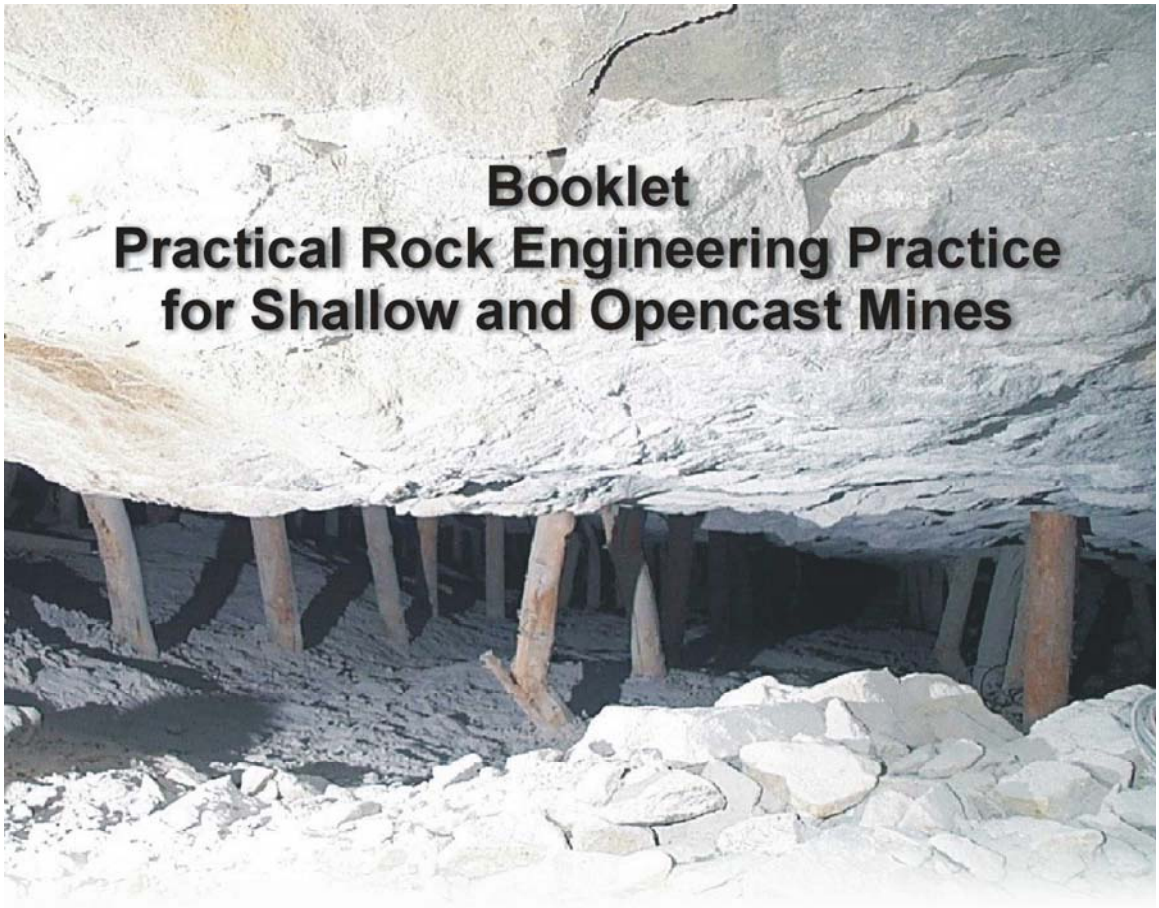

Booklet
Practical Rock Engineering Practice for Shallow and Opencast Mines

T.R.Stacey
School of Mining Engineering
University of the Witwatersrand
A.H.Swart
SRK Consulting

The Safety in Mines Research Advisory Committee (SIMRAC)
October 2001

MINE HEALTH AND SAFETY COUNCIL
SIMRAC

SRK Consulting
Engineers and Scientists

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CHAPTER 1

INTRODUCTION

Unsafe conditions result in mines when rock falls occur and/or when failure of excavations takes place. To ensure that safe openings can be excavated, mining personnel need the answers to the following questions:

- will the excavation I have planned be stable?
- what do I need to do' to its shape or design, or what support do I need, to make it sufficiently stable?

This booklet provides the means of evaluating stability and support requirements simply and quickly. Its scope of application includes "other mines" (both underground and surface). However, it will be applicable, at least in part, to gold, platinum and coal mines as well.

The booklet is the output of SIMRAC Project OTH602.When the application of the simple and quick methods given here are insufficient, reference should be made to the full report output for the project (available in www.simrac.co.za). Included with the full report is a simple computerised process for identifying rock related hazards. This process, and the evaluation of stability and support requirements, will serve as direct inputs to the Code of Practice to combat rockfalls. Pro-forma codes of practice, and a standardised system for auditing the implementation of a code of practice, are included with the full report.

The aim of this booklet is to provide a means of ensuring that sound rock engineering practices, which will minimise rock related accidents, are being followed. The booklet is deliberately brief, in the hope that it will be used by managers and others who have not specialised in the rock engineering field. It must be strongly emphasised that this should not be interpreted as replacing the requirement for input from qualified rock engineers. On the contrary, it should allow users to judge better when they need to call for specialist input.

The approach to excavation design and stability evaluation follows a straightforward path:

i) the purpose of the excavation determines its geometry and size, for example:

- development excavations such as haulages and crosscuts must accommodate vehicles and equipment, drawpoints must accommodate loaders etc;
- service excavations may be large to accommodate crushers, hoists, workshops etc:

- the mining extraction excavation geometry is dictated by the orebody shape and the chosen mining method.

ii) the practicality and stability of the excavation must then be evaluated in relation to the quality of the rock mass in which it is located:

- is it, or will it be, stable?
- what is the mode of identified instability, if any?
- can the instability be overcome by modifying the geometry and location of the excavation?
- what support, if any (quantity and type), is necessary to ensure that the desired stability is achieved?

The above process can be complicated in cases when instability is a requirement such as in caving. However, the approach remains unchanged - to cave, the opening must be larger than a critical dimension, and to be stable it must be smaller or be supported.

Appropriate support will depend on the risk associated with the excavation – when there is workforce access then stability is of the utmost importance. Conversely, when there is no workforce access, the important criterion may be the efficiency of the excavation process.

The booklet has been structured logically to cover firstly rock mass behaviour, followed by stability evaluation and then support considerations. Since rock mass behaviour is fundamental for the evaluation of stability of all types of excavations, rock mass characterisation is dealt with first. The recommended geotechnical data to be collected, the methods of collection, the recording of the data, the interpretation of the data, and the required outputs are all described. Rock mass characterisation provides the definition of geotechnical domains or control districts, which is an essential requirement in a Code of Practice to combat rock falls.

The evaluation of excavation stability, the rock engineering design of excavations, and the design of solid and artificial support, are dealt with separately for:

- tunnels and other horizontal operational excavations (eg drawpoints);
- service excavations, and vertical excavations (shafts, orepasses);
- underground mining extraction excavations (stopes), and
- surface mining excavations (pits, quarries).

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CHAPTER 2

ROCK MASS CHARACTERIZATION

The behaviour of excavations in rock will depend on the structure of the rock mass. Four conditions can be considered:

- massive rock condition, in which the structure in the rock mass has an insignificant effect on the excavation behaviour;
- major structural influence condition: features such as faults, dyke contacts and major joints provide surfaces on which deformation and failure may take place;
- jointed rock mass condition: well distributed jointing systems throughout the rock mass such that the behaviour of an excavation will be in response to the "overall" rock mass;
- weathered rock and soil.

Rock mass characterisation will determine which of these conditions is applicable, and which method of stability analysis or design is most appropriate. It is fundamental to the planning of any mining operation.

2.1 Collection of geotechnical data

2.1.1 Site investigations

To provide data for the satisfactory planning and mining of safe excavations, the sequence of investigations that would be required from pre-feasibility stage through to full production, is ideally as follows:

- study of available geological plans and similar material;
- remote sensing (satellite imagery);
- aerial photograph interpretation;
- specific field mapping;
- targeted exploration drilling, including specific geotechnical drilling, all based on the information obtained from the above investigations;
- evaluation and prediction of geological influences:
  - structural;
  - in situ and induced stresses;
  - groundwater;
  - quality and durability of the rock and rock mass;
  - control investigations during production, to identify conditions different from those on which the design was based.

The importance of a thorough understanding of the geological setting, and of the value of the early investigation stages in the above sequence cannot be over emphasised.

2.1.2 Geotechnical logging of borehole core

Exploration borehole core represents a source of geotechnical information. It will usually be necessary to re-log the core specifically for geotechnical purposes. For this purpose, a rapid geotechnical core logging technique[6] is recommended. The core is divided into separate geotechnical zones or design zones, within which the core displays similar geotechnical characteristics, and within which the rock mass is expected to perform uniformly in an excavation. The geotechnical core logging sheet in Figure 1 facilitates the activity.

The orientation of the borehole in relation to the jointing needs to be considered carefully. For example, if joint sets have sub-vertical dips, a vertical borehole is unlikely to intersect many joints. This situation must be taken into account in the interpretation of the borehole data.

2.1.3 Mapping of exposed rock surfaces

If jointing in the rock mass is judged to be such that excavation behaviour will be dictated significantly by the joint orientations and other joint characteristics, then **specific mapping of the joint parameters** will be required. If, however, the behaviour will be of a homogeneously jointed rock mass, then **rock mass classification** mapping will be appropriate.

To determine the potential for the formation of blocks and wedges (the geometric possibility of occurrence thereof), it is necessary to know the following parameters for each joint set:

- the orientation (dip and dip direction);
- the joint spacing;
- the joint length.

The recommended approach for rock mass classification mapping is to use a standardised rock mass description sheet, an example of which is shown in Figure 2. A sheet such as this, used during field mapping, acts both as a check list on the information to be collected as well as a physical data sheet.

2.1.4 Laboratory rock testing

The purpose of rock sample testing is to extend the data available from descriptions and index tests by providing real data on specific properties of the rock. The aim is to "tie down" the characterisation of the rock sufficiently, and the extent of the testing should accordingly be limited to the amount necessary.

2.2 In situ stress conditions

In situ stresses determine the confinement imposed on the rock mass. This can have three effects:

ORGANISATION : **PROJECT :** **HOLE NO :** **Page: of**

GEOTECHNICAL BOREHOLE LOG **SITE :** **INCLINATION :**

PROJECT No. : **FINAL DEPTH :** **Date:**

DRILLED BY :

Drilling Interval	Recovery			RQD		Geotechnical Interval	Rock Type	Rock Competence			Weathering 1-5	Hardness 1-5	Jointing Distribution				Joint Surface Condition			
	m Drill	m Rec	% Rec	m	%			Solid m	Matrix m	Matrix Type			0º-30º	30º-60º	60º-90º	Total	Micro 1-5	Macro 1-5	Infilling Type	Wall Alter 1-3

MATRIX TYPE
M1 Fault
M2 Shears
M3 Intense fracturing
M4 Intense mineralisation
M5 Deformable material

WEATHERING
1. Unweathered
2. Slightly
3. Moderately
4. Highly
5. Completely

HARDNESS
1. Very soft
2. Soft
3. Hard
4. Very hard
5. Extremely hard

JOINT SURFACE
MICRO ROUGHNESS
1. Polished
2. Smooth planar
3. Rough planar
4. Slickensided undultating
5. Smooth undulating
6. Rough undulating
7. Slickensided Steppped
8. Smooth stepped
9. Rough stepped/irregular

MACRO ROUGHNESS
1. Planar
2. Undulating
3. Curved
4. Irregular
5. Multi irregular

INFILLING TYPE
1. Gouge thickness > Amplitude of Irregularities
2. Gouge thickness < Amplitude of Irregularities
3. Soft Sheared Material - Fine
4. Soft-Sheared Material - Medium
5. Soft-Sheared Material - Coarse
6. Non-Softening Material- Medium
7. Non-Softening Material - Coarse

JOINT WALL ALTER
1. Wall > Rock Hard
2. Wall > Rock Hard
3. Wall < Rock Hard

Figure 1 Geotechnical core logging sheet

ROCK MASS DESCRIPTION AND CLASSIFICATION SHEET

DATE **MAPPED BY** **SITE LOCATION**

ROCK TYPE

WEATHERING

INTACT MATERIAL STRENGTH

soil_____very soft rock_____
soft rock_____hard rock_____
very hard rock___extremely hard_____

GROUND WATER

BLASTING EFFECTS

CLASSIFICATIONS
Q = RQD/Jn x Jr/Ja x Jw/SRF

Bieniawski RMR = UCS+RQD+spacing+cond+water+orien

Adjusted MRMR = (UCS+ FF + cond(water)) x adj

ROCK MASS DISCONTINUITIES

Joint set Number	Type	Orientation		Joint spacing				Joint condition				dip Continuity strike			
		Dip	Dip dir	Min	Max	Average	No /m	Large	Small	Infill	Alt	Length	Ends	Length	Ends

Jc =

RQD equivalent
(RQD = 115 - 3.3 Jc where Jc = total number of joints/ set/ metre)

COMMENTS

RELATIVE JOINTING ORIENTATIONS

N

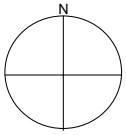

Figure 2 Rock mass classification field mapping sheet

- if the in situ stress is low, instability may occur since rock blocks may have the freedom to fall out;
- if the in situ stress is higher, the rock mass will be well confined and stability will be achieved;
- if the in situ stress level is sufficiently high (say > 15% of the UCS), it may induce fracture and failure of the rock and rock mass, and again instability will result.

A database of in situ stress measurements carried out in Southern Africa has recently been compiled[27] and relevant information is summarised in Figures 3 to 5.

2.3 Rock mass classification

A rock mass is generally weaker and more deformable 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.

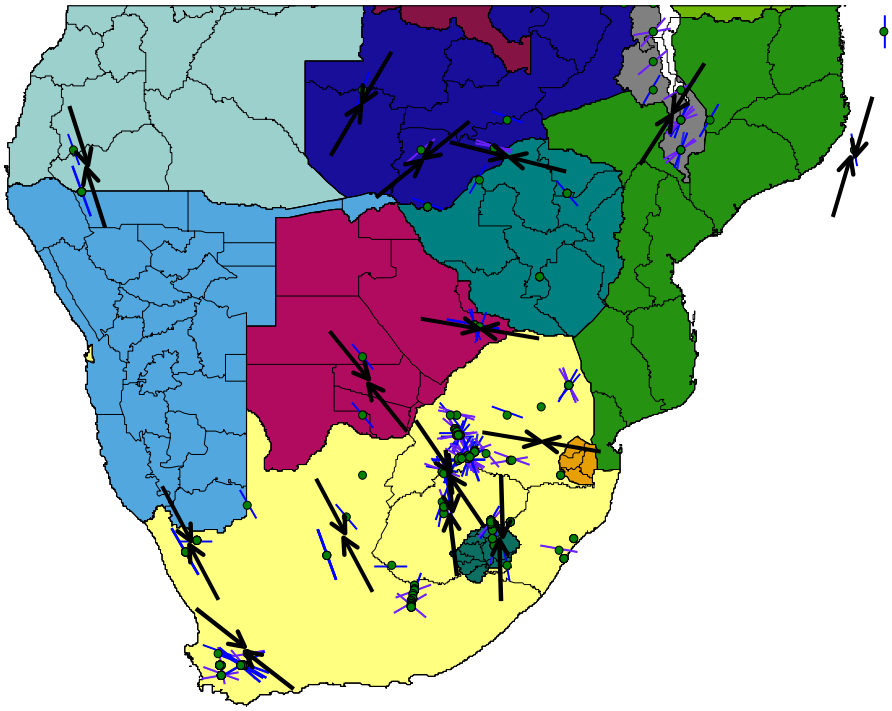

Figure 3 **Orientation of horizontal in situ stresses[27]**

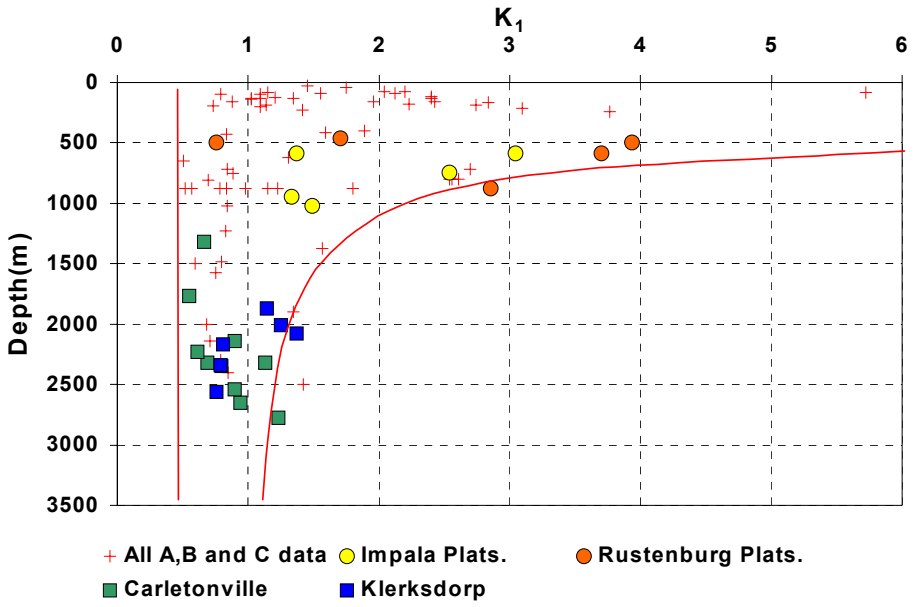

Figure 4 **Ratio K_1 of major horizontal in situ stress to vertical stress[27]**

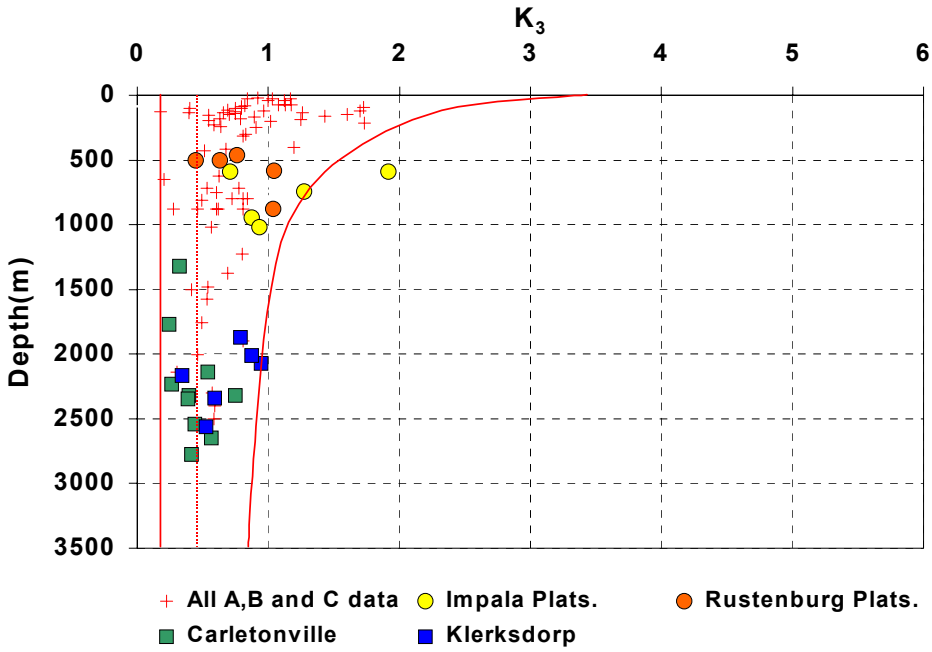

Figure 5 Ratio K_3 of minor horizontal in situ stress to vertical stress[27]

Quantitative classification of rock masses has become almost routine in rock engineering, since it provides a rapid means of quantifying the quality of a mass, comparing qualities, and assessing support requirements.

