T. R. Stacey / C. H. Page

Practical Handbook for Underground Rock Mechanics

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Preface

One often hears criticisms of "rock mechanics" from practical mining and civil engineers — that it is theoretical, esoteric, impractical. These criticisms are well founded in that, for the practical engineer, the subject is generally presented in a complicated manner which obscures the immediate usefulness of the subject and discourages further attempts at its use. This handbook is an attempt to redress the balance.

Generalisation and simplification can be overdone and we shall, no doubt, be accused of such. Our aim has been to present "rock mechanics" in a form which is practical and uncomplicated. If this allows practical engineers to apply more design in their activities and so obtain a better appreciation of the complexity of the rock mass, then we believe that our aim will be satisfied.

We hope that this handbook will be used in the spirit in which it is intended.

T.R.S.
C.H.P.

Contents

List of Figures

Chapter 1

Introduction

Rock mechanics is an extremely complex field which has only fairly recently become a science. It still remains a bewildering mixture of art, advanced analysis techniques and "rules-of-thumb". As a result, appropriate evaluation procedures and design methods are not always easy to identify within current textbooks. This is not a reflection on these textbooks, but is rather a result of the complexity of the subject. In addition, rock mechanics is a field which relies heavily on experience. This handbook is not a substitute for experience nor for detailed textbooks. Rather, it sets out to identify those procedures and methods which are appropriate to feasibility evaluations with very limited data. It is intended for use as an *everyday tool* — as a user manual rather than a summary text book. Hence we have termed it a handbook and see its use entirely in that context. We have assumed that the user will have a general rock mechanics awareness, but do not believe that use of the handbook should be limited to rock mechanics personnel. A civil contractor for example, can make use of the handbook to assist in understanding and evaluating an excavation design; a mine manager may use it to prepare pertinent questions and to evaluate technical input to the everyday operation of his mine.

We expect that a major use of the handbook will be for *pre-feasibility evaluations*. It is a very common failing in many projects that exploration, investigation and design activities proceed too far before a practical evaluation is carried out. This handbook can be used prior to the acquisition of any detailed geotechnical data. For example, a potential orebody may have been identified, but the profitability of mining it may depend on the feasibility of a particular extraction method. That feasibility should be assessed at a very early stage, and though not accurate, the assessment will be sufficient to permit a decision to be made on whether or not to proceed with further extremely expensive exploration. In the civil field, the feasibility of a hydroelectric scheme may depend on the practicality of a certain size of underground chamber. That practicality must be determined as early as possible.

In both of these examples it may be necessary to carry out a stability evaluation and provide a preliminary support design, using incomplete data and with the site possibly even inaccessible. This handbook provides a tool with which a meaningful answer can always be obtained by an experienced user. As further information becomes available the evaluation can be updated using the handbook to provide more accurate analyses.

The handbook is a collection of simple formulae, empirical relationships, simple data acquisition techniques, rock property data, estimates, opinions and guesses, extracted from published literature and our own experience. We are well aware that we may be accused of over-simplification, but we have attempted to avoid inappropriate complications and many alternative procedures. We have restricted the approaches and analyses presented to what has, in our experience, worked adequately for the purposes of preliminary evaluation.

The *approach to underground excavation design* follows a relatively straight-forward path :

- the purpose of the excavation will determine the shape and size requirements, for example :
  - — routing of services such as rail and road tunnels, mine extraction haulages
  - — accommodation of equipment or material such as underground power stations, oil storage caverns, nuclear repositories, mine storage bunkers
  - — extraction of an orebody; the openings are determined by the orebody shape and the chosen mining method.

Initially an "ideal" excavation will be considered which best satisfies the purpose. The practicality of this "ideal" opening must then be considered in relation to the properties of the rock mass in which it will be located:

- will the opening be stable?
- what is the mode of any identified instability?
- can the instability be overcome by changing the size, shape, orientation or location of the excavation, and therefore modifying the "ideal" opening?

The above process is often complicated in a mining environment where instability may be essential, such as for caving design, but the approach remains unchanged. To cave, the opening must be larger than a certain critical dimension; to be stable it must be smaller, or be supported.

Appropriate support will depend on the risk associated with an excavation. When the public uses an excavation, or when potential instability may affect the public, then safety is of the utmost importance. Conversely, instability in a mining excavation may represent no danger to personnel and the important criterion will simply be the efficiency or effectiveness of the extraction process. In essence this handbook presents practical guidelines for stability evaluation and subsequent support estimation.

In following the above approach, we have structured the *layout and contents* of the handbook so that the logical sequence of stability evaluation, assessment of support requirements and support design can be followed. The following two chapters contain all the necessary evaluation and design techniques. Each is described in sufficient detail to justify its use, but the theoretical basis is limited to identification of a reference. Chapter 2 on **Stability** ends with a summary in which the techniques and data requirements are identified.

In the Appendix are described the ways in which the required data can be collected, measured, estimated or inferred depending on site accessibility and previous investigatory work. This Appendix is limited to providing data essential to the requirements of Chapters 2 and 3. It does not need to be read, simply accessed as required by reference to its sections in the text of the two Chapters.

We have specifically not dealt with methods of excavation, nor with the choice of mining methods and related underground excavation design, but have provided the "tools" with which this choice or design can be made and evaluated.

Chapter 2

Stability

The evaluation of stability for a proposed underground excavation is the fundamental step in the design of that excavation. Depending on the purpose of the excavation, instability may be a necessity (mining by caving methods), short term stability may be required (temporary mining or civil excavations) and major civil excavations and important mining service excavations must be stable in the long term. Evaluation of the inherent or natural stability and the mechanism and mode of instability are a pre-requisite to the design of support systems. Instability can result from the following:

- failure of rock material or mass around the opening as a result of high stress to strength conditions
- movement and collapse of rock blocks as a result of the geological structure (structural instability)
- a combination of stress induced rock failure and structural instability
- instability of either type induced or aggravated by other factors such as seismic forces.

We have identified a number of methods of evaluating stability to cover the above situations. These are not the only methods available, but from our experience they have been found to provide satisfactory results. The types of instability which we have considered applicable are discussed before we proceed with the methods of analysis:

Rock Failure

This type of failure refers to cases in which the failure takes place substantially through intact rock material. We have differentiated between brittle rock behaviour and yielding rock behaviour. In the former case instability occurs in the form of slabbing and spalling from excavation surfaces and splitting of rock pillars. Typical examples of this are the characteristic failures around excavations in the deep level mines of South Africa[48], "bursting" from tunnel surfaces in Norway[15] and New York[11], and splitting of pillars at Mt Isa Mine[19] and Nasliden Mine[13].

For non-brittle massive rocks such as chalk[29] and mudstone we consider a shear failure mechanism to be more applicable.

Rock Mass Failure

When the geological structure is such that the application of rock material strength parameters is no longer approximately correct, it is necessary to look at the mass rather than the material. The reduction of the material strength to the mass strength is achieved most realistically, we believe, by using rock mass classification results. Further, for a mass we feel that a failure criterion based on shear stress is probably most applicable.

Major Structural Instability

Faults, dykes and major joint directions may be identified from topographical maps, airphoto and Landsat interpretation and preliminary geological work. These planes may combine to form large unstable blocks which can be specifically identified. We suggest that an approximate stereonet approach can be used to assess instability potential. If this potential exists, a more sophisticated approach is then recommended.

A special case of major structural instability is that related to the stability of natural rock "beams", which occur in stratified or bedded rock masses, and constitute the roof of an excavation. In such cases we recommend that stability assessments should be based on beam theory.

Rock Mass Structural Instability

For rock masses with no identifiable major weak planes, rock mass classification techniques have been proved to be very successful in evaluating stability of underground openings. We strongly recommend the regular application of classification methods since the benefits can be enormous. Regular, formalised classification improves understanding and communication between technical and operating personnel, is a mechanism for control of mining operations and civil engineering contracts, and ultimately results in safer conditions, better planning and reduced overall costs.

We do not recommend that only a single method of stability assessment be applied in each case. Rather, several applicable methods should be used and the results compared before arriving at your final assessment using your own engineering judgement based on the results.

We suggest that you use the following sequence in carrying out the stability evaluation:

- test for stress induced failure around the opening. In many cases this will be a formality requiring nothing more than judgement — for example no failure is likely around a 10 m span cavern in unweathered granite at a depth of 100 m.
- test for instability of large blocks. Again this may be little more than a formality. In stratified and bedded rocks test for stability of roof or hangingwall beams.
- classify the rock mass and test for rock mass instability. Classification should be done in almost every situation, even when the mode of instability is stress-induced failure.
- optimise the opening with respect to location, orientation, shape and size. Loop back to the beginning of the stability evaluation procedure if you make any geometrical adjustment.
- assess the influence of external factors, such as seismic forces.

2.1 Rock Failure

The input data required for the assessment of rock failure are the in situ stress (Section A13)*, the strength properties of the rock material (Section Al) or mass (Section A15) and the stress distribution around the excavation (Section A14).

2.1.1 Massive Brittle Rock (Elastic Analysis)

For brittle rocks and masses which can be considered as massive, a failure zone delineated by a limiting extension *strain* has been found to work in practical situations[43]. The extension strain can be calculated from

$$\epsilon = \frac{1}{E} [\sigma_3 - \nu (\sigma_1 + \sigma_2)]$$

where:

ν is Poisson's ratio

E is the rock material modulus of elasticity

ν and E can be determined from the information in Section A6.

σ_1 and σ_3 are obtained from the stress distribution diagrams in Section A14 and σ_2 can be assumed to be the horizontal in situ stress (Section A13) normal to the plane of the stress distribution diagram. If the calculated ϵ is negative (compression positive convention) and its magnitude exceeds the typical values in the following table, slabbing or spalling failure can be assumed to occur.

Table 1: Critical Extension Strains for Massive Brittle Rock

Rock Material	Critical extension strain
Basalt, diabase, dolerite, gabbro	0.000300
Conglomerate reef	0.000160
Granite	0.000250
Quartzite, quartzitic sandstone	0.000200

* Indicates reference to sections in the Appendix.

The depth of the rock failure zone behind the excavation surface can be determined by substituting the corresponding values of σ_1 and σ_3, taken from the stress distribution diagrams at increasing depths, until the extension strain calculated from the above equation is less than the critical value.

2.1.2 Massive Brittle Rock (Empirical Approach)

For approximately square 3 m to 4 m tunnels in brittle rock it has been found[49] that the ratio of the major in situ stress to the uniaxial compressive strength, σ_c of the rock material can be used as a simple criterion for assessing potential failure.

Table 2: Empirical Instability Criteria for Massive Brittle Rock

σ_1/σ_c	Description of Condition
< 0.2	No particular problems
0.2—0.4	Spalling from surface parallel to σ_1. Heavier support required
0.4—0.5	Heavy support required. Major spalling
0.5—0.67	Very dangerous and difficult to keep open. Support heavy and costly.
>0.67	Impractical or extremely difficult to maintain open

2.1.3 Massive Yielding Rock

Where failure of massive rock is non-brittle a more appropriate criterion to apply is that based on a shear failure mechanism.

The following equation can be used[7]:

$$\frac{\sigma_{1s}}{\sigma_c} = 3.5 \left(\frac{\sigma_3}{\sigma_c}\right)^{0.75} + 1$$

σ_{1s} is the triaxial strength (major principal stress at failure)
σ_3 is the confining stress.

For ease of use, this equation is represented in Figure 1.

Use of this relationship should not result in errors of much greater than 10%.

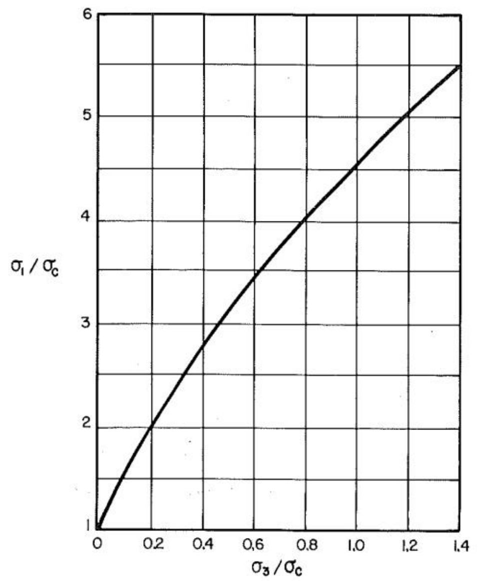


Fig. 1
Shear Failure Criterion for Massive Yielding Rock

The uniaxial compressive strength σ_c is found from Section Al. σ_1 and σ_3 are obtained from the stress distribution diagrams in Section A14 and σ_3 substituted in the above equation to calculate σ_{1s}. If σ_1 exceeds σ_{1s} rock failure can be assumed. The procedure is repeated for values of σ_1 and σ_3 further into the rock to determine the extent of the rock failure zone.

2.1.4 Yielding Rock Mass Failure

In cases in which the rock mass cannot be considered as massive, a rock mass failure criterion equivalent to that in the previous section can be used to evaluate the rock failure. The strength equation[23] is:

$$\sigma_{1s} = \sigma_3 + \sqrt{m \cdot \sigma_c \cdot \sigma_3 + s \cdot \sigma_c^2}$$

m and s are constants for the rock mass, (section A15), which can be determined from rock mass classification data. Values of σ_3 and σ_1 are read off the stress distribution diagrams in Section A14 and with m, s and σ_c (Section Al) substituted in the above equation to calculate σ_{1s}. If σ_1 exceeds σ_{1s} failure will occur. This procedure can be repeated for various points to define the extent of the failure zone.

2.1.5 Rock Mass Failure (Empirical Approach)

The following method is based on experience and is particularly applicable in a mining environment. The design rock mass strength *(DRMS)* can be established using the Mining Rock Mass Classification[27] (Section A16) and then modified to account for the degree of confinement. σ_1 and σ_3 stresses are read off the diagrams in Section A14. If σ_3 is tensile an adjustment of 80% should be applied and if $\sigma_3 > 0.1\ \sigma_1$ the adjustment is 120%. Interpolate between these adjustments for intermediate values of σ_3. If the resulting *DRMS* is less than σ_1 then failure can be assumed.

Repeat the procedure at various points to define the extent of the failure zone.

2.2 Major Structural Instability

Two categories are relevant, one involving rock blocks and the second involving rock beams.

2.2.1 Discrete Blocks and Wedges

Where the occurrence of faults or major joints is known or expected and discrete blocks or wedges may result, specific stability analyses can be carried out. This is more relevant in civil excavations and long-term mining service excavations. To obtain a quick assessment of whether such specific analyses need to be considered, the polar stereonet in Section A8 can be used for an approximate analysis.

Plot the dip angles and dip directions of the three planes as three points on the stereonet as shown in Figure 2. Straight lines joining the three points should be sketched in. If the centre of the stereonet is inside the resulting triangle, the three planes can form a wedge which may fall clear out of the roof.

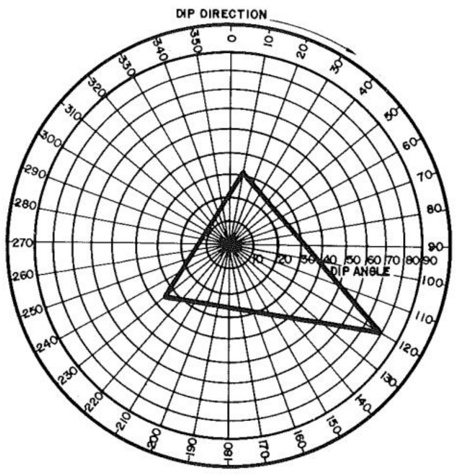


Fig. 2
Polar Stereonet Showing Potentially Unstable Wedge

If the centre of the stereonet is not inside the triangle, then to fail, the wedge will have to slide rather than fall. In this case the shear strength of the joints will have a greater influence on the stability. The shear strength can be represented by the effective angle of friction. In the case of clean joints with strong rock surfaces, the effective angle of friction is likely to be greater than 70°. For softened joint wall surfaces and joints with gouge infill, the effective angle of friction will be lower. The assessment of joint shear strength is dealt with in Section A9.

As shown in Figure 3 the angle of friction can be drawn on the stereonet as a circle concentric with the stereonet and radius corresponding to the friction angle using the dip angle radial axis. This friction circle should correspond to that joint whose pole plots outside the circle, but closest to the circle.

If any part of the triangle is outside the friction circle there is a possibility of sliding failure. The greater the area outside the circle, the greater the likelihood of failure.

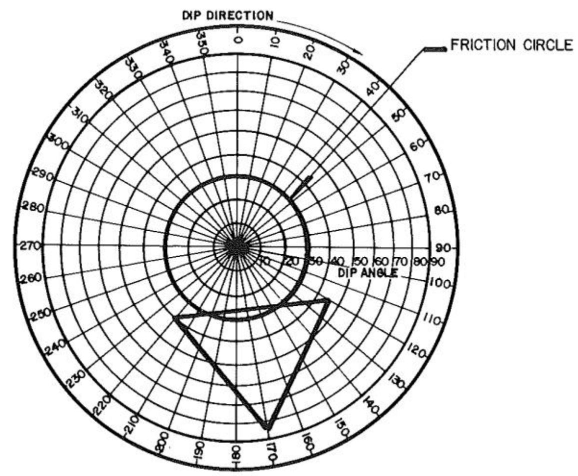


Fig. 3
Polar Stereonet Showing Friction Circle and Potentially Unstable Wedge

The sketches in Figures 2 and 3 illustrate various conditions for explanation purposes. The steps described represent an approximate analysis for quick assessment of stability for feasibility purposes. Should major wedges be identified as potentially unstable, more sophisticated approaches[23] will be required to define the specific support requirements.

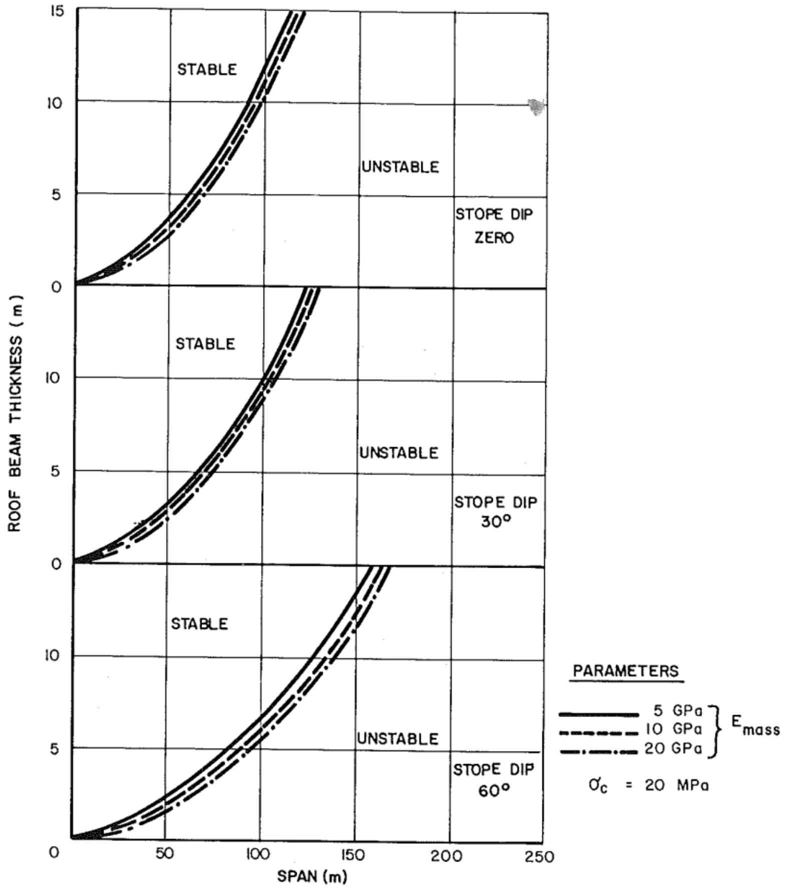


Fig. 4
Roof Beam Stability — σ_c = 20 MPa

2.2.2 Rock Beams

The curves in Figures 4 to 7 provide a rapid means of assessing the stability, or alternatively, the maximum size of unsupported spans for openings in bedded or stratified deposits.

These curves are based on the assumption of plane strain conditions[6] and will give conservative estimates of stable spans for openings with length to width ratios less than 3.

