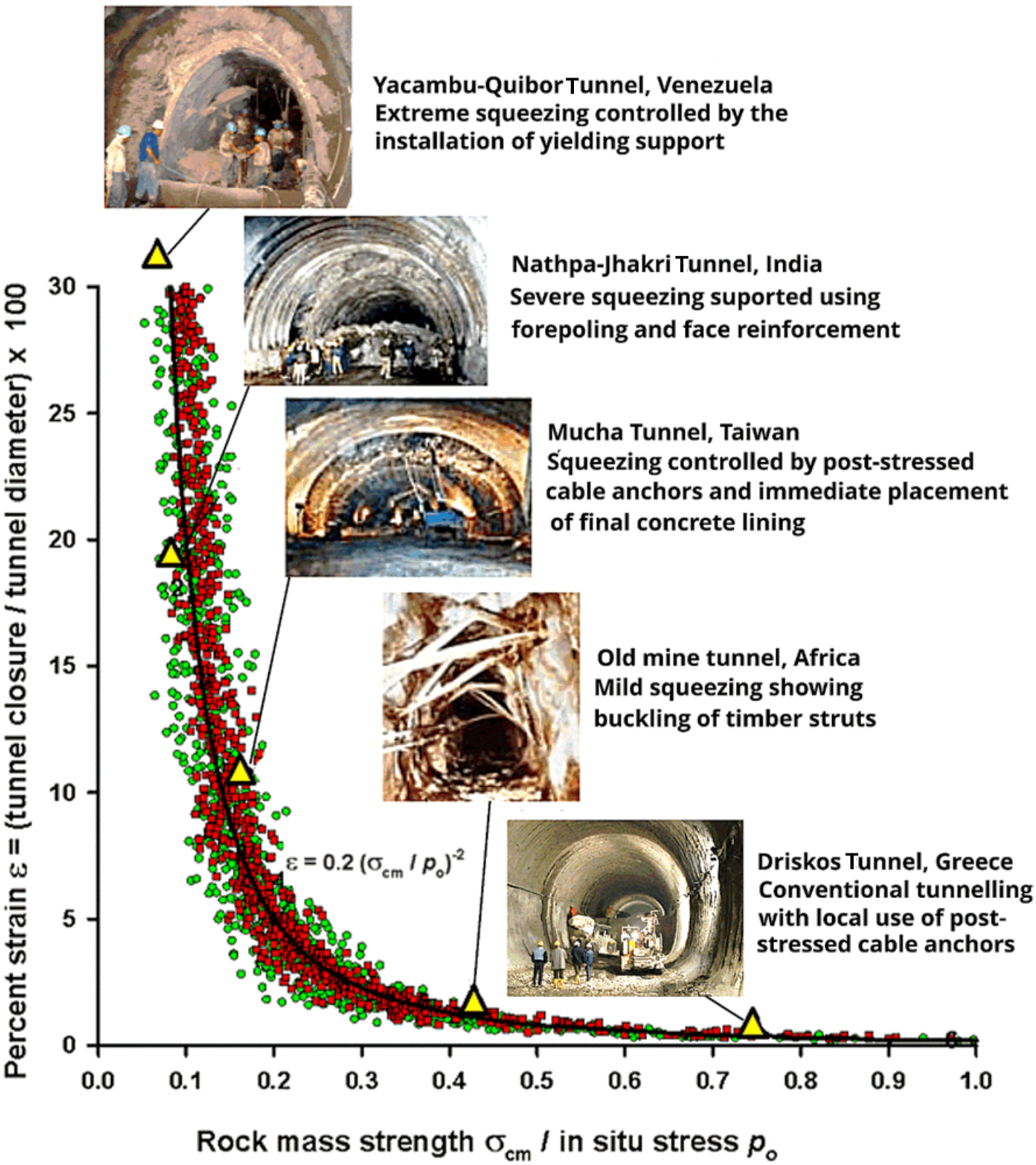



Figure 10: Approximate relationship between tunnel strain and the ratio of rock mass strength to the in-situ stress magnitude, together with comments on geotechnical issues and support solutions associated with different strain levels. Note that the plot is for unsupported tunnels.

**Driskos Tunnel on the Egnatia Highway in Greece**

To demonstrate the processes described above, an example involving one of the tunnels on the Egnatia Highway in Greece is presented in the following discussion. This highway runs for 680 km across Northern Greece. It meets all the requirements set out by the European Union for highways forming part of the Trans-European Highway Network. The highway includes 72 two-lane tunnels, with an internal diameter of 11 m and a total length of approximately 100 km. Fifty interchanges and 1,650 bridges, with a total length of about 64 km, are also included in the highway, which was completed in 2009. The Driskos Tunnel was excavated through a range of different rock types, the weakest of which was sheared flysch like that illustrated in the centre of the photograph in Figure 11.

Figure 11: Appearance of sheared flysch in a surface outcrop along the Egnatia Highway.

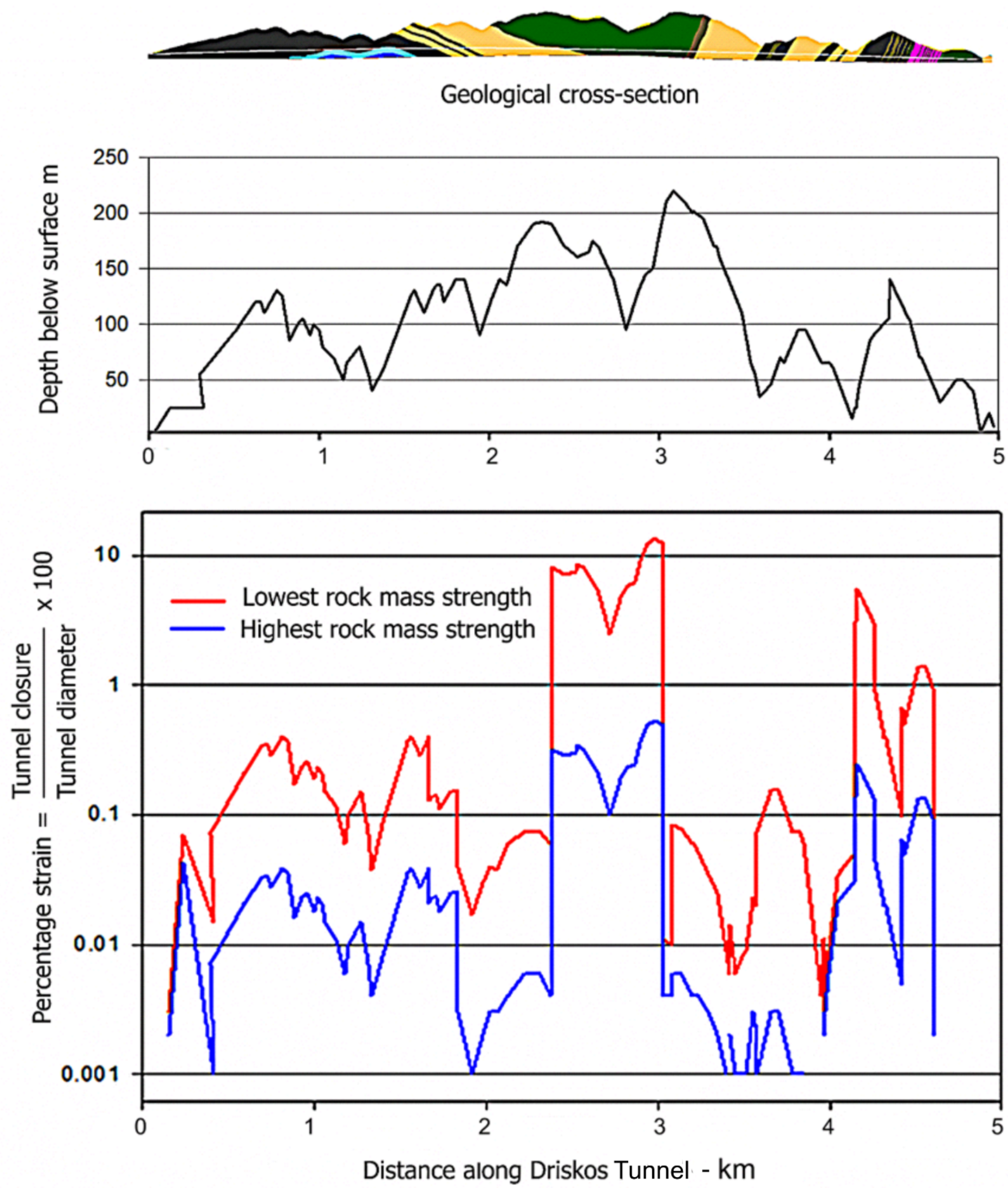



Figure 12: Results of an analysis of tunnel closure of the Driskos Tunnel, based on the stresses induced by the depth of the tunnel and the estimated rock mass strength of rock masses along the tunnel.