Two classification methods have stood out, the *Q* System[1] and the Geomechanics Classification System[2]. A system specifically for mining applications, the MRMR system[17], was based initially on the Geomechanics Classification System, but is now independent. These three systems are used in the booklet. The output of each of the classification methods is a number which represents the quality of the rock mass. These three values, which are used in this booklet are:

- the rock mass rating *RMR*[2]*;*
- the *Q* value[1], and
- the mining rock mass rating, *MRMR*[17].

The three classification methods will not be described, but instead a short cut approach is given to enable very quick estimates of classification values to be obtained - Figure 6 can be used to estimate a value of *RMR* based on visual assessment of the rock mass. This value must be adjusted for the favourability of the strike and dip orientations of joints encountered in the rock mass being classified, using the following tables (adjustments to *RMR* are in bold).

Adjustment for strike normal to excavation axis

Drive with dip		**Drive against dip**	
Dip 45° to 90°	Dip 20° to 45°	Dip 45° to 90°	Dip 20° to 45°
0	**-2**	**-5**	**-10**

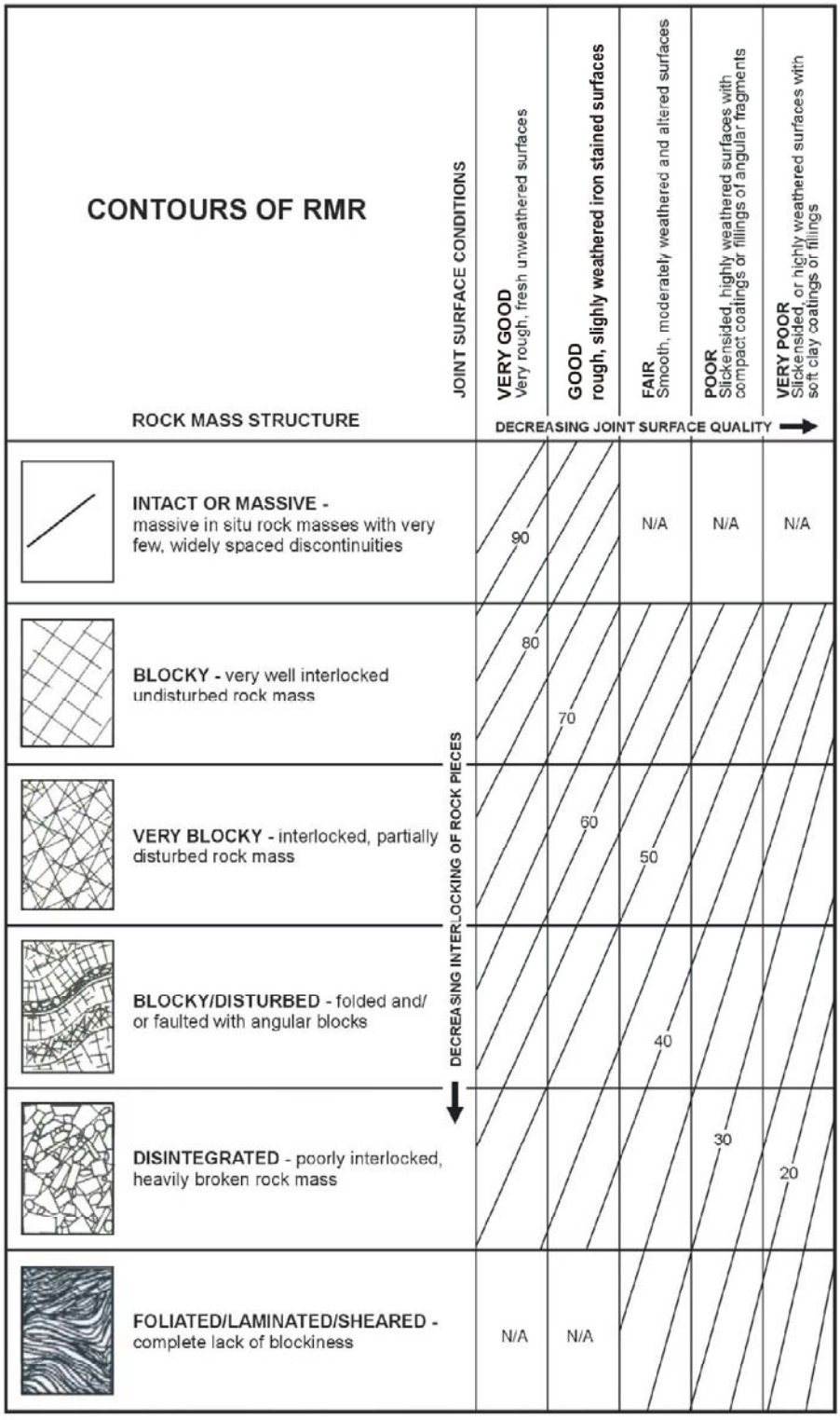

Figure 6 **Estimation of *RMR* from visual assessment of the rock mass (modified[10])**

Adjustments for strike parallel to excavation axis

Dip 45° to 90°	Dip 20° to 45°
-12	**-5**

An adjustment of **-5** is applicable when the dip is less than 20°, irrespective of the strike.

A *Q* value can then be calculated from the correlation equation between *RMR* and *Q*:

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

For guideline purposes, typical values of *RMR* and *Q* for rock masses occurring commonly in some of the South African diamond and base metal mines are given in Table 1.

Table 1 Typical *RMR* values for some Southern African rock masses

Rock type	*RMR*	*Q*
Amphibolite	55 – 67	3.4 – 12.9
Banded Ironstone	55 – 75	3.4 – 31.3
Chromitite	47 – 54	1.4 – 3.0
Dolerite	57 – 63	4.2 – 8.3
Dolomite	68 – 71	14.4 – 20.1
Granite/gneiss	60 – 84	5.9 – 85.2
Kimberlite	35 – 55	0.37 – 3.4
Massive sulphide	63 – 74	8.3 – 28.0
Metagabbro	50 – 66	1.9 – 11.5
Pyroxenite	53 – 80	2.7 – 54.6
Quartzite	58 – 80	4.7 – 54.6
Sandstone	38 – 58	0.51 – 4.7
Schist	25 – 55	0.12 – 3.4
Shale	55 – 69	3.4 – 16.1
Ventersdorp lava	55 – 72	3.4 – 22.4

The above adjustments are not applicable in determining an *MRMR* equivalent, which will be between 5 and 10 points lower than the *RMR* determined from Figure 6.

Table 2 Adjustment to *MRMR* for weathering

Rate of weathering and adjustments (%)					
Description of weathering extent	**6 months**	**1 year**	**2 years**	**3 years**	**4 + years**
Fresh	100	100	100	100	100
Slightly	88	90	92	94	96
Moderately	82	84	86	88	90
Highly	70	72	74	76	78
Completely	54	56	58	60	62
Residual soil	30	32	34	36	38

Table 3 Adjustment to *MRMR* due to joint orientation

Number of joints defining the block	**Adjustment (%) Number of faces inclined away from the vertical**				
	70	75	80	85	90
3	3	-	2	-	-
4	4	3	-	2	-
5	5	4	3	2	1
6	6	5	4	3	2 or 1

Table 4 Adjustment to *MRMR* for blasting effects

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

Adjustments to the *MRMR* are made to take into account the effects of weathering, joint orientation and blasting using the values in Tables 2, 3 and 4 (note that, to avoid confusion with the Geomechanics Classification System[2] *RMR, MRMR* is used in this booklet. The *adjusted MRMR* referred to in the following corresponds with the Mining Rock Mass Rating System[17] *MRMR).* In addition, the *MRMR* is adjusted to account for the confining stress - good confinement enhances stability, and the maximum positive adjustment is 120%. Poor confinement, associated with numerous, closely spaced joint sets, does not promote stability, and the maximum negative adjustment is 60%. These adjustments are cumulative, being applied as multipliers to the *MRMR.* For example, if the *MRMR* value was 72, after adjustments of 82% for weathering,

85% for joint orientation, 110% for mining induced stresses, and 80% for blasting effects, the adjusted *MRMR* is:

Adjusted *MRMR* = 72 x 0.82 x 0.85 x 1.1 x 0.8 = 44

In spite of the suggested use of a shortcut method in this booklet, the recommended approach in critical situations is that the rock mass classification procedures are carried out in detail. This will ensure that factors which will affect the behaviour of the rock mass are not overlooked. It is also recommended that more than one rock mass classification method is used to give added confidence to the interpretation of conditions.

CHAPTER 3

STABILITY

In this Chapter, the evaluation of stability of both underground and open excavations will be dealt with in turn.

3.1 Evaluation of the stability of underground excavations

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 excavations, benches) and important mining excavations (shafts, ramps) 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 for 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;
- failure of "beams" as a special case of the above. This could be either footwall or hangingwall, or both, depending on the dip.

3.1.1 Evaluation of failure of intact rock material and rock mass

Test for stress induced failure around the opening. In many cases this will be a formality requiring nothing more than judgement and experience. This type of failure refers to cases in which the failure takes place substantially through intact rock. Failure through intact rock is expected to be uncommon on most of the mines for which this booklet is appropriate. This is due to the relatively shallow mining depth, and to the strength of the intact rocks encountered in most mines in South Africa.

3.1.2 Evaluation of structural instability

Rock mass characterisation will have defined the orientations of planes of weakness. These may be major planes such as faults and dyke contacts, or joint sets. Combinations of these planes may define blocks or wedges which may be unstable. A simple computer program BlockEval, on the CD included with this booklet, can be used to determine whether these blocks and wedges are unstable or not. The use of this program is as follows:

- visualise one or more geometries of blocks that might occur;
- the stability of five different block shapes can be analysed, with from 3 to 5 planes of weakness being taken into account. These alternative block shapes are shown in Figure 7, which is a plot of one of the windows from the program;
- click on the appropriate block type, and a second window will appear, as shown in Figure 8;
- input data should be entered as indicated on the window, and clicking on "calculate" produces the result.

This analysis is conservative since it only considers the geometrical "removability" of the block, and assumes that the weakness planes are cohesionless, but have a friction angle of 30°.

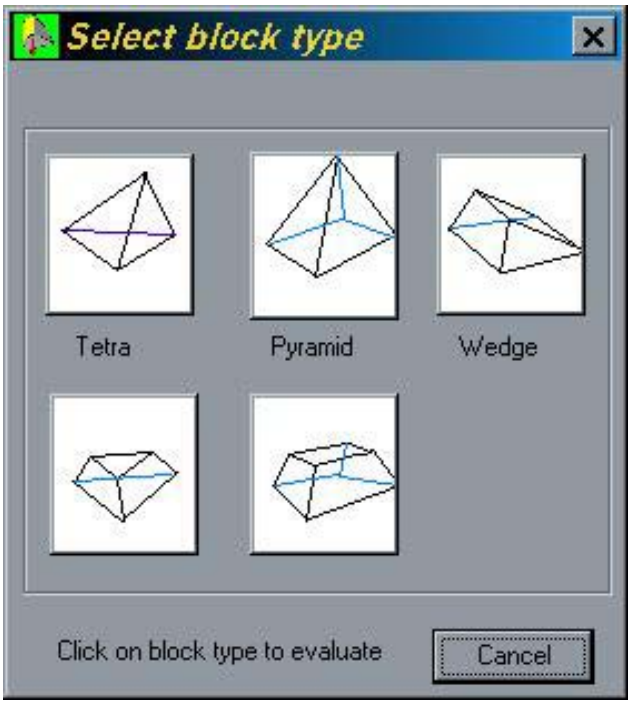

Figure 7 Block shapes catered for in BlockEval

3.1.3 Evaluation of the stability of rock beams

In some rock masses the geological structure may be such that significant "flat" spans may be formed. This is the case in stratified and pseudo-stratified deposits such as occur in the chrome and manganese mines. Hangingwalls and footwalls of mining excavations may be defined by this structure. Depending on the dip of the strata, it may be necessary to evaluate the stability of both hangingwalls and footwalls. Stability can be evaluated using the graphs in Figures 9 and 10. In these graphs, strong rock refers to a rock such as pyroxenite, and weak rock refers to a rock such as mudstone. Dips shallower than 30° fall into the flat category and those greater than 60° into the steep category. Interpolation should be used for intermediate rock strengths and inclinations. It must be emphasized that the stability of beams is

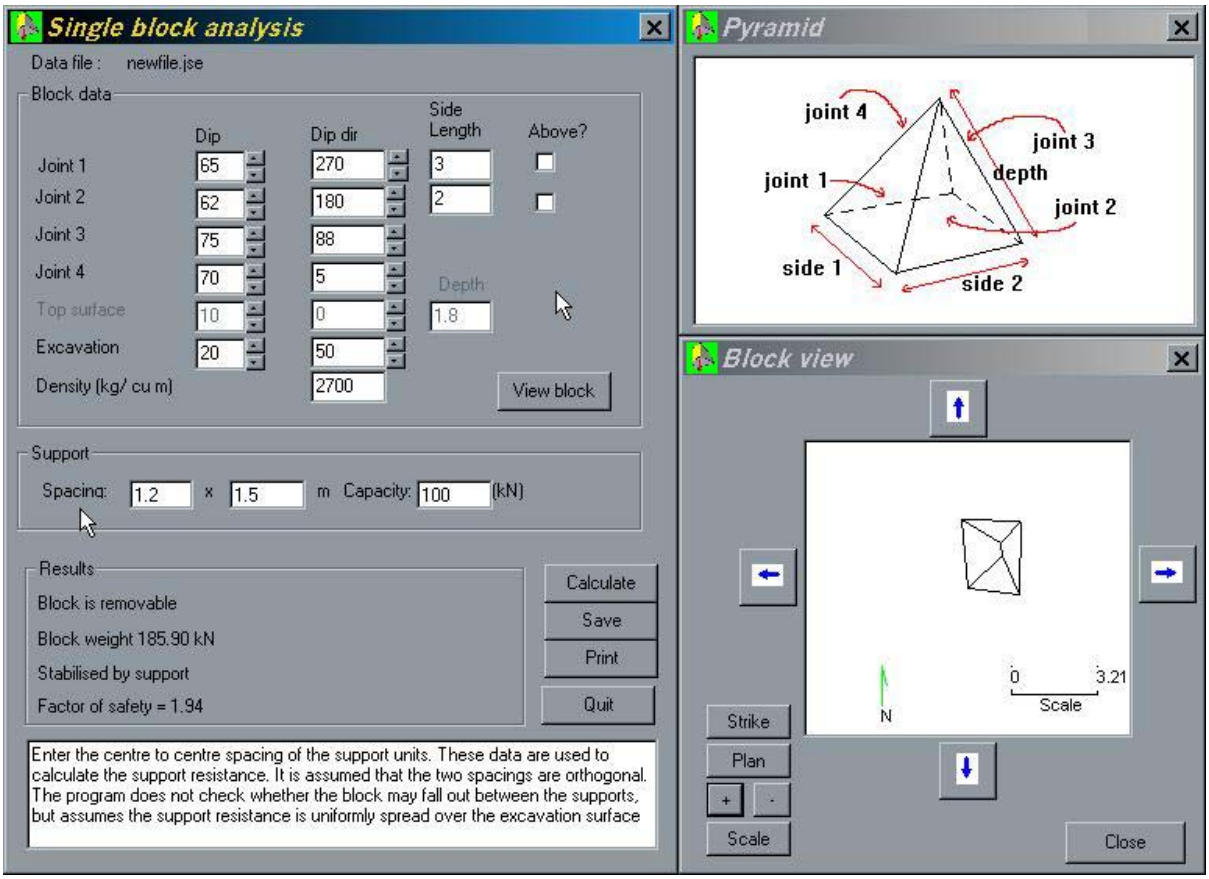

Figure 8 BlockEval windows for input data, output results and block visualization

critically dependent on the continuity of the beam. The graphs in Figures 9 and 10 provide guideline values regarding stability - if they indicate that stability might be marginal, then more comprehensive analyses should be carried out. If beams contain any inclined joints, then Figures 9 and 10 will tend to overestimate stable spans and they should only be used with great caution.

3.1.4 Use of rock mass classification approaches for the evaluation of rock mass stability for specific classes of excavations

In the following sections, excavations that fall into different classes are dealt with separately. For example, haulages and drill drives are both tunnels, but haulages are long term excavations whereas drill drives may have a very short life. The two types of tunnels therefore have different requirements regarding stability.