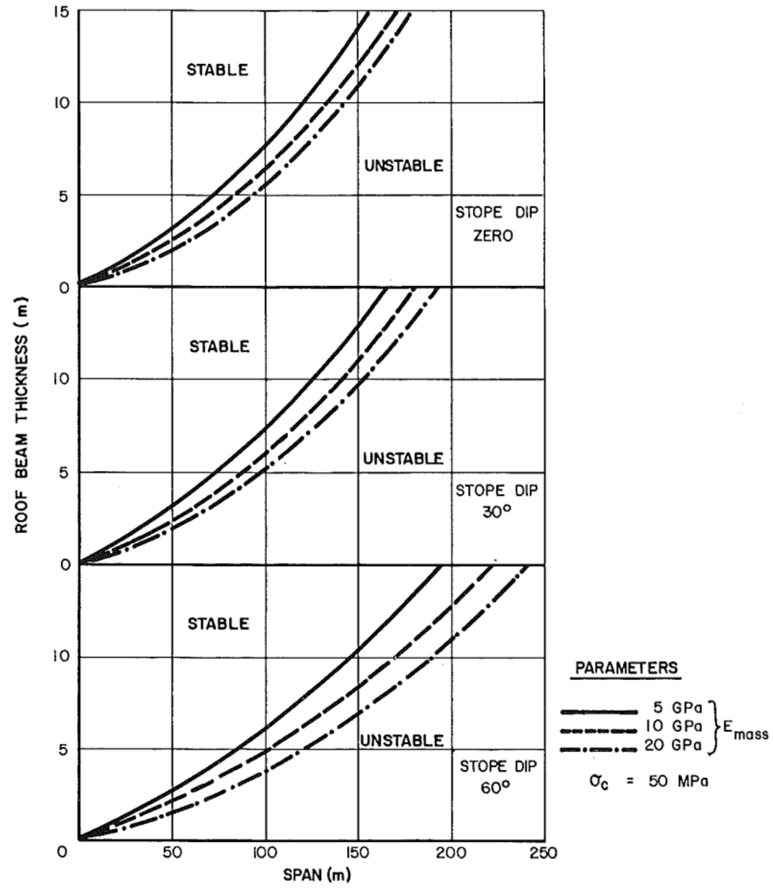


Fig. 5
Roof Beam Stability — σ_c = 50 MPa

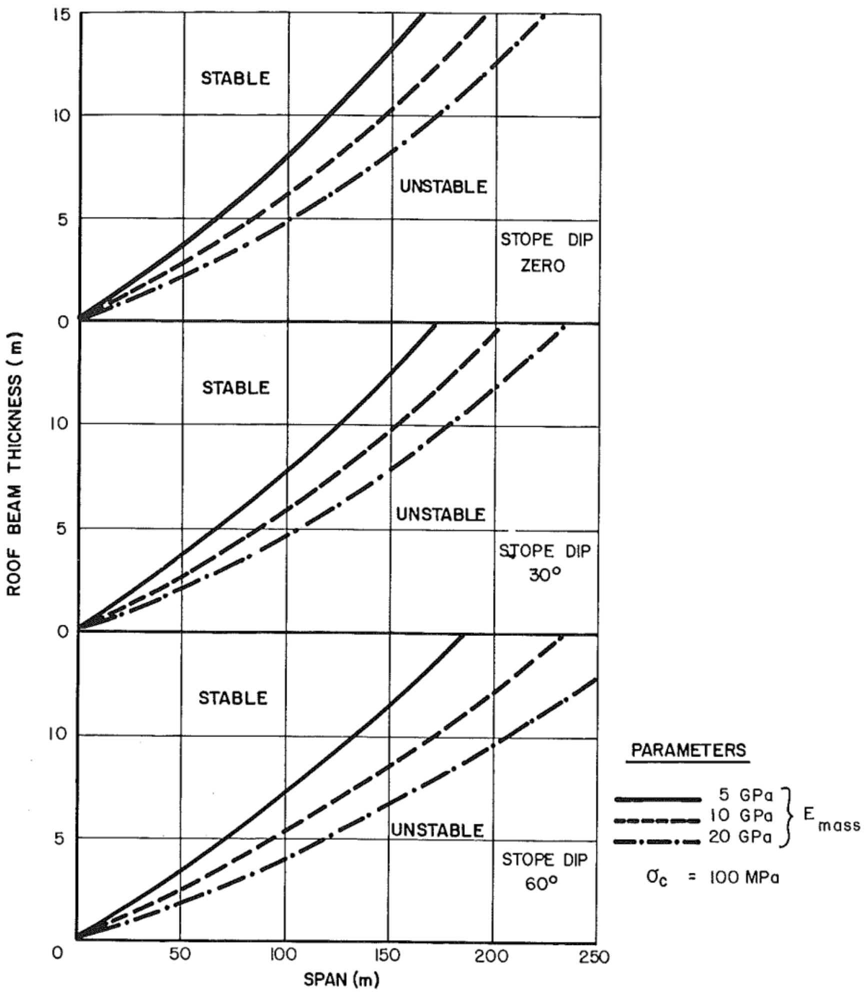


Fig. 6
Roof Beam Stability — σ_c = 100 MPa

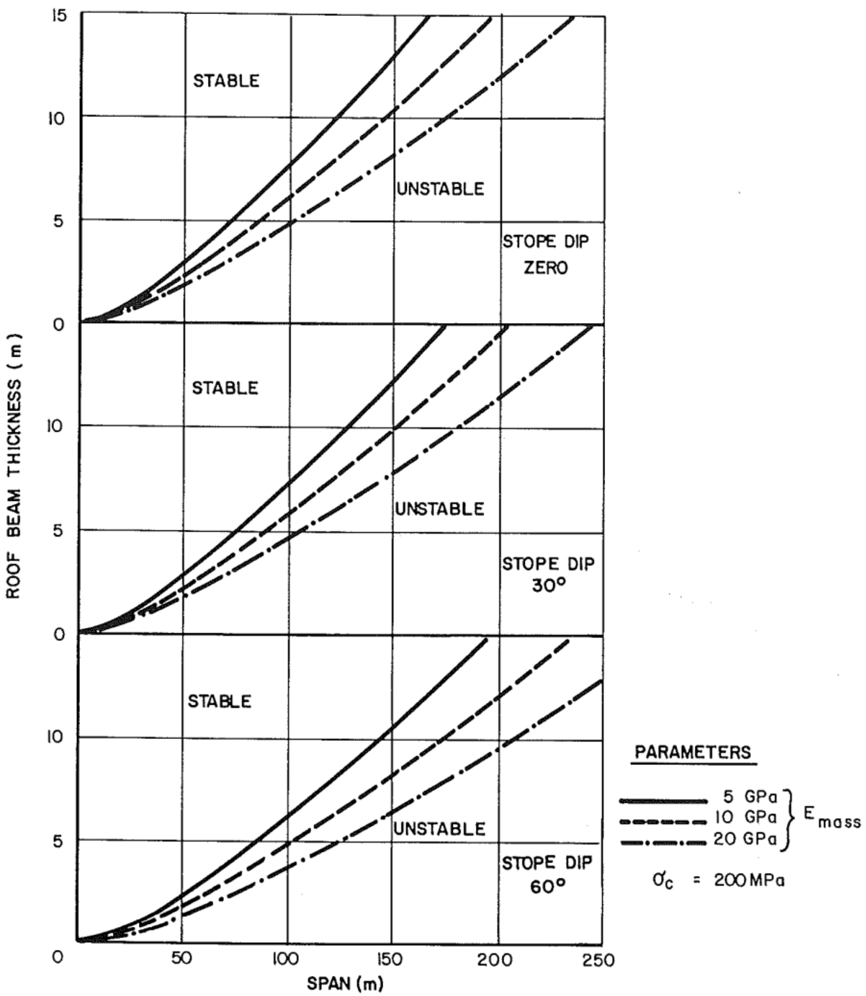


Fig. 7
Roof Beam Stability — σ_c = 200 MPa

2.3 Rock Mass Structural Instability

A rock mass is generally weaker than its constituent rock material as the mass contains structural weakness planes such as joints and faults. The stability of an excavation in a jointed rock mass is influenced by many factors including:

- strength of rock material
- frequency of jointing
- joint strength
- confining stress
- presence of water.

The best practical way in which these weakening/strengthening effects can be taken into account is by applying rock mass classification methods.

Quantitative classification of rock masses has become very popular in the last decade, and justifiably so, since it provides a rapid means of assessing the quality of a mass, comparing qualities, and assessing support requirements. Classification applied on a routine basis can have tremendous value in mines, and has been shown to result in consirable economic benefit to the operation[27].

Two classification methods have stood out, the Q System developed by Barton et al[5] and the Geomechanics Classification System developed by Bieniawski[9]. These two systems are substantially for civil engineering applications. A system specifically for mining applications, based initially on Bieniawski's method, but now independent, has been developed by Laubscher and Taylor[28] and refined by Laubscher[27].

It is our opinion that although application of the systems is straight forward, considerable insight, intuition, experience and engineering judgement are required, and this should be borne in mind. The information contained in the Sections A10, All and A12 will provide you with the basic procedures required for classification.

2.3.1 *Q* System

The Q System classification is based on three aspects:

- rock block size (RQD/J_n)
- joint shear strength (J_r/J_a)
- confining stress (J_w/SRF)

where:

RQD	is the rock quality designation
J_n	is the joint set number
J_r	is the joint roughness number
J_a	is the joint alteration number
J_w	is the joint water reduction factor
SRF	is the stress reduction factor.

The description of these parameters, and the corresponding values to be substituted into the equation for Q below, are given in Section A10. Some modifications[26,30] to Barton's original data have been made to simplify presentation.

The rock mass quality number Q is calculated from:

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$$

The Q system does not take the rock material strength into account explicitly, although it is implicitly included in arriving at the SRF assessment. The orientation of joints is also not taken into account since it is considered that the number of joint sets, and hence the potential freedom of movement for rock blocks is more important.

The range in values of Q is from 0.001 for extremely poor rock to 1000 for excellent rock.

A quantitative assessment of the stability of unsupported excavations can be obtained using the Q System[2] which has been used to differentiate between stable and unstable cases. The graph in Figure 8 represents a suggested quantification[24] for the factor of safety of unsupported excavations. Owing to the variations in the method of excavation and in the rock mass parameters, stability will not be precisely defined. For civil engineering applications, we suggest that factors of safety greater than 1.2 will be required if omission of support is to be considered.

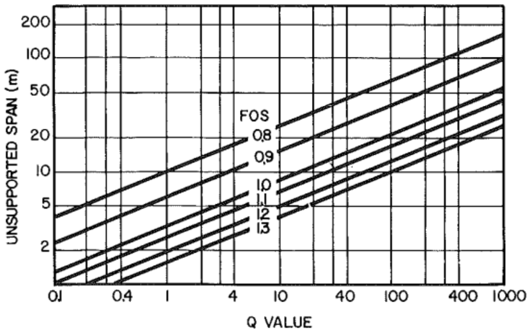


Fig. 8
Relationship between Unsupported Span and *Q* Value
(Redrawn after Houghton and Stacey[24])

2.3.2 Geomechanics Classification

The Geomechanics Classification System derives a rock mass rating (*RMR*), obtained by summing 5 parameter values and adjusting this total by taking into account the joint orientations. The parameters included in the system are:

- rock material strength (σ_c)
- *RQD*
- joint spacing
- joint roughness and separation
- groundwater.

The descriptions and corresponding ratings for these parameters are given in Section A11.

The *RMR* value can range between zero and 100, and with its 5 finger parameter scale, this system is conceptually easier to apply than the *Q* System.

The Geomechanics Classification does not take account of the confining stress present in the rock mass, nor explicitly the number of joint sets. Considerable weight is given to block size since both *RQD* and joint spacing are classification parameters.

A relationship has been found between *RMR* and *Q* as follows[9].

$$RMR = 9 \ln Q + 44$$

The stability of an excavation can be estimated in terms of stand-up time from the *RMR* value[9] using the graph in Figure 9. The accuracy of this stand-up time estimate is doubtful since it is influenced by excavation technique and the in situ stress, effects which the classification system does not take into account. In addition it is extremely important to consider weathering of non-durable rocks such as dunite, kimberlite, and slaking mudrocks (Section A5). In these rocks, conditions may be very stable immediately after excavation, but may deteriorate as a result of exposure, and collapse may ensue. Durability is not accounted for in the classification. Stand-up time considerations can be useful, however, particularly for comparative purposes.

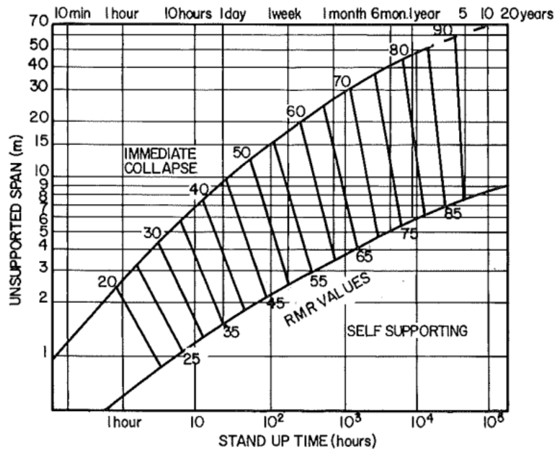


Fig. 9
Relationship between Unsupported Span, Stand-up Time and *RMR* Values
(redrawn after Bieniawski[9])

2.3.3 Mining Rock Mass Classification

This system takes into account the same parameters as the Geomechanics system, but combines the groundwater and joint condition, resulting in the four parameters:

- rock material strength (σ_c)
- *RQD*
- joint spacing
- joint condition and groundwater.

Cumulative adjustment techniques are provided for the joint spacing rating for multi-joint systems and for the joint condition and groundwater rating. Parameter descriptions, corresponding ratings and adjustments are detailed in Section A12.

The mining rock mass rating *MRMR* value is obtained by summing the four parameter ratings. The range of *MRMR* lies between zero and 100.

The mining rock mass classification is better suited to real stability assessment since it is also concerned with cavability. Figure 10 based on published information[28], gives an indication of the cavability (inversely, stability) related to the size of excavation or the plan area of extraction (hydraulic radius is the plan area divided by the perimeter). This figure gives a qualitative estimate of the possible maximum size of unsupported excavations or extraction areas prior to total collapse.

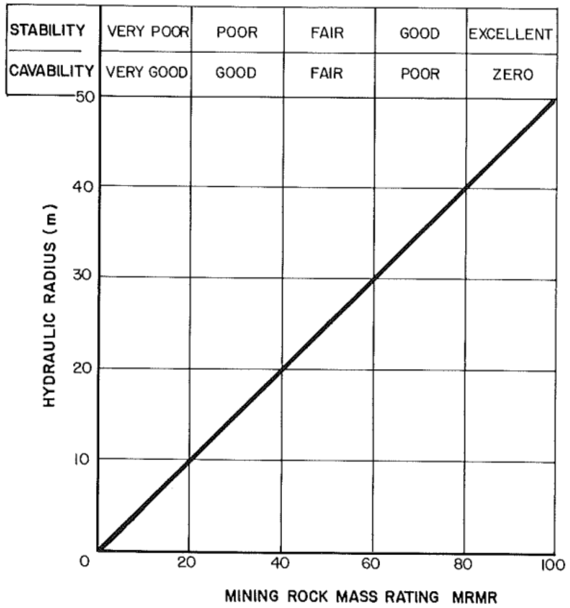


Fig. 10
Relationship between Unsupported Excavation Size, Cavability and *MRMR* Values

2.4 Rockburst Potential

The prediction of instability due to very high stresses can be based on energy. The rate of energy release has become a standard empirical measure of rockburst potential in the deep level mines of South Africa. In simple terms the rate of energy release corresponding to an elemental volume of ore to be mined is half the product of the stress in that volume before it is mined and the closure that takes place across it as a result of mining it out. The value of the rate of energy release is very dependent on the geometry of the mining layout. However, for feasibility purposes the summary data in Figure 11 can be used. These results apply for isolated stopes. A higher rate of energy release will result when stopes interact and the "effective" span becomes greater.

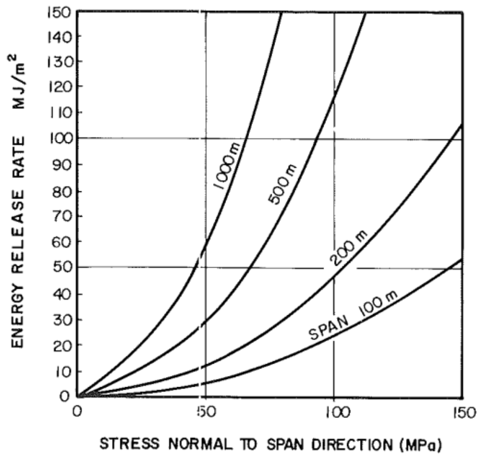


Fig. 11
Rate of Energy Release as a Function of Stress and Span

The graph in Figure 12 summarises the empirical correlation between energy release rate and rockburst occurrence for the South African gold mines. This is particularly related to longwall, narrow stoping height operations.

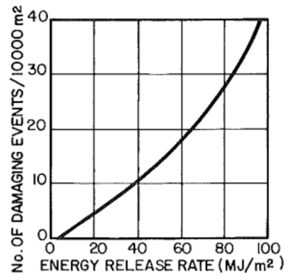


Fig. 12
Correlation between Seismic Activity and Energy Release Rate
(Redrawn after Ortlepp[36])

In these mines rockbursts have been found to become a serious problem when the calculated energy release rates are greater than 40 MJ/m^2. This is a guide value, however, and is only suitable for drawing attention to potentially serious conditions. In other parts of the world, Canada in particular, rockbursts have frequently occurred at much lower values, but data are insufficient to produce empirical relationships.

2.5 Geometrical Optimisation

Rearrangement of the geometry of an underground excavation can often increase/decrease stability. We recommend that geometry changes be considered in the following order. If any geometrical optimisation is made it will be necessary to return to the earlier sections to reassess stability for the new geometry.

2.5.1 Location

It will often be possible to relocate the excavation at the planning stage to take advantage of improved rock mass quality at a slightly different position. Consideration should also be given to cases in which the roof of the excavation can be located in a competent strata layer to minimise stability problems (Figure 13).

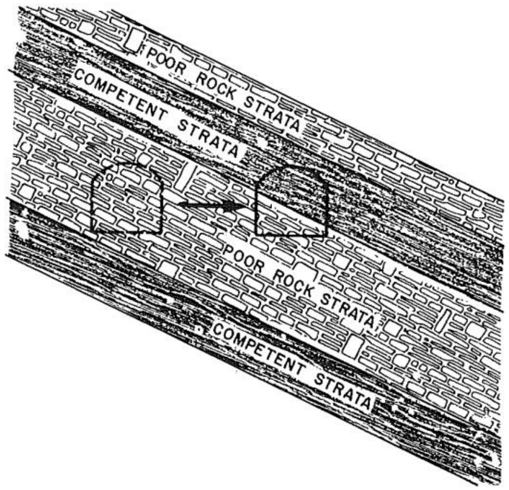


Fig. 13
Relocation of Excavations in Competent Rock

2.5.2 Orientation

The geological structure often has such a significant influence on stability that reorientation of the excavation in plan can result in considerable optimisation (Figure 14). For stability the following should be attempted:

- minimise the number and/or volume of potentially unstable roof wedges
- do not have high excavation walls parallel to major joint directions
- do not plan development directions, bord/room and pillar layouts and extensive stope face or longwall orientations parallel to major joint directions. In inclined strata, development parallel to bedding strike is far less stable than development across bedding.

In addition to the geological structure, the orientation of the in situ stress field can have a significant influence on stability. This is perhaps of particular significance in mining since the clamping action of a high horizontal stress may severely inhibit caving.

It will be necessary to plan the layout of panels and stopes to optimise performance under the conditions pertaining.

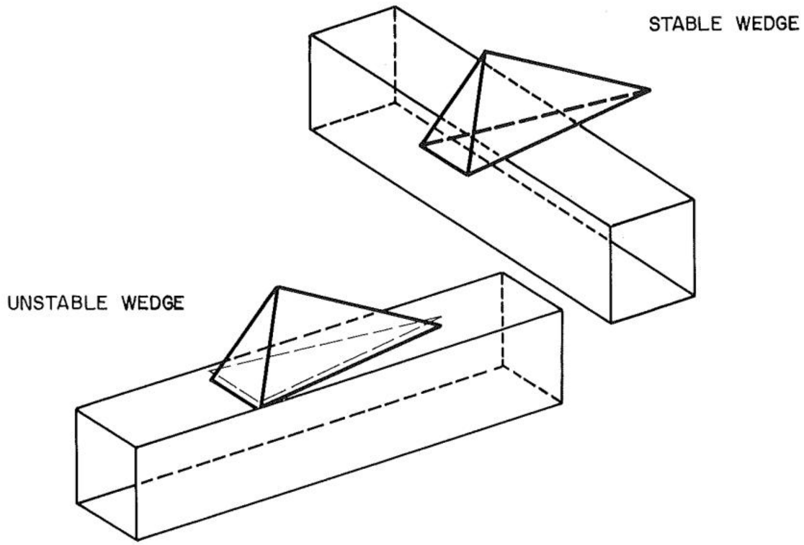


Fig. 14
Optimisation of Stability by Changing Excavation Orientation

2.5.3 Shape

In the past many underground excavations have been constructed with arched roof profiles. It is rare, however, for the rock mass to break naturally to this profile. In such cases part of the installed support action is simply to maintain the profile when a slightly different profile, probably still acceptable, will require less support. We recommend that from both a safety and an economy point of view, as much use as possible is made of the natural stable shape which tends to result in a particular rock mass (Figure 15).

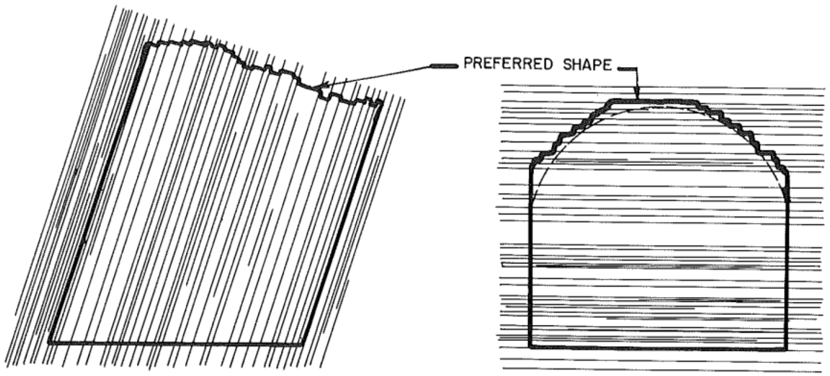


Fig. 15
Excavation Shape Controlled by Geological Structure

Shape can also be optimised with respect to the stress field to minimise rock failure instability. The use of an elliptical shape oriented in the directions of the principal stresses (Figure 16) will minimise the stress magnitudes around the perimeter of the excavation[29]. If the rock is sufficiently strong this will maximise stability against rock failure. If, however, the field stresses are high or the rock relatively weak, and rock failure is impossible to contain, then it is better to change the shape as shown in Figure 17, allow rock failure to occur and then to contain this failed rock.

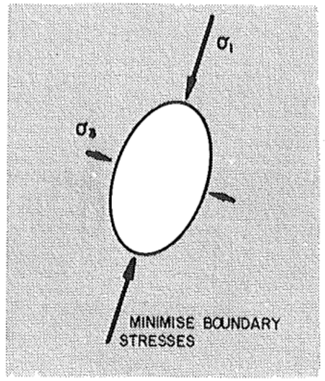


Fig. 16
Minimisation of Stress Failure

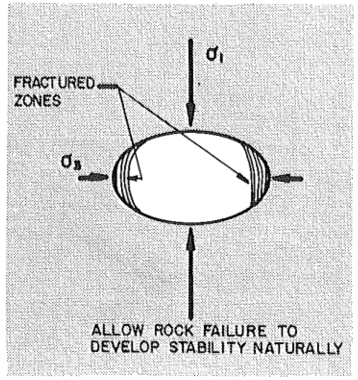


Fig. 17
Promotion of Stability under Severe Stress Failure Conditions

2.5.4 Size

The larger an excavation, the greater the number of structural weakness planes that will be exposed in the surfaces of the excavation and hence the less stable the excavation. This conclusion comes out very clearly from the stability assessments based on rock mass classification dealt with in Section 2.3. If you have optimised the location, orientation and shape of the excavation, then the only option left to improve the natural stability is to reduce the size or free span of the excavation if this is possible. In some cases it may be possible to decrease the span and increase the height, providing the same excavation volume, for example, for oil storage. To induce caving it will be necessary to increase the size of the undercut to reduce the natural stability.

2.6 Stability Evaluation Summary

The following is a summary of the various alternatives and steps we recommend for quick evaluation of excavation stability. It is a repetition of material contained in the previous sections. However, once you are familiar with the detail there, it will only be necessary for you to make use of this summary. Reference to the data required and their source in the Appendix, is given for each evaluation.

Massive Brittle Rock Failure

$$\epsilon = \frac{1}{E}[\sigma_3 - \nu(\sigma_1 + \sigma_2)]$$

Data required:

Section A14: Principal stresses around excavation σ_1, σ_2, σ_3 — Compression positive.

Section A6: Rock material modulus of elasticity E

Section A6: Rock material Poisson's ratio ν

If calculated ϵ is negative and its magnitude exceeds following guide values, rock failure can be assumed.

Rock Material	Critical Extension Strain
Basalt, diabase, dolerite, gabbro	0.000300
Conglomerate reef	0.000160
Granite	0.000250
Quartzite, quartzitic sandstone	0.000200

Massive Brittle Rock Failure (Empirical Approach)

Data required:

Section A13: Major in situ principal stress σ_1

Section A1: σ_c

σ_1/σ_c	Description of Condition
< 0.2	No particular problems
0.2—0.4	Spalling from surface parallel to σ_1. Heavier support required
0.4—0.5	Heavy support required. Major spalling
0.5—0.67	Very dangerous and difficult to keep open. Support heavy and costly.
>0.67	Impractical or extremely difficult to maintain open

Massive Yielding Rock

$$\sigma_{1s} = \sigma_c \left[3.5\left(\frac{\sigma_3}{\sigma_c}\right)^{0.75} + 1\right]$$

Data required:

Section A14: Principal stresses σ_1 and σ_3 around excavation

Section A1: σ_c

If σ_1 exceeds calculated σ_{1s} rock failure can be assumed.

Yielding Rock Mass Failure

$$\sigma_{1s} = \sigma_3 + \sqrt{m \cdot \sigma_c \cdot \sigma_3 + s \cdot \sigma_c^2}$$

Data required:

Section A14: Principal stresses σ_1 and σ_3 around excavation
Section A 1: σ_c
Section A15: Rock mass constants m and s.

If σ_1 exceeds calculated σ_{1s} rock failure can be assumed.

Rock Mass Failure (Empirical Approach)

Data required:

Section A 1: σ_c
Section A12: Mining Rock Mass Rating *MRMR*
Section A16: Design Rock Mass Strength *DRMS*
Section A14: Major principal stress σ_1 around excavation

If σ_1 exceeds *DRMS* failure can be assumed.