Figure 12 presents the results of an analysis of potential squeezing in the unsupported Driskos Tunnel, indicating that strains of approximately 10% were anticipated between distances of 2.3 km and 3 km, in poor quality flysch at depths between 100 and 200 m below surface. As shown in Figure 10, minor squeezing of the tunnel could be anticipated at this depth.

To check on the adequacy of conventional support systems to stabilize this tunnel, the analysis was re-run with a uniform support pressure of 1 MPa. This is typical of the support pressures that can be generated in a 12 m span tunnel with rockbolts, lattice girders or steel sets used in combination with shotcrete lining (Hoek, 1999). The results of this analysis suggested that the strains in the tunnel would be reduced by approximately 50% which suggested that stabilization of the tunnel by means of relatively conventional support measures would be entirely feasible.

Figure 13 shows that minor squeezing did occur as predicted and that it was necessary to supplement the support locally by the installation of patterns of 10 m long post-stressed cables. The tunnel was excavated successfully, and shown in Figure 14, with a shotcrete layer to complete the temporary support system. A final concrete lining was placed after completion of the excavation, using the components illustrated in Figure 15.

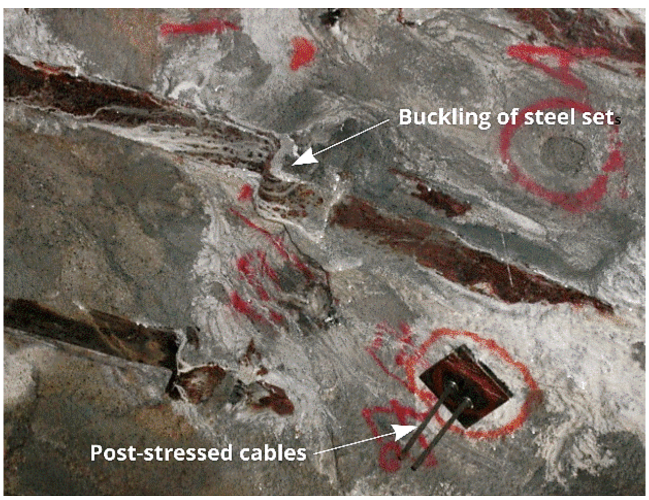



Figure 13: Deformation of steel sets because of mild squeezing in the Driskos Tunnel at the location predicted in Figure 12. Post-stressed cables of 10 m length were used to supplement local support.

Figure 14: Driskos Tunnel being advanced by top-heading and bench drill and blast methods. A 20 cm thick shotcrete lining was added to the steel sets and rockbolts to provide support pending the installation of the final concrete lining.

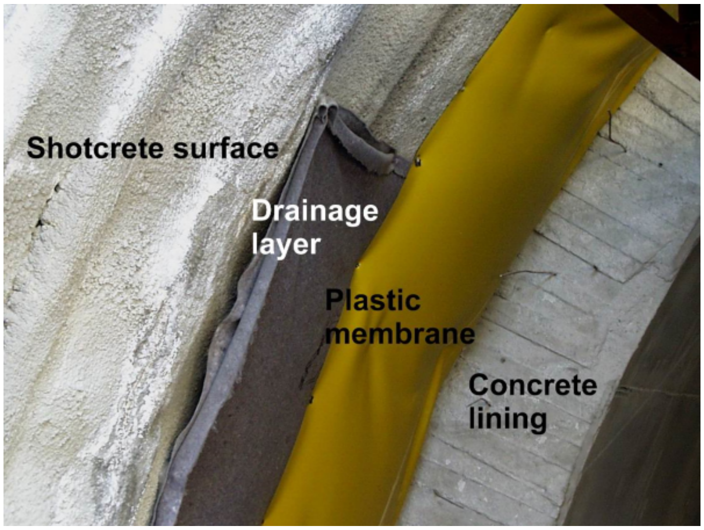



Figure 15: Details of placement of the final concrete lining.

**Decision making in Rock Engineering**

The example of the Driskos Tunnel presented above shows that, even in the case of the excavation of a relatively simple tunnel, several decisions must be made by the designer and the contractor to ensure that the tunnel can be constructed safely. It is necessary to ensure that enough information is available to make these decisions at appropriate times during the design and construction process. This requires careful planning at the start of the project.

Table 1 presents a summary of the steps involved in the design and construction of a typical rock engineering project such as a tunnel, a dam or bridge foundation, a rock slope in an open pit mine or a highway. Each project is unique, and details of the steps outlined must be chosen and refined to meet the requirements, constraints, and budgets for each case. Figure 16 is a simplified presentation of the increase in confidence in the design for each of the steps presented in Table 1. The general aim in this process is to minimize the risk of failure to the lowest possible level, given the inherent uncertainties associated with the properties and behaviour of naturally occurring rock masses. While strenuous efforts have, and continue to be, made to improve the techniques and tools for predicting rock mass properties and their behaviour when used as engineering material in civil and mining engineering structures, it is highly improbable that we will be able to reach the level of confidence associated with the use of man-made materials such as steel or concrete.

The examples presented in the following pages of this document are intended to present actual projects in order to demonstrate some of the methods used to arrive at practical, economical and safe solutions to projects in which rock and rock masses are used as engineering materials.

Table 1: Steps involved in the design and construction of a rock engineering project.

| Step | Examples | Processes |
| --- | --- | --- |
| Definition of problem | Mine layout, tunnel route selection, open pit mine layout, large underground cavern configuration, foundation specifications | Creation of preliminary geological model, walk-over surveys, preliminary assessment of anticipated problems |
| Pre-feasibility study | Definition of investigations and studies required for design of slopes, tunnels, foundations, and other components. Analysis of functional and budgetary requirements and constraints. | Drafting of project layout and alternative solutions, if available. Drilling of exploration boreholes at critical locations, description and classification of rock outcrops |
| Feasibility study | Execution of defined investigations and studies, including creation of detailed geological maps and cross-sections, diamond drilling for rock samples, laboratory testing of intact rock and discontinuities, detailed classification of rock and rock mass to estimate rock mass properties, collection of groundwater data to create groundwater model. Detailed analyses, using empirical and numerical tools, of critical project details to establish ranges of practical and cost-effective methodologies and designs. | Interpretation and analysis of information collected, refinement of project layout options and selection, construction sequences, contractual details. In some cases, access roads, accommodation for designers and contractors, construction of pilot or access tunnels and installation of drainage systems can be carried out in an initial contract, followed up by construction of project components in the final design and construction stage. |
| Final design | Use of pilot tunnels or trail excavations to confirm predicted rock mass behaviour. Detailed analyses of defined problems by experienced engineers and geologists, using most sophisticated tools available, to finalize choices and sequencing of excavation methods, temporary and final support designs, drainage and ventilation measures. Ongoing monitoring, back-analyses and evaluation of stability issues and deformations of completed project components. | Construction and completion of the project which may involve refinement of design and construction components, based on observed and measured behaviour and re-analysis of refined estimates of rock mass properties, in situ and induced stresses, groundwater and drainage conditions and, in some cases, the performance of excavation and support methodologies. |