Long term service excavations

Long term service excavations refer to those mining excavations whose life is not limited to the temporary life of an operating component such as a stope or a boxhole. Tunnels, shafts, orepasses, and the larger excavations such as pump chambers, crusher stations etc, fall into the category of long term service excavations. Since these excavations must be stable for long periods of time, their stability must be conservatively assessed. In this they are some what equivalent to civil engineering

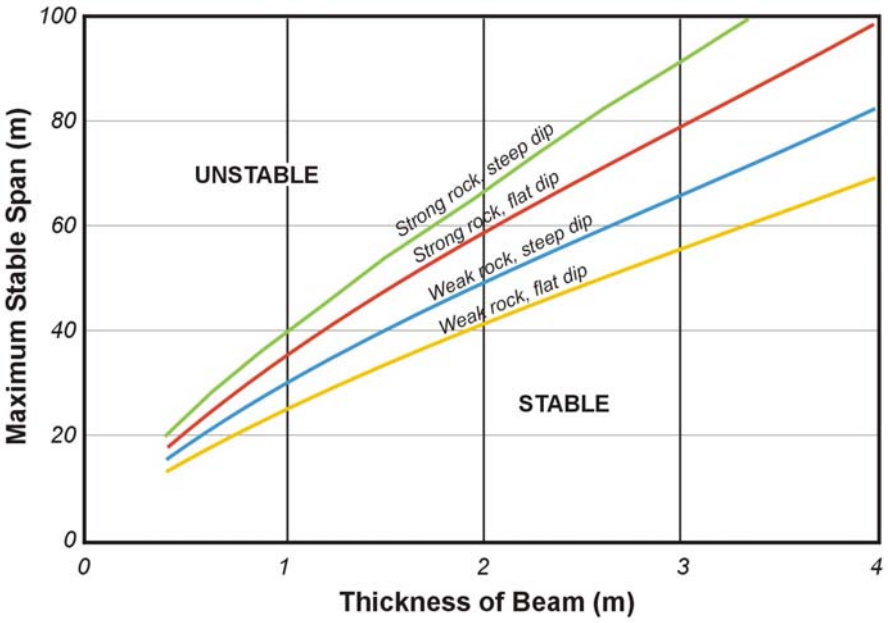

Figure 9 Stability of beams - long excavation (modified[13])

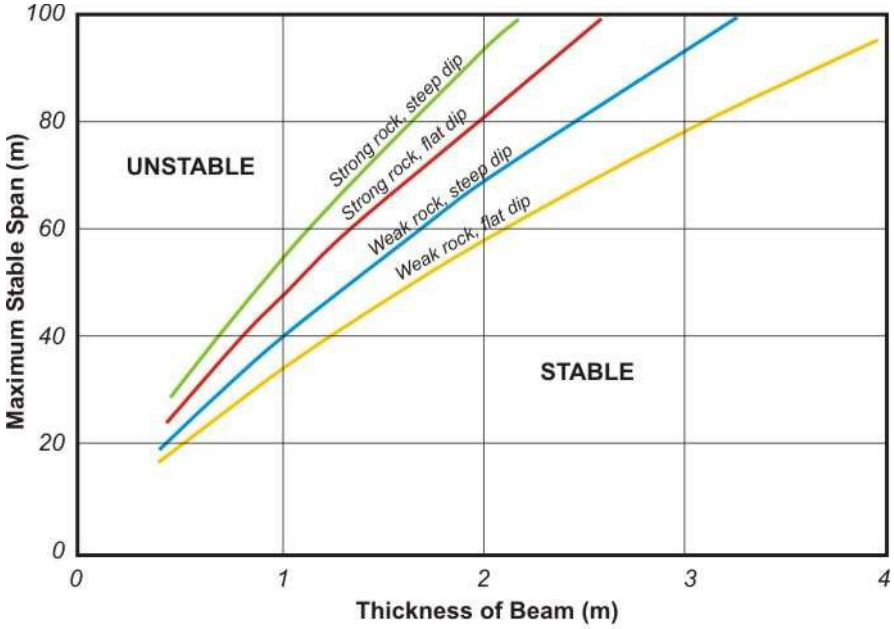

Figure 10 Stability of beams - square excavation (modified[13])

excavations. Since orepasses are subjected to on-going mechanical impacts and abrasion, they will be dealt with separately below.

The *Q* System[1] and the *RMR* System[2] are applicable. These methods have both been developed from a civil engineering base and are therefore inherently conservative. The unsupported spans of long term service excavations can be determined from Figures 11 and 12.

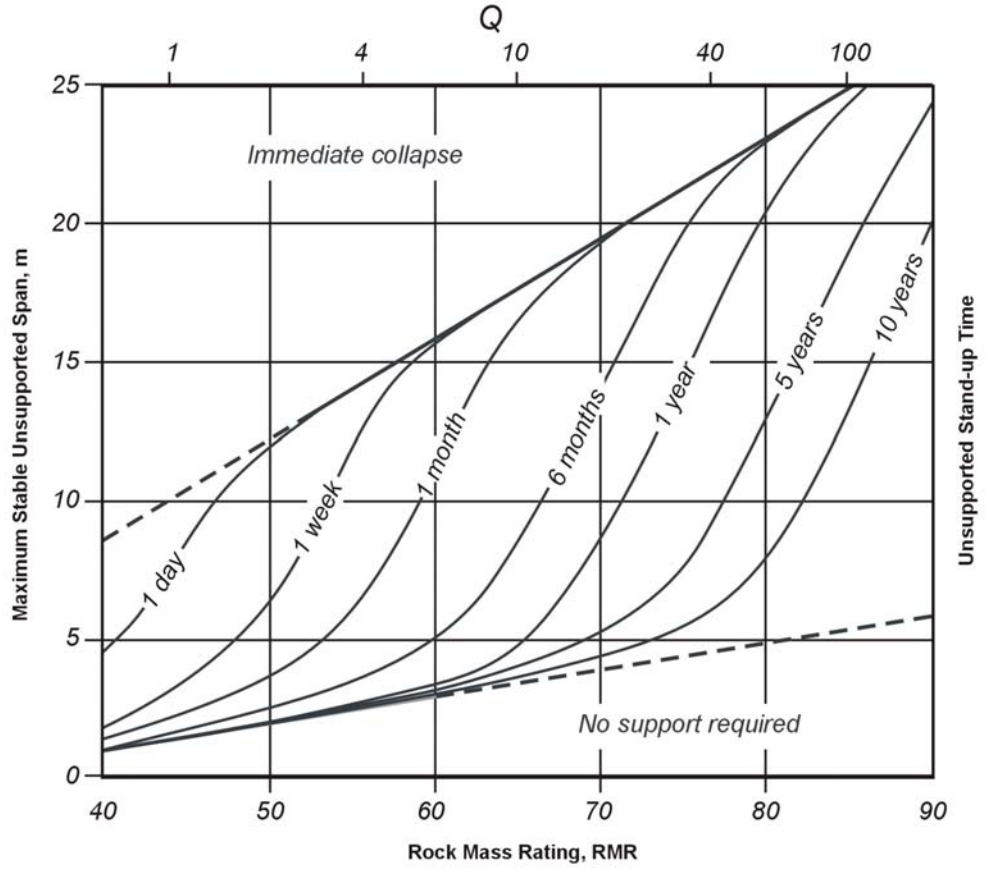

Figure 11 Unsupported span vs *RMR* relationship[13]

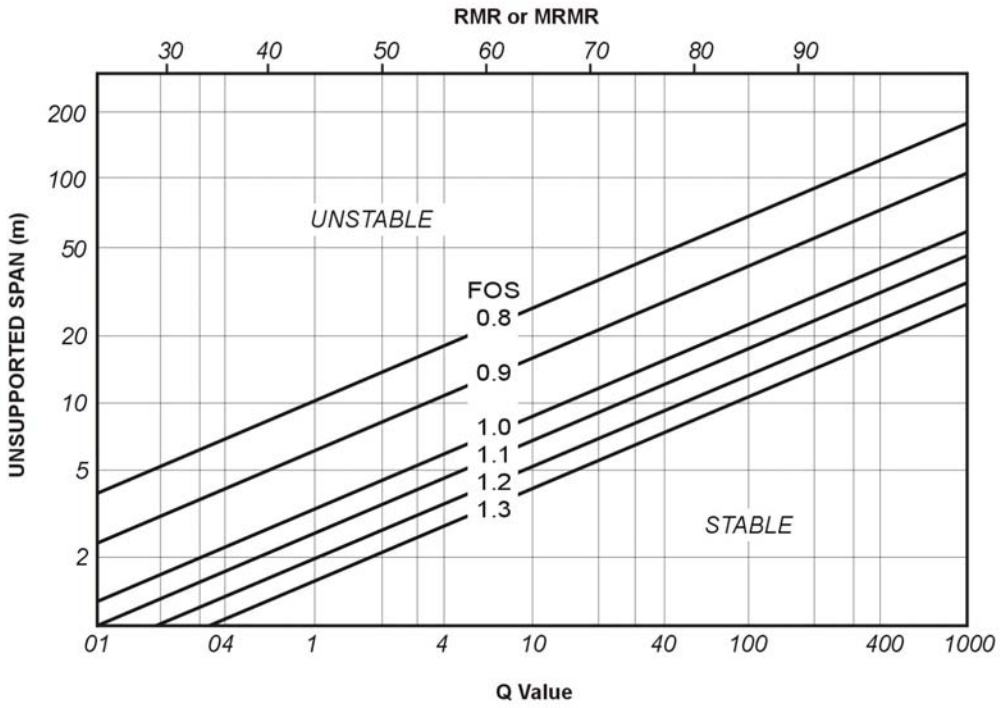

Figure 12 Unsupported span vs *Q* relationship[12]

Figure 11 is based[13] on the *RMR* system and Figure 12[12] is an adaptation of the *Q* System "no support" curve, with suggested factors of safety as additions. Comparing these two figures, for example, for a 4m span and a *RMR* of 65 (corresponding *Q* of 10), a 5 year stand up time corresponds with a factor of safety of about 1.3. If the rock mass quality and excavation span required plot within the factor of safety < 1.3 or the stand up time < 5 years, then support of the excavation needs to be considered. From experience with the application of Figure 12, caving or collapse only occurs when the factor of safety is about 0.7 to 0.8, which corresponds with a span of about 25m. The use of Figure 11 will result in more conservative results.

In cases in which stress induced failure of rock does not occur, vertical walls of excavations, and vertical excavations such as shafts, are usually more stable than the spans of excavations. This is not considered directly in the *RMR* System, but is taken into account in the *Q* System by assuming that Q_{wall} = 5 x Q_{roof} when Q > 10, that Q_{wall} = 2.5 x Q_{roof} when 0.1 < Q < 10, and that Q_{wall} = Q_{roof} when Q < 0.1. Thereafter, stability of the vertical surfaces can be evaluated in the same way as described above.

A method of assessing the geotechnical risk for large diameter raise bores[21] is used by contractors in South Africa, and widely used in the mining industry in Australia. The method is also applicable to the stability evaluation of conventionally excavated shafts. The method is based on the *Q* System, with adjustments applied to take account of orientation of joints and weathering. These adjustments are given in Tables 5 and 6 below.

Table 5 Adjustment for joint orientations

Face Orientation Adjustment				**Wall Orientation Adjustment**			
No of flat dipping (0^o-30^o) major joint sets	1	2	3	No of steep dipping (60^0-90^0) major joint sets	1	2	3
Adjustment	0.85	0.75	0.60	Adjustment	0.85	0.75	0.60

Table 6 Adjustment for weathering

Weathering Description	**Slight**	**Moderate**	**Severe**
Adjustment	0.9	0.75	0.5

A wall adjustment different from that indicated above for the general *Q* System is applied: Q_{wall}= 2.5 x Q when Q > I and Q_{wall}= Q when Q < 1.

The adjustments are cumulative as illustrated in the following example for a raisebore sidewall, with two steeply dipping joint sets:

$Q_{original}$ = 4.2
Wall adjustment = 2.5
Orientation adjustment = 0.75
Weathering adjustment = 0.75
Raisebore rock quality Q_R = 4.2 x 2.5 x 0.75 x 0.75 = 5.88.

The reliability of the raisebore can be assessed using Figure 13. A probability of failure of 5% (reliability of 95%) is considered to be acceptable. For a Q_R of 5.88 it can be seen that an acceptable raise diameter is about 5m. In assessing the stability of the **face** of a raisebore, a higher probability of failure is acceptable since the face of a raisebore is temporary.

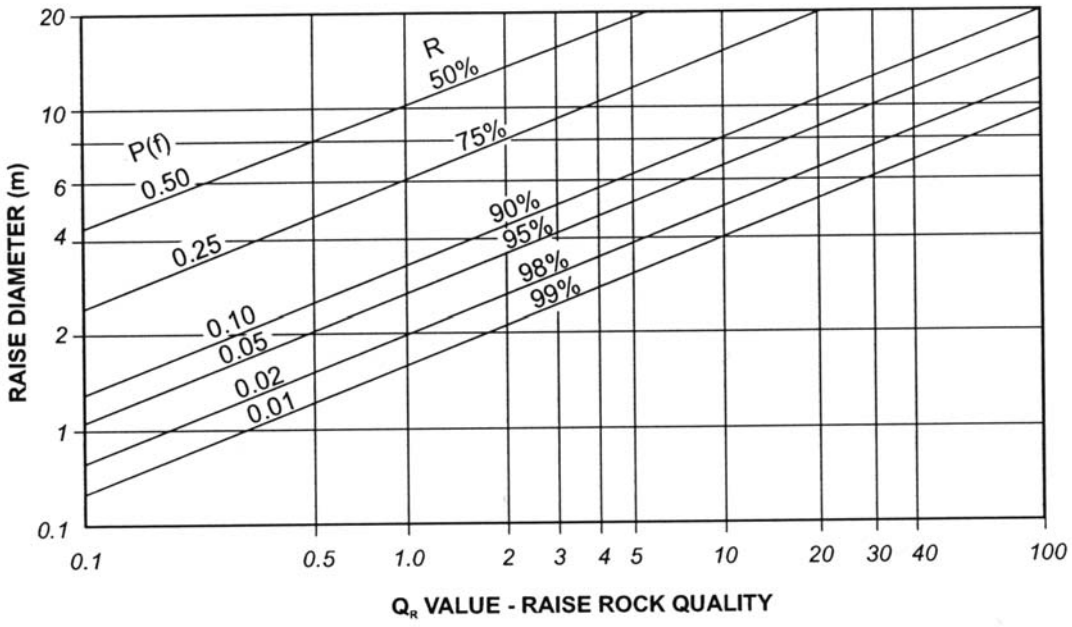

Figure 13 Raisebore reliability chart[21]

Production associated excavations

Production oriented excavations are those short term excavations whose life is limited to the time required for ore extraction at that location. They include, for example, drill drives, undercut drifts, drawpoints, boxholes and stopes.

The evaluation of stability of the **non-stope excavations** can be carried out using Figures 11 and 12. However, the shorter term stability requirement must be taken into account as well as changing stress conditions to which the excavation may be subjected over its life. The latter is particularly important in the case of production level drawpoints and drifts in block cave mining, since stress conditions vary from virgin during development, to abutment loading during undercutting, to relaxation and point loading during drawing, and finally to relaxation as the cave is exhausted. It is

most important that the stability of the excavation, and the corresponding support requirements, are determined for the worst conditions to be encountered over the life of the excavation. The aim must be to install support that will ensure stability for the full life of the excavation, not only during development.

Stability can be evaluated using Figure 12 with a factor of safety of 1.0. Note the value of *Q* is reduced to a third of its in situ value at intersections and half its value at portals. It may be reduced further owing to variations in the mining induced stress. At a block cave drawpoint, for example, which represents an intersection, the *Q* value could be many times less than it would be at the same location for a normal tunnel.

Figure 11 is increasingly conservative at lower values of *RMR,* and its use for production oriented excavations is not recommended.

Stability evaluations for **stope excavations** in more jointed rock masses can be based on rock mass classifications. Two approaches are suggested and it is recommended that both are applied in each case:

- the use of the adjusted *MRMR,* which has been correlated empirically with the hydraulic radius, as shown in Figure 14. The hydraulic radius is defined as the plan area of the stope excavation divided by its perimeter;
- the use of Figure 12, with a factor of safety of 1.0 or less, depending on exposure of personnel.

It is appropriate to mention the hazard of air blasts - the sudden collapse of the back or hangingwall of a stope, or significant parts of it, can cause air to be displaced at high velocity. Such air blasts can be extremely hazardous, even remote from the stope, when there is interconnection for air flow via tunnels, shafts and other openings.

Orepasses

Orepasses are special excavations that are a key component of a mine's production system. Failure or blockage of a pass can have a severe effect on production, and the clearing or rehabilitation of the pass can be a safety hazard. Evaluation of pass stability can be carried out as described above for shafts and tunnels, provided that a conservative approach is adopted to cater for the dynamic loading due to the rock being passed. Good planning and design will probably provide the greatest benefit to pass stability. As part of SIMRAC Project OTH303, a booklet was produced on design and operational aspects of orepasses, and important rock engineering related aspects are:

- passes should preferably be located in a strong rock mass to minimise instability;
- passes should be orientated as close to normal to the strata dip as possible, also to minimise instability;
- the size of passes should be at least 5 times the largest dimension of the largest rock blocks to be tipped, to avoid hang-ups. Sizes of branches and main passes should be compatible;
- the recommended pass inclination is between 55° and 70°, to allow free flow and to minimise impact damage and compaction of ore;

- the length of the pass should be minimised to avoid hang-ups and the difficulty of clearing hang-ups. Passes with leg lengths of less than 50m have rarely had problems;
- bends in passes should have an included angle of greater than 120°;
- water in passes should be avoided, since it can lead to compaction, sticky ore, hang-ups and hazards due to mudrushes.

Cavability

Caving methods of mining require that the back caves readily, and it is important in the planning of the operation that the cavability of the orebody and rock above it are predicted with reasonable accuracy. With the currently available knowledge, it is considered that cavability is best predicted using the empirical relationship in Figure 14. The breadth of the transitional zone in this graph illustrates the uncertainty regarding prediction. To ensure cavability, the planned hydraulic radius should therefore plot well into the designated caving area.

The graph in Figure 12 may also be useful for giving a quick indication of cavability, provided that a factor of safety of about 0.7 is used.