Major Structural Instability

Data required:

Section A8: Fault, joint etc. dip and dip directions
Section A9: Joint shear strength

For three-plane wedges, plot dip angles and dip directions on stereonet in Figure A5 and join points with straight lines. If centre of stereonet is inside triangle, wedge can fall out, if outside then sliding on joints must occur.

Draw friction circle. If any part of the triangle is outside the circle, failure is possible. The greater the area outside, the greater the likelihood of failure.

Rock Mass Structural Instability

This evaluation of stability is based on rock mass classification.

Q System
Section A10: All data requirements.

Geomechanics Classification
Section A11: All data requirements.

Mining Rock Mass Classification
Section A12: All data requirements

Hydraulic radius is plan area of extraction or excavation divided by its perimeter.

Excavation Geometry

Optimise location, orientation, shape, size.

Data required:

Section A 8: Geological structure
Section A13: In situ stress

Repeat stability evaluation process if you have made a change to the geometry.

Chapter 3

Support

Once the excavation has been adjusted for location, shape, orientation and size and parts or all of the excavation are still found to be unstable, then the next stage is to prevent or contain that instability. This will depend on the following questions:

- is the instability structural and gravity controlled or does it constitute failure of intact or rock mass material?
- is the excavation for civil or mining purposes?
- what are the associated risks and therefore the appropriate level of support?
- what is the practicality of the identified support and is it economic?

One can draw the broad conclusion that structurally controlled instability can be prevented by correctly installed support but that stress controlled instability (rock failure) can only be contained and controlled but *not* prevented. This statement has some bearing on the different requirements within the civil and mining fields:

Civil Engineering

— structures tend to be permanent and characterised by high economic risk if instability is not prevented

— support cost is generally a small cost element and practicality is usually the main constraint

— it is probable that, at the stage of a pre-feasibility assessment, a design which resulted in stresses which overloaded the rock mass would be found unacceptable.

Mining Engineering

— mining is normally characterised by two sets of excavations; service excavations, such as haulages, whose cost is small compared with the overall mining cost and where a high level of support may be acceptable; and stoping excavations in which the primary extraction operation takes place and where instability, or prevention of instability by support, may have significant economic consequences.

— considerable stress changes result from mining, and service excavations are often situated in severe stress fields. The main consideration is whether it is practical to maintain access, and not necessarily whether it is economic to do so.

Support methods fall into a number of broad categories:

- rapid or slow acting (sometimes termed active or passive)
- external support (outside the rock surface and not in intimate contact)
- rock reinforcement (placed within the rock and with intimate contact on the excavation surface).
- solid support, where rock in the form of pillars is left in situ so that it can provide support.

External support is generally 'slow acting' since the rock mass has to deform significantly to load the support. Supporting forces can only develop when this deformation has taken place. Conversely, rock reinforcement is rapid acting since only the slightest deformation causes a large reaction in the support system.

When considering support methods, it is also important to differentiate between structural instability and rock mass failure due to overstress:

Structural Instability

— the support system is designed to support blocks of ground, bounded by joints or bedding planes, which may fall or slide under the action of gravity. In this case support should be rapid acting and be installed as soon after excavation as possible. If not, then the rock mass may progressively loosen and the amount of rock which has to be supported will increase.

Failure Due to Overstress

— the support is designed for the case where the stresses around the excavation are greater than the rock mass strength. Normally it is impractical to prevent failure and the function of the support is to contain the failed material. Effective containment enables a stress to be built up through the failed and partially failed material. This stress will increase further from the excavation surface providing increasing confinement. Once this confinement reaches a certain level further failure is prevented. If the failure zone is extensive, then the support will have to be extremely flexible if it is to accept gross deformation and retain its support potential.

Within the broad categories of support types identified above there are many variations, some of which are specific to mining excavations. The object of this handbook is not necessarily to provide an accurate support design, but rather to identify an *appropriate support type* and the *level of support intensity*. This will enable the determination of approximate costs, in terms of the overall excavation costs, and practicability of support installation. Therefore we have severely restricted the variety of support measures considered. These are identified for specific purposes:

Excavation Type	Mode of Instability
1. Civil excavations and permanent mining service excavations	Instability mainly structurally controlled
2. Mining service excavations and extraction development within a high stress, or stress change, zone	Instability mainly stress controlled
3. Mining extraction openings (stopes)	Instability both structurally and stress controlled

For these three situations we have identified the following limited number of applicable support types and corresponding methods of assessing the required levels of support:

Support Type		Method of Assessment of Support Level
Grouted bolts Grouted bolts + shotcrete Grouted bolts + shotcrete + mesh	Civil and Mining Service Excavations	Empirical estimation method using *Q* system[5]. This method was developed for civil excavations and has seen wide application in this field.
Grouted bolts Grouted bolts + mesh Grouted bolts + mesh + shotcrete Grouted bolts + mesh + 'lacing' Yielding steel arches set in shotcrete Any of the above + cables	Temporary Mining Excavations	Empirical estimation method[27] using *DRMS* (design rock mass strength) and the stress environment. This method was developed in a mining environment under severe stress and operating conditions.
Rock pillars	Mining Extraction Openings	Pillar design relationship
Timber props (sticks) and packs		Standard procedures
Backfill		Empirical design methods and material identification

The above only includes reference to timber support in mining extraction openings. In mining there is still a tendency to use timber support in service excavations since either the materials or labour, or both, are relatively inexpensive. However, timber is not an effective means of support since it requires considerable rock mass deformation before it provides significant restraint. By the time this restraint has been provided the zone of rock mass requiring support will have increased considerably. Timber support is adequate for superficial surface instability only, and unfortunately is often installed for cosmetic purposes. However, timber has some specialised applications, such as in longwall hard rock mining and some coal mining situations.

There is a general trend away from passive support to rapid acting support, and this is reflected in our evaluation methods. The *importance of engineering judgement and experience in using the following systems cannot be over-emphasised.*

3.1 Support Estimation

3.1.1 Civil and Permanent Mine Excavations

The Q system of rock mass classification[5] is described in Section A10. The Q value has been compared with the actual support installed in over 200 civil excavations and this comparison has formed the basis for a method of support estimation.

The type and level of support is determined from the Q value of the rock mass and the *modified* dimension of the excavation:

$$\text{Modified Span or Wall Height} = \frac{\text{Actual Span or Wall Height}}{\text{Modification Factor } (MF)}$$

Appropriate modification factors[5] are given in Table 3.

Table 3: Span Modification Factors for Different Types of Excavations

Type of Excavation	Modification Factor MF
Temporary mine openings	3 to 5
Permanent mine openings, low pressure water tunnels, pilot tunnels, drifts and headings for large excavations	1.6
Storage chambers, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	1.3
Power houses, major road and railway tunnels, civil defence chambers, portals, intersections	1.0
Underground nuclear power stations, sports and public facilities, factories	0.8

The above values for the modification factors are only guide-lines. They provide a means for qualifying the cost of reducing risk by increasing support. Lower modification factors might also be

appropriate where there is considerable doubt regarding the reliability of the data used to derive the Q value.

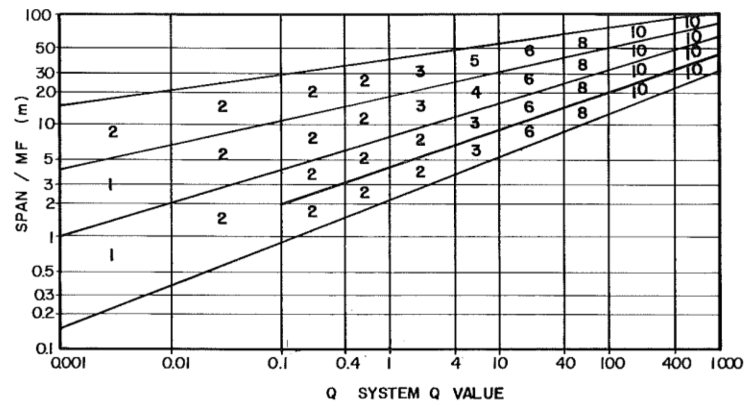
In order to estimate support requirements for the walls of a large excavation the wall dimension must be converted to an equivalent roof (span) dimension. We are mainly considering structural instability where the disturbing force is gravity and therefore a wall will usually be far more stable than a roof. This "improvement" is achieved with the following modifications to the value of Q[5]:

For $Q > 10$ take $Q_{wall} = 5\,Q$

$0.1 < Q < 10$ take $Q_{wall} = 2.5\,Q$

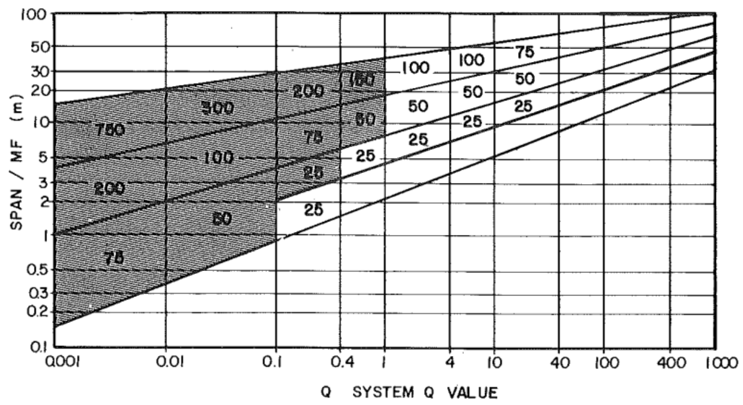
$Q < 0.1$ take $Q_{wall} = Q$

We can now proceed to the support design charts in Figures 18 and 19. These are simplifications of the original data[5] and give bolt spacing, shotcrete thickness and mesh requirements.



BOLT SPACING — m² OF EXCAVATION SURFACE PER BOLT
(where the area per bolt is greater than 6 m², spot bolting is implied)

Fig. 18
Bolt Support Estimation Using the Q System



SHOTCRETE THICKNESS IN mm
(mesh reinforcement in shaded areas)

Fig. 19
Shotcrete and Wire Mesh Support Estimation Using the Q System

The support intensity given in the design charts is appropriate primary support for civil excavations and permanent support for mining excavations. For long term civil excavations, the design chart output should be modified as follows:

To give permanent civil support

— divide area per bolt by 2
— multiply shotcrete thickness by 2.

The support estimates given on the design charts are conservative by mining standards and reflect the increased economic consequences should instability result. Note that the very thick applications of shotcrete indicated will not be practical, but the numbers are included for completeness. For these conditions cast concrete arches will be more appropriate.

The length of rockbolts or cables can be calculated from the following simple formulae using the modification factor *MF* from Table 3.

Roof — Bolts : $L = 2 + 0.15\ B/MF$
— Cables : $L = 0.4\ B/MF$ $(B = \text{span})$

Walls — Bolts : $L = 2 + 0.15\ H/MF$
— Cables : $L = 0.35\ H/MF$ $(H = \text{wall height})$

L in metres

3.1.2 Mining Service and Extraction Development — Severe Stress and Operating Conditions

The extensive extraction of orebodies will, unavoidably, result in extreme changes to the stresses around these excavations. Mining is often characterised by considerable stress failure and collapse and these zones of distress can be so extensive around the ore extraction zone that they include the service and extraction development. The classification system used to assess rock mass failure[28], in Section A12, was specifically developed to assess conditions in a mining environment. This classification system has recently been extended[27] to quantify support requirements in a severe stress environment. It is largely based on personal, as opposed to documented experience but that experience is, none the less, very wide.

The design charts shown in Figures 20 and 21 tend to be more applicable to the direct extraction development, or drawpoints, where not only stress changes but also considerable secondary blasting takes place. Gross deformation of mining development is often perfectly acceptable as long as access is maintained. In this situation there is no need to use the intensity of support suggested in the design charts in Figures 20 and 21 and a 50% reduction is appropriate. In general this reduction can be made by not including the use of shotcrete and cables where these are recommended in the charts. We have taken considerable liberty in interpreting and simplifying the original charts and support methods[27], and feel justified in doing so, since the aim is to assess support type and intensity for feasibility purposes, rather than to reach a definitive design.

The design charts are based on the following criteria:

- maximum induced stress
  - resulting in stress failure where the *DRMS* (design rock mass strength) is less than the mining environment stress
- minimum induced stress
  - leads to opening of joints, consequent reduction of confinement and joint shear strength, and ultimately falling out of blocks

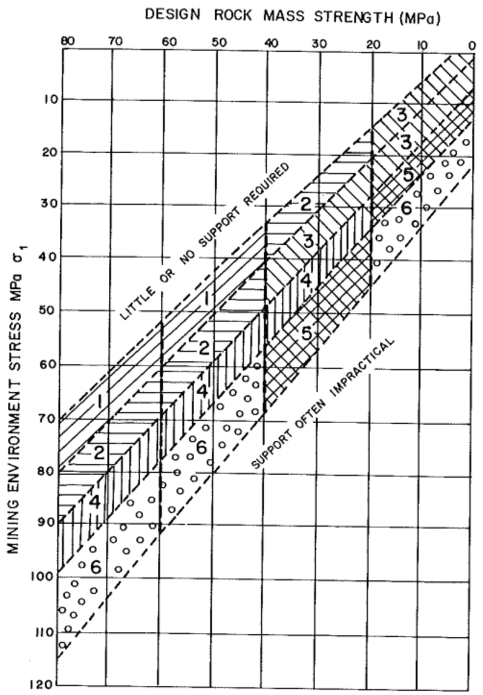


Fig. 20
Support for Mining Excavations Considering Maximum Stress and Failure
(Modified after Laubscher[27])

The stresses σ_1 and σ_3 can be identified from the stress distribution diagrams (Section A14). Care must be taken to define the total induced stress:

(i) evaluate the in situ stresses (Section A13)

(ii) determine the induced stresses around the stoping excavation (Section A14)

(iii) place extraction development in these induced stresses which now represent a pseudo in situ stress field.

(iv) determine the induced stresses around the development using this pseudo in situ stress field (Section A14 again).

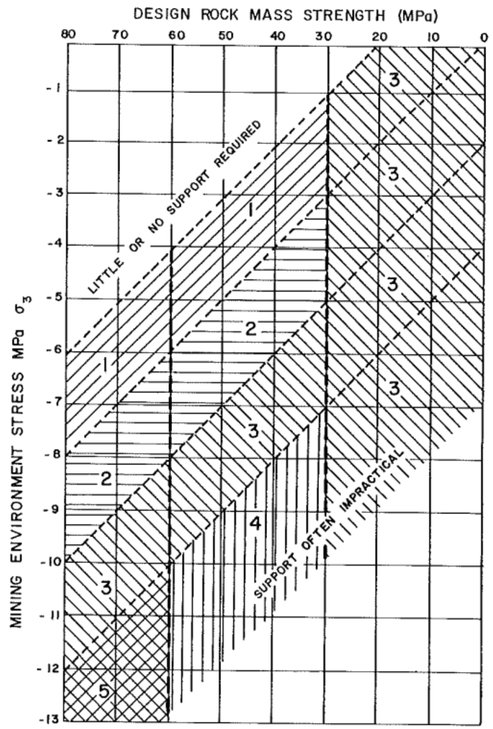


Fig. 21
Support for Mining Excavations Considering Tension and Loosening
(Modified after Laubscher[27])

Once the stress environment and the *DRMS* have been calculated then these values can be entered in the charts and the appropriate support identified. The support types used in the charts in Figures 20 and 21 are:

Type	Description
1	Grouted bolts with a nominal 1 m spacing and 1.8 m length
2	Type 1 + mesh (or straps)
3	Type 2 + shotcrete at least 25 mm thick
4	Type 2 + rope lacing between the bolts
4b	Yielding steel arches set in shotcrete (as an alternative in drawpoints where moving rock and secondary blasting would damage the lacing type support)
5	Type 3 or Type 4 + cables to provide lateral restraint (cable length = 2.5 r + 1 m where r is the radius of the excavation). Shotcrete to be at least 50 mm thick
6	Type 5 + floor support or bracing between the legs of the arch support

3.1.3 Dynamic Loading

Dynamic events such as earthquakes, rockbursts or heavy blasts impose additional forces on support elements. The moving force in this situation is the generated ground velocity v. This imparts a kinetic energy to the 'loose' mass m of rock or rock blocks being supported in the failure zone. This energy is balanced approximately by the support force F in the support elements and the extension d of the support elements. An *approximate* energy balance equation is:

$$\frac{1}{2} mv^2 = Fd$$

From this it follows that the greater the extension possible in the support, the less the support force need be, i.e. for dynamic loading, support with considerable extension capability is required. Appropriate support types include grouted end-anchored smooth shafted bolts, grouted cables, wire mesh, shotcrete and rope lacing. Rigid support with little extension capability, such as fully grouted rebar should not be used.

3.2 Pillars

Stability evaluation, as described in Chapter 2, may have shown that a particular span is likely to be unstable. A solution to this problem is to divide the excavation into two or more smaller excavations. This implies that pillars could be formed between the excavations. In a mining environment these pillars are usually in ore and, to minimise loss of revenue as a result of "frozen" ore reserves, every effort must be made to minimise the size of the pillars if they are to be permanent. If they are to be extracted at a later date then care must be taken to ensure that they are in a condition conducive to efficient extraction at this later stage. They should be large enough not to be highly stressed or they should have been designed to fail partially, consequently relaxing the stresses, whilst maintaining their integrity.

Pillars can therefore be designed to yield as long as this does not lead to an unstable condition and the possibility of uncontrolled pillar collapse. This depends, to a certain extent, on the ratio of pillared area to depth below surface.

A further consideration is the consequence of a pillar failure and this will depend on:

- the use of the excavation
  - — will men be working in the excavations supported by pillars?
- the purpose of the pillar
  - — is it a barrier pillar to act as an abutment around a mining area, or a regional support pillar, water barrier etc.?
- the life of the pillar
  - — is it permanent or temporary (to be extracted later or perhaps the excavation is to be backfilled)?

Finally, it is conventionally accepted that once a pillar has a width to height ratio (W:H) of 10:1 it can be considered to be indestructible[47]. However, the load it will, or can, accept will depend on the strength of the floor and roof material. If foundation failure occurs then the pillar height effectively increases, with the consequence that the pillar geometry becomes less favourable and the pillar strength reduces.

The above aspects are considered under the following headings:

- pillar strength
- foundation strength
- pillar stress
- yielding pillars
- factor of safety.

3.2.1 Pillar Strength

Pillar strength will depend on:

- the rock mass strength of the pillar material
- the shape and size of the pillar defined by its width and height
- gross structural features such as clay bands, unfavourably orientated faults and joints.

There are numerous empirical pillar design methods although most of them relate only to coal. However, there are only two methods which have been tested against sufficient field data to be statistically reliable. The two relationships[41,22] are given below:

- Salamon: Pillar strength $(P_s) = 7.2 \dfrac{W^{0.46}}{H^{0.66}}$

  derived for square coal pillars in South Africa

- Hedley: Pillar Strength $(P_s) = 133 \dfrac{W^{0.50}}{H^{0.75}}$

  derived for extremely competent strong rock rib pillars in the Elliot Lake area of Canada.

  In the above formulae

  W = pillar width in metres
  H = pillar height in metres
  P_s = pillar strength in MPa

What is remarkable about these two relationships is the similarity of the shape factor (W^a/H^b) even though they are for entirely different materials. In addition the recommendations regarding appropriate factors of safety for room and pillar mining are very similar, 1.6 from Salamon and 1.5 from Hedley. The values of 7.2 MPa and 133 MPa are well below the intact strength of the two rock types and can be related to the design rock mass strength, *DRMS*, described in Section A16, which takes into account the rock mass quality, unfavourable joint orientations and the excavation method.

Pillars can have many different shapes and the relationships above are specifically for square and long rib pillars respectively. A suggested method for estimating an *effective* pillar width based on the 'hydraulic radius' is:

$$W_{\text{eff}} = 4 \frac{A_p}{R},$$

where A_p is the plan area of the pillar and R is the perimeter.

This is logical since W_{eff} then approaches 2 x W for very long pillars where the majority of the material in the pillar is affected by only 2 pillar walls, whereas in a square pillar it is affected by 4 pillar walls.

A further adjustment must be considered for very large pillars as the application of the two pillar design relationships above has been limited to a relatively small range of W:H ratios. These relationships also indicate a decreasing rate of strength increase for greater W:H ratios, and do not take into account the strengthening effect of increased confinement with increased W:H ratio. This limitation has been recognised[47], and from model pillar tests with over two hundred samples of sandstone using W:H ratios varying from 1:1 to 8:1, the following relationship has been suggested for W:H ratios greater than that at which the increase in strength starts to accelerate:

$$P_s = k \frac{R_o^c}{V^d} \left\{ \frac{c}{e} \left[\left(\frac{R}{R_o} \right)^e - 1 \right] + 1 \right\}$$

where:

R_o = W_{eff}: H ratio at which the increase in strength starts to accelerate
R = W_{eff}: H ratio greater than R_o

e = rate of strength increase beyond R_o
V = volume of the pillar
c and d are the constants derived from converting the relationship

$$K\frac{W^{a}_{eff}}{H^{b}} \quad \text{to} \quad K\frac{R^{c}}{V^{d}}$$

where:

$$R = \frac{W_{eff}}{H} \text{ and } V = \text{pillar volume}$$

As the pillar volume is a measure of the confinement it is appropriate to use $V_{eff} = W^2{}_{eff} \times H$

In the model studies it was found that the transition $W{:}H$ ratio (R_o) was approximately 4.0 to 4.5 and e was 4.5. In the absence of field data we believe that there is sufficient justification for using the values derived from the model tests for a feasibility assessment. A more conservative view is suggested by Wagner and Madden[47] with e as 2.5 and R_o equal to 5. Our recommended simplified design approach is:

For $W{:}H$ ratio less than 4.5:

$$\text{Pillar Strength } P_s = k\,\frac{W_{eff}{}^{0.5}}{H^{0.7}}$$

For $W{:}H$ ratio greater than 4.5:

$$\text{Pillar Strength } P_s = k\,\frac{2.5}{V^{0.07}}\left\{0.13\left[\left(\frac{R}{4.5}\right)^{4.5} - 1\right] + 1\right\}$$

where:

$$R = \frac{W_{eff}}{H}$$

$$V = W^2{}_{eff} \cdot H$$

k = $DRMS$ (Section A16) in MPa

$$W_{eff} = 4 \times \frac{\text{pillar area}}{\text{pillar perimeter}}$$

and where all dimensions are in metres.

The above relationships are represented graphically in Figure 22.