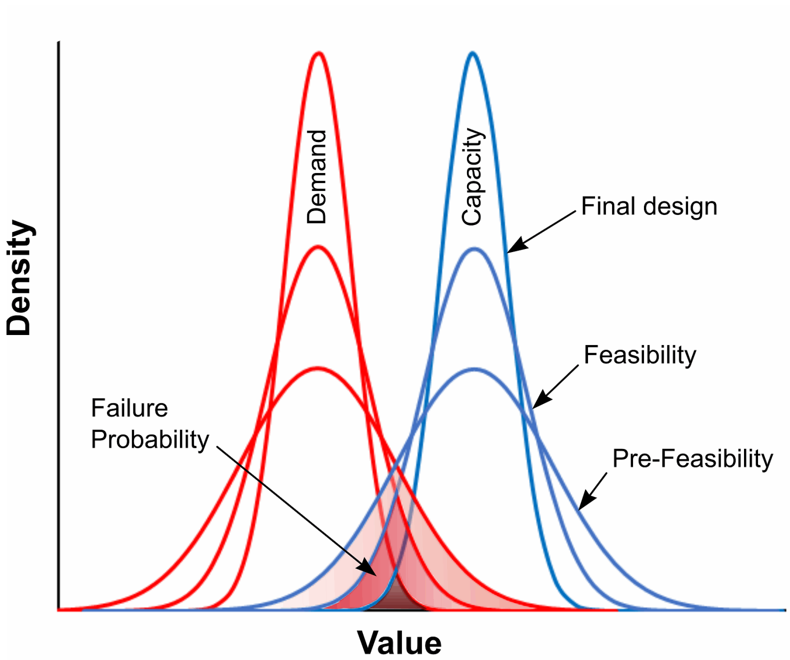



Figure 16: Reliability of the design at different stages of investigation, design, and construction. (Hoek, 1991, 1993).

A set of hypothetical distribution curves representing the degree of uncertainty, associated with available information on shear strength parameters and disturbing stresses for different stages in the design of a rock or soil structure, is illustrated in Figure 16. The failure probability is defined by the overlap of the Capacity and Demand plots.

During preliminary design studies, the amount of information available is usually very limited. Estimates of the shear strength of the rock or soil are generally based upon the judgement of an experienced engineer or geologist which may be supplemented, in some cases, by estimates based upon rock mass classifications or simple index tests. Similarly, the disturbing forces are not known with very much certainty since the location of the critical failure surface will not have been well defined and the magnitude of externally applied loads may not have been established. In the case of dam design, the magnitude of the probable maximum flood, which is usually based upon probabilistic analysis, frequently remains poorly defined until very late in the design process.

For this case, the range of both available shear strength and disturbing stresses, which must be considered, is large. If too low a factor of safety is used, there may be a significant probability of failure, represented by the section where the distribution curves overlap in Figure 16. To minimise this failure probability, a high value for the factor of safety is sometimes used. For example, in the 1977 edition of the US Bureau of Reclamation Engineering Monograph on Design Criteria for Concrete Arch and Gravity Dams, a factor of safety of 3.0 is recommended for normal loading conditions when …'only limited information is available on the strength parameters'. This value can be reduced to 2.0 when the strength parameters are … 'determined by testing of core samples from a field investigation program or by past experience'.

During detailed design studies, the amount of information available is usually significantly greater than in the preliminary design stage discussed above. A comprehensive program of site investigations and laboratory or in situ shear strength tests will normally have been carried out and the external loads acting on the structure will have been better defined. In addition, studies of the groundwater flow and pressure distributions in the rock mass, together with modifications of these distributions by grouting and drainage, will usually have been carried out. Consequently, the ranges of shear strength and driving stress values, which must be considered in the design, are smaller and the distribution curves are more tightly constrained.

### Changes in the size and scope of projects involving rock engineering

In the past fifty years, there have been dramatic changes in the type and scale of civil and mining engineering projects involving the use of rock masses as the principal construction material. Tunnels have become significantly longer and deeper, foundations larger and more complex and excavated open pit mine slopes with heights more than one kilometre, have become relatively common. To deal with these changes, improvements have been made in design methodologies for existing problems involving deformation driven tunnel failures, rockfalls, groundwater control and time dependent deformations in materials such as evaporites.

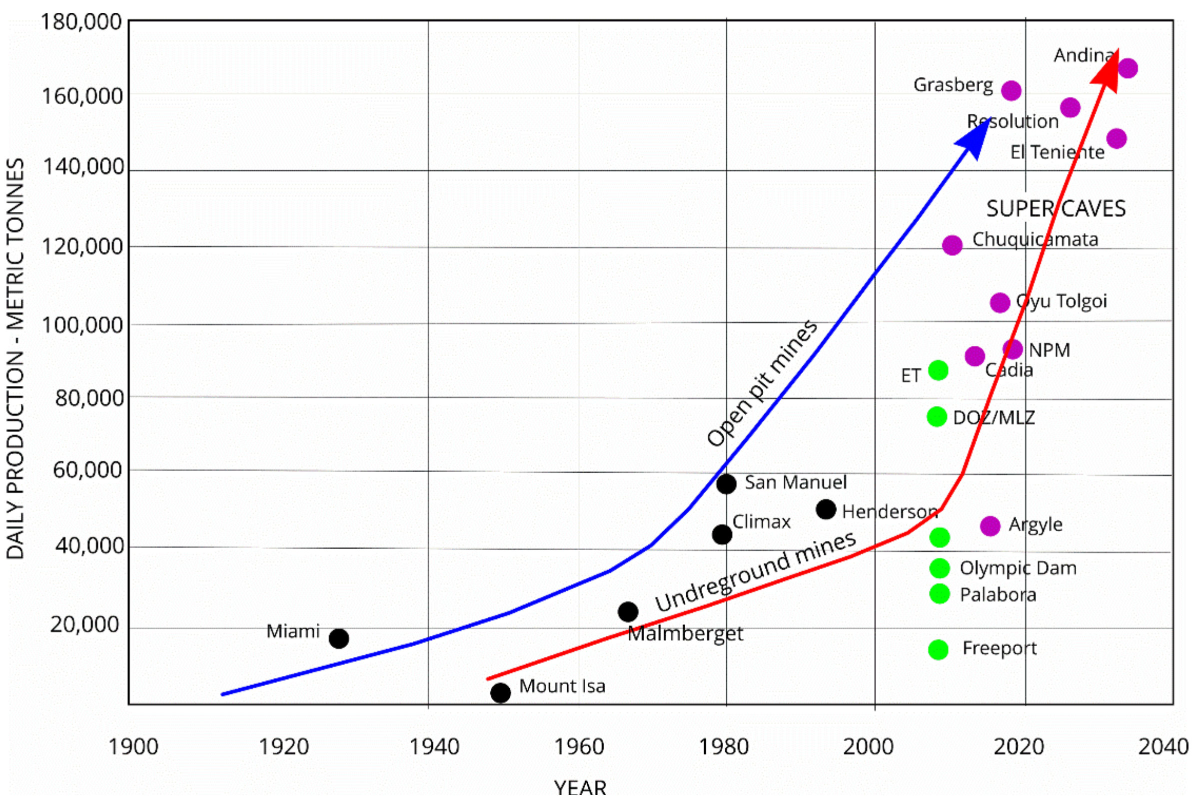



Figure 17: Changes in the production rate of open pit and underground mines (after Brown (2004) and Eberhardt (2019)).

An example of the rapid changes in open pit and underground mines is presented in Figure 17, which shows changes in the daily production rates during the 1970s in open pit mining and in the period from 2000 onward for underground mines. The locations of some of the mines, included in Figure 17, are shown in Figure 18.