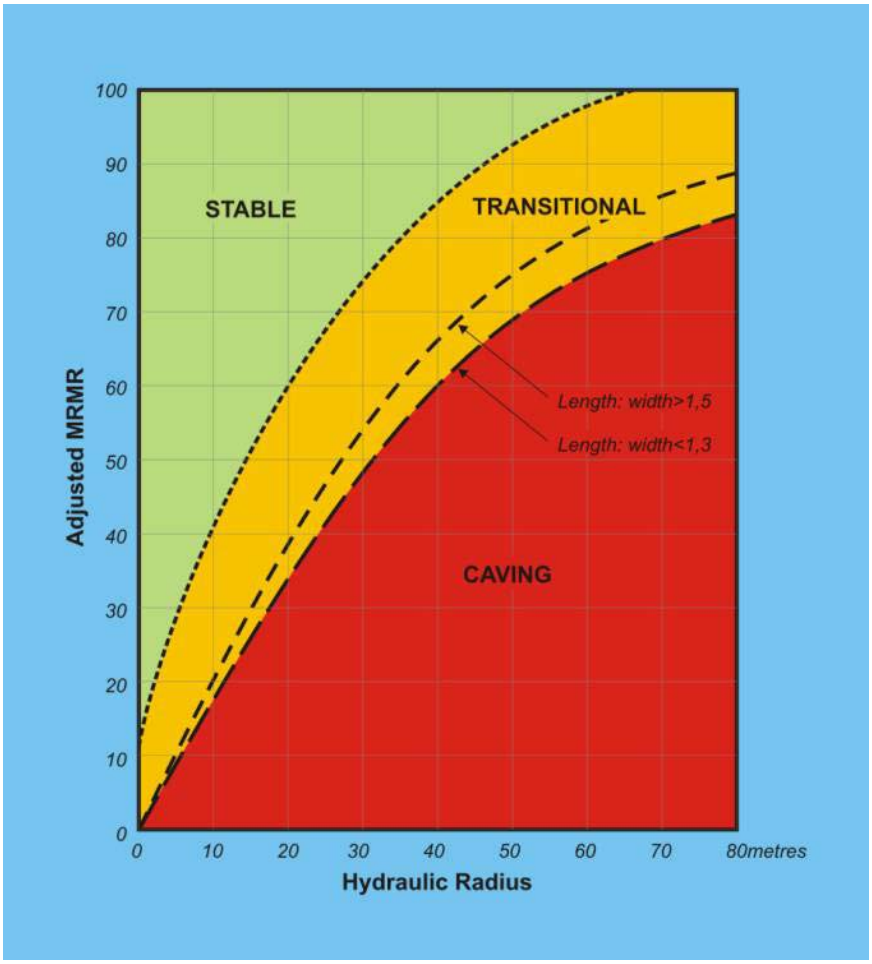

Figure 14 Stability diagram[19]

3.1.5 Improvement of stability by changes in geometry

If the evaluation of stability indicates that stability might be adverse, then it may be possible to avoid the instability problem by:

- altering the location of the excavation, if appropriate and if possible, so that it is placed in stronger rock and is hence more stable;
- altering the orientation of the excavation such that:
  - geometric instability of potential wedges and blocks is minimised, as illustrated in Figure 15;
  - the orientations of long sidewalls are not parallel or sub-parallel to faults, dykes or significant joints;
  - the shape of the excavation corresponds with the naturally stable shape, as illustrated in Figure 16;
- altering the geometry of the excavation - for example, if the span of the excavation is likely to be unstable, reduce the span and increase the height, if appropriate. The larger the size of an excavation, the less likely it is to be stable;
- altering geometry or orientation when stress induced failure is a possibility. The orientation, shape and size of the excavation have important controls on stability:
  - long excavations should be orientated parallel to the major stress if possible;
  - for intermediate stress situations, it may be possible to provide stability by optimising the stress distribution around the excavation (for example, by orientating the long axis of an elliptical excavation in the direction of the major stress);
  - the smaller the size of the excavation, the less likely it is that uncontrolled stress induced instability will occur.

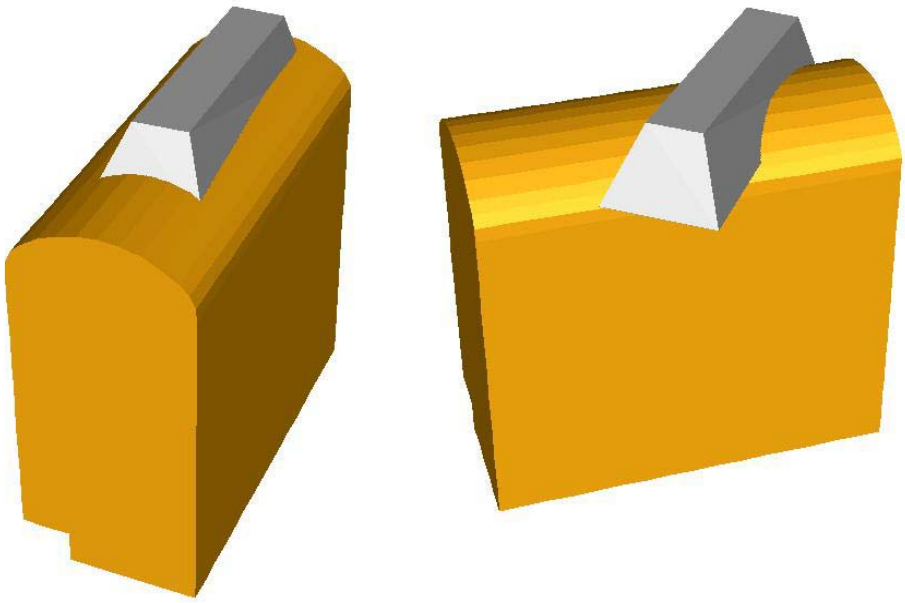

Figure 15 Minimising instability with respect to major geological structures

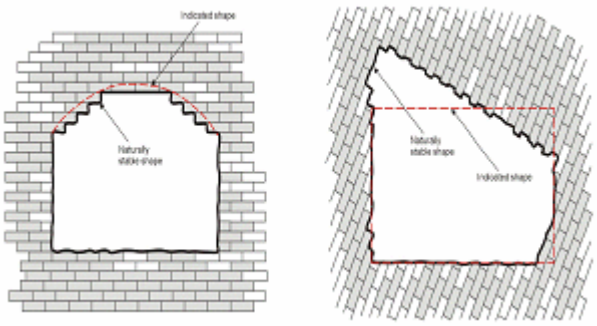

Figure 16 Minimising instability with respect to local geological structure

3.1.6 Surface subsidence and caving

Subsidence due to underground mining can take two forms:

- continuous subsidence resulting from regular closure of underground workings and competent or continuum hangingwall behaviour;
- discontinuous subsidence resulting from collapse of underground workings, failure of pillars or a combination of both. Intentional subsidence, such as occurs in caving mining methods, also falls into this category.

If continuous subsidence associated with metalliferous mining in South Africa does occur, it is probably imperceptible, and is therefore of little interest. In contrast, there are numerous examples of discontinuous subsidence, such as that which occurred at Bafokeng South Mine[15], in which the mine hospital was damaged; the ongoing subsidence in the former open pit diamond mines, now operating underground using caving methods; the hangingwall "block" collapse which occurred at Winterveld chrome mine; and the subsidence which occurs locally, 'associated with very shallow and outcrop chrome mining. The questions to be answered are:

- will subsidence occur and if so what will be its magnitude?
- what form will the subsidence take?
- when will it occur, how will it progress and what will be its extent or limit?

If surface damage cannot be tolerated, then the design of the underground workings and their support must be such that they will not induce any subsidence. This will require the evaluation of stability of stope excavations, and the design of these excavations to ensure that any instability which may occur will not affect the surface. Solid support or backfill may be required to prevent surface movements.

The standard restrictions on mining (or the restrictions on surface development) imposed by the Department of Minerals and Energy for seam type deposits are given in Figure 17. Implementation of these guidelines will give a conservative result.

If surface damage is acceptable, then it may be necessary to determine the potential for subsidence, and to predict the extent of the subsidence and its further development as mining progresses or with time. The methods of prediction for continuous subsidence are not applicable to the metalliferous mining which takes place in South Africa. There are no established methods of subsidence prediction for this type of mining, and it is necessary to make use of several different approaches.

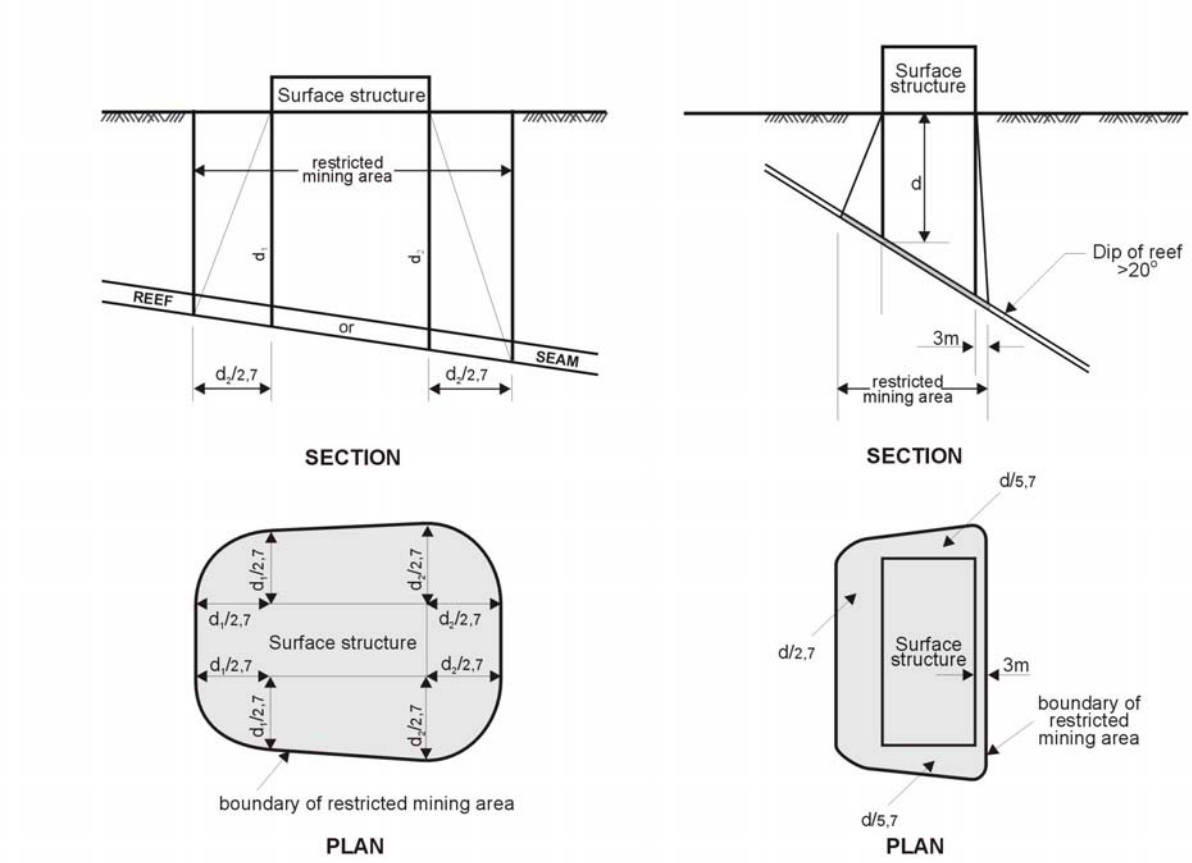

Figure 17 Restrictions on undermining of structures

Empirical prediction based on previous experiences

A mine that has records of subsidence and cave crack occurrence, correlated with mining geometry and time, will be in a very good position to use this information for prediction of future subsidence behaviour. The behaviour of the rock mass above a caving operation is very dependent on the competence of the rock mass. Geological structure does not appear to have a strong influence unless the orientations of major features are close to those of the cave cracks. Minimum cave crack angles observed in a massive hard rock environment are about 50° to the horizontal, but the angles can be much greater than this. When geological structure has an influence, they can be lower, however.

Empirical prediction based on rock mass classification

Adjusted *MRMR* values[18] have been correlated with estimated cave angles for caving situations, as given in Table 7.

Table 7 Cave angles corresponding with rock mass classification[18]

Condition	Depth (m)	Adjusted *MRMR*				
		90	70	50	30	10
No lateral restraint	100	80	70	60	50	40
	500	70	60	50	40	30
Lateral restraint from caved material	100	90	80	70	60	50
	500	80	70	60	50	40

The values in the table are for "uniform" rock mass conditions, and will vary somewhat depending on rock mass characteristics. It will therefore be necessary to adjust the predictions of caving angles to take into account these characteristics.

Informed prediction based on inspection and interpretation of rock mass characteristics

This approach is an extension of the above two approaches and introduces "reasoned" estimates of subsidence behaviour based on interpretation of how the rock mass will deform in relation to its structure and geometry. For example:

- in dipping strata, the influence of the subsidence on the up-dip side will be less than on the down-dip side;
- fault and dyke contact planes can inhibit or extend the area of subsidence, depending on their orientations;
- the plan shape of the mining will also influence the extent of caving – when the plan radius of curvature of the cave zone is less than the depth of the mining, the predicted cave angle can conservatively be 5° steeper. Conversely, if there is a significant section of the cave zone, which has a convex shape, the cave angle should be taken to be 5° shallower.

Numerical analysis of subsidence

The ready availability of sophisticated computer codes for analysis of stresses and deformations often results in this approach being the first choice for the prediction of subsidence. If no pre-existing data exist regarding subsidence behaviour at a mine, this theoretical approach may be the only option. Such predictions can be seriously in error, since they are very dependent on the deformation and strength parameters used, and also on the representation of jointing in the model.

3.2 Evaluation of the stability of open pit and quarry slopes

The slope architecture of the open pit or quarry should be matched with the particular geotechnical environment. The term architecture refers to the spatial relationship, face inclinations and geometrical dimensions of each of the individual elements:

- individual bench face: height and face angle;
- bench stack or inter-ramp slope: height versus angle relationship;
- bench spill berm width;
- ramp or geotechnical berm width;

- limiting overall slope by design sector

There are three categories of stability evaluation, which follow directly from the rock mass characterisation approaches described in Chapter 2. These are:

- soil and overburden materials for which soil mechanics based stability evaluations are appropriate;
- locally homogeneous rock mass situations (homogeneous within a domain) for which rock mass classification based stability evaluations are appropriate;
- behaviour dominated by geological structure, in which cases, kinematic stability analyses will be required. Toppling instability is a special case of kinematic instability.

The methods described in the following sections are simple and will be adequate in most cases. If stability is shown to be critical, more sophisticated analyses may be necessary.

3.2.1 Evaluation of the stability of slopes in soil and deeply weathered rock

For the purposes of this booklet, the most convenient method of "soil" slope stability analysis is a set of charts corresponding with different groundwater conditions[11].

- dry conditions;
- saturated conditions, to take account of a worst case situation;
- a partially saturated condition intermediate between the dry and saturated conditions.

These conditions are included on the slope stability chart in Figure 18. The input data required for the use of the chart are as follows:

c cohesion of the soil mass (kPa)
γ density of the soil mass (kN/m^3)
φ angle of internal friction of the soil mass (º)
H height of the slope (m)

Table 8 gives descriptions of the consistency of soils and corresponding typical values for the above parameters. These values do not take account of any soil structure that may be present in the soil mass.

The method of use of the charts to determine the factor of safety of a slope is as follows:

i) Calculate the value of $c/(\gamma H.\tan\varphi)$ and find the corresponding point on the circumference of the chart.
ii) Translate radially inwards on the chart from this point to meet the required slope angle isoline.
iii) For this intersection point, read off the corresponding ordinate value $\tan\varphi/F$ (or the abscissa value) and hence calculate the value of the factor of safety F.

The chart is a means of determining slope factors of safety very rapidly.

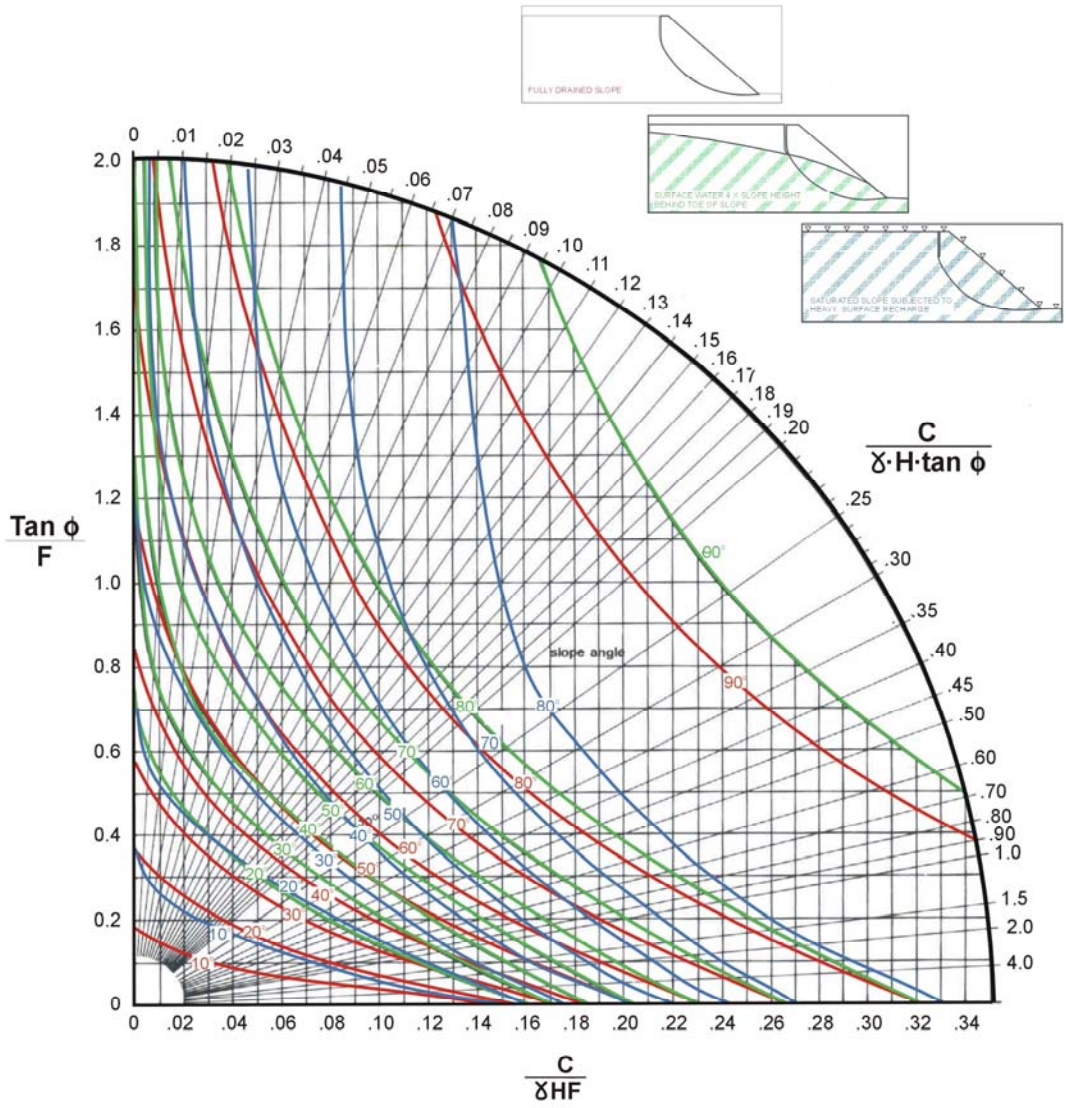

Figure 18 Stability and design chart for soil and weathered rock slopes[11]

Table 8 Typical values of soil parameters

Soil description	Density (kN/m^3)	Cohesion (kPa)	Friction angle (^{o})
Loose sand/gravel	16	Zero	35
Medium dense sand/gravel	18	Zero	37
Dense sand/gravel	20	Zero	40
Loose silt	16	2	29
Medium dense silt	17	5	30
Dense silt	18	10	31
Soft clay	16	5	23
Moderately stiff clay	17	10	24
Stiff clay	18	25	25

3.2.2 Evaluation of the stability of rock slopes in homogeneous rock mass domains

The use of a rock mass classification approach is valid in situations in which the control on global or overall stability is not dominated by major structural features. The correlation between adjusted *MRMR* values and rock slope angles[9] in Table 9 has been selected for inclusion in this booklet since it makes use of a standard rock mass classification system (see Chapter 2), and its validity has been proved in application to a variety of mining rock slopes.