It is important to note the use of W and W_{eff} in the above relationships. The use of W_{eff} where $W{:}H$ is greater than 4.5 implies that the strength of a rib pillar increases at a faster rate than with a square pillar. This is unlikely to be the case since, as the pillars get larger, the difference in the strength of the pillar material contained in a square pillar and a rib pillar, will decrease. It must be realised that there is, as yet, no empirical or scientific justification for assuming the validity of a single design procedure for pillars of all shapes, sizes and material

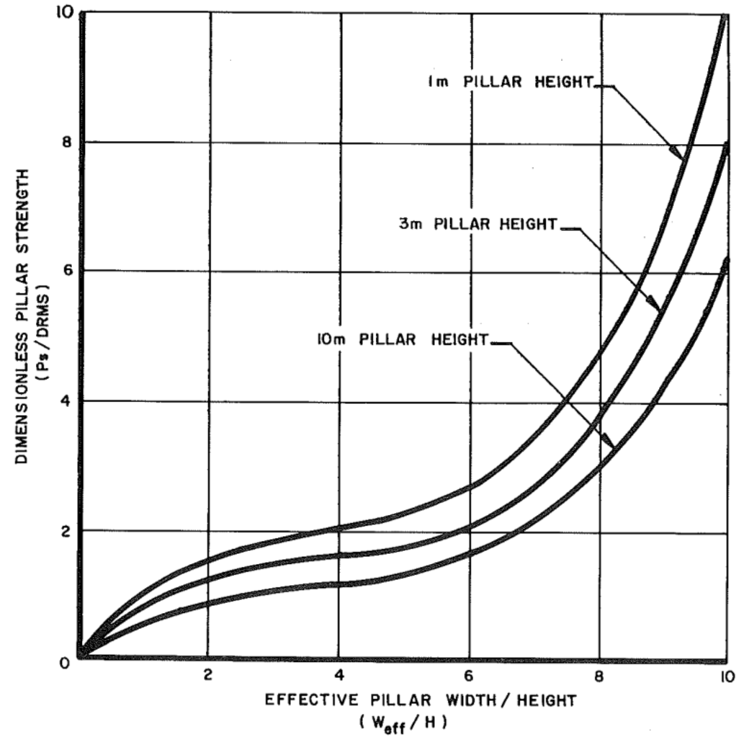


Fig. 22
Change in Pillar Strength with Change in Width to Height Ratios

types. However, for the purposes of a feasibility study we believe there is sufficient indirect justification as long as the user realises the limitation in terms of accuracy.

The above relationships assume an average rock mass for a pillar and do not consider major structural features such as a clay band through a pillar or very weak material at the contact between roof and floor. There is very little published data on the effect of large structural weaknesses on pillar strength and we have therefore resorted to theoretical analyses and experience[37] to derive the strength reductions shown in Figure 23.

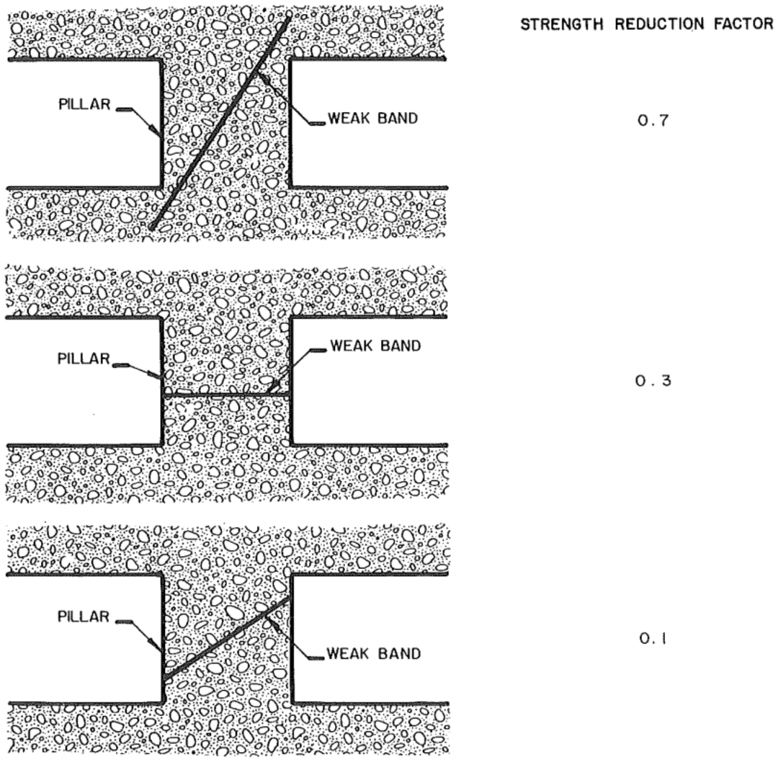


Fig. 23
Pillar Strength Reduction Caused by Gross Structural Weakness
(Redrawn after Page et al.[37])

The ultimate strength of very large pillars may be irrelevant in certain situations:

— if stress failure at the walls of a pillar is such that its support is impractical and travelling ways cannot be maintained.

— if, in the case of boundary or regional pillars, foundation failure of the roof or floor material results.

3.2.2 Foundation Strength

Once the strength of the pillars has been determined it is necessary to consider the strength of the roof and floor as these form the pillar foundations (Figure 24).

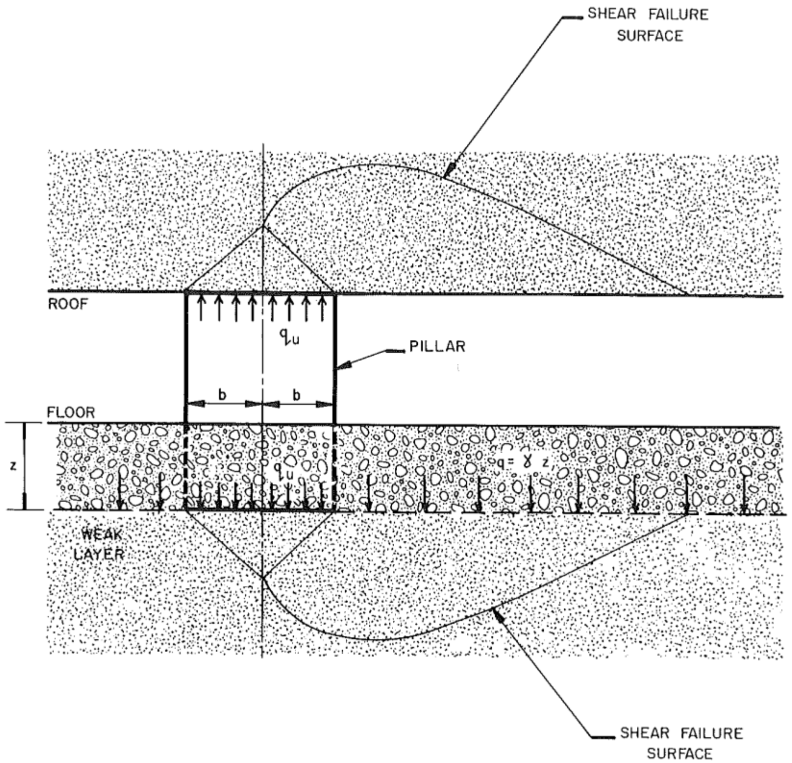


Fig. 24
Foundation Strength for Pillar Stability

Classical foundation design is commonly the province of soil mechanics since if rock is involved there is usually very little problem with foundation strength. However, with strong pillars this is not necessarily the case and in coal mining 'floor heave', which is a result of foundation failure, can be a very serious problem.

Terzaghi's method is the most widely used for determining bearing capacity. Referring to Figure 24, the foundation strength q_u is given by:

$$q_u = c\,N_c + q\,N_q + \gamma b N_\gamma$$

where:

c = cohesion = 0.16 x *DRMS* (design rock mass strength) (Section A16) in MPa

q = as shown in Figure 24 will normally be zero unless the failure is likely to take place in a weak bed some distance below or above the floor or roof contact, MPa

γ = specific weight of the material, MN/m^3

b = half the pillar width, m

N_c, N_q and N_γ are bearing capacity factors which depend on the angle of friction of the material. Approximate values of these factors can be read off the graph in Figure 25. There are differences between

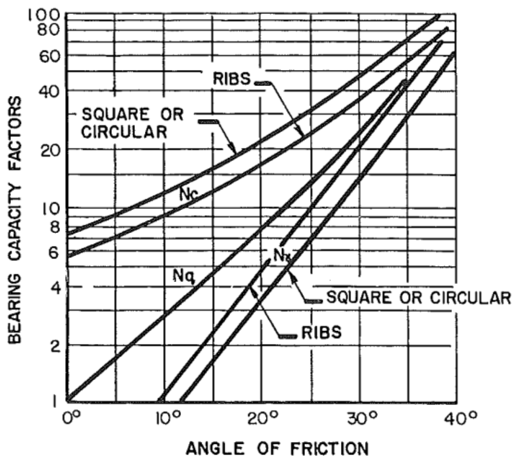


Fig. 25
Foundation Bearing Capacity Factors

ribs, square and circular footings which are taken into account in Figure 25.

The above relationship can be simplified for assessment of the immediate contact material where the material is moderately strong:

Foundation strength (MPa) = 13 x *DRMS* + pillar width.

DRMS is obtained from Section A16 and the pillar width is in metres.

If the pillar and foundation consist of the same material then once a pillar reaches a $W{:}H$ ratio of approximately 7:1 foundation failure rather than pillar failure can be expected.

3.2.3 Pillar Stress

The standard approach to the calculation of pillar stress is from tributary area theory. For horizontal seams:

$$\text{Pillar Stress} = \frac{\sigma_v}{1-e}$$

where:

σ_v = vertical field stress

e = extraction ratio ($100e$ is the percentage extraction)

For inclined pillars the above relationship becomes:

$$\text{Pillar Stress} = \frac{\sigma_v \cos^2\alpha + \sigma_h \sin^2\alpha}{1-e}$$

where:

α = the dip angle in degrees

σ_h = horizontal field stress

When the mining area is significantly smaller than the depth below surface the above relationship gives conservative values since it does not take into account the stress carried by the abutments. Therefore tributary area theory gives the upper limit of pillar stress. If men will be working in the excavations it is advisable to use tributary area theory at the feasibility stage.

In situations where men are remote from the excavations (certain open stoping methods) tributary area theory will be too conservative. In this case some allowance must be taken of pillar yield and transfer of stress to the abutments. However, without the use of sophisticated analysis techniques it is difficult to calculate pillar stresses, particularly if a yielding system is designed. This problem can be partially overcome by using 'pressure arch' theory in which the rock mass is assumed to have a maximum distance over which it can arch and transfer stress to the abutments as shown in Figure 26. It has been suggested[35] that twice this maximum transfer distance can be used as a limit to the size of a panel in which yielding pillars might be successful. There is very little theory or empirical data to support this and what empirical data there is comes mostly from coal mines and mines located in soft rocks. Although the 'pressure arch' concept is simplistic we believe it is appropriate to a pre-feasibility assessment, enabling identification of situations in which yielding pillars may be feasible.

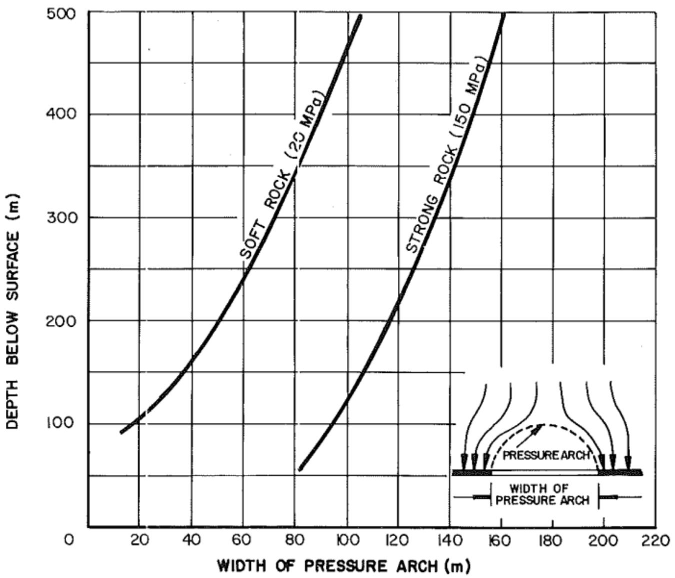


Fig. 26
Width of "Pressure Arch"
(Modified after References 1 and 35)

The stress in abutment pillars separating extraction areas should be calculated from tributary area theory and the load carried by the yielding pillars should be ignored.

3.2.4 Yielding Pillars

A yielding pillar is simply a pillar whose peak strength has been exceeded and the following assumptions are made in terms of design:

- that elastic closure is unavoidable
- that the elastic closure across the mining area will take the pillars past their maximum strength
- but that the pillars will retain a certain degree of integrity and sufficient residual strength to prevent gross inelastic closure, bed separation and loosening of the rock mass in the hangingwall.

The most important criterion is that the pillars remain stable and this will depend on their size, shape, elastic properties, rock mass quality and the amount of deformation. It will also depend on the properties of the roof and floor rocks and the ratio of span of the yielding zone to depth below surface. These considerations determine the 'stiffness' of the loading system[41].

Size: If the pillar walls are very high then considerable failure could take place before the pile of failed material around the pillar starts to exert a confining stress to the pillar core.

Shape: The wider the pillar the more likely that a confined core can be created. Even if it is of failed material it will have significant strength. $W{:}H$ should be at least 2:1 in strong rock (σ_c greater than 100 MPa) and at least 4:1 in a weak rock (σ_c less than 50 MPa).

Elastic Properties: Stiff strong rocks will accept less deformation before failure than weak rocks. They will also show more signs of stress slabbing failure.

Rock Mass Quality: When failure of a pillar takes place additional fractures are created and movement takes place on existing joints reducing the shear strength of the joint surface. In effect, the rock mass quality and *DRMS* (design rock mass strength) are reduced.

Deformation: The greater the deformation, the greater the reduction in rock mass quality of the pillar material and *DRMS*.

In the absence of numerical analysis techniques it is difficult to design yielding pillars with any certainty and at a pre-feasibility stage any conclusions would be restricted to whether or not the design was feasible.

3.2.5 Pillar Design Factor of Safety

The factor of safety is the strength divided by the stress. Since there is usually considerable uncertainty regarding both the strength and the stress there is a risk that the estimated strength may be below the stress. If the probability of either being correct is estimated then the risk associated with a design can be quantified. However, this has not been particularly acceptable to the user since it implies an acceptance of inherent risk whilst the use of a 'factor of safety' implies the opposite. This consideration has tended to restrict empirical relationships to factor of safety terminology with different factors of safety being applied depending on the requirements of a pillar and the consequences of failure:

- men working in the excavations
  — factor of safety greater than 1.5 (where the pillars may, ultimately, be extracted the factor of safety should be greater than 2.0 so that an effective abutment can be formed)
- men remote from the excavations
  — factor of safety greater than 1.1 (if a yielding system is proposed or pillar failure is necessary for future extraction of the pillars using backfill then the factor of safety should be less than 0.5 having regard for the comments in Section 3.2.3)
- barrier or abutment pillars
  — factor of safety greater than 2.5 (unless the barrier is to protect other mining areas from collapse, in which case it is usually designed to be "indestructible" with W:H greater than 10:1).

3.3 Shaft Pillars

A shaft pillar may be required in a number of mining situations to ensure stability and serviceability of the shaft:

- where the orebody is comparatively flat, thin and very extensive and where the shaft is required at the centre of the operations and must therefore pass through the orebody
- where the shaft is maintained in the plane of the orebody as an inclined shaft either in the ore or just beneath the ore
- where the orebody is irregular, discontinuous or steep and the shaft is vertical and well outside the orebody.

These situations are illustrated in Figure 27. Shafts will require protection from:

- subsidence effects from the fully extracted ore zones
- stress effects — when mining depths are considerable it is uneconomic to design a shaft pillar on the basis of subsidence

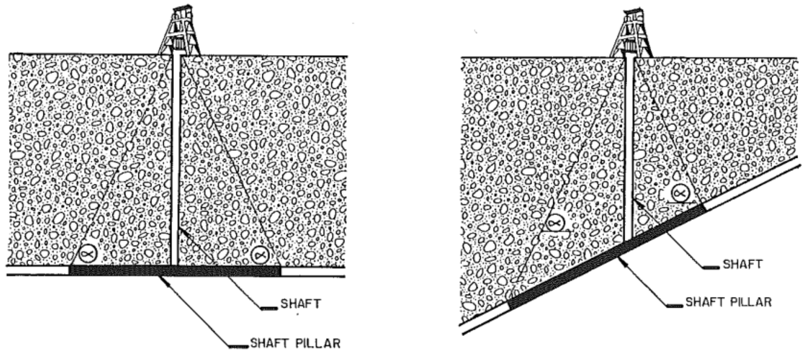
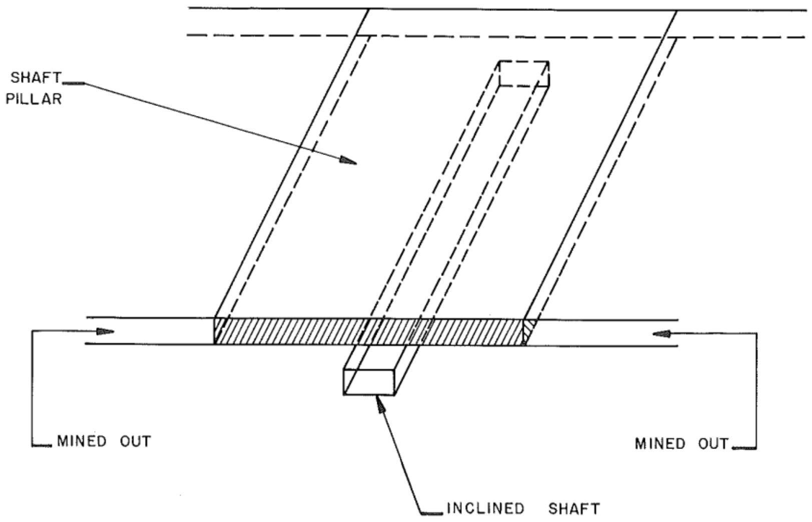
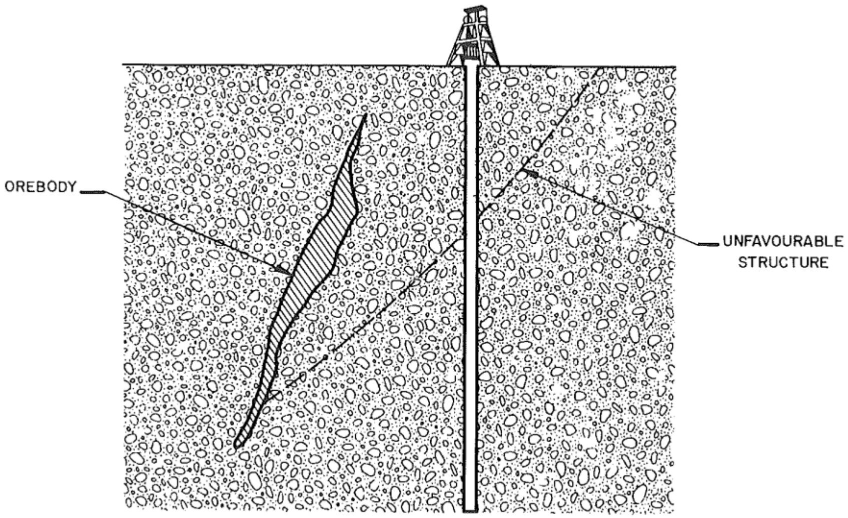


Fig. 27
Shaft Pillar Requirements:
a) Central Location through Thin Tabular Orebody



b) Inclined Shaft in, or just beneath, Tabular Orebody



c) Away from Orebody, but Affected by Loosening when Ore is Extracted

effects. A smaller pillar may be highly stressed making it difficult to maintain stable service excavations within the shaft pillar and causing unacceptable strains and deformations of the shaft

- relaxation effects — once the orebody has been extracted adjacent to a shaft, the loosening of the rock mass between the shaft and orebody may result in movement on unfavourable geological structures. Care must be taken to ensure that the supporting pillar does not result in the transmission of very high stresses.

3.3.1 Subsidence Protection

A first approximation for the design of a shaft pillar would be to use the subsidence limit angle, shown as α in Figure 27(a), with a value of 55°.

3.3.2 Stress Protection

As the depth increases the large strain effects of the subsidence become less pronounced and the shaft is increasingly able to accommodate the reduced strains. In addition the shaft pillar will become uneconomically large if it is to protect the shaft totally from subsidence. In this case the size of the pillar can be assessed from numerical modelling, by applying limiting strain and tilt criteria. A reasonable estimate, for feasibility purposes, can also be made from the relationships in Figure 28. Although approximate, this figure is applicable for both shafts in the plane of the orebody (pillar width) and vertical shafts (pillar radius). If the orebody is inclined relative to the major principal stress, then the shaft pillar can be reduced in size. The pillar size obtained from Figure 28 can be reduced as a linear function of the angle between the σ_1 direction and the plane of the orebody to a practical minimum limit, corresponding to the value of σ_3, when the angle is 90°.

It will be found that when the in situ major principal stress exceeds about one third of the intact rock compressive strength ($\sigma_1 > \sigma_c/3$) the pillars have to be made unacceptably large. In such cases extraction of the orebody in the shaft region at an early stage of mining eliminating a shaft pillar, should be considered.

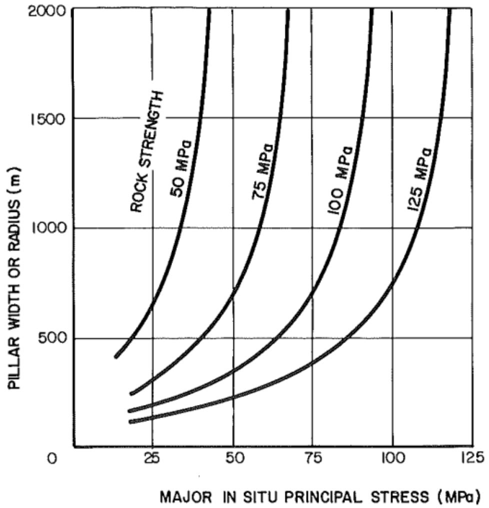


Fig. 28
Shaft Pillar Sizing (Modified after Wagner[45])

Where the shaft is within the footwall it is not usual to use a shaft pillar unless the shaft is relatively close to the extraction area which could result in high stresses being induced in the shaft excavation. In this case, however, the most practical approach is to ensure that the shaft is overmined early in the extraction schedule so that high stresses cannot be transmitted. Support within the shaft excavations should then be designed to cater for relatively large changes in stress and some loosening.

3.4 Passive Support

Passive timber support, in the form of props (sticks) and packs (cribs), was identified in previous discussion as having a place in longwall extraction methods and where large spans are formed in a well bedded deposit. In both these situations the main function of the timber is to restrict bed separation in the roof and maintain a continuous beam as shown in Figure 29. Should blocks of rock comprising the beam be allowed to displace significantly, the integrity of the beam will be reduced or lost entirely.

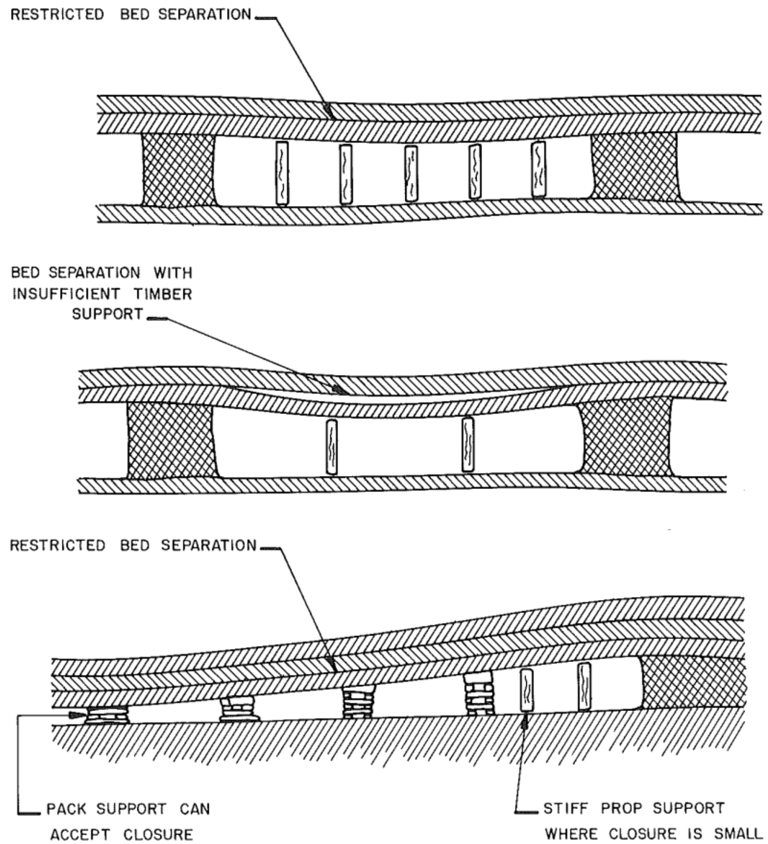


Fig. 29
Principles of Timber Support in Stopes

The timber support must accept the convergence, or closure, between roof and floor without losing support strength. The differences between the prop and the pack are:

- props are stiff and therefore accept little closure, less than 2%, before failing. They can be placed tightly against the roof and therefore can exert a support load early in the life of the excavation
- packs tend to be soft and will take almost unlimited convergence, due to their method of construction, shape and inherent stability, whilst maintaining a support pressure against the roof. Considerable convergence has to take place before the pack provides a significant support resistance.