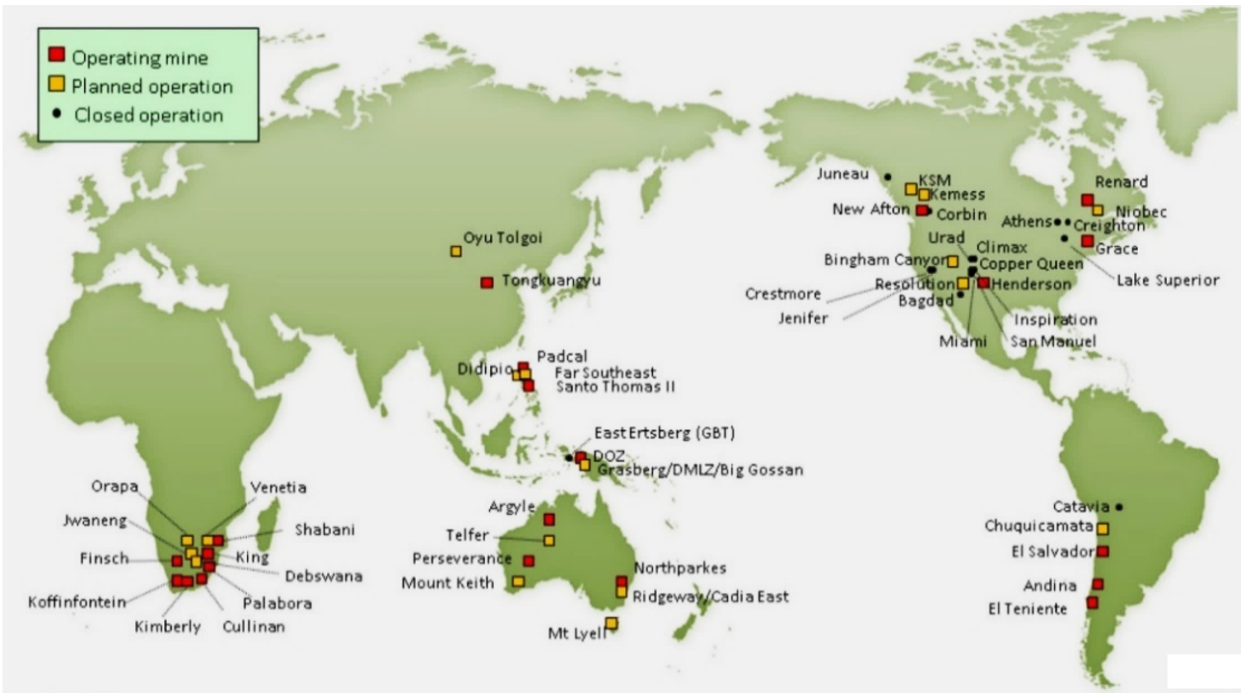



Figure 18: Major open pit and underground mines around the world, after Woo et al (2013).

The development of very large open pit mines, such as the Chuquicamata Copper Mine in Chile illustrated in Figure 19, became feasible because of an improved understanding and experience in the behaviour of rock masses and improvements in equipment and methodologies used in excavation.

Hoek and Martin (2014) have described a detailed stability analysis of the east wall of the Chuquicamata Mine. This is a good illustration of the application of many of the most advanced theoretical tools available to the solution of a very practical problem involving the stability of a conveyor transfer station located in the rock mass behind the slope. Most of the crushed ore mined in the pit passed through this transfer station on its way to the processing plant, making its stability a critical factor in the overall operation of the mine.

Figure 19: The Chuquicamata open pit mine in Chile in 2013. Before conversion to an underground block caving operation in approximately 2019, the pit was 4 km long, 3 km wide and 1 km deep.

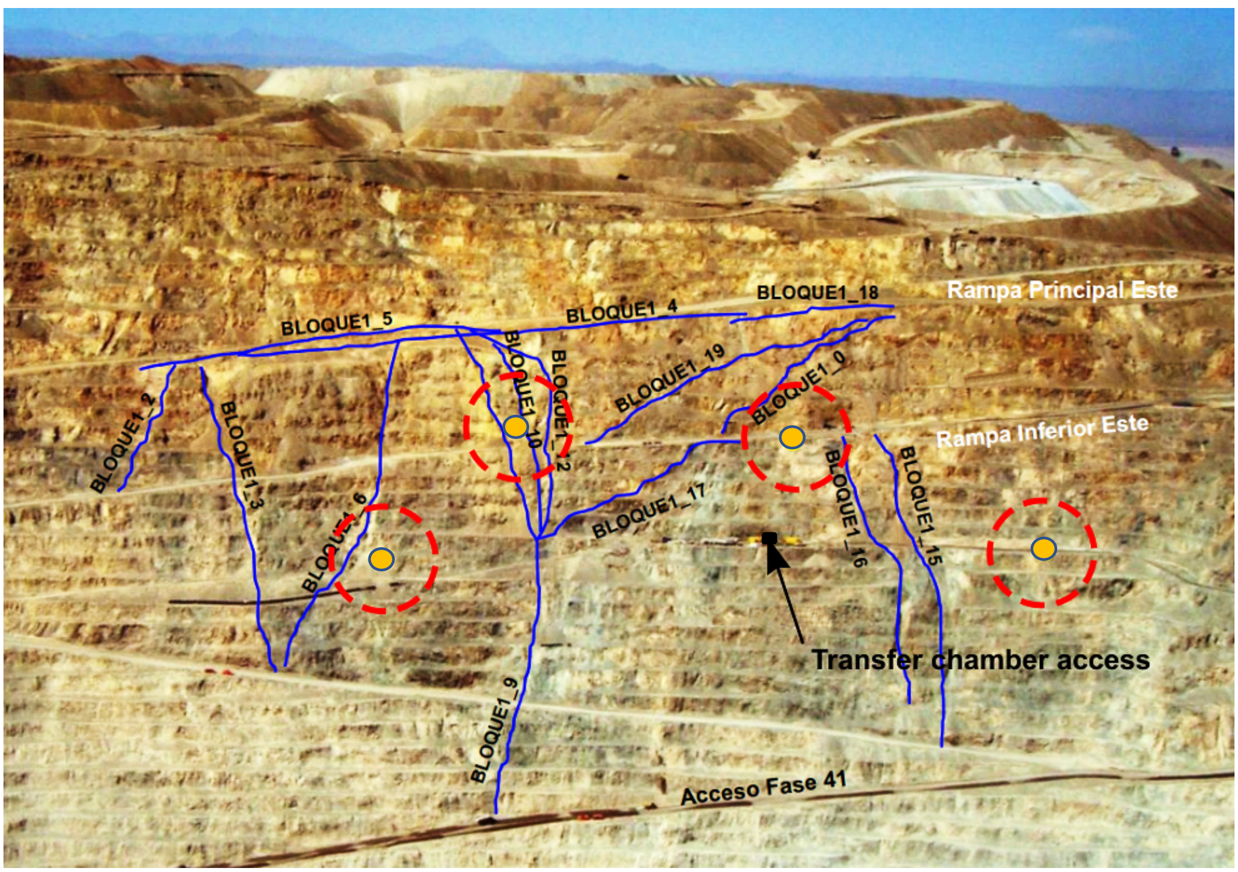



Figure 20: View of the Chuquicamata Mine east face showing locations of the conveyor transfer chamber, major structural features and optical targets for distance measurement (yellow spots circled in red).

Figure 20 illustrates the East Wall and the location of the entrance to the underground conveyor transfer station. A detailed 3DEC[1] analysis was carried out, by Pedro Varona of Itasca and Felipe Duran of Chuquicamata, in 2012/13 assuming that the overall rock mass could be treated as homogeneous, using the rock mass properties based on the Hoek-Brown criterion and the Geological Strength Index (Hoek and Brown, 2019), with super-imposed major discontinuities (shown in blue). The yellow dots are locations of some of the 1000 laser distance measurement targets that surround the open pit. An underground conveyor transfer chamber is located behind the potentially unstable wedge, defined by the coloured contours in the centre of the east face in Figure 21.