Table 9 Adjusted *MRMR* values and recommended overall rock slope angles

Adjusted *MRMR* rating	100	90	80	70	60	50	40	30	20	10	0
Overall Slope Angle	>75°	75°	70°	65°	60°	55°	50°	45°	40°	35°	<35°

This table is best suited to the preliminary determination of overall slope angles.

It is necessary to take the slope height into account as well and, based on case history data, the empirically based chart in Figure 19, correlating slope height, slope angle and adjusted *MRMR* values has been developed[9]. This chart is better suited to the selection of inter-ramp or bench stack slope angles than Table 9.

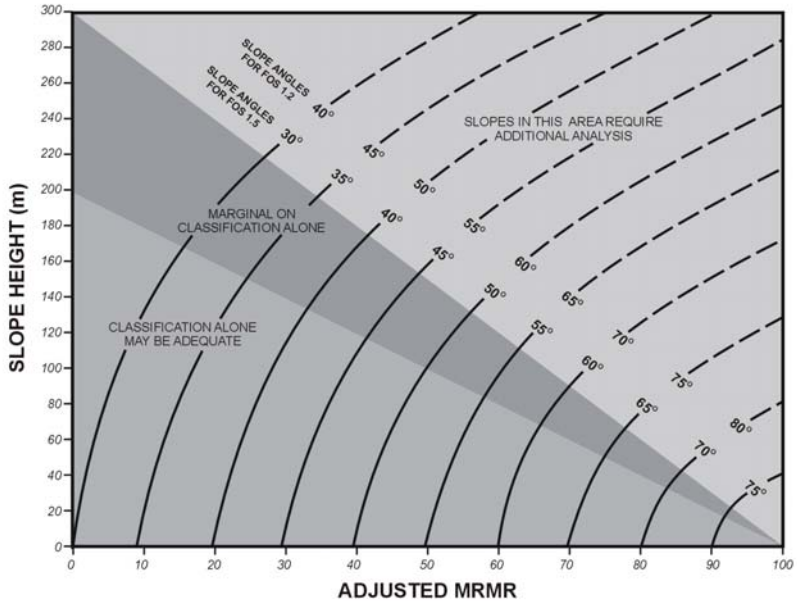

Figure 19 Empirical slope design chart[9]

3.2.3 Evaluation of the stability of rock slopes in rock masses containing major structural features

The presence of major structural features such as faults, major joint planes, unfavourably orientated bedding planes or schistosity fabric, may have an overriding influence on the stability of rock slopes.

Evaluation of stability of rock slopes exposed to potential planar failure

Planar failure is a simple case, and the stability of a planar wedge or block, such as that shown in Figure 20, can be determined using the following equation:

$$F = \{cA + (W\cos\psi_f - U - V\sin\psi_a + T\cos\psi_a)\tan\varphi\}/\{W\sin\psi_f - V\cos\psi_f - T\sin\psi_a\}$$

where: F is the factor of safety
- c is the cohesion on the failure plane
- A is the area of the base of the wedge or block
- W is the weight of the wedge or block
- ψ_f is the angle of inclination of the failure plane
- U is the uplift force on the failure plane due to water pressure
- V is the horizontal force on the wedge due to water pressure in a tension crack
- T is the tensile force in a cable anchor
- ψ_a is the angle of inclination between the anchor and the normal to the failure plane
- φ is the friction angle on the failure plane.

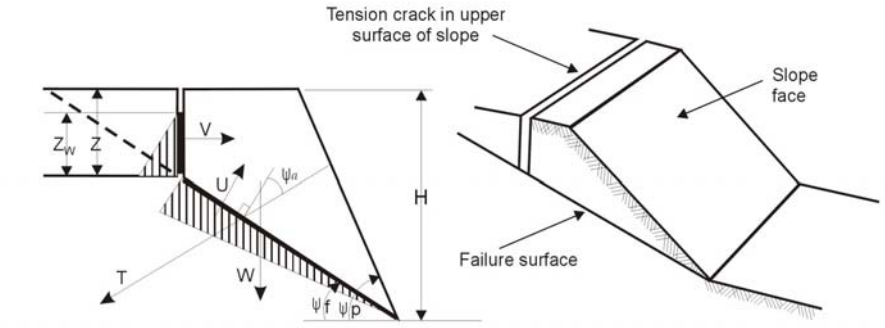

Figure 20 Planar wedge mode of slope instability[11]

Evaluation of stability of rock slopes exposed to potential wedge failure

A set of stability charts[11] is recommended for rapid stability evaluation. These charts take into account frictional strength on the planes of weakness only, and cohesion and water pressures are ignored. Cohesion will contribute to stability whilst water pressures will be detrimental to stability. The general geometry of the wedge considered, with sliding taking place on two planes, is shown in Figure 21.

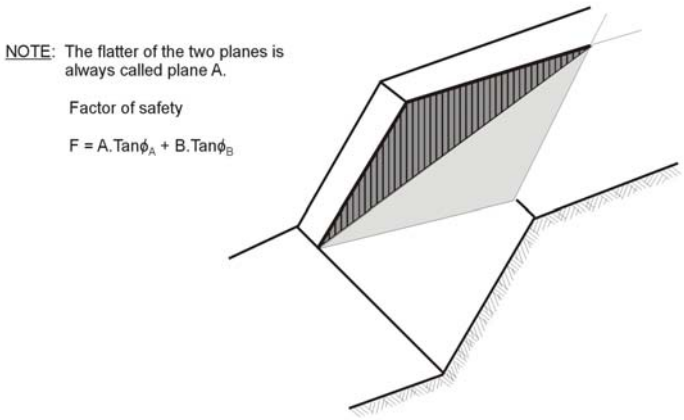

Figure 21 Three dimensional wedge model of slope instability[11]

The wedge stability charts are presented in Figures 22 to 25. In these charts the *A* plane is always the plane with the flatter dip angle. To use the charts, the difference in dip angles of the two planes is determined and the appropriate chart then chosen. Using the difference in dip direction angles and the dips of the two planes, the factors *A* and *B* are read off the charts. The factor of safety of the slope is then calculated using the equation:

$$F = A.\tan\varphi_A + B.\tan\varphi_B$$

where φ_A and φ_B are the friction angles of the two planes.