The first decision in choosing timber support is to identify the degree of possible and acceptable closure. In room and pillar workings the closure will be very small and therefore prop support is indicated. In longwall total extraction methods the closure may be total. In this case packs are indicated. Often a mixture of support is required; stiff props at the face where closure is small and packs further back where the closure is greater. In this situation the props fail and the packs take over the support duties. Props can be modified to accept greater closure (50%) by tapering the ends (profile props) and by encasing them in steel (pipe props).

The choice of timber size and frequency of support unit can be assessed as follows:

- the use of curves in Figures 4 to 7 (Rock Beams Section 2.2.2) to estimate the thickness of a stable beam. This thickness of beam can be taken as a minimum dead weight load which the props must be able to carry with no distress, with the object of preventing any bed separation within this beam
- the frequency of the support units must also be sufficient to ensure that blocks of rock do not fall out between the supports. Figure 18 can be used to estimate the unit spacing if the Q value is known. For a value of $Q < 1$ headboards will be required to give increased support coverage.

Typical load-deformation behaviour for various types of props (sticks) and packs (cribs) is given[44] in Figure 30.

It is only in well-bedded deposits that timber support can be used effectively.

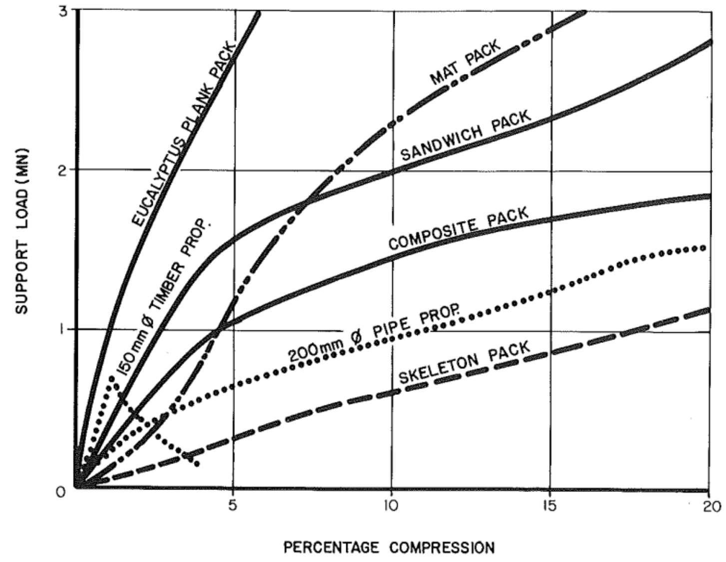


Fig. 30
Deformation Properties of Timber Support
(Redrawn after Wagner[44])

3.5 Backfill

The use of backfill is the most comprehensive strata control method. No collapse can take place although controlled closure can continue until the backfill is sufficiently compressed to accept appreciable load. Backfill can be used if the desired excavation (stope size) is unstable and conventional support methods or the use of pillars are either impractical or uneconomic. Certain mining methods, such as sub-level and block caving, have been developed to avoid the necessity for fill, but these methods can lead to high dilution and loss of ore. In essence, the use of backfill reduces the area of exposed rock surface at any stage in the excavation process.

The assessment of a suitable backfill depends on:

- there being a suitable source of material; usually fill is the product of a concentration process, but in certain circumstances may be quarried sand or rock
- whether a free-standing backfill wall is to be created in which case the addition of cement is usually necessary.

Backfill is only used for mining purposes and in most cases a concentrator will be relatively close to the mining operations. Recent advances in the use of hydraulic fill at very high pulp densities has enabled uncycloned fine tailings to be used, and consideration of grain size and percolation rates is not critical. Hence, for the purposes of a feasibility assessment it can be assumed that if there is a concentrator then a suitable fill exists.

The required strength of the fill will depend on the size of the exposed face and the amount of closure that will take place across the exposed face before the excavation is filled. An approximate estimation of the required strength of the fill can be gained from the following relationship [32]:

$$\sigma_f = \frac{\gamma H}{(1 + H/L)}$$

where:

σ_f = fill compressive strength, MPa
γ = fill density, MN/m^3
H = height of exposure, m
L = width of exposure, m

The amount of cement necessary to achieve the required strength can be estimated from Figure 31. A considerable range in strength is shown. The range depends on the quality of the fill in terms of in situ density. This in turn depends on the grading of the fill, a well graded fill having the right proportion of fine and coarse material to result in as dense a fill as possible. It should also be placed at as high a pulp density as possible. These requirements may conflict with the rate at which the fill will drain and consolidate, although extremely high pulp density fill does not contain any free water. If aggregate is added to the hydraulic fill then proportionally less fill is required. All these factors make it extremely difficult to identify the correct portion of the graph in Figure 31 to use, and we recommend that you err on the conservative for a feasibility analysis.

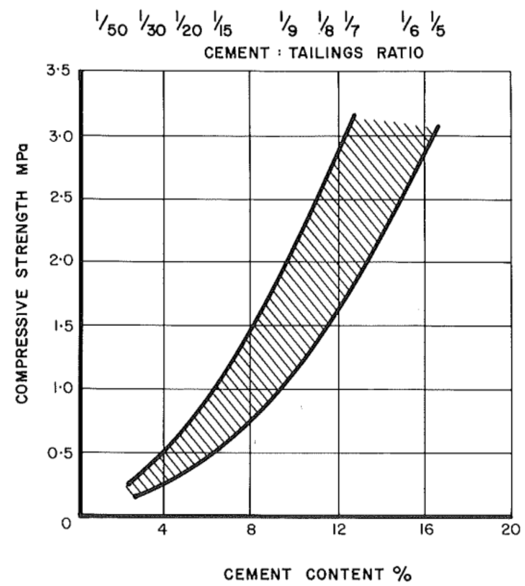


Fig. 31
Compressive Strength Properties of Backfill
(Modified after Singh and Hedley[42])

Appendix

Data Acquisition

This Appendix contains the numbers, or the means of obtaining them, to substitute into the equations, graphs and charts involved in the stability evaluation, estimation of support requirements and pillar design. It is not intended for reading, but more as a sort of "workshop manual" — for you to be able to enter into one section without reference necessarily to any others, find out the information required and then exit again to make use of that information.

An attempt has been made to arrange the Appendix in a sequence in which information required in latter sections has already been dealt with in former sections. Unfortunately this has not always been possible.

We have tried to be as practical and simple as possible, and apologise to any readers who feel we may have overdone it, or believe that there are better sources of information. In such cases we hope that the information provided in this handbook will form a basis for you to add to — in fact we hope that this is the way that it *is* used.

A1

Uniaxial Compressive Strength

The most commonly measured rock strength property is the uniaxial compressive strength σ_c. It is often not the most appropriate property to know, but because it is probably the easiest "quality" test to carry out, there is a large volume of published data available. Since this available set of data forms the basis for preliminary assessment of conditions, we shall develop the approach from this basis.

There are several different levels of rock strength testing to be considered:

(i) If suitable core samples and specimen preparation facilities are available, *direct measurements* of the σ_c can be obtained[25].

(ii) If you are on site you may have the rocks, but no laboratory to carry out the testing. However, the site equipment may include a *point load* tester. This will allow you to carry out tests in a short time, on either rock cores or lumps of rock[25]. It is necessary to convert these measured point load strength index (*PLSI*) values into equivalent values for a 50 mm diameter core, using the chart in Figure Al. The equivalent σ_c can then be calculated as:

$$\sigma_c = 24\ PLSI_{50}$$

For example a measured PLSI of 8 MPa on a 38 mm diameter core corresponds to a $PLSI_{50}$ of 6.6 MPa. The equivalent σ_c is then 158 MPa. To convert MPa to lb/in² (psi) approximately, multiply by 150.

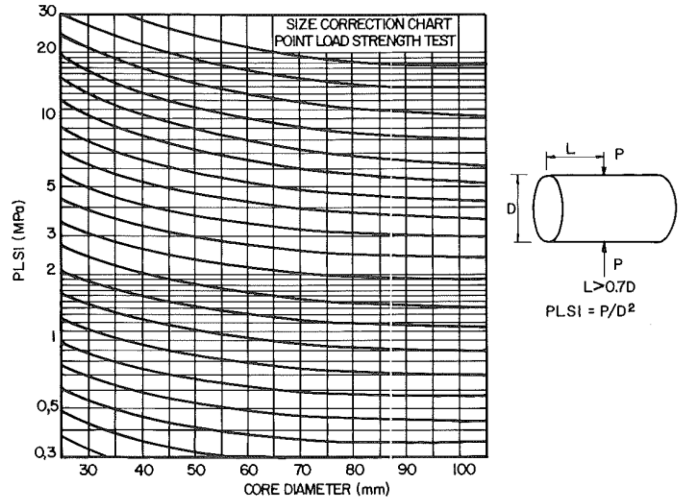


Fig. A1
Point Load Strength Test Size Correction Chart
(Redrawn after Broch and Franklin[14])

The multiplication factor of 24 has been established from many empirical results[14,25,8]. However, there is considerable scatter in the data and the factor can vary between 15 and 35. As a guide, a higher factor should be used for rocks stronger than about 150 MPa and a lower factor for the weaker ones.

(iii) A Schmidt hammer is a very useful and portable item of testing equipment that will enable tests to be carried out very quickly on rock cores, lumps of rock or exposed rock surfaces. A correlation has been derived[31] between the rebound number for an L-type hammer and the σ_c as shown in the chart in Figure A2.

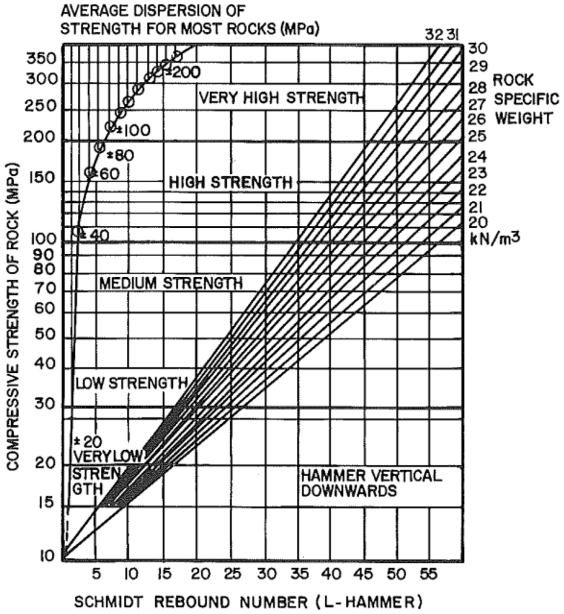


Fig. A2
Correlation between Schmidt Rebound Number (L-Hammer) and σ_c
(Redrawn after Miller[31])

(iv) If you have access to samples, but no testing equipment, you can assess the σ_c of the rock by applying the *descriptive approach*[39,40] in Table Al with the approximate equivalent values of σ_c. It will be necessary for you to use your judgement for interpolating between the categories in the table.

Table A1: Rock Strength from Descriptive Approach

Descriptive Rock Strength	Equivalent σ_c MPa	Equivalent σ_c psi
Easily moulded in fingers; shows distinct heel marks	0.05	5
Moulds in fingers with strong pressure; faint heel marks	0.07	10
Very difficult to mould in fingers; difficult to cut with hand spade	0.15	20
Cannot be moulded in fingers; cannot be cut with hand spade and requires hand-picking to dig out	0.5	70
Very tough and difficult to move with hand pick; requires pneumatic spade for digging	0.7	100
Crumbles under firm blows with sharp end of geological pick and can be peeled off with a knife; it is too hard to cut out a test specimen by hand	3	450
Can just be scraped and peeled with a knife; indentations up to 3 mm show in the specimen with firm blows of the geological pick point	7	1 000
Cannot be scraped or peeled with a knife; hand-held specimen can be broken with one firm blow of hammer end of geological pick	20	3 000
Hand-held specimen breaks under more than one blow with hammer end of geological pick	70	10 000
Many blows with geological pick required to break through intact specimen	200	30 000

(v) When none of the above approaches is applicable, and all you have is the name of the rock type, the published results for common rock types in Table A2 may be useful. The σ_c values in this table are typical values for *fresh unweathered and unaltered* dry rock. They tend to be upper limit strength values. If the rock type in question does not appear in the table, a geologist's opinion will be essential in selecting a similar rock type which does appear.

Table A2: Typical Uniaxial Compressive Strength Values for Unweathered Unaltered, Fresh, Dry Rock

Rock Type	σ_c MPa	σ_c psi	Rock Type	σ_c MPa	σ_c psi
Amphibole	210	30 000	Mudstone	30	4 300
Andesite	240	35 000	Norite	220	32 000
Anorthosite	240	35 000	Porphyry	250	36 000
Basalt	230	33 000	Pyroxenite	150	22 000
Chalk	4	600	Quartzite	240	35 000
Chert	300	44 000	Rocksalt	40	6 000
Coal	40	6 000	Salt	35	5 000
Diabase	240	35 000	Sandstone (Porous)	80	12 000
Dolerite	240	35 000			
Dolomite	100	14 500	Sandstone (Quartzitic)	200	29 000
Gabbro	280	41 000			
Gneiss	220	32 000	Sandstone (Hawkesbury)	40	6 000
Granite	220	32 000			
Greywacke	50	7 250	Schist*	150	22 000
Gypsum	20	3 000	Shale*	120	17 000
Haematite	270	39 000	Siltstone	140	20 000
Limestone	180	26 000	Slate*	210	30 000
Magnetite	100	14 500	Syenite	250	36 000
Marble	140	20 000	Tuff	200	29 000

* Will exhibit very marked strength anisotropy

This table contains gross generalisations regarding the σ_c of the rock types. All of the rock types can vary considerably in their strength properties, depending on many factors, such as grain size, porosity and cementing agents. For example, kaolinisation, chloritisation and seritisation will greatly reduce strength. These factors will have to be taken into account and judgement applied in arriving at a value for σ_c based on the values in the table.

The strength of rock is affected by many factors. The two most relevant for the present purposes are the moisture content and the degree of weathering. The wetter the rock the lower will be its strength. If the underground conditions are such that the rock can be considered as saturated, take the strength as 0.75 of the value you arrived at

previously (based on Colback and Wiid[17]). For intermediate moisture contents interpolate between reduction factors of 0.75 and 1.0.

Weathering has a more dramatic effect on the strength of rock, which must be taken into account depending on the depth of the excavation and the effects of intrusion, faulting etc. The graph in Figure A3 can be used to estimate the corresponding strength reduction from a descriptive basis[40,51].

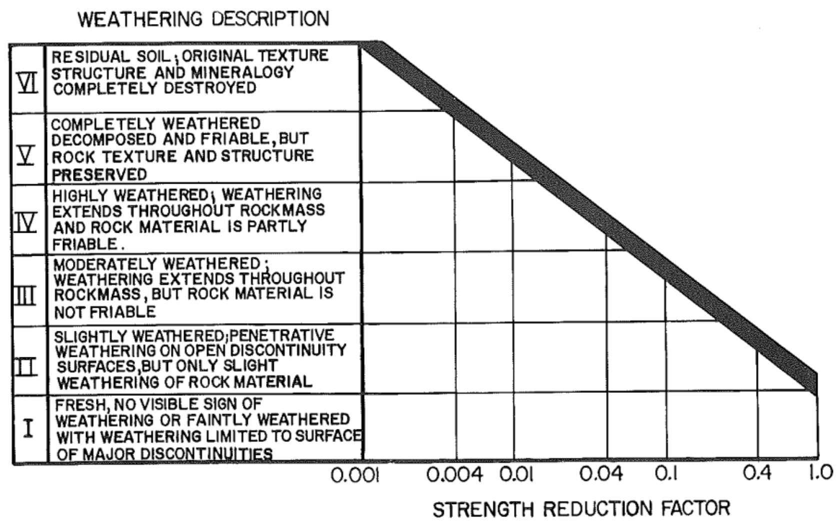


Fig. A3
Strength Reduction as a Function of Weathering

A2

Triaxial Compressive Strength

This property is not referred to specifically in the text of the handbook, but is included here for interest. Confinement increases the strength of rock. When such conditions are applicable it is possible to *estimate* the triaxial strength from the uniaxial compressive strength with reasonable accuracy. In fact the value of σ_{1s} obtained from the equation in Section 2.1.3 is the triaxial strength of the rock. The error in the estimated triaxial strength should generally not exceed 10%[7,33].

A3

Tensile Strength

Tensile strength of intact rock is easily measured by means of the indirect Brazilian test. This again requires laboratory equipment for preparation and testing of specimens, however. Since the tensile strength of intact rock is not usually of great importance, it is sufficiently accurate for practical purposes to assume that the tensile strength is 10% of σ_c.

A4

Shear Strength

The shear strength of intact rock is also not a property often required. However, it may be required when considering "punching" of a pillar into roof or floor for example. For practical purposes, the cohesive strength (shear strength at zero confinement) of the intact rock can be taken[40] as 16% of σ_c. This will usually be a conservative value since the shear strength increases with confinement.

A5

Rock Material Durability

A very important aspect which must be borne in mind is the susceptibility of some rock types to slaking (deterioration on exposure). Mudrocks are particularly susceptible, and others that may cause problems are shale, siltstone, dunite, dolerite and kimberlite. Slaking can rapidly reduce the strength of apparently competent rock and turn initial stability into collapse. Significant swelling often results, which could cause failure of support and concrete linings.

If you have samples of rock available, which you suspect may deteriorate, a quick assessment can be obtained by means of a jar-slake test. This test is qualitative with six descriptive degrees of slaking determined from visual observation of oven dried samples soaked in tap water for 24 hours. These six descriptions and suggested strength reduction factors are listed in Table A3.

Table A3: Jar Slake Test Descriptions

Descriptive Behaviour	Estimated Corresponding σ_c Strength Reduction Factor
Degrades into a pile of flakes or mud	Zero
Breaks rapidly and/or forms many chips	0.01
Breaks rapidly and/or forms few chips	0.05
Breaks rapidly and/or forms several fractures	0.2
Breaks slowly and/or develops few fractures	0.5
No change	1.0

The test specimen should consist of an irregularly shaped particle weighing approximately 20 grams. The observation of slaking action is aided by placing the specimen suspended above the bottom of the jar on a grate with 3 mm to 6 mm size openings. The slaked material will fall through the grate to the bottom of the jar.

Reaction to the jar-slake test usually occurs within the first 10 to 30 minutes, and a standard of 24 hours is recommended as a convenient maximum time for initial testing of a large number of samples. As experience is gained with a particular formation, the maximum time can be reduced to 2 hours or less.

We recommend that you test a minimum of 10 samples for slake durability classification.

The jar-slake test can be extended to provide more useful information when describing the behaviour and hardness of soaked pieces when attempts are made to indent with the fingernail and break apart, crumble, or snap in two with the thumbs and fingers. Many shales that do not slake can be broken apart or crumbled. This softening indicates that the shale could be crushed at low loads if soaking occurred.

A6

Rock Material Deformability

If rock cores and laboratory testing facilities are available, the modulus of elasticity can be measured using instrumented uniaxial compression samples. Alternatively, if hand grab samples are available, or if tests can be carried out on outcrops on the site, then a Schmidt hammer can be used, the rebound number being correlated with the modulus of elasticity[31] as shown in Figure A4.

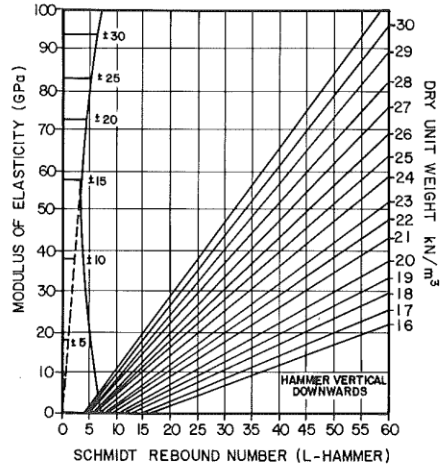


Fig. A4
Correlation between Schmidt Rebound Number (L-Hammer) and Modulus of Elasticity
(Redrawn after Miller[31])

If you have no access to samples then Table A4 gives typical values of the modulus of elasticity E and Poisson's ratio for a few common fresh rock materials.

Table A4: Typical Elastic Properties for Unweathered, Unaltered Rock Material

Rock Material	Modulus of Elasticity E (GPa)	Poisson's Ratio ν
Andesite, Basalt	60	0.20
Diabase, Dolerite, Gabbro	90	0.20
Coal	3	0.42
London Clay	0.1	0.50
Diorite	80	0.26
Dolomite	70	0.15
Gneiss	60	0.24
Granite	60	0.22
Limestone	70	0.30
Quartzite, Quartzitic sandstone	80	0.17
Sandstone	20	0.15
Shale	15	0.10

Judgement must be exercised to interpolate between these values for other rock material types, and to take into account the effect of weathering and alteration.

A7

Anisotropy of Rock

In the previous sections no attention has been given to the variation in strength and deformation properties of rock in different directions. In some rock types, for example slate, this variation can be extreme, particularly when confinement is low, such as in rock pillars and adjacent to the walls of excavations. In confined situations, the effect of rock anisotropy is unlikely in practice to result in a strength or deformability reduction in the weakest direction of more than 2:1. In unconfined situations, strength reduction effects may be greater than 10:1.

A8

Jointing in Rock Masses

Jointing or the presence of discontinuities within a rock mass plays a critical role in determining the extent of any likely instability surrounding a proposed excavation.

The discontinuities within a rock mass *in general* conform to recognisable patterns and can be grouped into sets or families, easily seen when plotted on a stereonet (Figure A5).

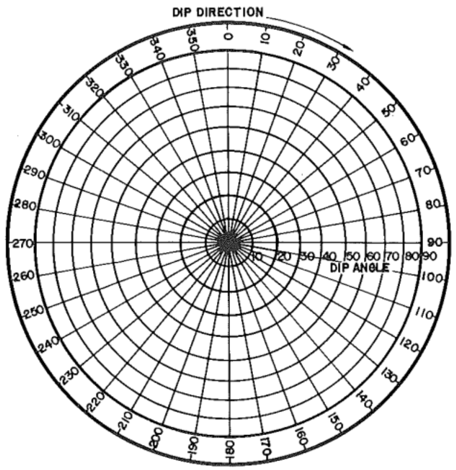


Fig. A5
Polar Stereonet

Each set usually has a distinct orientation which in turn is related to the structural evolution of the rock mass. The three geological rock types of igneous, metamorphic and sedimentary can exhibit recognisable fracture patterns within a distinct structural environment. Further to this, it is sometimes possible to identify whether the discontinuity was formed in tension, compression, shear or from sedimentary deposition by examining the surface properties related to mineral coating, infilling and roughness.

The five properties controlling the three-dimensional framework of a rock mass are as follows:

- dip angle } orientation of joint plane
- dip direction
- strike length } continuity of joint plane
- dip length
- spacing between individual joints.