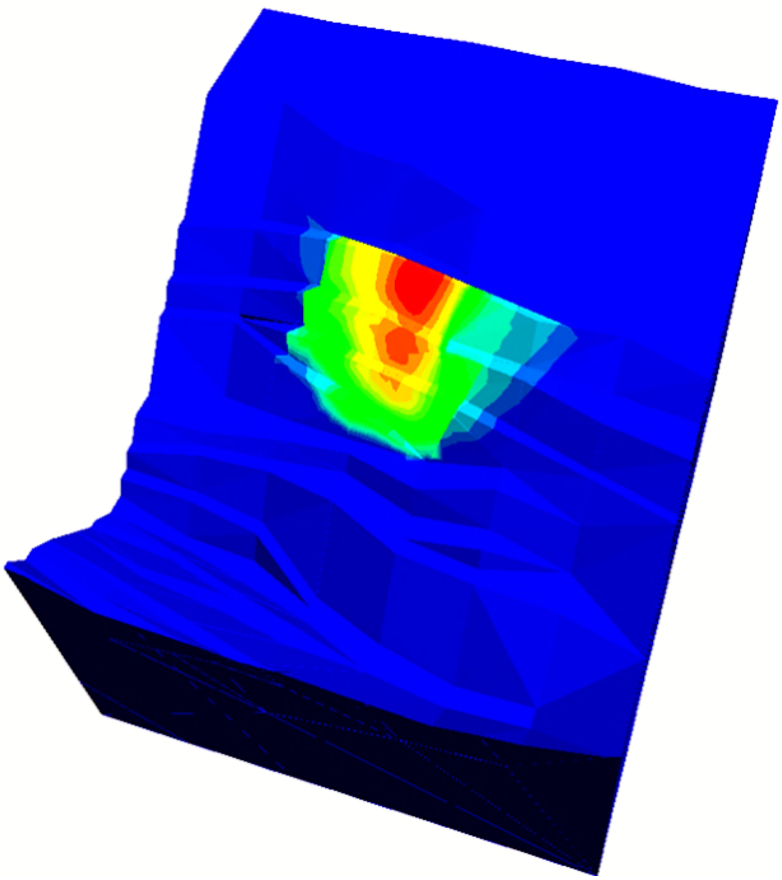


Figure 21: Displacement contours in the east face generated in an Itasca 3DEC model. The colours, from blue to red, show increasing displacement in the rock mass.

There are many other examples, in both open pit and underground mining, where practical decisions have been made based on sound rock engineering principles. Some of these decisions have incorporated detailed analytical studies, but most of them are made based on practical rock engineering experience acquired from years of empirical observation and cautious trial and error development.

[1] A three-dimensional discrete element numerical model in which individual elements, defined by intersecting structural features in the rock mass, can be incorporated (Itasca, 2002).

Underground mining is an unknown world to many of us. An excellent introduction to different methods used to mine ore can be found in Brady and Brown's book, Rock Mechanics for Underground Mining (Brady and Brown, 2004). Well-illustrated Chapters 12 and 15 on underground mining methods and longwall and caving methods, respectively, are particularly useful in helping the reader to distinguish between a huge array of complex ore recovery methods.

As shown in Figure 17, a spectacular step change around 2010 resulted from advances in block-caving technology. This is a very old mining technique, originally developed in the underground coal mines in the United Kingdom (Sainsbury, 2012), involving undercutting an orebody and allowing it to cave, in a controlled manner, into draw points where the coal or ore is collected. The principal features of a caving operation are illustrated in Figure 22 while details of the support required to stabilize draw-points are given in Figure 23. The most used approach for estimating caveability, developed by Laubscher (Diering and Laubscher 1987, Laubscher 1990, 1994, 2000) is based on a compilation of caving case histories – largely derived from low strength, kimberlitic deposits in South Africa.

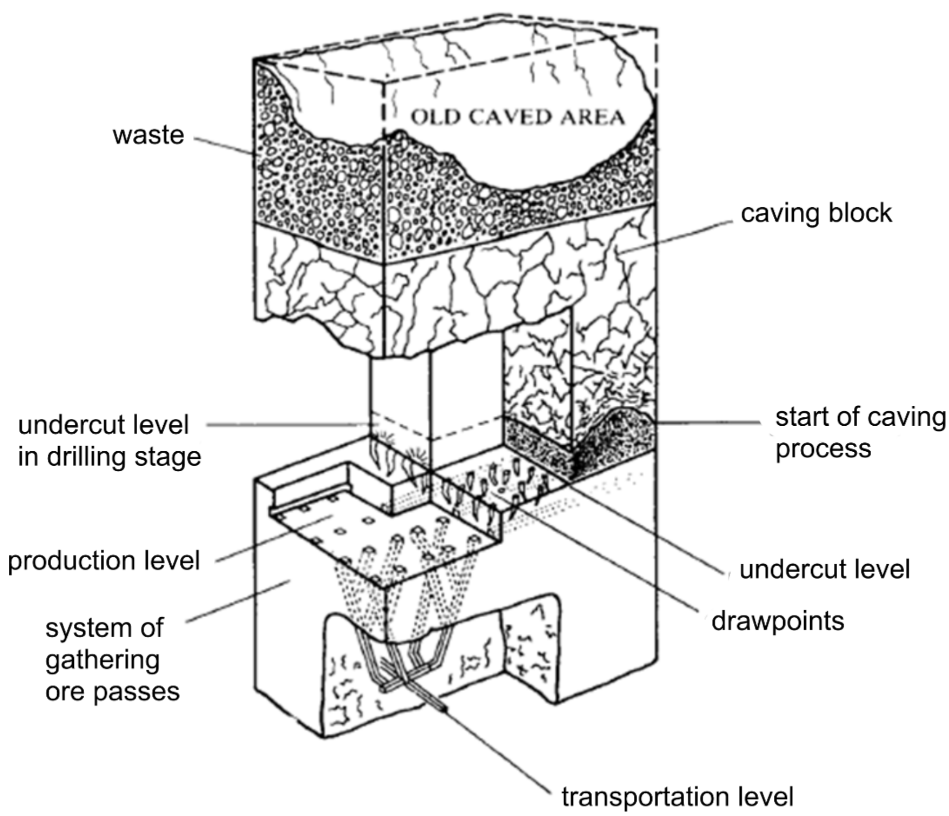



Figure 22: Principal features of a caving operation (Borquez, 1981).

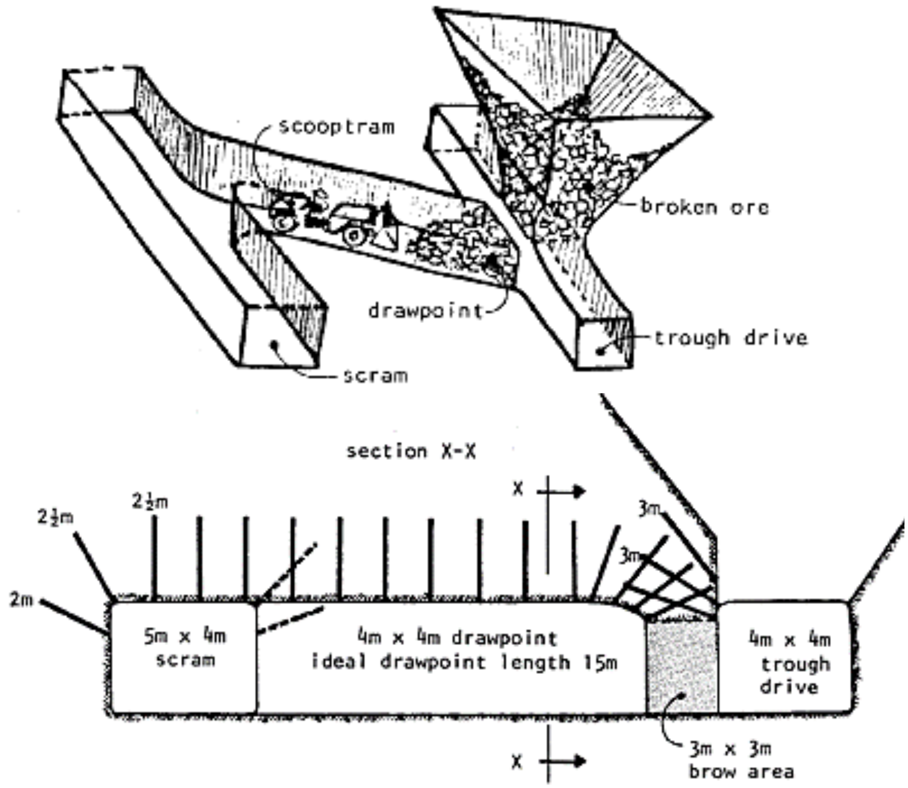



Figure 23: Placement of grouted bars to pre-reinforce a draw-point in a large, mechanised mine. The shaded area is blasted last, after the reinforcement has been installed from the draw-point and the trough drive (after Hoek and Brown, 1980).