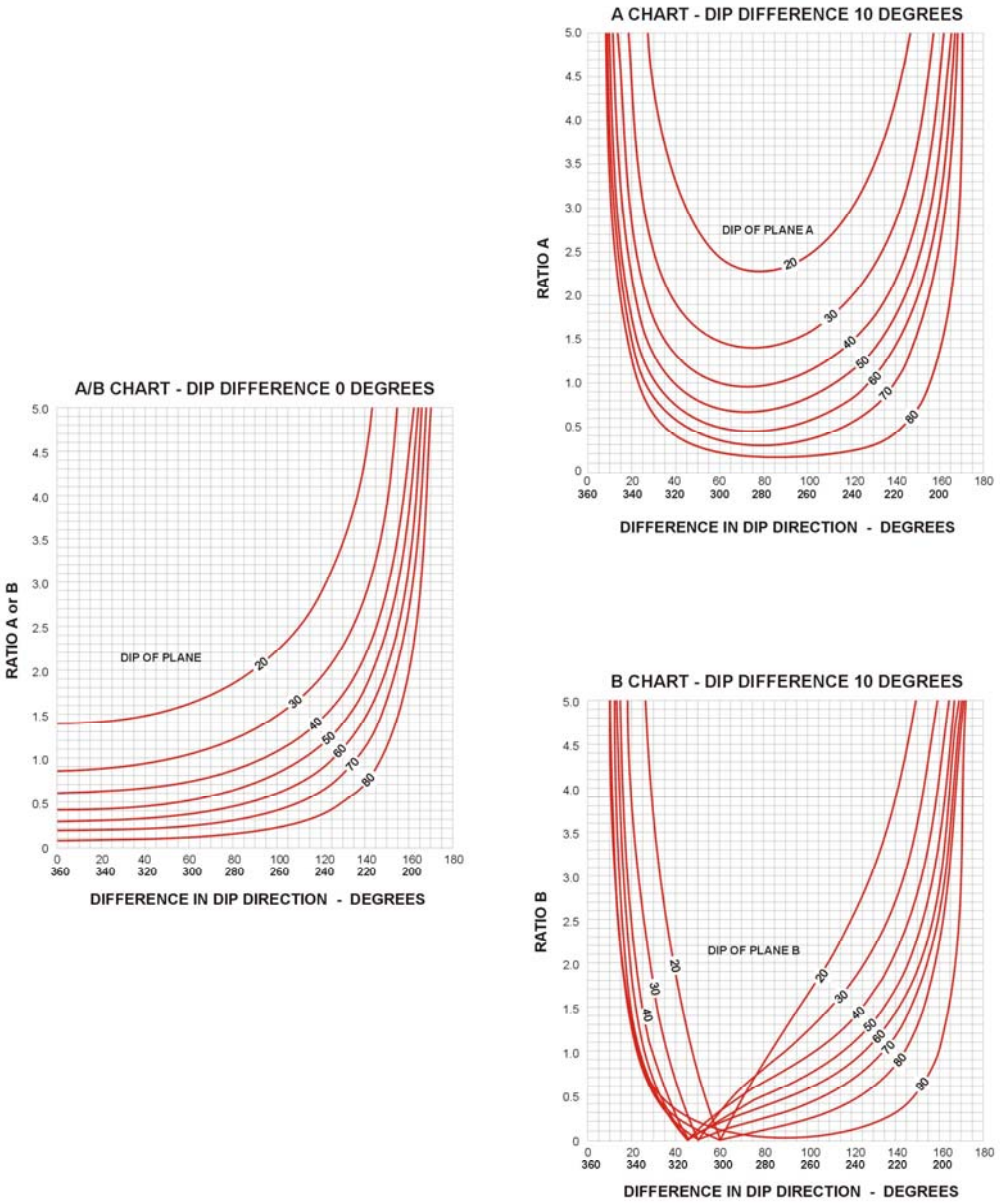

Figure 22 Slope stability and design charts[11] - dip difference 0° and 10°

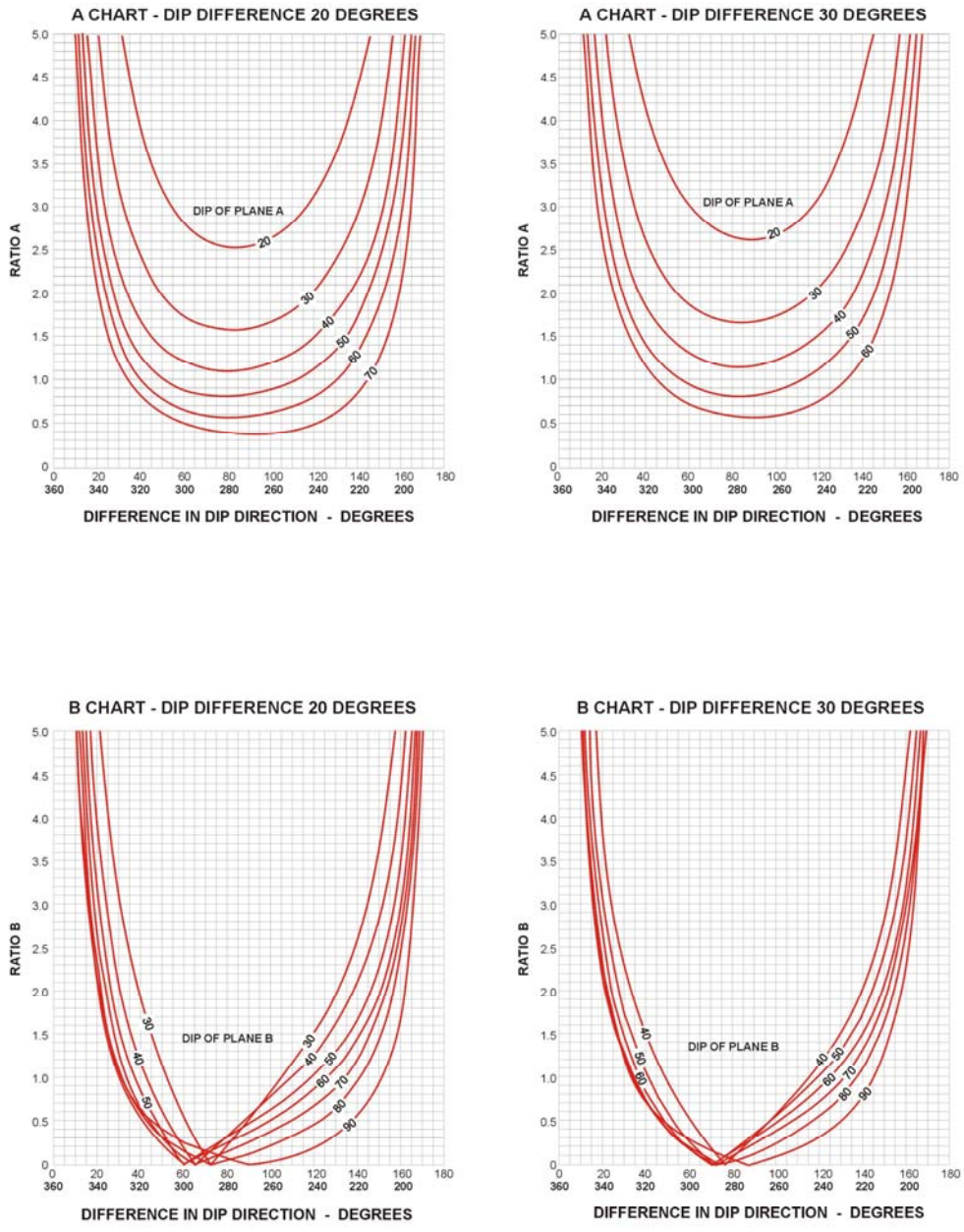

Figure 23 Slope stability and design charts[11] - dip difference 20°and 30°

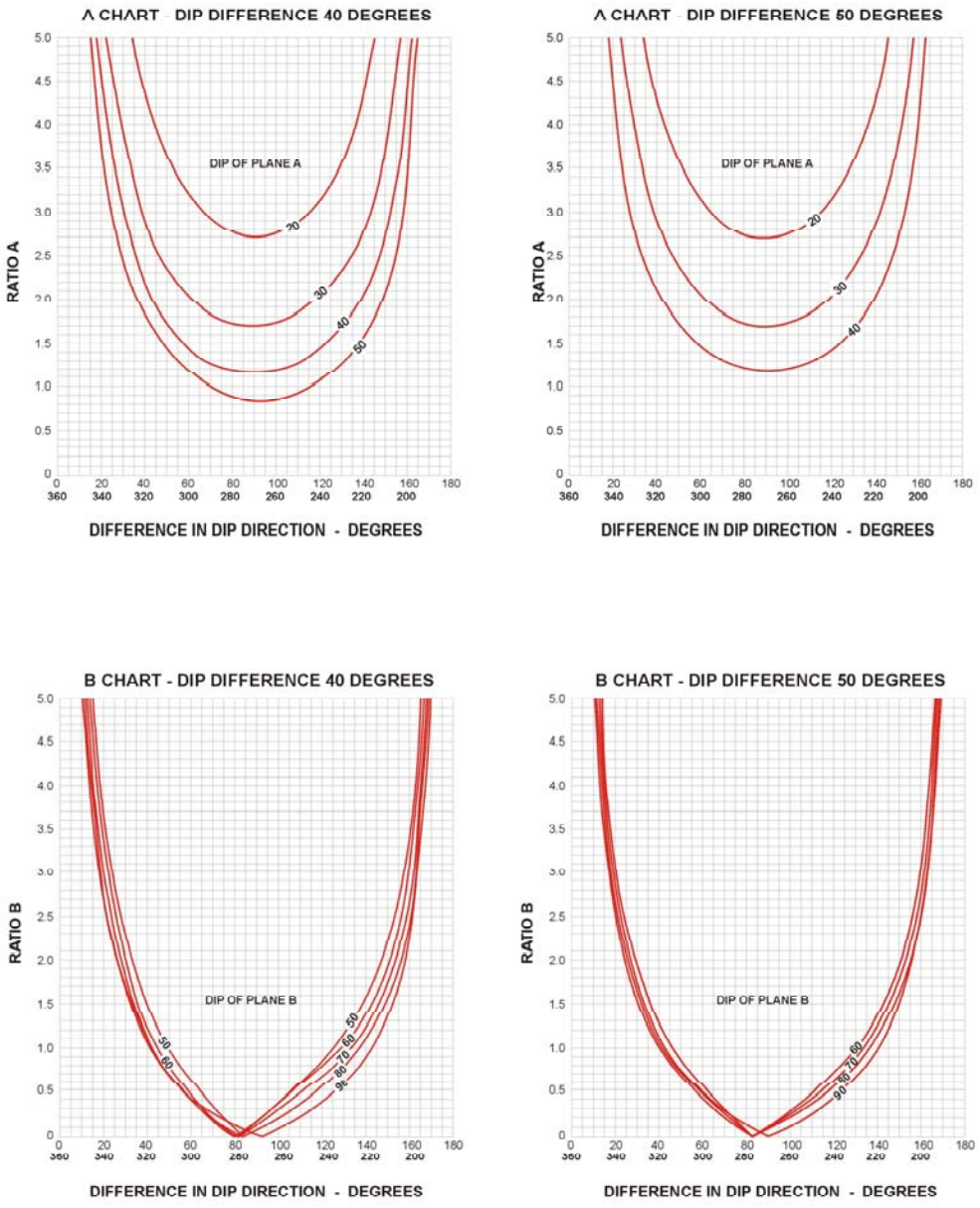

Figure 24 Slope stability and design charts[11] - dip difference 40° and 50°

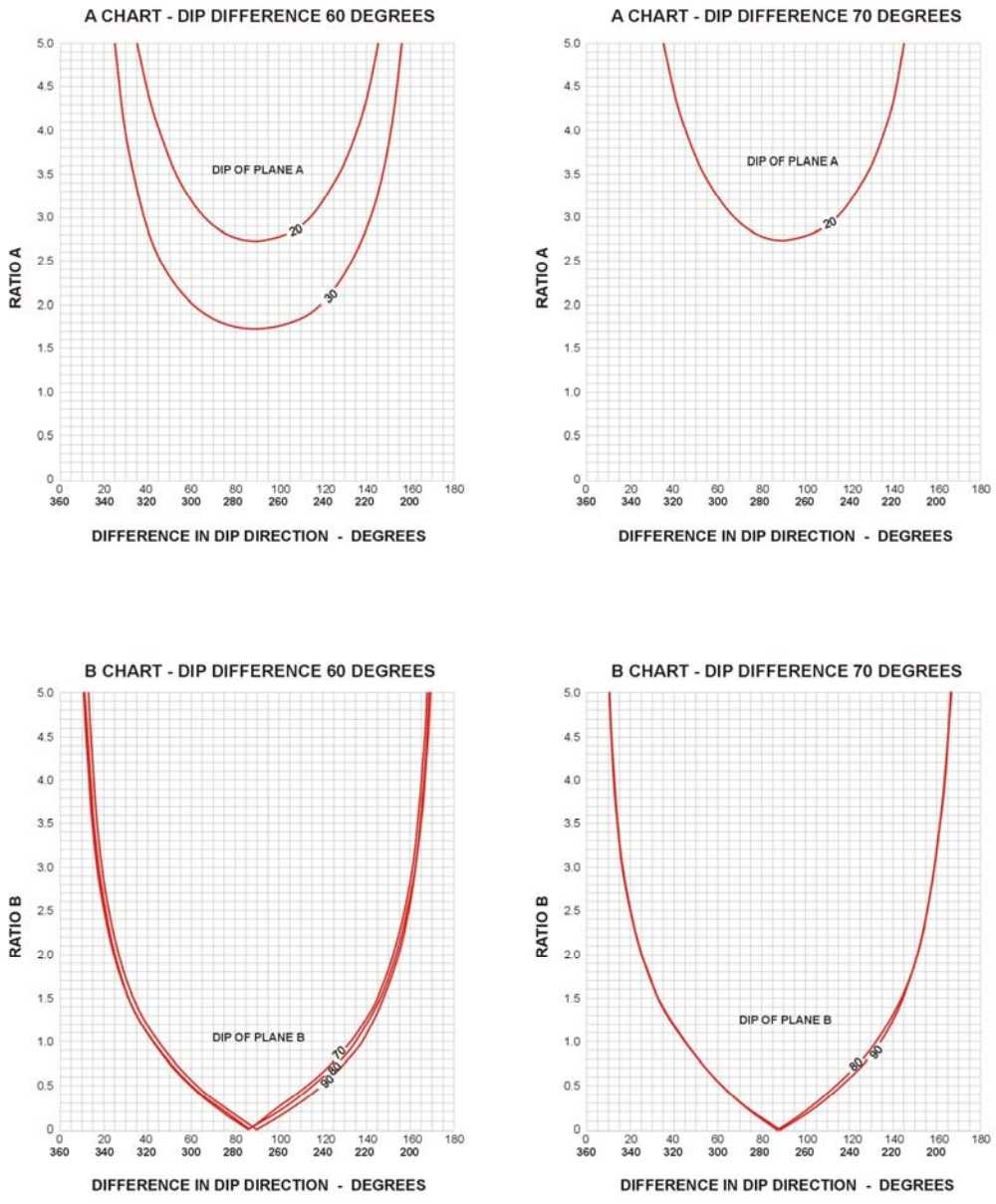

Figure 25 Slope stability and design charts[11] - dip difference 60° and 70°

Since cohesion and water pressure have been omitted from the analyses, conservatism is necessary in the interpretation of the results as indicated by the recommended actions in Table 10.

Table 10 Stability condition from wedge stability analyses

Calculated Factor of Safety	Stability Condition	Recommended Action
$FOS > 2.0$	Stable	None
$1.0 < FOS < 2.0$	Marginal	Analyse stability rigorously
$FOS < 1.0$	Unstable	Revise design or stabilise

Evaluation of rock slopes exposed to potential toppling

Large scale toppling failures are unlikely to occur in open pits or quarries, and, should they develop, a long period of warning will occur, with evidence such as the gradual opening up of tension cracks adjacent to the crest of the slope. Small scale toppling failures from bench faces represent greater hazards. If the dip of joints is between 60° and 95°, then there is toppling potential. In areas in which toppling potential is identified, benches or spill berms should be sufficiently wide to ensure that boulders cannot bounce over the crest.

3.2.4 Influence of various factors on slope stability

The most important factors of influence on slope stability are dealt with in the following sub-sections.

Geology and geological structure

The importance of these factors is obvious, but they are often ignored to meet short term requirements. It is essential, not only for safety, but also from an economic point of view, that the mine operator has a thorough understanding of the geology and geological structure, and their potential effects on the mine.

Groundwater

Not only is water a nuisance since it causes erosion of roads and the accumulation of mud in the bottom of the pit or quarry, but water within the rock or soil mass decreases stability considerably. If instability develops in a slope, drainage is one of the first measures that should be considered for stabilisation.

Blasting

Poor quality blasting is detrimental to slope stability. Not only do the vibrations cause some dynamic loading of the slopes, but, more importantly, the rock behind the slope face can be fragmented and loosened. It is estimated that the stable slope angle for a good quality rock mass could, in some cases, be reduced by about 15% by poor blasting.

Slope geometry

The plan shape affects both the potential for failure and the consequences of failure. Concave slopes are preferred:

- the confinement inherently provided by the concave geometry enhances stability;
- if instability occurs, the volume of the failure is less.

When the radius of curvature of a concave slope is less than the height of the slope, the slope angle can be 5° steeper than determined by conventional slope stability analysis, which is a two dimensional analysis with a plan radius of curvature of infinity. Conversely, for a convex slope, with the radius of curvature less than the slope height, the slope angle should be 5° flatter than conventionally determined.

Earthquake loading

Earthquake loading should be taken into account in the evaluation of slope stability if the slopes are in areas of significant earthquake or mine induced seismic risk. Information on such risks can be obtained from the Council for Geoscience.

3.2.5 Slope management

In open pit mining operations the use of artificial or installed support to stabilise slopes which have become unstable can rarely be justified. Rather than consider stabilisation directly the recommended approach is to adopt a slope management process. In this approach, the geometry of the slope, the mining sequence and the variation in this sequence, and the planning of alternative or dual haul road systems, are all part of the process. Regular and routine monitoring of movements by survey methods, installed instruments, seismic or microseismic systems, or even satellite based global positioning systems, are appropriate. Early knowledge of potential instability, and the consequent ability to implement a plan of action, will be much less hazardous and costly than if a slope failure occurs unexpectedly.

There are two controllable factors which have a significant contribution to slope stability - blasting and groundwater. In many cases of unstable slopes, stability can be ensured, or at least improved, by improving the quality of the blasting, and by dewatering of the slopes.

CHAPTER 4

SUPPORT

The consideration of support is in two separate areas - large scale or regional type of support such as pillars and backfill, and small scale installed support such as rockbolts, wire mesh and mine poles.

4.1 Solid support

Solid support, in the form of pillars of ore or waste rock, is very commonly used to provide stability in many mining methods. Crown pillars, shaft pillars and barrier pillars are examples of those pillars required to promote overall mine stability. Panel pillars, sill pillars and crush pillars are examples of pillars used to satisfy the more local stope stability requirements.

4.1.1 Design of solid pillars

The design of pillars for support of mine openings requires the determination of two aspects:

- the pillar strengths, and
- the stresses acting on the pillars.

Pillar strength

The strength of pillars is mainly a function of:

- the strength of the intact rock of which the pillar consists, suitably downrated to take into account the scale effect;
- the geometry of the pillar, including both its shape and its width to height relationship.

The pillar strength formula recommended for use in design and stability evaluation of pillars with a width to height ratio of about 5 or less is:

$$P_{strength} = DRMS.\ W_{eff}^{0.5}/H^{0.75}$$

where: *DRMS* is the Design Rock Mass Strength[17]
W_{eff} is the effective pillar width
H is the pillar height

The above formula is a general one, and its applicability to particular mining cases needs to be verified by observation of pillar behaviour and correlation of this behaviour with the application of the formula. It should be noted that a much more sophisticated approach to pillar design has been proposed in GAP 334. It is not well established yet, however, and is considered to be too involved for the purposes of this booklet.

The effective pillar width takes the variations in pillar geometry into account.

W_{eff} = 4 x pillar area/pillar perimeter

Experience has shown that the *DRMS* usually falls in the range of 20% to 50% of the *UCS* of the intact rock, and is commonly about 30% of the *UCS*. For convenience, Figure 26 provides a guideline relationship between *DRMS* and *RMR* for different values of *UCS*.

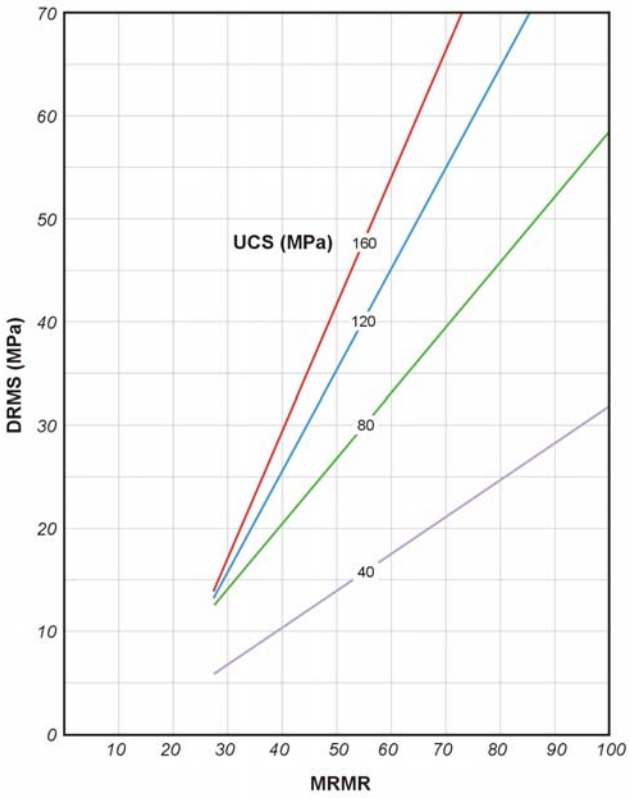

Figure 26 Guideline relationship between *DRMS* and *MRMR*

As the width to height ratio of a pillar increases, the pillar becomes progressively stronger. The pillar strength formula above begins to underestimate the strength when the width to height ratio exceeds 5. It can be assumed that, when the width to height ratio is greater than 10, the pillar will not fail unless it contains weaknesses such as soft bands or layers.

Confinement of pillars by backfill increases the strength of the pillars. For good quality backfill, an adjustment of 120% can be used.

The presence of jointing in a pillar will decrease its strength since the joints represent defects. Adjustments to the pillar strength (as determined from the above pillar strength formula) for the effect of jointing may be determined from Figures 27 and 28. These figures[7] take into account the orientation of the jointing, the intensity of the jointing, and the pillar width to height ratio.

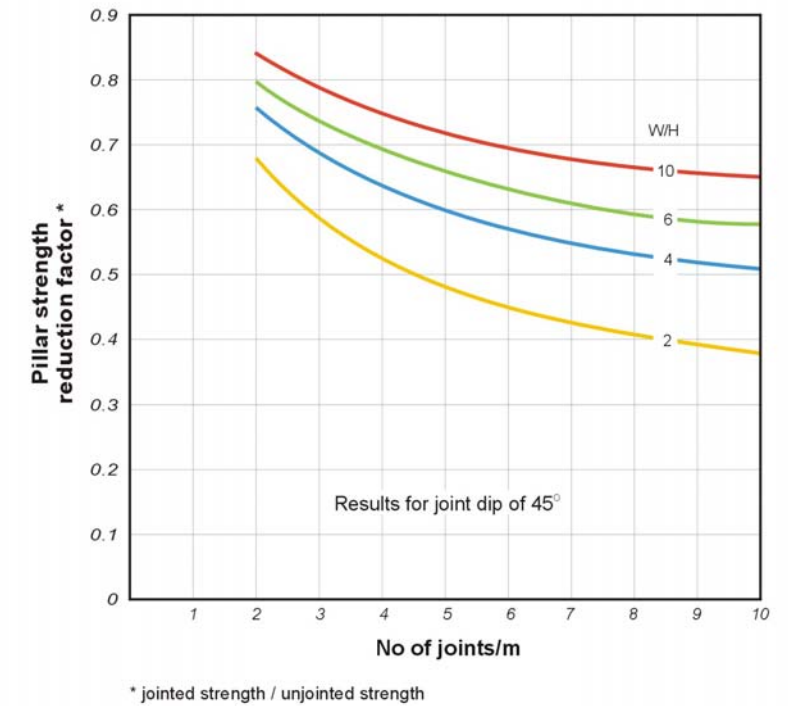

Figure 27 Pillar strength reduction factors for various pillar width to height ratios[7]

Recent research work carried out into the design of pillars in areas of variable topography[28] has shown that dimensions determined for pillars at shallow depth using the above formula can result in under-designed pillars owing to the shear loading introduced by the topography. To ensure the necessary stability in such situations, it is recommended that the extraction be decreased as indicated in the guideline sketch in Figure 29. For example, if the pillar formula suggests that an extraction of 90% would be acceptable, then for mining at the points marked X on Figure 29, this extraction should be reduced to 82.5%.

Pillar stress

The tributary area theory assumes that the pre-mining stresses are evenly distributed on the pillars taking into account the extent of mining extraction. This is an approximation that is most applicable to regular pillar layouts. If σ_v the pre-mining vertical in situ stress, the average pillar stress P_{stress} in a sub-horizontal mining layout is given by:

$$P_{stress} = \sigma_v/(1 - e)$$

where:
e is the extraction ratio (percentage extraction is 100 x e)

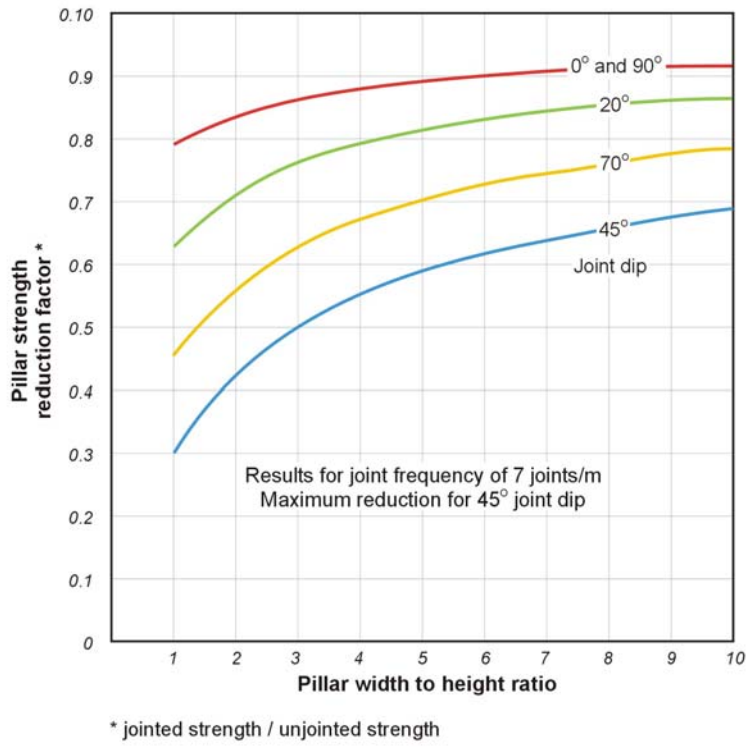

Figure 28 Pillar strength reduction factors for various joint dip angles[7]

When the mining horizon is inclined, the average pillar stress equation becomes:

$$P_{stress} = (\sigma_v.\cos^2\theta + \sigma_h.\sin^2\theta)/(1 - e)$$

where

σ_v is the vertical in situ stress
σ_h is the horizontal in situ stress
θ is the dip angle of the mining horizon

In massive mining, layouts are generally not sufficiently regular or extensive for the tributary area theory to be applicable. In such cases it is necessary to determine pillar stresses by other means. The only way in which this can be achieved realistically for complex pillar and mining geometries is to use numerical stress analyses.