The range of these properties and therefore the minimum, maximum and average values are significant if a complete understanding of the overall structure is to be achieved.

The surface properties of each set, as well as being a guide to the origin and formation of the discontinuities, are partly responsible for developing the shearing resistance between individual blocks during movement of the rock mass surrounding the excavation.

When collecting joint field data we recommend the use of a standardised approach and have found that the use of a data collection sheet such as that in Figure A6 is most successful.

As the major use of this handbook will be for pre-feasibility evaluations it is possible that no site specific detailed geotechnical data will be available. Therefore it will be necessary to utilise a "general knowledge" of rock masses to obtain a "first impression" of the geological environment as input into a rock mass description and classification for initial stability analysis.

Subsequently, use must be made of the rock mass structure in evaluating the most efficient excavation size and shape. For example, in certain sedimentary strata it is beneficial to cut the excavation roof along prominent bedding planes thus producing a relatively stable con-

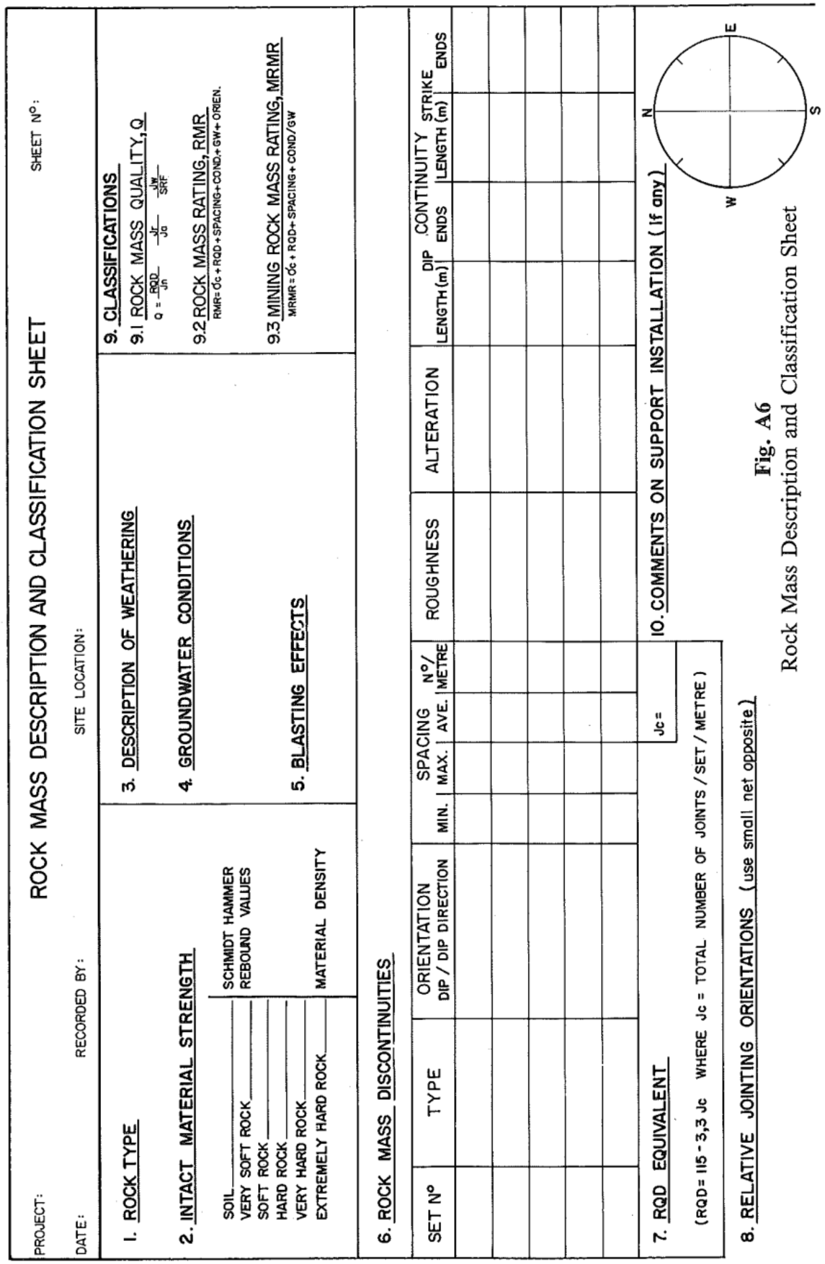


Fig. A6 Rock Mass Description and Classification Sheet

figuration. Conversely, within most igneous extrusive and intrusive rock masses very significant advantages can be achieved by cutting an arched roof thus taking full advantage of the numerous cooling joints which tend to have limited continuity.

The following criteria have been developed from experience:

- most rock types have at least three sets of discontinuities
- the number of discontinuity sets present will increase relative to the complexity of the geological history of a specific area
- the structural importance of discontinuities will decrease with depth below ground surface and hence also with degree of weathering and influence of groundwater. The spacing between joints is generally closer at shallow depth
- in general the shearing resistance of discontinuity surfaces will decrease with an increase in weathering
- the strike or trend of the subvertical discontinuities can generally be ascertained from an examination of aerial photographs or from the drainage patterns shown on a topographical map.

Typically within sedimentary rock types, for example, the three most commonly occurring discontinuity sets are as follows:

- bedding, generally fully continuous along dip and strike in relation to excavation dimensions
- cross joint, generally normal to the bedding, confined to the more massive bedding units and not continuous. Often two such sets at right angles to each other
- shear joint, generally fully continuous through the bedding and also generally normal to the bedding.

These three sets can, therefore, combine to form unstable blocks of rock controlled in general by the bedding spacing dimension.

In the following table, some examples of typical structural configurations taken from actual projects are given. These will provide some indication of typical jointing and joint conditions that can be expected in a range of rock types. Manipulation of these set configurations using a stereonet to match your own strata dip, for example, will allow you to assess the possibility of occurrence of unstable blocks in

Table A5: Characteristic, Measured Joint Properties

Rock Type	Discontinuity Set Dip Angle and Dip Direction	Average Spacing	Continuity	Surface Properties
1. Anorthosite or Pyroxenite	84°/074°	1.2 m	Controlled by cooling joint	Smooth, planar, calcite coating
	83°/184°	1.3 m	Fully continuous	Slickensided, undulating, chlorite coating
	9°/346° (cooling joint)	2.2 m	Fully continuous	Smooth, undulating clean surface
2. Gneiss	56°/262°	1.4 m	Controlled by shallow angle discontinuity	Smooth, undulating, calcite coating
	10°/210°	1.7 m	Fully continuous	Rough, undulating oxide stained
	80°/165°	1.4 m	Fully continuous	Rough, irregular, clean surfaces
	78°/023°	3.2 m	Limited continuity controlled by intermediate dipping discontinuity	Very rough, irregular, mica coating.
3. Tuff	65°/320° 60°/006° 70°/105° 50°/128°	0.25 m 0.95 m 0.85 m 0.35 m	All of similar continuity in the dip and strike directions	Slightly rough and undulating with iron oxide staining and pyrite crystals
4. Serpentinite	65°/171°	1.40 m	Limited continuity	Smooth, planar, talc coating
	80°/069°	1.10 m	Limited continuity	Smooth, undulating, slight talc coating
	60°/099°	0.80 m	Limited continuity	Smooth, undulating, slight talc talc
	75°/290° (foliation)	0.10 m	Fully continuous	Smooth, undulating with talc coating.

Table A5: Characteristic, Measured Joint Properties

Rock Type	Discontinuity Set Dip Angle and Dip Direction	Average Spacing	Continuity	Surface Properties
5. Dolomite	88°/006°	1.10 m	Fully continuous	Slightly rough and undulating, calcite and occasional graphite coating
	80°/258°	0.95 m	Continuity limited by bedding discontinuity	Slightly rough and undulating, calcite and occasional graphite coating
	88°/035°	1.25 m	Continuity limited by bedding discontinuity	Slightly rough and undulating, calcite and occasional graphite coating
	7°/002° (bedding)	0.40 m	Fully continuous	Rough, undulating, pyritic coating
6. Siltstone/ Sandstone	20°/265° (bedding)	0.25 m	Fully continuous	Rough and undulating, oxide staining
	80°/305°	0.55 m	Fully continuous	Rough and planar, oxide staining
	60°/195°	0.60 m	Controlled by bedding	Smooth and undulating, clean surface
	72°/085°	1.5 m	Limited continuity in dip and strike	Very rough and undulating, clean surface
7. Quartzite	11°/135° (bedding)	0.5 m	Fully continuous	Rough, planar, slight oxide staining
	80°/335°	1.6 m	Fully continuous	Smooth, planar, clean surface
	88°/045°	0.9 m	Controlled by bedding spacing	Rough, irregular clean surface
	87°/115° (random)	2.4 m	Partially controlled by bedding discontinuity	Slickensided, planar, clean surface

the roof and sidewalls of an excavation. You should note that tremendous variations can occur, so generalisation is not possible.

The determination of a rock mass classification during the pre-feasibility stage of a project requires a certain level of data for the following parameters (see Sections A10, All and A12):

- number and orientation of discontinuity sets
- spacing and orientation of discontinuity sets and therefore an equivalent *RQD*
- condition of the discontinuity surfaces
- groundwater conditions
- strength of rock material
- orientation of the major discontinuities relative to the proposed excavation geometry.

All of these parameters can be assessed to some degree from limited sources of information as has been described previously in this section. A sample rock mass description and classification sheet containing the three most widely used classification systems is shown in Figure A6.

A9

Joint Shear Strength

The shear strength of the joints will affect the stability of the excavations. If sufficient structural information is available to allow realistic analysis of stability, then some quantitative assessment of joint shear strength will be required. Measurement of joint strength by means of direct shear testing is not recommended. The shear strength can be estimated perfectly adequately[4] from the following formula, provided that exposed joint surfaces are available in situ, or at least in core samples.

$$\tau = \sigma_n \tan \left[JRC \log_{10} \left(\frac{JCS}{\sigma_n} \right) + \phi_b \right]$$

where:

τ	is peak shear strength
σ_n	is effective normal stress
JRC	is the joint roughness coefficient
JCS	is the joint wall compressive strength
ϕ_b	is the basic friction angle (obtained from residual shear test on flat unweathered surfaces).

The shear strength estimated using the above formula includes the apparent cohesion of the joint.

In practice ϕ_b may vary between about 25° and 35°, as shown[4] in Table A6 and the assumption of 30° will be adequate. This is confirmed by the distribution of residual friction angles measured in actual rock joints[52] in a variety of rock types shown in Figure A7.

Table A6:
Basic Friction Angles

Rock Types	Basic Friction Angle ϕ_b Wet	Basic Friction Angle ϕ_b Dry
Amphibolite		32
Basalt	31	38
Chalk	30	
Dolerite	32	36
Dolomite	27	31
Schistose Gneiss	23	29
Granite	29	35
Limestone	33	40
Sandstone	25	35
Shale	27	
Siltstone	27	31
Slate	25	30

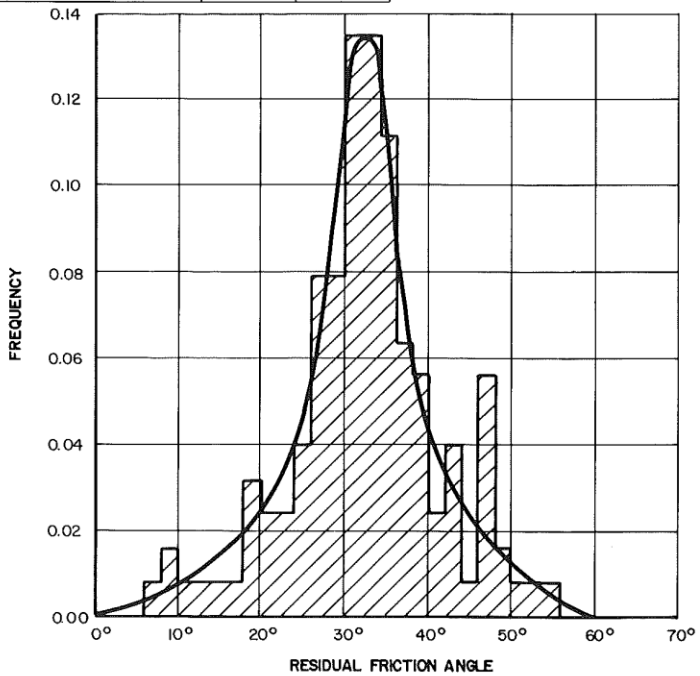


Fig. A7
Distribution of Residual Friction Angle of Joints
(Redrawn from Reference 52)

JCS should be determined using a Schmidt hammer (Section Al) and *JRC* can be interpolated from the roughness profiles in Figure A8.

A simpler, but more approximate means of estimating *frictional* shear strength (excluding apparent cohesion) is to take account of the roughness using only the descriptions[40] in Table A7 and add the basic friction angle.

Table A7: Roughness and Frictional Strength of Joints

Description of Roughness	Friction Angle (°)
Smooth	Basic + 2
Defined ridges	Basic + 6
Small steps	Basic + 10
Very rough	Basic + 14

Slickensiding on joint surfaces will reduce the angle of friction very considerably, and the presence of gouge or other infilling on the joints may totally control the joint strength. In such cases, take the angle of friction to be 15°.

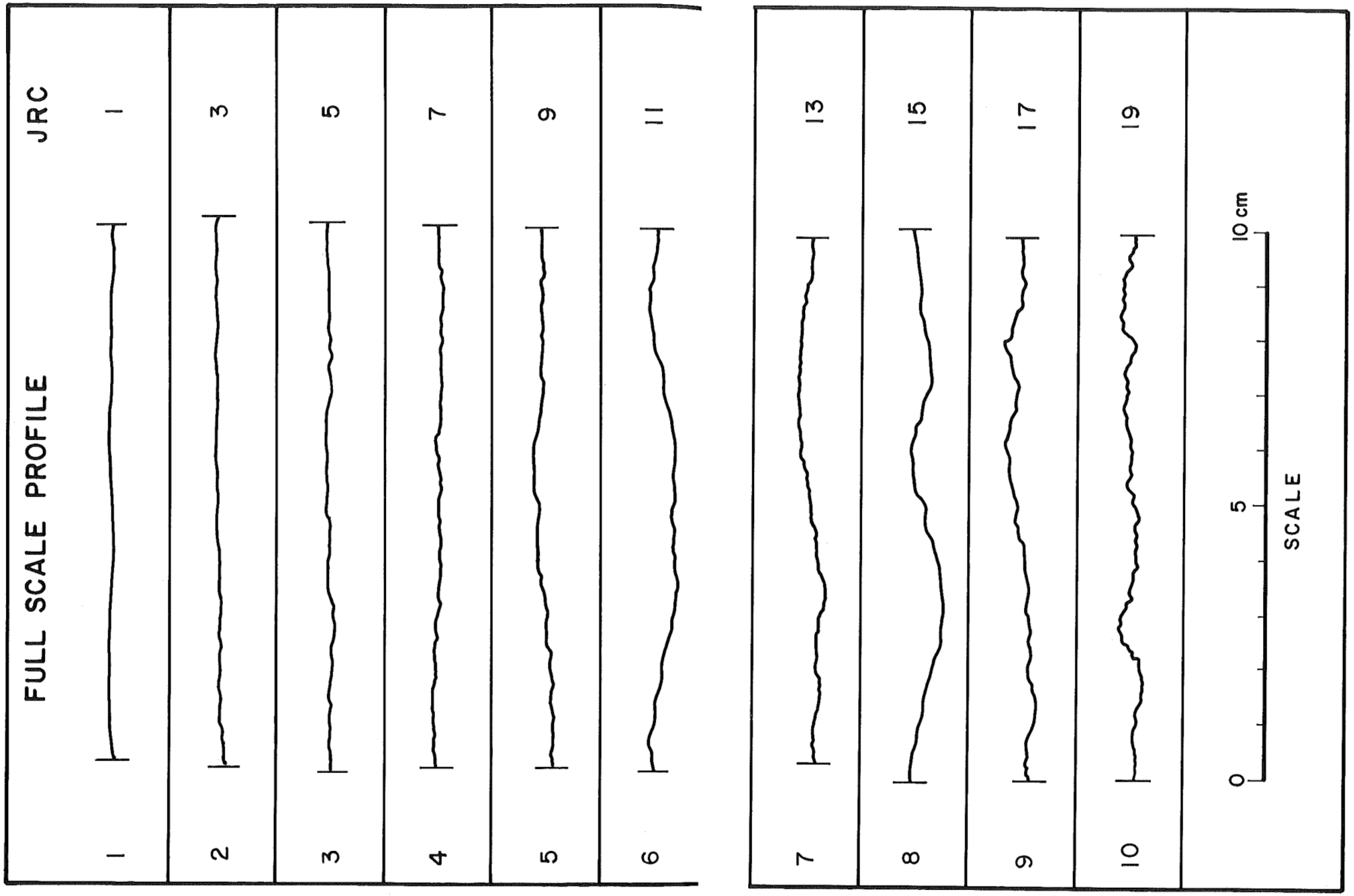


Fig. A8
Joint Roughness Profiles
(Redrawn after Barton and Choubey[4])

A10

Q System Rock Mass Classification[5]

$$\text{Rock mass quality } Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$$

where:

RQD is the rock quality designation
J_n is the joint set number
J_r is the joint roughness number
J_a is the joint alteration number
J_w is the joint water reduction factor
SRF is the stress reduction factor

These parameters and their corresponding values are described in the following sections. These descriptions are somewhat abbreviated[30] from the original presentation[5].

RQD — Rock Quality Designation

RQD is defined as the ratio of the cumulative length of sticks of NX size core more than 100 mm in length in a drill run to the total length of the drill run:

$$RQD = \frac{\text{Sum of length of core sticks more than 100 mm long}}{\text{Total length of drill run}} \times 100\%$$

Judgement must be exercised for poorly orientated boreholes. For example holes parallel to bedding in a sedimentary deposit may indicate very high values of RQD, whereas holes across bedding in the same rock may indicate much lower RQD's.

RQD can be estimated from inspection of exposed rock surfaces by determining the number of unhealed joint planes per m³ of rock. This may be done by counting the relevant number of joint planes (excluding blast fractures) which cross a 2 to 3 m length of tape held against the excavated wall. The number of joint planes divided by the relevant sample length gives the number of joints per metre. This process is then to be repeated for 2 additional directions. The sum of these three values gives J_v, the number of joints per m³, and hence RQD from the equation[5]:

$$RQD = 115 - 3.3\ J_v$$

Notes:

- Where RQD is reported or measured as less than 10, a nominal value of 10 is used to evaluate Q
- RQD intervals of 5, giving 100, 95, 90...10 are sufficiently accurate.

J_n — Joint Set Number

A numerical value is allocated corresponding to the number of joint sets present in the rock mass.

Table A8

Number of Joint Sets	Joint Set No. (J_n)
Intact, no or few joints	0.5 — 1.0
One joint set One joint set plus random joints	2 3
Two joint sets Two joint sets plus random joints	4 6
Three joint sets Three joint sets plus random joints	9 12
Four or more joint sets, random, heavily jointed, sugar cube, etc.	15
Crushed rock, earthlike	20

Notes:

- For intersections use $J_n = 3\ J_n$
- For portals use $J_n = 2\ J_n$

J_r Joint Roughness Number

Distinction is made between the large scale nature of planes and the small scale roughness as well as between continuous and discontinuous joints. Table A9 gives joint roughness number (J_r) values.

Table A9

Desciption of Joint Surface Roughness	Discon-tinuous	Undulat-ing	Planar
Rough	4.0	3.0	1.5
Smooth	3.0*	2.0	1.0
Slickensided	2.0*	1.5	0.5
Planes containing gouge thick enough to prevent rockwall contact	1.5*	1.0	1.0

* Data added to original sequence[5]

Note:

- Add 1.0 to J_r if the mean spacing of the relevant joint set is greater than 3 m.

J_a — Joint Alteration Number

The joint alteration number takes into account the weathering of, or coating on, joint surfaces and the thickness and nature of any gouge infill present in the joints. This parameter will determine the shear strength of the rock mass as well as its deformability and potential to squeeze or swell.

Table A10

Description of Gouge	Joint Alteration Number (J_a) for Joint Separation (mm)		
	< 1.0[1]	1.0—5.0[2]	> 5.0[3]
Tightly healed, hard, non-softening impermeable rock mineral filling	0.75	—	—
Unaltered joint walls, surface staining only	1.0	—	—
Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling	2.0	4.0	6.0
Non-softening, slightly clayey non-cohesive filling	3.0	6.0*	10.0*
Non-softening strongly over-consolidated clay mineral filling, with or without crushed rock	3.0 *	6.0[4]	10.0
Softening or low friction clay mineral coatings and small quantities of swelling clays	4.0	8.0*	13.0*
Softening moderately over-consolidated clay mineral filling, with or without crushed rock	4.0*	8.0[4]	13.0
Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock	5.0*	10.0[4]	18.0

* Figures added to original data[5] to complete sequence

Notes:

1. Joint walls effectively in contact
2. Joint walls come into contact before 100 mm shear
3. Joint walls do not come into contact at all upon shear
4. Also applies when crushed rock present in clay gouge and no rock wall contact

J_w Joint Water Reduction Factor

The joint water reduction factor allows for the water pressure on the joint walls, as well as the potential for the outwash and softening of joint gouge.

Table A11

Condition of Groundwater	Head of Water (m)	Joint Water Reduction Factor (J_w)
Dry excavation or minor inflow 5 litre/minute locally	< 10	1.0
Medium inflow, occasional outwash of joint/fissure fillings	10 — 25	0.66
Large inflow in competent ground with unfilled joints/ fissures	25—100	0.5
Large inflow with considerable outwash of joint/fissure fillings	25—100	0.33
Exceptionally high inflow upon excavation, decaying with time	> 100	0.2—0.1
Exceptionally high inflow continuing without noticeable decay	> 100	0.1—0.05

Notes:

- Last three categories are crude estimates. Increase J_w if drainage measures are installed
- Special problems caused by ice formation are not considered.

SRF — Stress Reduction Factor

(a) Weakness zones intersecting excavation which may cause loosening of rock mass when tunnel is excavated.

Table A12

Description	*SRF* value
Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
Single weakness zones containing clay or chemically disintegrated rock (depth of excavation < 50 m)	5
Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m)	2.5
Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5
Single shear zones in competent rock (clay-free) (depth of excavation < 50 m)	5.0
Single shear zones in competent rock (clay-free) (depth of excavation > 50 m)	2.5
Loose open joints, heavily jointed or "sugar-cube" etc (any depth)	5.0

Notes:

- Reduce these values of *SRF* by 25-50% if the relevant shear zones only influence, but do not intersect the excavation

(b) Competent rock, rock stress problems

Table A13

Description	σ_c/σ_1	σ_t/σ_1	*SRF* Value
Low stress, near-surface	> 200	> 13	2.5
Medium stress	200-10	13-0.66	1.0
High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10—5	0.66—0.33	0.5—2
Mild rock burst (massive rock)	5—2.5	0.33—0.16	5—10
Heavy rock burst (massive rock)	< 2.5	< 0.16	10—20

Calculation of *Q*

All selected values for the above six parameters, based on observed or estimated conditions are substituted into the equation

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$$

to obtain the value of the rock quality index Q.

A11

Geomechanics Classification[9]

All the parameters, descriptions and ratings are included in Table A14.

The *RMR* value for the rock mass is obtained by summing the five individual parameter ratings.