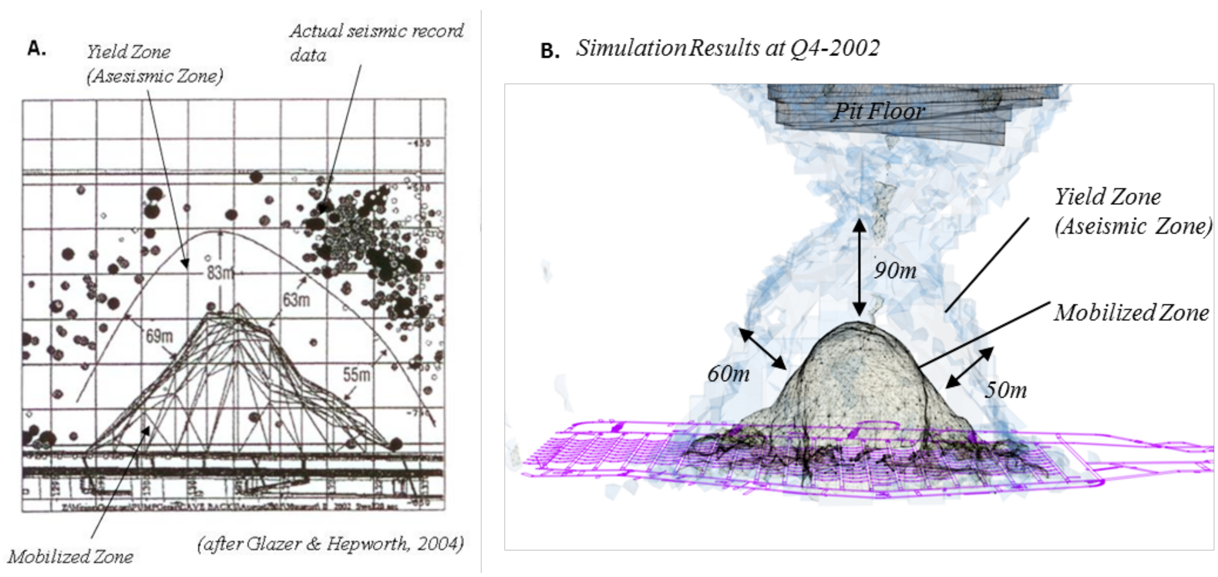



Figure 24: Observed seismic activity at the Palabora Mine during cave initiation and propagation (A) observed mobilized, yield and seismogenic zones during production (after Glazer and Hepworth, 2004); (B) numerical prediction of mobilized and yield zones during production at the Palabora Mine (after Sainsbury, 2012).

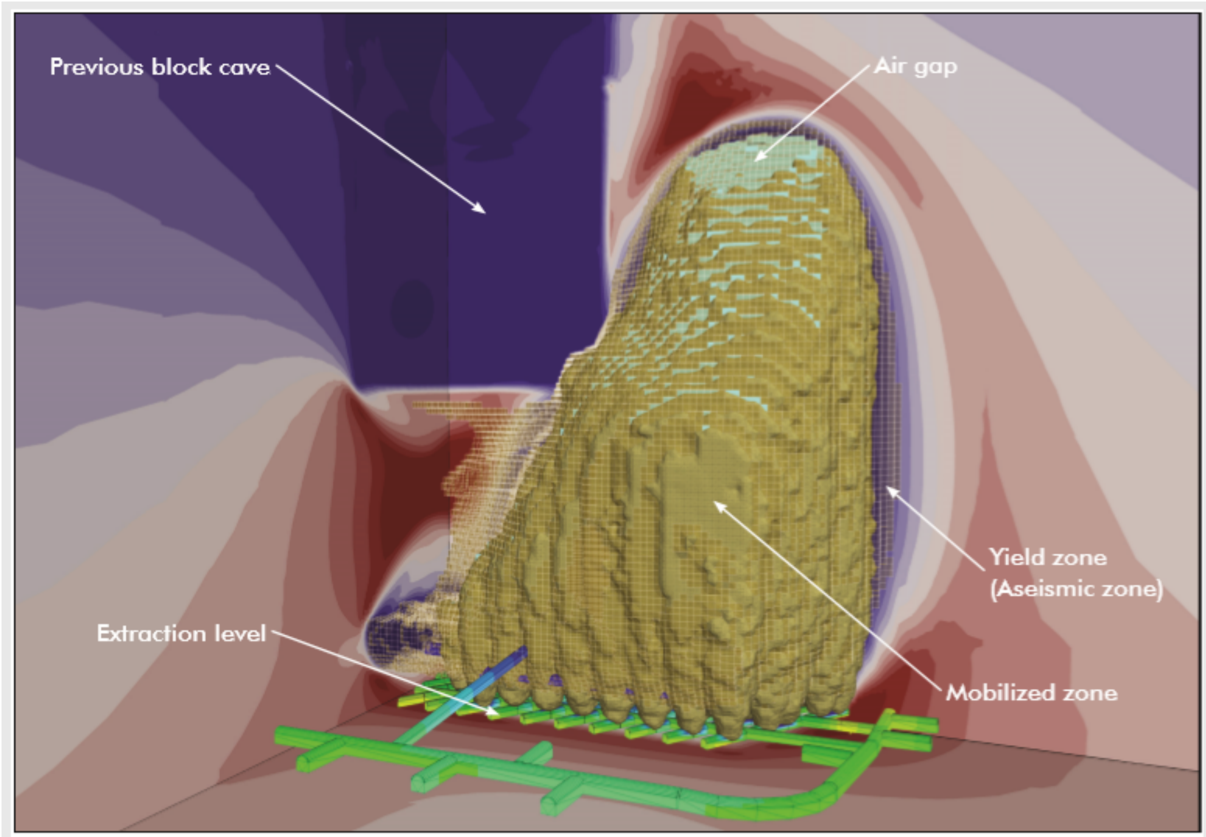



Figure 25: Coupled FLAC3D/REBOP block cave model indicating the predicted flow and yield (blasted/swell) zones and the formation of air gaps after 20 months of production at New Afton Mine, NewGold, Inc. Contours of the maximum principal stress are shown along octant cut-planes (blue for lower stress and red for higher stress). Image provided by Itasca Consulting Group, Inc**.,** with permission from NewGold, Inc.

Most of the mines designated "super caves" in Figure 17 are block caving operations. Control of the progressive caving and the movement of the broken rock, which is critical in maintaining a stable progressive mining operation, has benefitted greatly from the development of three-dimensional discrete element numerical tools. These methods, collectively named Synthetic Rock Mass Modelling tools, are generally accepted as the current state-ot-the-art in anisotropic rock mass behaviour analysis. An excellent discussion of this technology, with several case histories, can be found in a PhD thesis by Bre-Anne Sainsbury (2012). General discussion on caving by Lorig et al., 1995, Laubscher, 2000, Cundall, 2008, Sainsbury et al, 2008, and Mas Ivers et al, 2011, provide important background material on this topic. Several specific cases are discussed by Sainsbury and Lorig, 2005, Fuentes and Villegas, 2014 and Flores and Catalan, 2019.

Examples of the application of three-dimensional numerical analysis to the prediction of block caving in the Palabora Mine in South Africa and the New Afton Mine in British Columbia, Canada, are presented in Figures 24 and 25 respectively.

### Examples to illustrate the design principals used in rock engineering

Figure 26 shows a circular tunnel advancing through a rock mass in which in situ stresses exist due to the weight of the overlying rock mass and the lateral constraints provided by the surrounding rock mass. For the sake of simplicity, the three orthogonal principal stresses are shown as perpendicular and parallel to the tunnel axis. In fact, these stresses may be inclined at any direction; their magnitudes are dependent upon the tectonic history of the rock mass in which the tunnel is being driven.