Pillar design procedure

Once the pillar strength and pillar stress are known, the factor of safety *(FOS)* of the pillar can be determined as the ratio of the strength to stress:

$$\text{Pillar } FOS = P_{strength}/P_{stress}$$

The choice of the *FOS* value to be used for the design of the pillars and layout depends on the function of the pillars. The following are recommendations for various types of pillars commonly used in stoping operations:

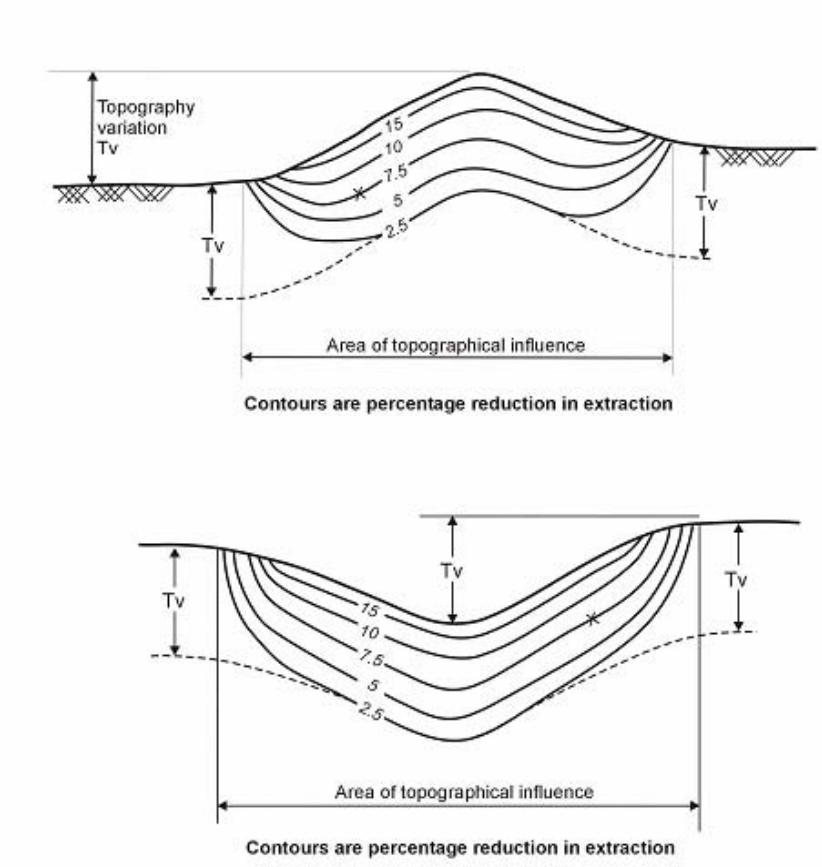

Figure 29 Guideline for reduction of extraction in areas of topographical variation

- barrier pillars are required to provide support such that they are a barrier to the progress of any failure of the stope past their location. Therefore, the *FOS* of the pillar must be sufficiently high to ensure that the pillar does not fail under these conditions. A recommended *FOS* for design of barrier pillars is 2.0;
- stope or panel pillars are required to provide stability to that section of stope between barrier pillars. A recommended *FOS* for stope pillars is 1.6;
- crush pillars provide the function of local support. They must allow closure to take place uniformly without breaking the back of the stope. Their width to height ratio will generally fall into the range of 1.5 to 2.5, and a recommended *FOS* is 0.7;
- yield pillars are intermediate between non-yield and crush pillars. They should be initially intact, but, as mining progresses and loading on them increases, they should be able to yield in a stable manner[24]. A recommended *FOS* is 1.0. Yield pillars should only be used with great circumspection.

It should be noted that the above considerations may be over-ridden by other requirements such as the necessity in certain instances to ensure that no subsidence occurs on surface. In such cases, the pillar systems will have to be designed so that regional stability is ensured.

4.1.2 Special pillars

There are numerous other pillars which can be considered to be special cases and whose design needs to be considered individually.

Water and boundary pillars

It is essential that these pillars do not fail. They are pillars on the boundary between adjacent mines, and pillars to prevent the uncontrolled flow of water from one part of the mine to another. In tabular mining deposits, pillars with width to height ratios of more than 20 are adopted, and, in terms of South African law, boundary pillars with a minimum width of 18m are required.

Sill pillars

The above pillar design techniques may be partially applicable in some cases of sill pillars. However, there are several specific factors which must be taken into account in the design of sill pillars.

- Sill pillars are applicable in inclined orebodies and therefore will be subjected to shear stresses as a combination of the in situ principal stresses. The likely stresses in the pillars will be determined best by means of numerical stress analyses.
- The effect of inclined jointing on the strength of the pillar must be taken into account as indicated in Figures 27 and 28.
- The stability of the pillar in beam action must be checked, as described in Section 3.
- The potential for blocks and wedges to detach from the under side of the pillar must be checked, using, for example the simple computer program BlockEval.
- Should any of the above considerations indicate that the pillar does not have the required stability, it will be necessary to increase the dimensions of the pillar or to install support, such as cable anchors, to enhance the stability to the required level.

Crown pillars

The design of crown pillars should follow similar procedures to those described for sill pillars. Crown pillars are critical support elements, however, and are usually located in more weathered rock close to the surface. Failure of a crown pillar can be catastrophic, may lead to air blast damage underground, and may expose underground workings to inflow of water from surface, mudrushes and dilution of ore. Surface effects may also be hazardous to mine infrastructure. These include the direct effects of the collapse to form a crater as well as the subsequent stability of the crater.

Failure of a crown pillar will usually involve a combination of mechanisms - spalling from the under side of the pillar, which could be due to kinematic instability, overstressing, or both; disruption of the competence of the pillar by sagging in beam/plate action; and shearing through the rock mass on the boundaries of the pillar. The design and evaluation of crown pillars must therefore include the use of stress analyses, beam and plate stability analyses, and block/wedge stability analyses. The results of the stress analyses must be used to determine the potential for rock and rock mass failure due to overstressing as well as the potential for shear failure through the rock mass and on joints or other planes of weakness.

Shaft pillars

Shafts provide access to the orebody and ventilation to the mine, and it is crucial that these excavations remain serviceable for the life of the mining operations. In extensive, tabular orebodies, where a vertical shaft is required at the centre of operations, shaft pillars are usually left to protect the shaft and, in many cases, other major service excavations. Pillars are also frequently used to protect inclined shafts, which are either in the plane of the (tabular) orebody or just beneath it. In massive orebodies, shafts are normally placed in the host rock, sufficiently remote from the orebody.

The major principal stress in the shaft pillar (σ_1) should be limited to the critical stress, which is one third of the intact rock compressive strength ($\sigma_1<\sigma_c/3$) to limit damage to excavations. This criterion can be increased to $\sigma_1<\sigma_c/2$, but more intensive support is then likely to be required. In addition to this stress criterion, the induced strain and tilt need to be limited to prevent damage to the steelwork and concrete lining.

The required pillar size can be determined from numerical modelling. All barrier pillars and known geological losses should be included in the numerical model as they have a significant effect on the shaft pillar stress - areas of unmined ground increase the stiffness of the surrounding rock mass, which reduces the load on the shaft pillar. The average mining span between areas of solid ground has the greatest influence on the stiffness of the rock mass.

A reasonable estimate of shaft pillar sizes, for feasibility purposes, can be obtained from the graphs in Figures 30 to 34. These analyses were carried out with a barrier pillar system where the average span between barrier pillars is half the depth. Separate analyses were carried out for inclined and vertical shafts.

Vertical shafts

Numerous limiting criteria for damage to shaft steelwork and concrete linings have been quoted, in terms of induced vertical stress (σ_z^i) induced vertical strain (ε_z^i) and induced tilt (t^i). These are summarised in Table 11. Where the shaft pillar is at a depth of less than 600m, the induced strain is the limiting criterion. It is likely that shaft pillar dimensions designed on the basis of these criteria will be conservative[23].

The required shaft pillar radius for a given depth and rock elastic modulus can easily be determined from Figure 30. The pillar sizes in this figure are based on the most conservative induced strain criterion ($0{,}2 \times 10^{-3}$). It is suggested that an elastic modulus equal to two thirds of the intact rock elastic modulus should be used to determine the minimum pillar radius. Figures 31 to 33 indicate the maximum strain for a range of shaft pillar radii at various depths for different elastic moduli.

The shaft pillar abutment stresses will be significantly greater than the stresses in the centre of the pillar and main service excavations will need to be sited away from the influence of these abutments. Where relatively small shaft pillars are used it may not

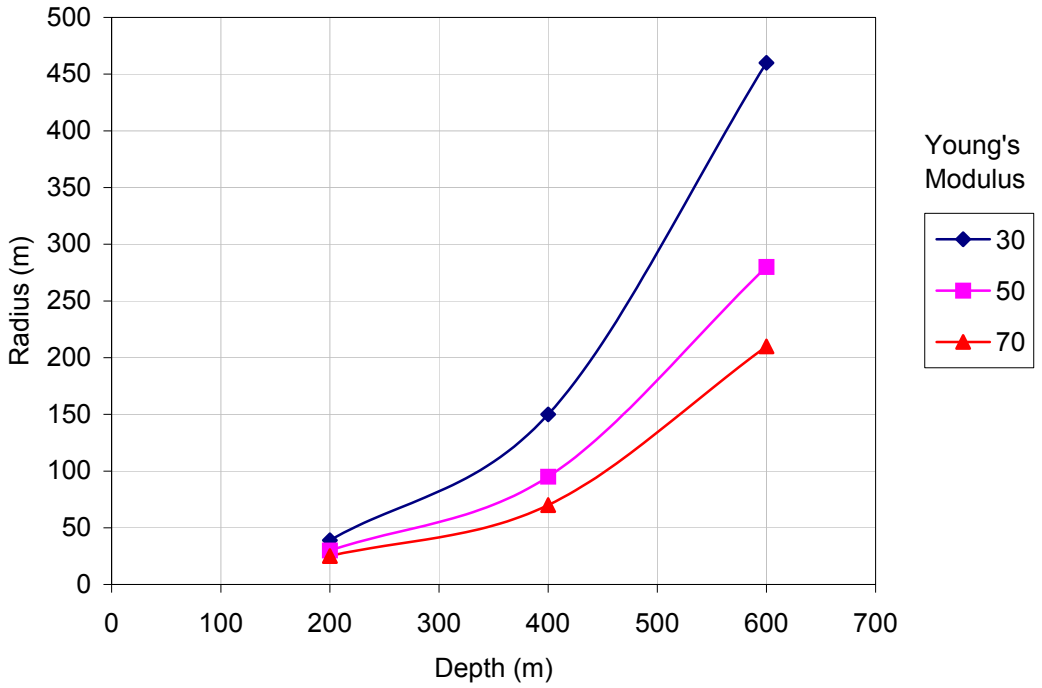

Figure 30 Vertical shaft pillar radius vs depth

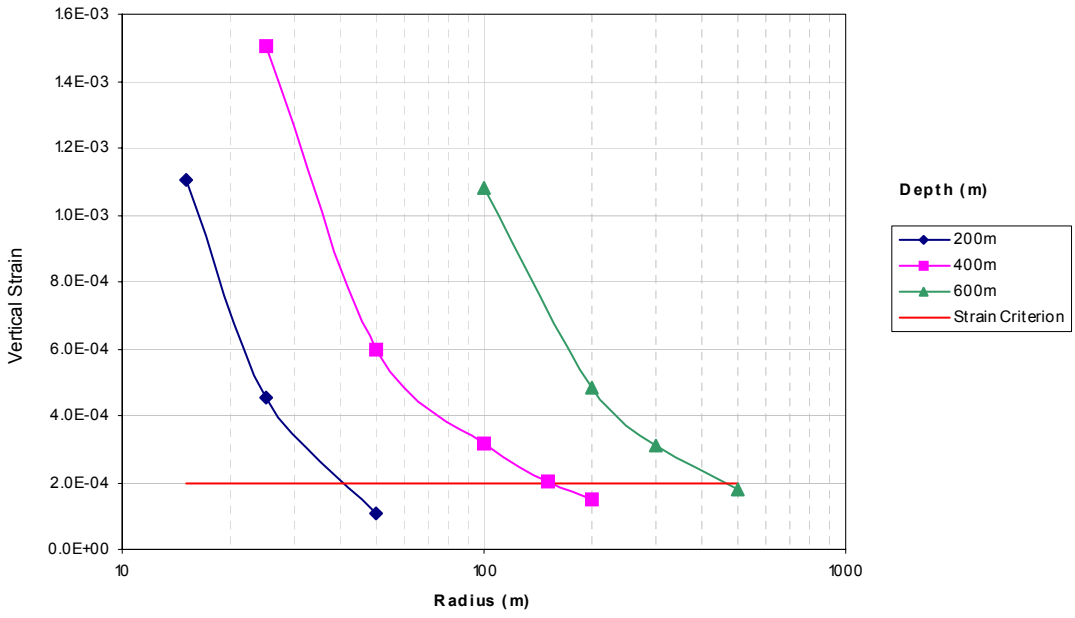

Figure 31 Strain in pillar vs radius - E = 30 GPa

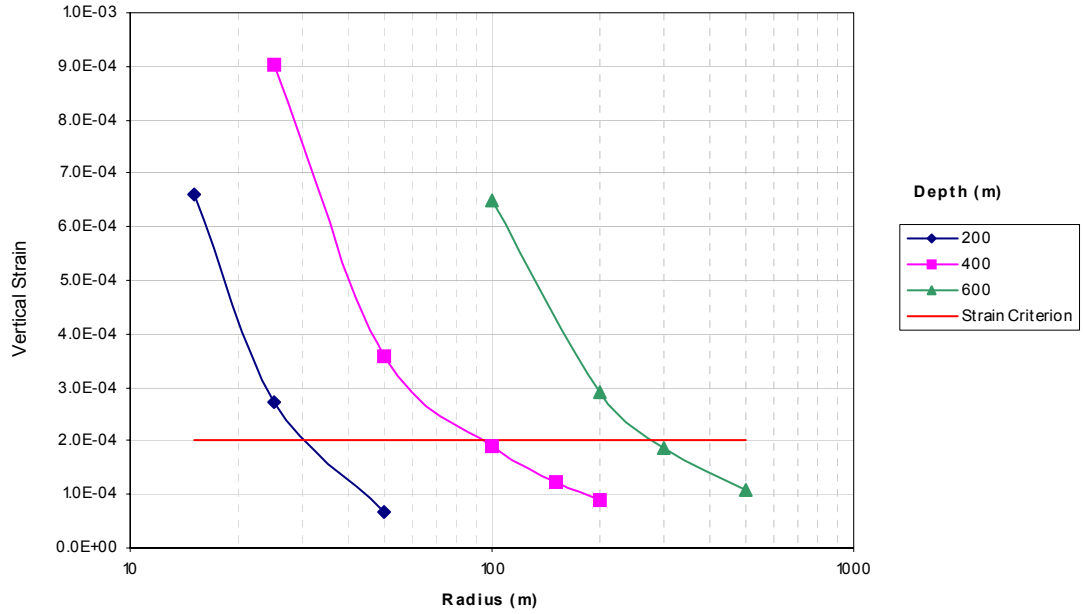

Figure 32 Strain in pillar vs radius - E = 50 GPa

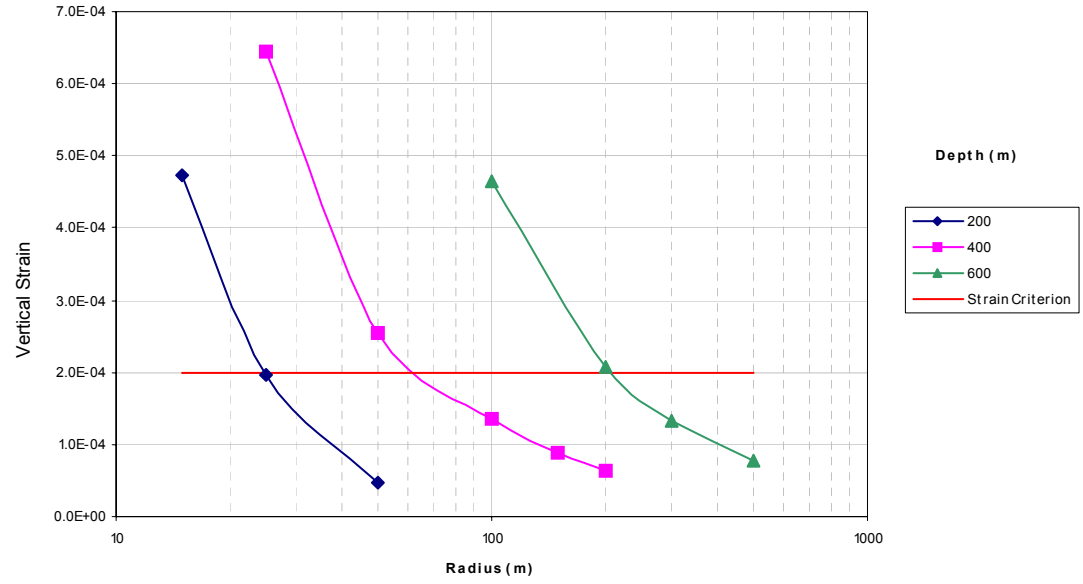

Figure 33 Strain in pillar vs radius - E = 70 GPa

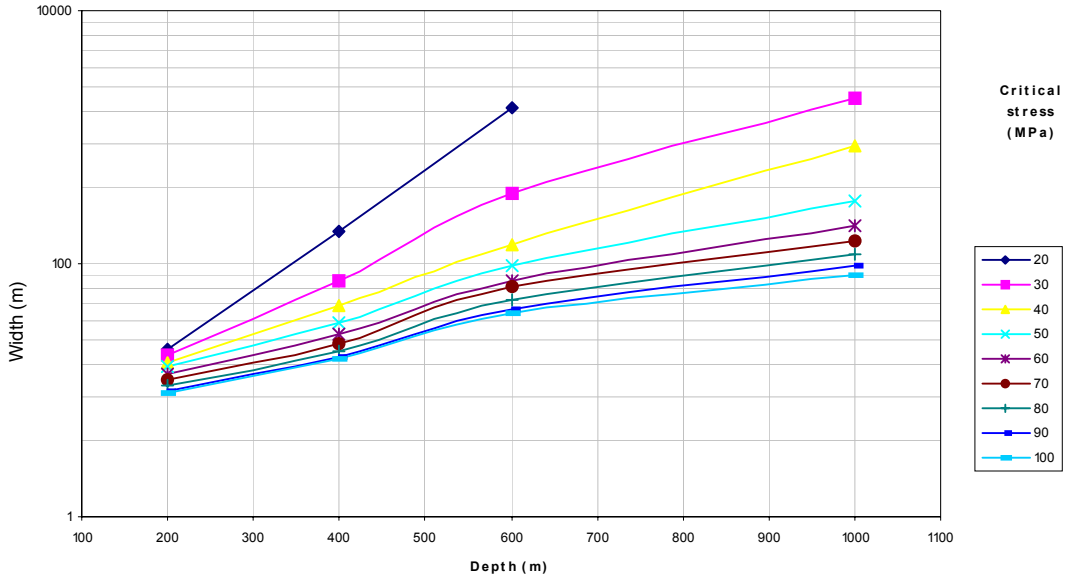

Figure 34 Inclined shaft pillar width vs depth

Table 11 Damage criteria for shafts[22]

Type of damage	Criterion	Reference
Unspecified damage	$\sigma_z^i < 17$ MPa	Wilson (1971)
Unspecified damage, criterion not based on back analysis	$\varepsilon_z^i < 1 \times 10^{-3}$	Salamon (1974)
	$\varepsilon_z^i < 1 \times 10^{-3}$	Wagner (1985)
	$\varepsilon_z^i < 1 \times 10^{-3}$	Daemen (1972)
	$t^i < 1 \times 10^{-3}$	Wagner (1985)
Steelwork damage and increased shaft maintenance	$\varepsilon_z^i < 0{,}2 \times 10^{-3}$	Wilson (1971)
	$\varepsilon_z^i < -0{,}4 \times 10^{-3}$	Van Emmenis and More O'Ferrall (1971)
	$\varepsilon_z^i < 0{,}4 \times 10^{-3}$	Esterhuizen (1980)
Tensile fracture of concrete lining	$\varepsilon_z^i < -0{,}51 \times 10^{-3}$ to $-0{,}3 \times 10^{-3}$	Kratzsch (1983)
	$\varepsilon_z^i < -0{,}2 \times 10^{-3}$	Budavari (1986)
Compressive fracture of concrete lining	$\varepsilon_z^i < 0{,}7 \times 10^{-3}$	Jager and Ryder (1999)

be possible to site main service excavations within the shaft pillar. These service excavations can be sited in ground that is destressed through under or over stoping. Alternatively the excavations can be sited away from the reef horizon, beyond the influence of the shaft pillar abutment.