Table A14: Geomechanics Classification

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point load strength index	> 8 MPa	4—8 MPa	2—4 MPa	1—2 MPA	For this low range — uniaxial compressive test is preferred		
		Uniaxial compressive strength	> 200 MPa	100—200 MPa	50—100 MPa	25—50 MPA	10—25 MPa	3—10 MPa	1—3 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90 %—100 %	75 %—90 %	50 %—75 %	25 %—50 %	< 25 %		
	Rating		20	17	13	8	3		
3	Spacing of joints		> 3 m	1—3 m	0,3—1 m	50—300 mm	< 50 mm		
	Rating		30	25	20	10	5		
4	Condition of joints		Very rough surface Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1 mm Hard joint wall rock	Slightly rough surfaces Separation <1 mm Soft joint wall rock	Slickensided surfaces OR Gouge <5 mm thick OR Joints open 1—5 mm Continuous joints	Soft gouge >5 mm thick OR Joints open >5 mm Continuous joints		
	Rating		25	20	12	6	0		

Table A14: Geomechanics Classification

	Parameter		Ranges of Values				
5	Groundwater	Inflow per 10 m tunnel length	OR	None	OR <25 litres/min	OR 25—125 litres/min	OR >125 litres/min
		Ratio joint water pressure / joint prin. stress	OR	0	OR 0.0—0.2	OR 0.2—0.5	OR >0.5
		General conditions	Completely dry		Moist only (interstitial water)	Water under moderate press.	Severe water problems
	Rating		10		7	4	0
6	Strike and dip orientations of joints*		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
	Rating		0	−2	−5	−10	−12

* The Effect of Joint Strike and Dip Orientation in Tunneling

Strike perpendicular to excavation axis				Strike parallel to excavation axis		Dip 0°—20° irrespective of strike
Drive with dip		Drive against dip				
Dip 45°—90°	Dip 20°—45°	Dip 45°—90°	Dip 20°—45°	Dip 45°—90°	Dip 20°—45°	
Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	Fair	Unfavourable

A12

Mining Rock Mass Classification[27,28]

This method is summarised in Table A15. It allows the ratings for joint spacing and joint condition to be allocated quantitatively. (Figure A9).

Adjustments for joint parameters *A* to *D* in Table A16 are cumulative. For example, a dry straight joint with a smooth surface and fine soft-sheared joint filling would have a minimum rating of 40 x 70% x 60% x 60% = 10.1.

The mining rock mass rating *MRMR* is obtained by summing the four individual parameter ratings.

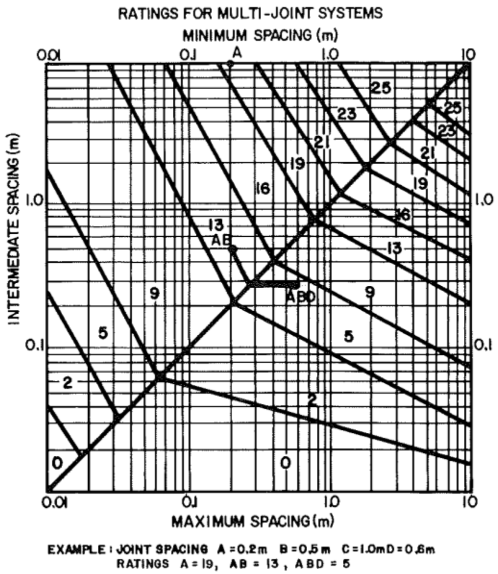


Fig. A9
MRMR Joint Spacing Ratings (Redrawn after Laubscher[27])

Table A15: Mining Rock Mass Classification

	Parameter	Range of Values										
1	RQD	100—97	96—84	83—71	70—56	55—44	43—31	30—17	16—4	3—0		
	Rating (= RQD x 15/100)	15	14	12	10	8	6	4	2	0		
2	UCS (MPa)	185	184—165	164—145	144—125	124—105	104—85	84—65	64—45	44—25	24—5	4—0
	Rating	20	18	16	14	12	10	8	6	4	2	0
3	Joint Spacing	Refer Figure A9										
	Rating	25 —— 0										
4	Joint Condition Including Groundwater	Refer Table A16										
	Rating	40 —— 0										

Table A16: Adjustments for Joint Condition and Groundwater

Parameter	Description		Dry condition	Wet Conditions: Moist	Wet Conditions: Moderate pressure 25—125 l/min	Wet Conditions: Severe pressure $>$ 125 l/min
A Joint expression (large scale irregularities)	Wavy	Multi-directional	100	100	95	90
	Wavy	Uni-directional	95 90	95 90	90 85	80 75
	Curved		89 80	85 75	80 70	70 60
	Straight		79 70	74 65	60	40
B Joint expression (smale scale irregularities or roughness)	Very rough		100	100	95	90
	Striated or rough		99 85	99 85	80	70
	Smooth		84 60	80 55	60	50
	Polished		59 50	50 40	30	20
C Joint Wall alteration zone	Stronger than wall rock		100	100	100	100
	No alteration		100	100	100	100
	Weaker than wall rock		75	70	65	60
D Joint filling	No fill — surface staining only		100	100	100	100
	Non-softening and sheared material (clay or talc free)	Coarse sheared	95	90	70	50
		Medium sheared	90	85	65	45
		Fine sheared	85	80	60	40
	Soft sheared material (eg talc)	Coarse sheared	70	65	40	20
		Medium sheared	65	60	35	15
		Fine sheared	60	55	30	10
	Gouge thickness $<$ amplitude of irregularity		40	30	10	
	Gouge thickness $>$ amplitude of irregularity		20	10	Flowing material 5	

A13

In Situ Stresses

The in situ stress field prior to creation of an underground opening is the basis for the assessment of the effects of stress redistribution around the excavation. Measurements of stress have been carried out in many parts of the world. These results can be used as the basis for estimating the in situ stress field acting at your site. The following simple equations are suggested for the estimation of stress magnitudes in MPa, where depth is in metres:

$$\frac{\sigma_{Hor}}{\sigma_{Vert}} = 3 - \frac{\text{Depth}}{500} \quad (\text{Depth} < 1000 \text{ m})$$

$$\frac{\sigma_{Hor}}{\sigma_{Vert}} = \frac{9}{8} - \frac{\text{Depth}}{8000} \quad (\text{Depth} > 1000 \text{ m})$$

The vertical stress can be estimated using the relation:

$$\sigma_{Vert} = \frac{\text{Depth in metres}}{40} \quad \text{MPa}$$

However, in situ stresses are subject to significant local variations, and the assessment of a competent structural geologist will be very useful. For example, high horizontal to vertical stress ratios are known to occur in Scandinavia. Topography also plays an important role. For example, an excavation near the foot of an escarpment will be affected by local stresses rather than by the general stress trend.

The orientations of stresses can often be inferred from the trends of structural features, river courses etc. Failing this, the orientations determined from in situ stress measurements in Scandinavia[34], Australia[50], Southern Africa[20] and the United States of America[21] summarised in Figures A10 to A13 show the general trends of the horizontal stresses, which can be assumed if topographical information is not available.

Fig. A10
Horizontal Stress Trends in Scandinavia
(Redrawn after Myrvang[34])

Fig. A11
Horizontal Stress Trends in Australia
(Redrawn after Worotnicki and Denham[50])

Fig. A12
Horizontal Stress Trends in Southern Africa
(Redrawn after Gay[20])

Fig. A13
Horizontal Stress Trends in North America
(Redrawn after Haimson[21])

A14

Stress Distributions around Excavations

The series of figures relevant to this section presents the stress distributions around a circular opening and around rectangular openings with various length to width ratios. These shapes have been chosen since they are representative of the majority of underground openings.

Instead of contouring the stresses as is commonly found in the literature, corresponding values of the major stress σ_1 and the confining stress σ_3 are given at particular points. We believe that this approach is most practical since the purpose of the stress distributions is to provide a means of assessing:

- likely depth of failure zone, if any
- length of support elements required
- possible extent of waste hangingwall dilution.

For a particular rock mass environment it will be necessary to consider only a few particular locations and therefore, for brevity, values have only been presented in several directions. The stress distribution diagrams are given in Figures A14 to A37. The magnitudes of the stresses are in the dimensionless form σ_1/Q and σ_3/Q. The directions of the field stresses P and Q are shown on the diagrams.

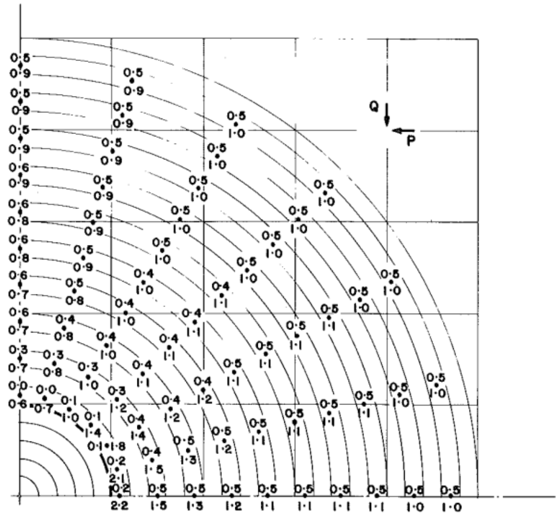


Fig. A14
Stresses around a Circular Excavation — $P/Q = 0.5$

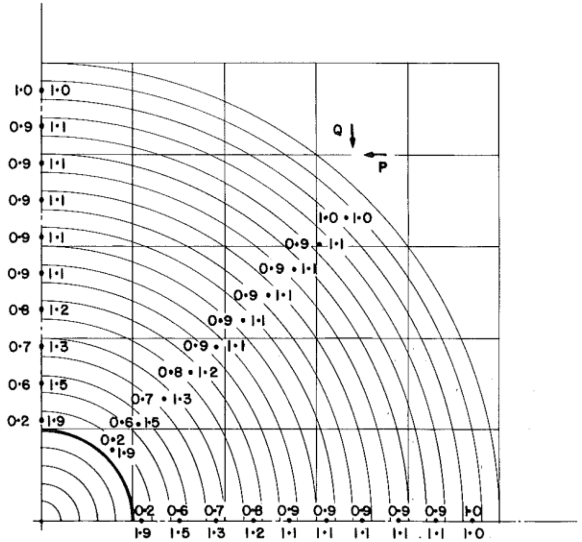


Fig. A15
Stresses around a Circular Excavation — $P/Q = 1$

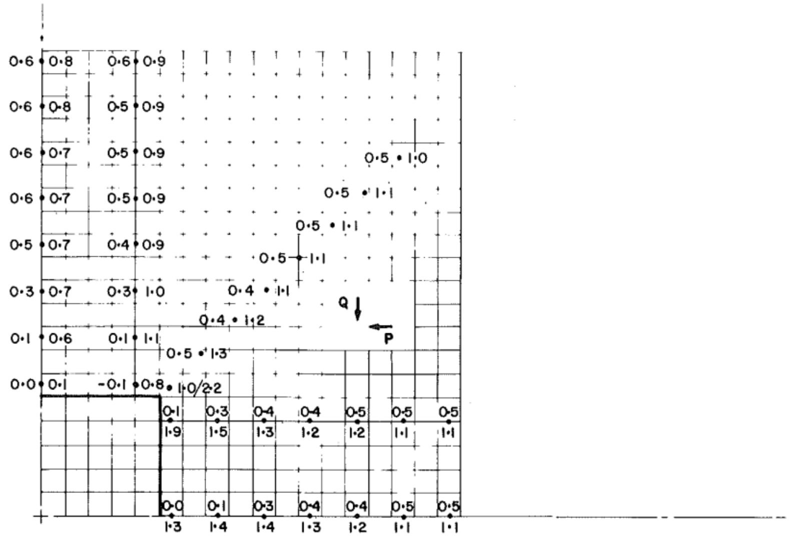


Fig. A16
Stresses around a Square Excavation — $P/Q = 0.5$

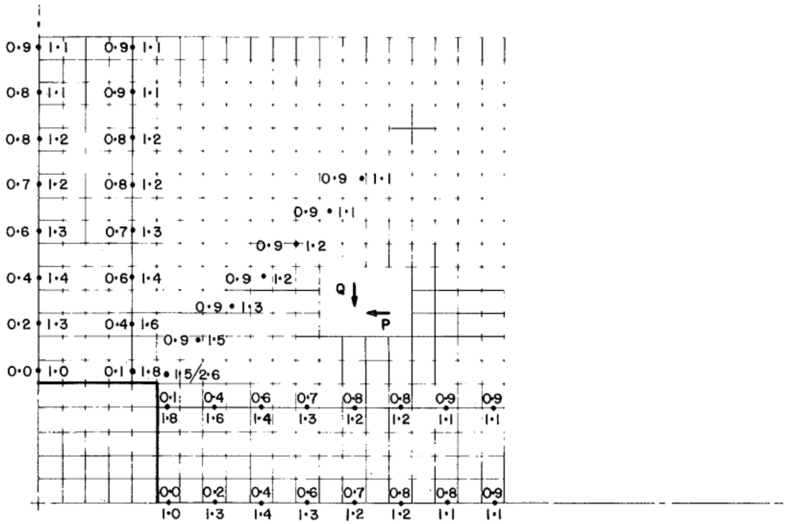


Fig. A17
Stresses around a Square Excavation — $P/Q = 1$

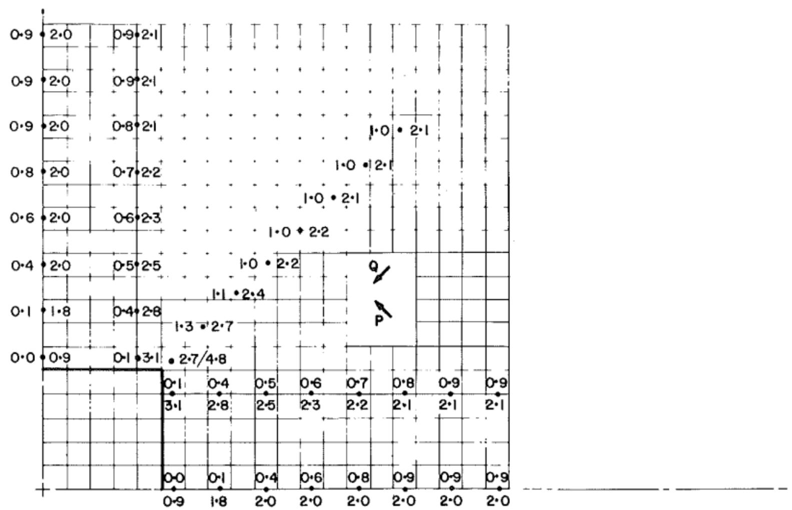


Fig. A18
Stresses around a Square Excavation Orientated at 45° to the Principal Stress Directions — $P/Q = 0.5$

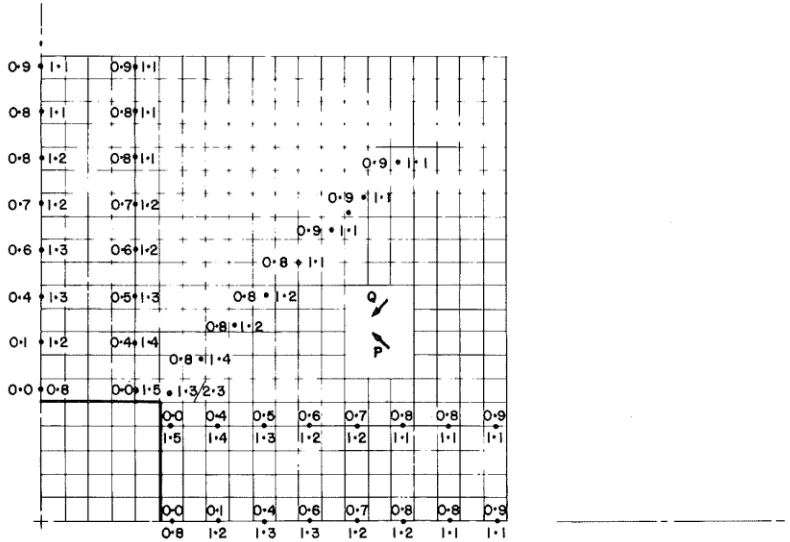


Fig. A19
Stresses around a Square Excavation Orientated at 45° to the Principal Stress Directions — $P/Q = 1$

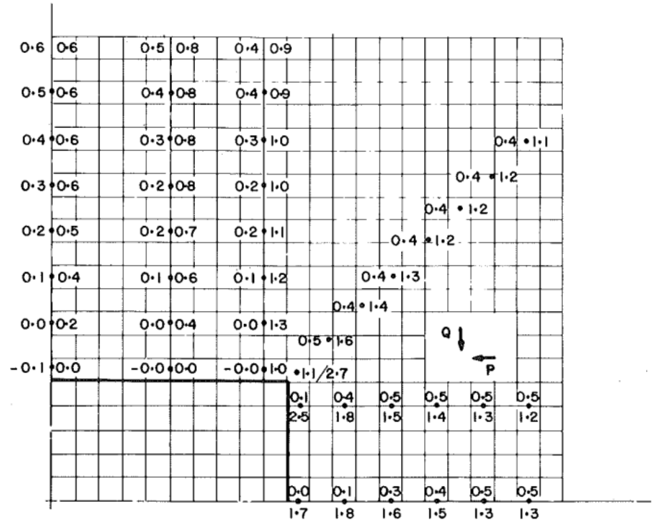


Fig. A20
Stresses around a 2:1 Rectangular Excavation — $P/Q = 0.5$

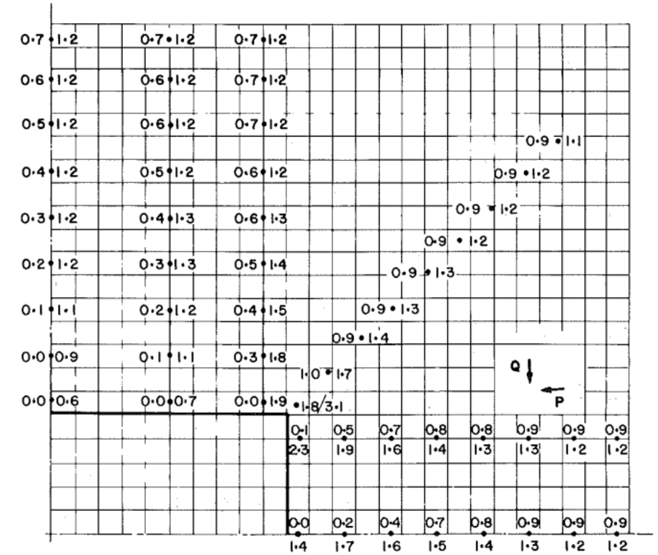


Fig. A21
Stresses around a 2:1 Rectangular Excavation — $P/Q = 1$

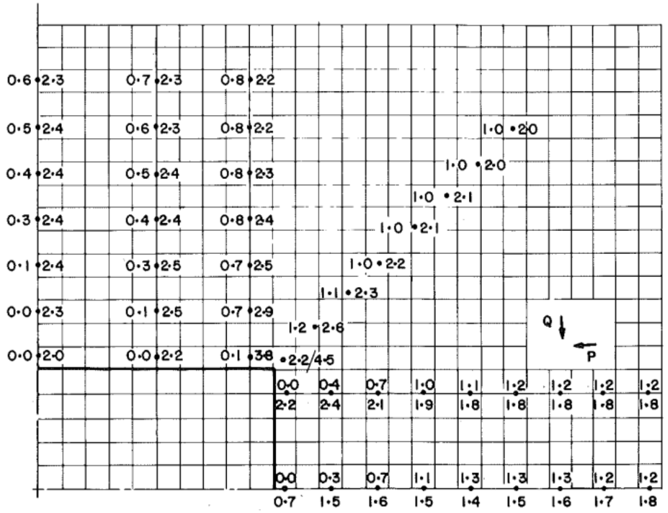


Fig. A22
Stresses around a 2:1 Rectangular Excavation — $P/Q = 2$

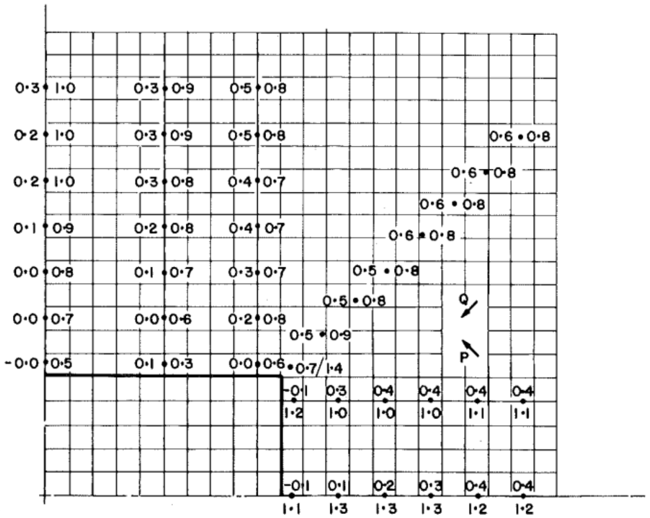


Fig. A23
Stresses around a 2:1 Rectangular Excavation Orientated at 45° to the Principal Stress Directions — $P/Q = 0.5$

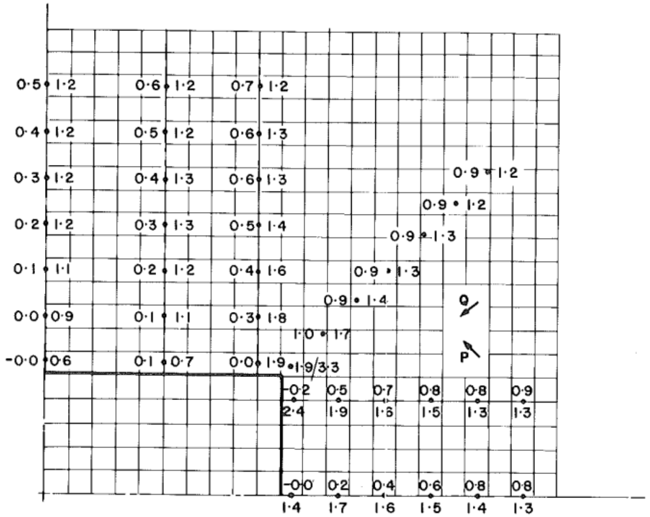


Fig. A24
Stresses around a 2:1 Rectangular Excavation Orientated at 45° to the Principal Stress Directions — $P/Q = 1$

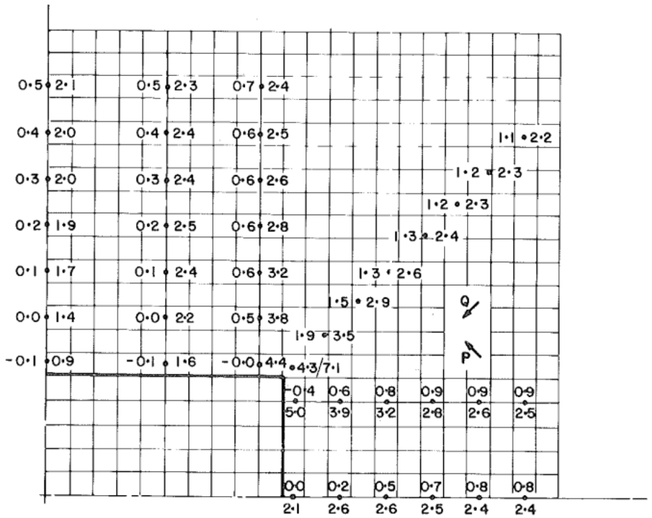


Fig. A25
Stresses around a 2:1 Rectangular Excavation Orientated at 45° to the Principal Stress Directions — $P/Q = 2$