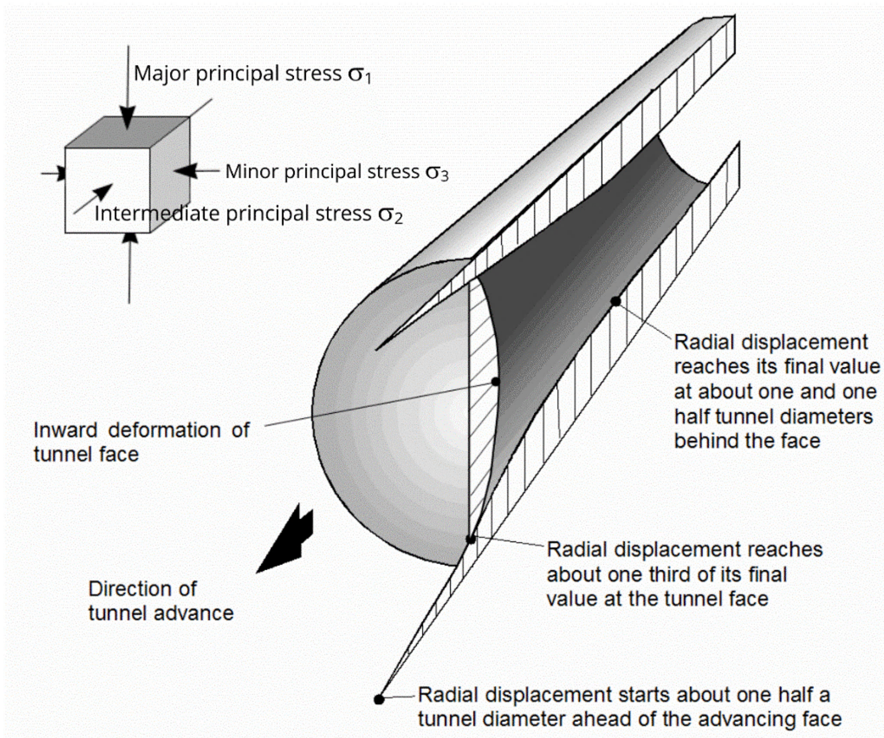



Figure 26: Advance of a circular tunnel through a stressed rock mass.

The boundary displacements, shown in Figure 26, result from the stresses induced in the rock mass surrounding the tunnel which, in turn, depend upon the depth below surface and the tectonically induced lateral stresses.

In situ stresses, expressed in terms of the vertical stress and the ratio k of horizontal to vertical stress, are plotted in Figure 27, reported by Brady and Brown (2004), based on 900 determinations of the pre-mining in situ state of stress at numerous locations of mining, civil and petroleum projects around the world. The line in the left-hand plot in Figure 27 is calculated on the basis that the vertical stress is a product of the depth below surface and the unit weight of the rock mass, assumed to be 0.027 $MN/m^3$. While the measured data are scattered around this line, the plotted line can be considered a reasonable average relationship between vertical in situ stress and depth below surface.

The right-hand plot in Figure 27 shows that the ratio of horizontal to vertical stress varies widely at shallow depth, with $k > 6$ having been reliably reported in some locations. With increasing depth, the variability decreases and the upper bound tends towards unity. A probable explanation for this is that the rock mass is not able to sustain shear stresses at great depth due to time-dependent viscoplastic flow, resulting from progressive failure of defects.

Brady and Brown (2004) also discuss the highly variable stress distribution which can be anticipated in jointed and fractured rock masses and quote several cases in which the state of stress is related to the locally dominant geological structure. They conclude that the viscoplastic flow, discussed above, together with the influence of geological structures, means that the virgin stress state in a rock mass is not amenable to calculation by any known method, but must be determined experimentally.

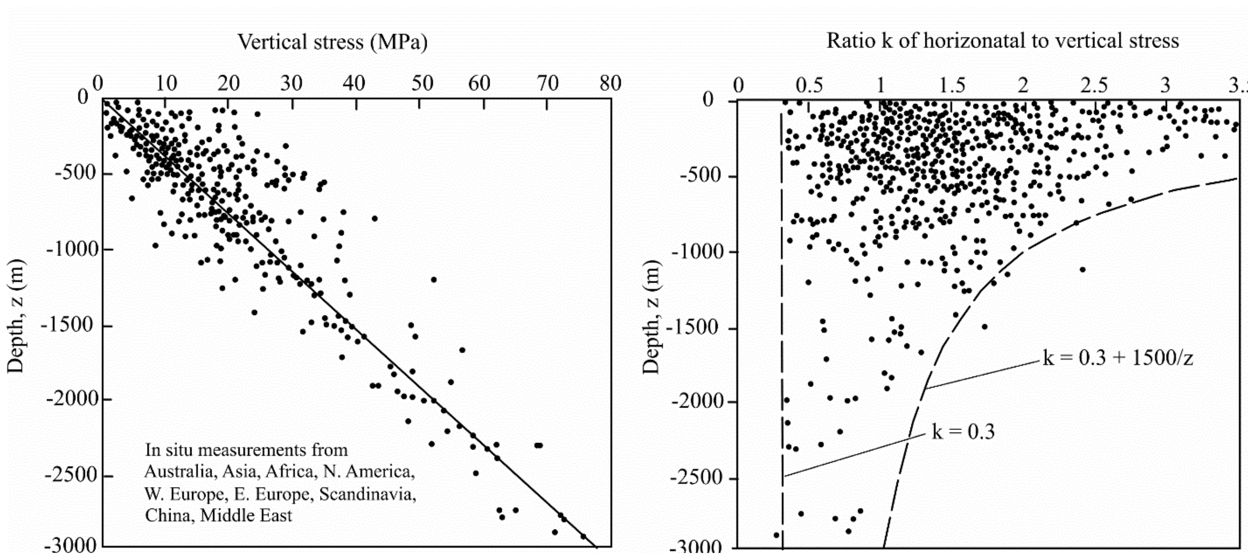



Figure 27: Variation of in situ vertical stress and the ratio of horizontal to vertical stress (k) with depth below surface (after Brady and Brown, 2004).

Measurement of these in situ stresses has been, and remains, an ongoing challenge in rock engineering (Leeman and Hayes,1966, Worotnicki and Walton, 1976). Fortunately, recent advances by Mills (1997), Mills and Puller (2017) have resulted in instrumentation that can be used to measure the three-dimensional in situ stress state at depths of up to 1 km below surface in competent rocks. In weak rocks such stress measurements are practically impossible, and stresses must be estimated based on engineering geology reasoning.

From a design perspective, before the commencement of any excavation, the most practical approach is to use all stress measurement information that is available, including that from adjacent projects, to make an estimate of the range of possible values of in situ stresses that could be encountered. The proposed excavation, such as a tunnel or an underground cavern, is then analysed for a variety of possible in situ stress regimes to determine whether conceptual designs can be created that would accommodate any stress conditions within the range considered.

Figure 28 shows that, when excavating tunnels or shafts in competent rocks at significant depths below surface, spalling failure of the rock in the excavation perimeter can occur at a depth (H) defined by a combination of the uniaxial compressive strength (UCS) of the rock and the ratio of horizontal to vertical stresses (k). For example, as shown by the red dot in Figure 28, an intact rock with a UCS value of 150 MPa will spall at a depth of H = 500 m when the ratio of horizontal to vertical stress is k = 2.

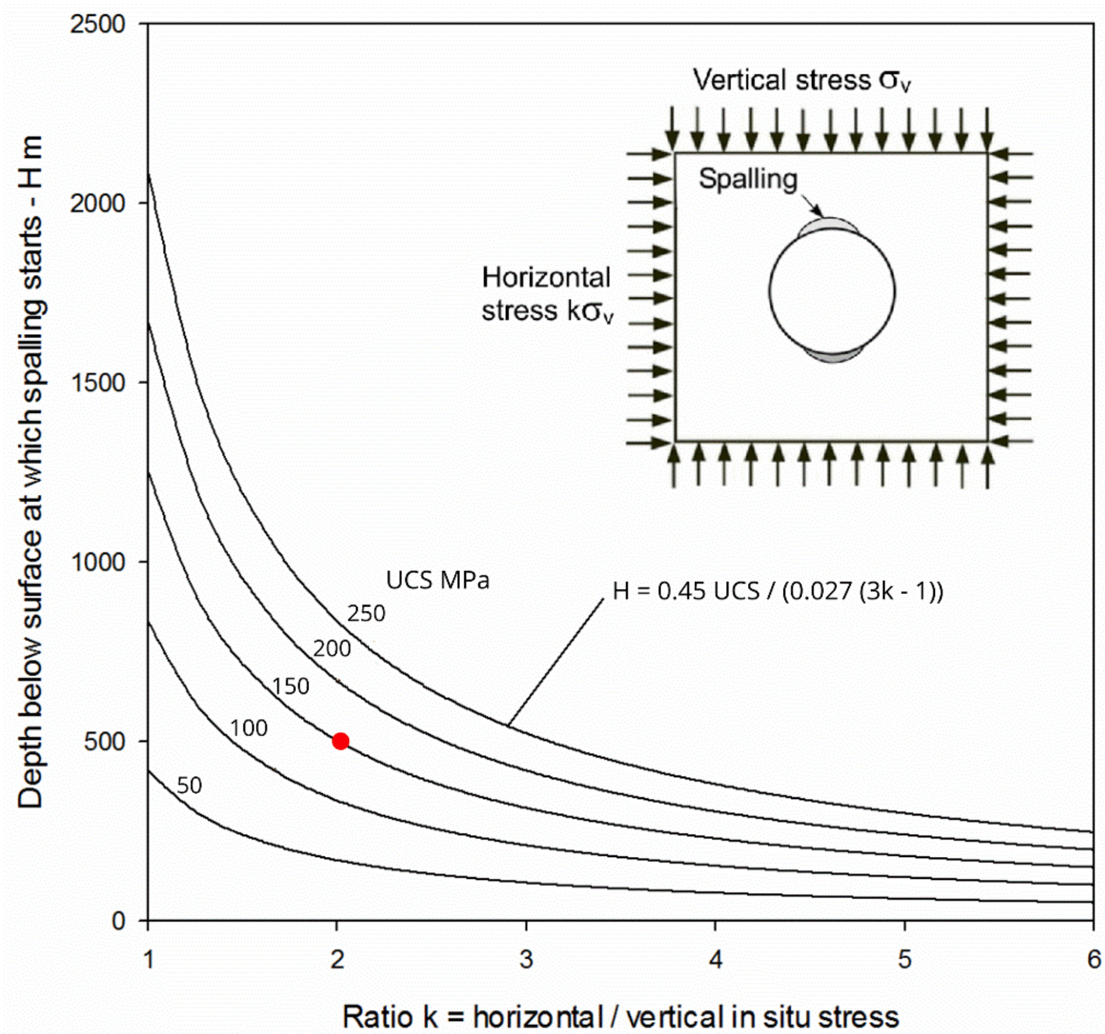



Figure 28: Approximate depth at which spalling occurs in a circular tunnel in intact rock.

Spalls, of the type and size illustrated in Figure 29 do not pose a significant threat since they propagate a limited distance, depending upon the strength of the rock and the in-situ stresses, and then stabilize. Small rockfalls can be contained by a layer of wire mesh supported by a pattern of rockbolts. A final lining consisting of a layer of shotcrete is generally adequate in such cases. The approximate depth of the spalls in a circular tunnel of a shaft can be estimated from the plot reproduced in Figure 30, in which the maximum boundary stress $\sigma_{max}$ is calculated by simple elastic theory.

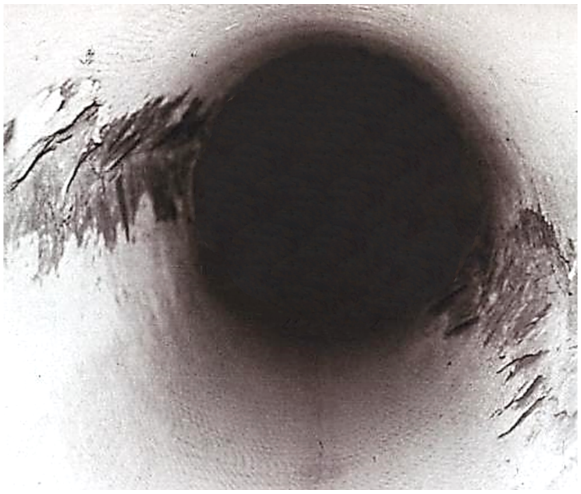


Figure 29: Spalling in the walls of a bored vertical shaft in competent rock at a depth approximately 600 m below surface.

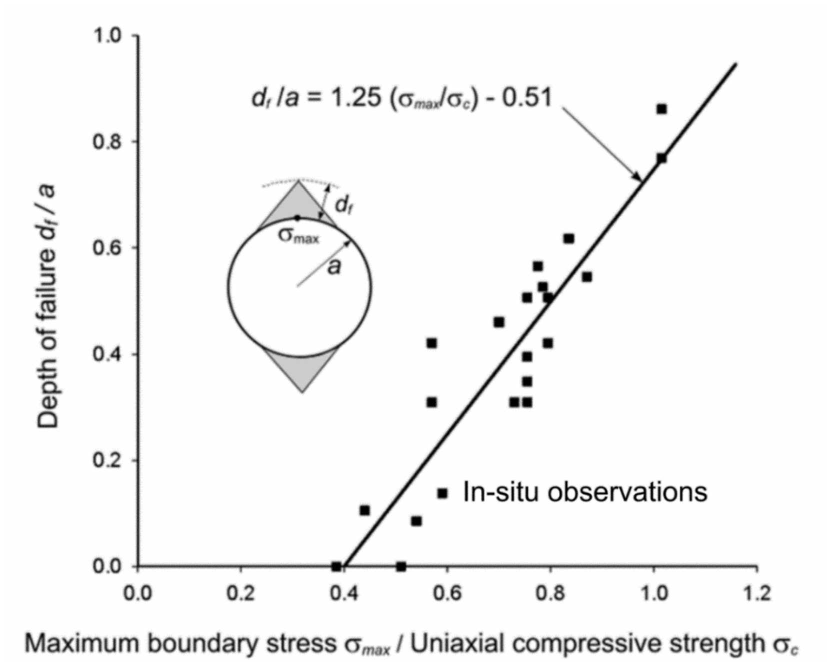



Figure 30: Estimated depth of spalls in a circular tunnel based on field measurement of spalls in competent rock.

When the maximum boundary stress $\sigma_{max}$ exceeds the uniaxial compressive strength $\sigma_c$, in Figure 30, the potential exists for the spalling to occur with a violent release of energy. This phenomenon is known as rockbursting, which can result very dangerous working conditions in the excavation. Figure 31 illustrates the damage due to a severe rockburst in hard rock in a deep level gold mine in South Africa in about 1955.

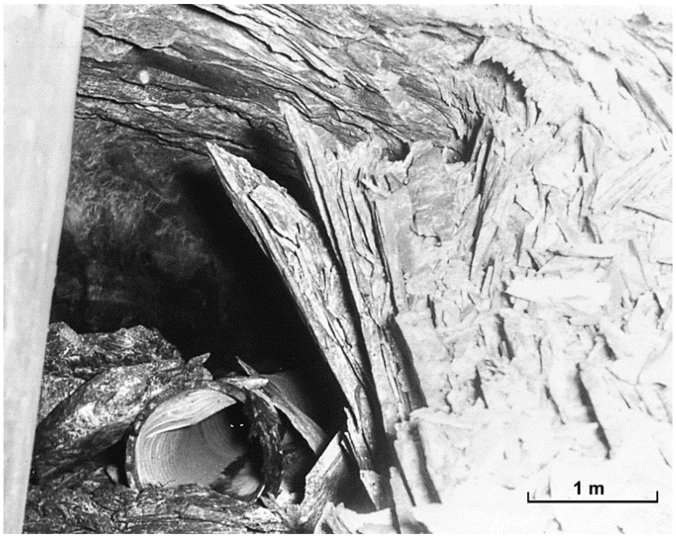



Figure 31: Damage due to a rockburst in a deep level gold mine in South Africa. Photograph from the files of the late Professor Alastair Black of the Royal School of Mines in London.

The problems of spalling and rockbursting in very deep level mines has been familiar to miners for many years. In 1942, Morrison prepared a report on the rockburst situation in the Ontario mines in Canada. Cook et al (1966) published a paper on rock mechanics applied to the study of rockbursts, in deep level gold mines in South Africa, which discussed extensive research into this problem dating back to the 1950s.

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