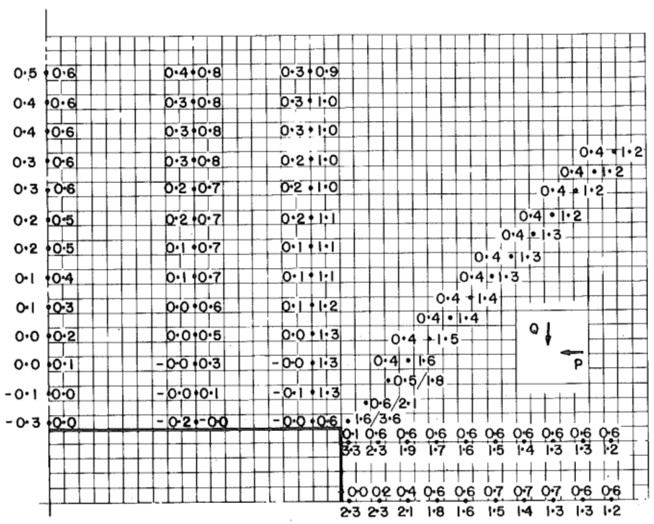


Fig. A26
Stresses around a 4:1 Rectangular Excavation — $P/Q = 0.5$

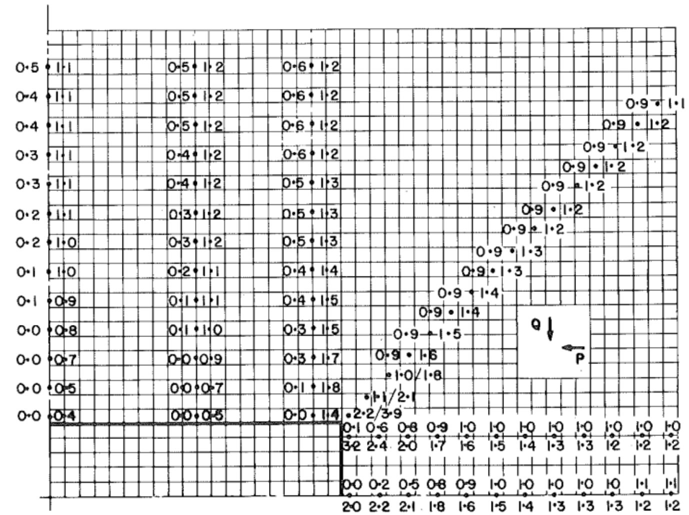


Fig. A27
Stresses around a 4:1 Rectangular Excavation — $P/Q = 1$

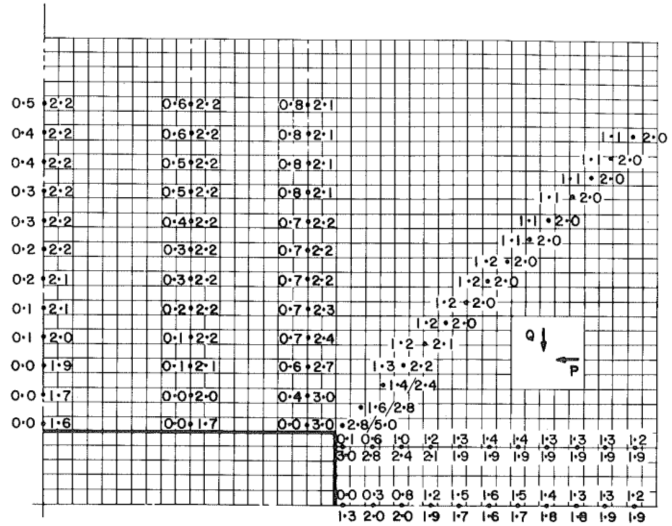


Fig. A28
Stresses around a 4:1 Rectangular Excavation — $P/Q = 2$

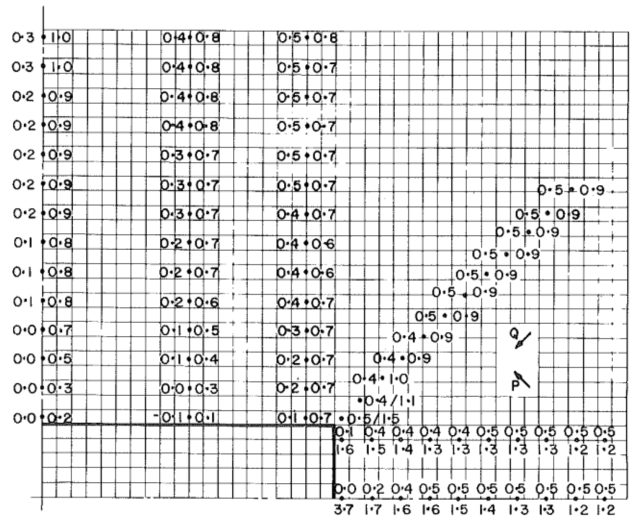


Fig. A29
Stresses around a 4:1 Rectangular Excavation Orientated at 45° to the Principal Stress Directions — $P/Q = 0.5$

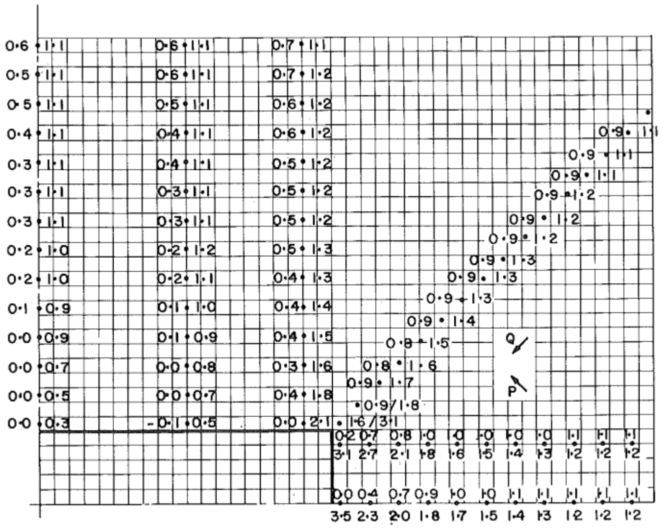


Fig. A30
Stresses around a 4:1 Rectangular Excavation Orientated at 45° to the Principal Stress Directions — $P/Q = 1$

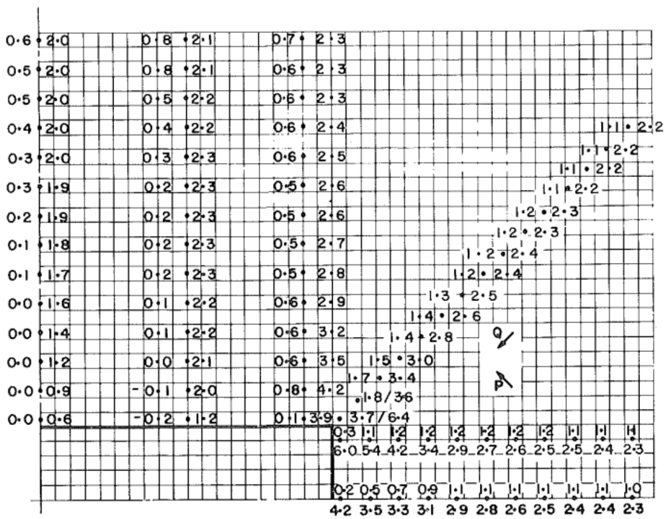


Fig. A31
Stresses around a 4:1 Rectangular Excavation Orientated at 45° to the Principal Stress Directions — $P/Q = 2$

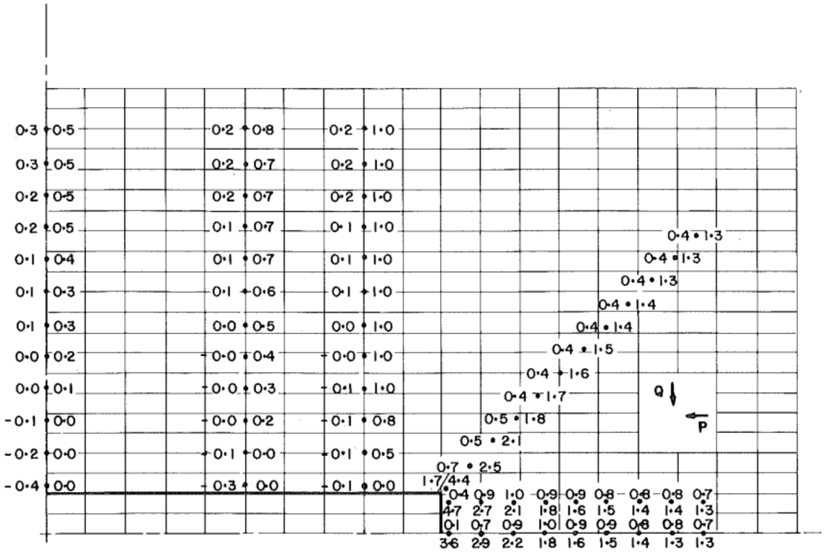


Fig. A32
Stresses around a 10:1 Rectangular Excavation — $P/Q = 0.5$

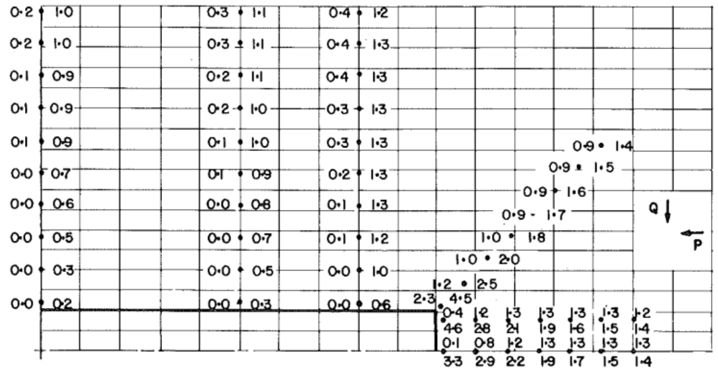


Fig. A33
Stresses around a 10:1 Rectangular Excavation — $P/Q = 1$

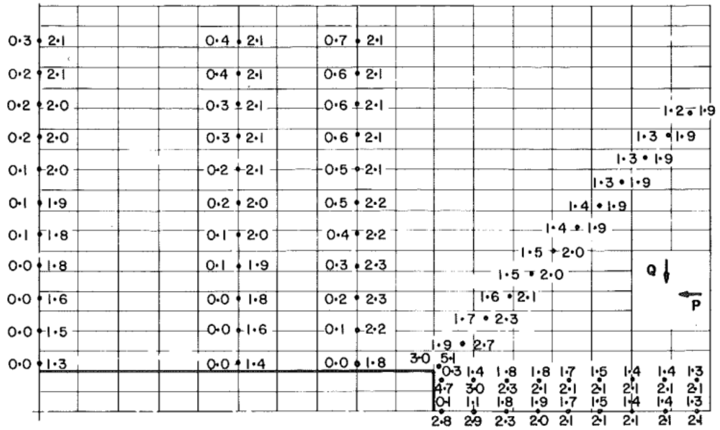


Fig. A34
Stresses around a 10:1 Rectangular Excavation — $P/Q = 2$

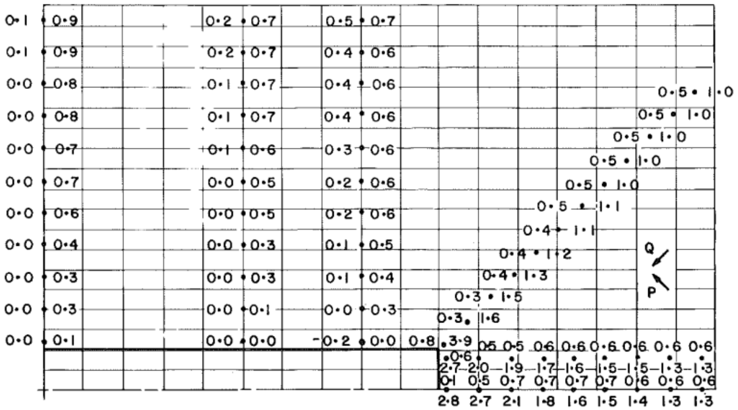


Fig. A35
Stresses around a 10:1 Rectangular Excavation Orientated at 45° to the Principal Stress Directions — $P/Q = 0.5$

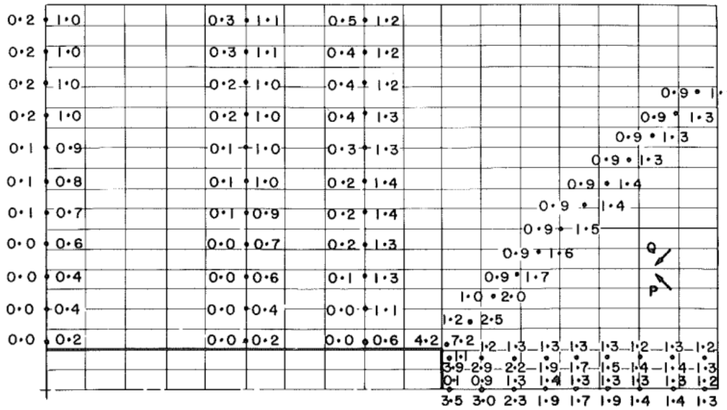


Fig. A36
Stresses around a 10:1 Rectangular Excavation Orientated at 45° to the Principal Stress Directions — $P/Q = 1$

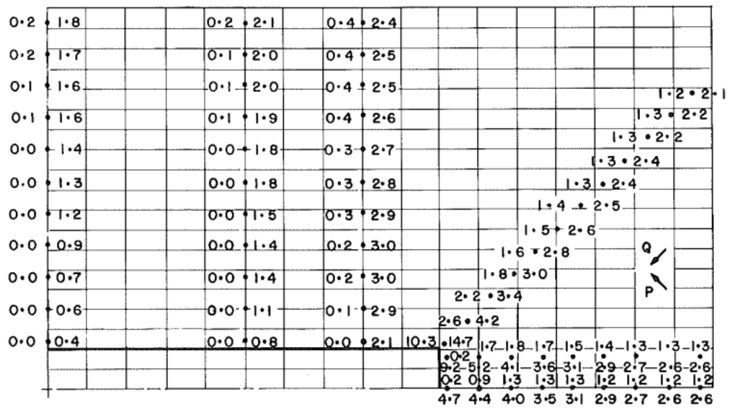


Fig. A37
Stresses around a 10:1 Rectangular Excavation Orientated at 45° to the Principal Stress Directions — $P/Q = 2$

A15

Rock Mass Strength

The methods of rock mass classification presented in Sections A10, A11 and A12 provide a quantitative means of describing rock mass quality. They can also be used as a means of estimating the strength of the rock mass. The following equation has been proposed[23] to define the strength of a rock mass:

$$\sigma_{1s} = \sigma_3 + \sqrt{m \cdot \sigma_3 \cdot \sigma_c + s \cdot \sigma_c^2}$$

where:

σ_{1s} is the strength (major principal stress σ_1, at failure)
σ_3 is the minor principal stress at failure
m, s are constants which depend on the properties of the rock and degree of jointing or fracturing. For intact rock $s = 1$.

It is possible to estimate the values of m and s from the results of rock mass classification[23] as shown in Figure A38.

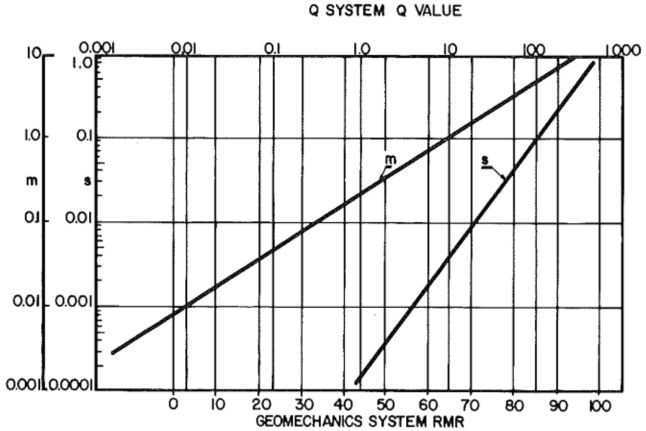


Fig. A38
Correlation between Q Value, RMR Value and m and s Rock Values

The values of m obtained from this graph are applicable to rocks such as mudstone, shale and siltstone. The following adjustments should be applied for other rock types as follows:

Limestone, dolomite	:	reduce m by 10%
Sandstone	:	increase m by 10%
Quartzite, dolerite, diabase	:	increase m by 20%
Gabbro, granite, gneiss	:	increase m by 30%

σ_c can be evaluated from Section A1.

The confining stress σ_3 can be estimated from the stress distribution diagrams in Section A14. The strength of the rock mass σ_{1s} can then be calculated from the above equation.

A16

Rock Mass Strength (Empirical Approach)

An alternative approach[27] is based on the Mining Rock Mass Classification system. A series of adjustments are made to the value of σ_c to downrate it to a design rock mass strength *DRMS*. The empirical adjustments applied in arriving at the value of rock mass strength have been developed out of a practical mining environment.

Values of rock material strength σ_c are obtained as described in Section A1.

For the rock mass σ_c is adjusted, using the set of curves in Figure A39 to take account of a mass containing strong and weak bands or zones.

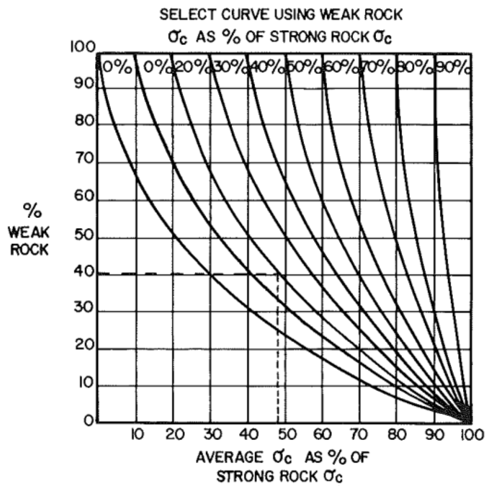


Fig. A39
Reduction of σ_c by Weak Bands (Redrawn after Laubscher[27])

The resulting average σ_c rating (see Table A15) is then applied as follows to give the rock mass strength, *RMS*.

$$RMS = \sigma_c \times \left(\frac{MRMR - \sigma_c \text{ rating}}{80} \right) \times 0.8$$

The factor 0.8 takes account of the fact that rock material strength decreases with increasing size of specimen.

A series of cumulative adjustments is now applied to the *RMS* to give the design rock mass strength *DRMS:*

Weathering: for rocks that deteriorate readily in a mining time span, adjust *RQD* rating by 95%, σ_c rating by 95% and joint condition rating by 82%, giving up to 75% adjustment to *RMS*.

Joint orientation: gravity is assumed to be the most significant force on rock blocks and the instability of blocks is assumed to depend on the number of joints inclined away from the vertical. Table A17 gives the adjustment percentages.

Table A17

No. of Joints Defining Block	No. of Faces Inclined away from Vertical and Adjustment Percentages				
	70%	75%	80%	85%	90%
3	3		2		
4	4	3		2	
5	5	4	3	2	
6	6		4	3	2 or 1

If shear zones or faults are present the following adjustment percentages are applicable:

Table A18

Dip of fault/shear zone	0-15°	15-45°	45-75°	75-90°
Adjustment	76%	84%	92%	100%

If the direction of advance is against the dip of such features, apply a further adjustment of 90%. Different joint orientation adjustments are applicable for pillars or sidewalls, as in Table A19.

Table A19

Joint condition rating	Plunge of joint intersection (degrees) and adjustment percentage		
0—5	10—30 = 85%	20—40 = 75%	>40 = 70%
5—10	10—20 = 90%	20—40 = 80%	>40 = 70%
10—15	20—30 = 85%	30—50 = 80%	>50 = 75%
15—20	30—40 = 90%	40—60 = 85%	>60 = 80/75%
20—30	30—50 = 90%	>50 = 85%	
30—40	40—60 = 95%	>60 = 90%	

A final adjustment to the *DRMS* is made to take into account the effects of *method of excavation* as summarised in Table A20.

Table A20

Excavation Technique	Adjustment
Boring	100%
Smooth wall blasting	97%
Good conventional blasting	94%
Poor conventional blasting	80%

A17

Rock Mass Deformability

The modulus of elasticity of a rock mass is usually considerably lower than the modulus of the rock material. Estimates of this modulus can be obtained in several ways, for example from the *RQD*[18]. However, we recommend that any estimate is based on rock mass classification.

Measured rock mass moduli have been correlated with the Geomechanics rock mass classification[10]. However, this relationship does not cater for poorer quality rock masses when $RMR < 50$. The same data have been inserted approximately on the graph in Figure A40 to cater for a wide range of rock mass quality. This practical suggestion should be accurate enough for preliminary purposes, and is the recommended means of assessing E_{mass}.

A considerable variation in the range of values of deformation modulus can be expected[3]. An indication of the dispersion is given by:

Mean $E_{mass} \approx 25 \log Q$
Min $E_{mass} \approx 10 \log Q$
Max $E_{mass} \approx 40 \log Q$

where Q is the Q system rock mass classification Q value.

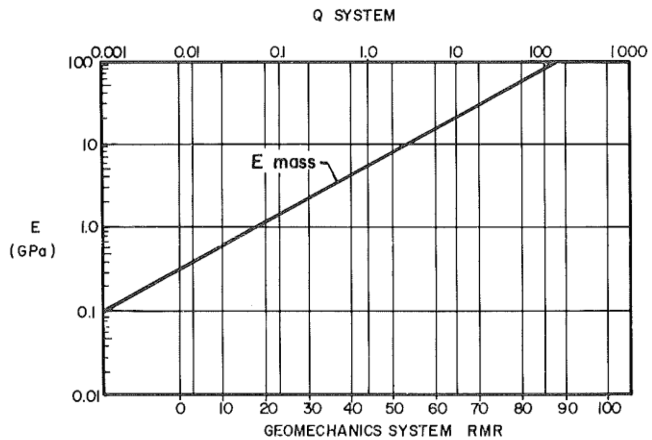


Fig. A40 Relationship between Rock Mass Classification and Rock Mass Deformation Modulus

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The list of publications we have referred to is by no means exhaustive and we apologise to readers if we have omitted references which they believe to be critical.

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Dr. T.R. (Dick) Stacey graduated initially in Mechanical Engineering, and followed this with a masters degree in the field of photoelasticity. His early exposure to rock mechanics was gained at the Council for Scientific and Industrial Research (CSIR), and it was for his work there on the stability of rock slopes, in particular involving stress analysis, that he was awarded his doctorate. After a year as an Academic Visitor in the Department of Engineering Geology of the Imperial College of Science and Technology, he moved into the field of consulting engineering. For the past ten years he has been with the international firm of geotechnical, mining and civil engineers Steffen, Robertson and Kirsten, in which he is now a Principal.

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Subject Index

Practical Handbook for Underground Rock Mechanics

This handbook is not a substitute for experience nor for detailed textbooks. Rather, it sets out to identify those procedures and methods which are appropriate to feasibility evaluations with very limited data.

This book is intended for use as an *everyday tool* — as a user manual rather than a summary textbook. It is assumed that the reader will have a general rock mechanics awareness, but the use of this handbook should not be limited to rock mechanics personnel. A civil contractor, for example, can make use of the handbook to assist in understanding and evaluating an excavation design; a mine manager may use it to prepare pertinent questions and to evaluate technical input to the everyday operation of his mine.

A major use of the handbook will be for *prefeasibility evaluations*. It is a common failing in many projects that exploration, investigation and design activities proceed too far before a practical evaluation is carried out. This handbook can be used prior to the acquisition of any detailed geotechnical data.

Dr. T.R. Stacey graduated in Mechanical Engineering and was awarded his doctorate in rock mechanics for work on the stability of rock slopes, in particular involving stress analysis, while employed by the CSIR. For the past ten years he has been with the international firm of geotechnical, mining and civil engineers Steffen, Robertson and Kirsten, in which he is a Principal.

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