Where multiple reef extractions are carried out, deformations could be significantly increased and larger shaft pillars may be required to reduce these deformations. Shafts should, as far as possible, be sited in geologically undisturbed ground. This will avoid wedge failures in the shaft sidewalls and reduce the amount of additional support required.

Accommodating large deformations in vertical shafts: large deformations in shafts can be accommodated by modifications to the steelwork and by the use of fibre reinforced shotcrete linings in zones where the deformation exceeds the limiting criterion. Fibre reinforced shotcrete and modifications to the steelwork represent a considerable additional cost in the equipping of the shaft.

Avoiding a shaft pillar: shaft pillars can be avoided by sinking beyond the sub outcrop or crosscutting through the reef horizon and continuing with a sub vertical shaft. Sinking a shaft beyond the sub outcrop has the disadvantages of additional development to access the reef and longer travelling times to the deeper parts of the orebody. A crosscut through the reef horizon represents the main access to the orebody and therefore needs to be protected for the life of the mine.

Early extraction of the shaft pillar: early extraction of the shaft pillar has several advantages. Revenue can be obtained at an early stage in the project and main service excavations can be placed close to the shaft in destressed ground. However, large deformations in the shaft can be expected under these circumstances, but can be reduced by using backfill or satellite pillars. The zones in the shaft which are still prone to large deformations will require modified steelwork and steel fibre reinforced shotcrete lining. With inclined reefs, the effects of tilt and horizontal dislocation will also need to be accommodated.

Inclined shafts

When inclined shafts are sunk in the orebody it is essential to leave a pillar to protect the shaft. The resulting major principal stress in the shaft pillar must be less than the critical stress for the rock in which it is sited. Required pillar widths for various depths can easily be determined from Figure 34. When the shaft is sited below the orebody, owing to the 45° protection rule, larger pillars may be required unless the shaft is close to the reef horizon. Destressing by overstoping should be considered, where the shaft is sunk beneath the orebody, and this overstoping should be carried out before any other mining to ensure that the shaft is properly destressed.

4.1.3 Pillar foundation stability

In certain situations the stability of a pillar support system may be dictated by the pillar foundation conditions rather than the pillars themselves. In "other mines" in South Africa, pillar foundations are unlikely to be a source of pillar system failure. As a conservative guideline, to ensure that foundation failure is avoided, pillars should be designed such that the average pillar stress is less than twice the *UCS* of the foundation rock.

4.2 Backfill

The reasons for using backfill as a support medium include the following: ..

- to permit safe and economical mining;
- to provide a working surface for mining operations, such as in cut and fill mining;
- to improve ventilation control;
- to achieve maximum extraction of the orebody;
- to benefit the environment by disposal underground of waste rock and/or tailings.

The structural action of backfill is the following:

- it reduces the exposed surface area of excavations during the mining process;
- it provides confinement to stope walls, and to pillars, and thus increase their stability and strength.
- in its bulk function, it fills the mining voids such that any loosened or failed rock has "nowhere to go". In this function it prevents or reduces deformations that would otherwise occur;
- it reduces or eliminates the risk of surface subsidence and caving since the void volume due to mining remains small;
- in some mining geometries such as narrow tabular stoping, it may have a very minor effect on the transfer of stresses in the rock mass.

Types of backfill

The advantages and disadvantages of five alternative backfill systems that can be considered for mining operations are summarized in Table 12.

Backfill material properties

The desired backfill characteristics depend upon the type of support required. To reduce water and mud in stopes to a minimum, and to minimise shrinkage of the backfill after placement, backfill should be placed at the highest possible slurry relative density. The optimum backfill slurry relative density normally lies between 1.75 and 1.80.

Typical porosities are: deslimed tailings 52%; full plant tailings 48%; tailings 45%; crushed waste fill 42%.

Typical moisture contents for different types of backfill are: deslimed tailings 26%; full plant tailings 32%; belt-filtered tailings 20%; crushed waste fill 5%.

Pulp density, which is the converse of moisture content, should be maximised for the following reasons:

- the capacity of the pipelines is maximised;
- pipe line wear is minimized;
- velocity changes, which cause line hammer, are reduced;
- less stope dewatering is required;

Table 12 Advantages and disadvantages of different types of fill

System	Advantages	Disadvantages	Remarks
Crushed Waste Fill	Available waste rock on surface can be used if tipping through a pass system is appropriate Hoisting time on shaft is reduced if waste is crushed underground Immediate access is possible on the fill by heavy equipment	High cost of crushing underground High cost of transport Confinement provided to pillars not as good as with some other types of backfill	Sufficient waste must be available or generated
Classified cycloned tailings (CCT)	Simple method of conveying large volumes of fill from surface directly into areas to be filled Has good drainage characteristics	Large volumes of excess water have to be handled underground Use of only the coarser fraction, leaving the finer fraction for tailings disposal creates potential problems with the building of slimes dams More difficult to pump than full plant tailings	Sufficient backfill material must be available
Cemented Full Plant Tailings	Simple method of conveying large volumes of fill from surface directly into areas to be filled Maximum use of tailings product Very good pumping characteristics	Poor drainage characteristics High moisture content requires greater quantities of binder	The binder material assists by hydrating the excess water retained by the fines
Paste Fill	Relatively small amounts of cement (3% to 5%) produce stiff backfill (1.5 to 3.5MPa) Reduced tailings impoundment requirements Good support properties Reduced spills underground The entire tailings stream can be placed underground	Costly system which requires expensive pumps, pipes and dewatering equipment Conveying distances limited due to high pressure gradients (approximately 1km horizontally) Good quality control is necessary	Pumping difficulties can be alleviated by pumping the fill underground as a slurry and then dewatering it close to the stopes
Slurry Fill	Very little run-off water Can be used with full plant tailings The total cost of this system could be less than for cemented CCT	Requires availability of a special binder Accelerator is costly and requires accurate dosing	Technically, this system is good

- less particle classification at placement;
- ground support potential increases;
- strength of cemented fills increases;
- probability of liquefaction is smaller.

The critical pulp density is the density up to which the viscosity increases reasonably steadily, but beyond which the viscosity increase is pronounced. This critical pulp density is normally 75%, but will depend on the length of the pipeline and the pipe diameter. The operating pulp density should be kept below the critical value by some suitable margin, incorporating a factor of safety suited to the particular operation. For CCT, this density is approximately 65% for a slurry relative density of 1.7tons/m^3.

Particle size distribution is one of the most important parameters in the design of any backfill system, as this is related not only to the backfill support capabilities, but also to the hydraulic transportation and placement behaviour of the backfill. A well-graded material will give a dense mass of interlocking particles, with high shear strength, low compressibility and low permeability. A uniformly graded material cannot be tightly packed, has a lower shear strength, higher compressibility and greater permeability. The -45μm size fraction controls the permeability of the material. However, it is considered that removal of the -10μm provides sufficient permeability to the fill material.

Backfill can be described as a low cohesion, granular medium whose shear strength is a function of the pore-water pressure. Drainage of excess water from backfill results in an increase in the mean effective stress. It is essential that the strength gain occurs as rapidly as possible after placement has been completed. The rate of drainage of excess water from the backfill is thus of major importance. Normally there is very little strength gain until the water: solids ratio drops to below about 0.23. Thereafter, there is a rapid gain in strength. The following are the most pertinent factors which affect the load-bearing characteristics of fills:

- increasing water content reduces the strength;
- the greater the porosity at placement, the weaker the fill;
- the addition of flocculants weakens a fill;
- in shallow mines and in stapes wider than 2m, confinement and strain rates are likely to be low. The most economical way to improve the stress-strain characteristics of the fill is by adding cement to increase the cohesion. Cement also increases the friction angle;
- the elastic modulus and compressive strength of cemented fills increase with curing time;
- reactive materials used in the metallurgical extraction process can adversely affect the curing of cemented fills.

Just as compressive strength is the most important property of cemented fills, permeability is the most important property of uncemented fills. A minimum percolation rate of 100 mm/h is normally required. If total tailings are used, artificial dewatering will be required.

Choice of type of backfill

The type of backfill required will depend on the mining method, on available material, and on the relative costs of the different types. Cut and fill mining will require fill that is trafficable soon after placement. Waste rock fill, or a fill capping approximately 0.5m thick, with a binder content of at least 10%, could be satisfactory for this. When mining of pillars is to take place adjacent to backfill, the fill must have inherent strength, and fill with a cement or other binder is therefore required.

With regard to costs, if paste fill is considered to be 100, typical relative costs for other types of fill are:

- paste fill 100
- cemented CCT 90
- cemented full plant tailings 85
- slurry fill 80
- uncemented CCT 70
- crushed waste fill 60
- waste rock 50

4.3 Installed support

Mining openings with different purposes have different requirements for installed support. The support requirements are determined by the time for which they must remain open, the variation in stresses to which they will be subjected, the mechanical damage to which they may be subjected, whether they have access for mine workers, and so on. The support of various types of openings is dealt with in the sub-sections below.

Distinction is made between active and passive support. Active support is support which applies a positive force or stress to the rock, for example, a tensioned rockbolt, a prestressed pack, a rapid yielding hydraulic prop. In contrast, passive support is support which only develops a resisting force when deformation of the rock takes place, for example, an untensioned grouted rockbolt (shepherd's crook), non-prestressed packs, timber poles.

Distinction is also made between temporary support, primary support and secondary support. *Temporary support* is that support, for example a mechanical prop, which is installed to ensure stability and safety in the working area so that other support and construction activities, such as the installation of primary support, can continue. Temporary support is removed when it has fulfilled its purpose. *Primary support* is the initial permanent support, for example a pattern of rockbolts, which should maintain adequate protection to the development crew during further operations. Additional support, which is installed at a later stage than the primary support, to ensure that the excavation achieves the desired long term stability, is termed the *secondary support.*

4.3.1 Support of tunnels and underground chambers

Four approaches to the evaluation of support requirements are given. The first is the use of precedent practice, the second the use of rock mass classification, the third the

determination of support for identified blocks and wedges, and the fourth is the use of stress analysis techniques.

Precedent practice: the determination of support requirements in a mine, based on what has worked well in the past, is a very pragmatic approach. It also has the advantage that mine personnel are familiar with the support elements and the methods of support installation. It is advisable that the support is reviewed, however, to ensure that it is appropriate, that safety is not compromised, and conversely that the rock is not being oversupported. Re-evaluation of the support methods is particularly applicable if geotechnical conditions change, and if new or changed mining methods are used.

Rock mass classification: the use of rock mass classification techniques to determine rock support requirements has now been practised for many years and is a sound approach. The following points should be noted:

- rock mass classification techniques are applicable to rock mass behaviour. They have limited application to massive rock and to stratified rock with limited cross jointing;
- since they are based on observed behaviour, the support derived from them in their standard form is conservative;
- since they are applicable to general rock mass behaviour, they do not take into account particular mechanisms of rock instability.

They provide a very quick and convenient method of determining support requirements. The *Q* System, in particular, facilitates this, and has been used to prepare the charts in Figures 35 and 36, which show the required rockbolt density and the shotcrete thickness. Figure 35 is generally applicable for fully grouted (cement or resin), untensioned rockbolts, and in Figure 36, wire mesh can be considered as an alternative to the shotcrete. These charts are applicable for medium to long term mining tunnels and chambers in reasonable rock conditions and in situ stress conditions (typical of conditions in many mines in the "other mines" category in South Africa). For poor rock and seismic conditions, and for particularly important excavations, more specific support design calculations should be carried out.

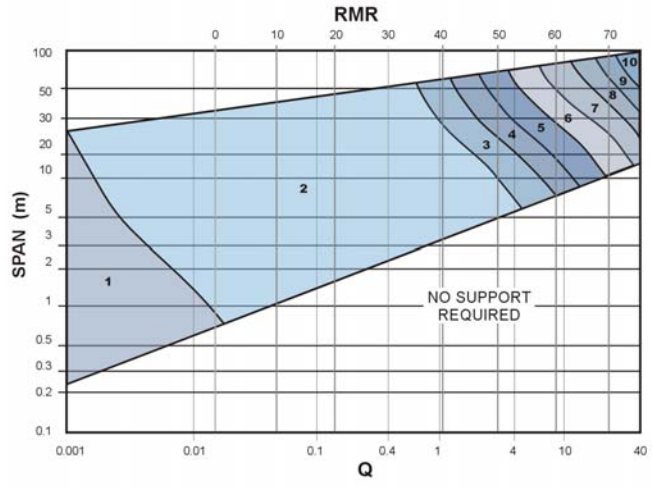

Figure 35 Rockbolt support design chart (modified[1])

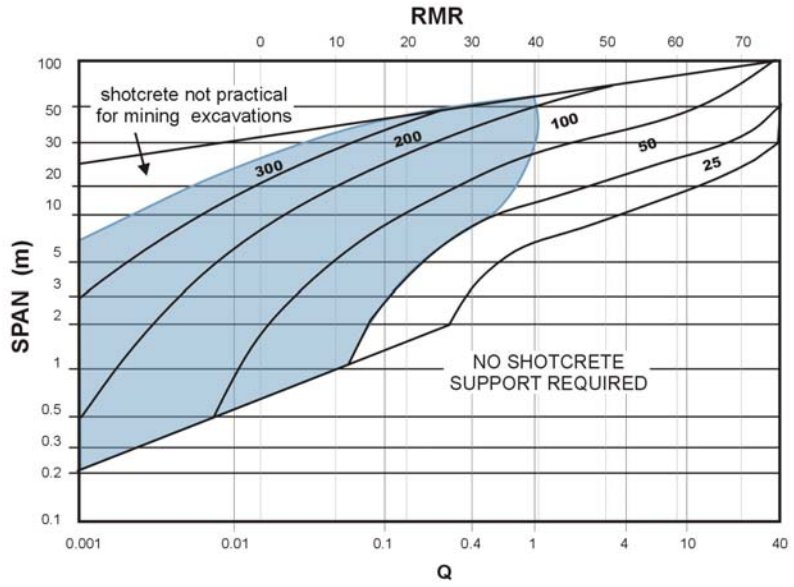

Figure 36 Shotcrete support design chart (modified[1])

The required *length of rockbolts* is usually a function of the dimensions of the opening. As a rule of thumb, for reasonable rock conditions, the length of bolts should be 0.33 times the span or the wall height. For very good rock mass conditions, this multiplier could be as low as 0.25, and for a poor quality rock mass, could be as high as 0.5 (or even higher for very poor and squeezing conditions). Cables may be required in larger excavations, and typical lengths would be 0.4 times the span or the wall height. When the hangingwall geometry results from stratification, its stability will depend on the stability of the beam of hangingwall rock. The length of rockbolts and cables used for stabilisation should ensure that the thickness of the stable beam is achieved. This length must be sufficient to ensure adequate anchorage in the stable rock above the defined beam thickness.

With regard to the required *capacity (diameter) of rockbolts,* the charts in Figures 35 and 36 are based on rockbolts with a 14 ton working load. This corresponds with 20mm diameter bolts in common use. In smaller tunnels with a span of 3.5m or less, bolts with a diameter of 16mm would be appropriate.

Shotcrete is indicated in the chart in Figure 36 as the surface support. It provides quickly installed support which prevents or at least reduces loosening of the rock mass. The use of shotcrete in mining is increasing, but many of the smaller mining operations are not geared up for routine application of shotcrete as rock support. In such cases, when the use of plain shotcrete (no mesh or fibre reinforcement) is indicated, as in the unshaded area of the chart, wire mesh, or possibly straps, may be used as an alternative surface support.

Kinematic analysis: the determination of support requirements for blocks and wedges follows directly from the evaluation of stability of such blocks and wedges. The simple computer program BlockEval can be used to evaluate the stability when rockbolt support has been installed, and hence to determine the required rockbolt support to ensure stability.

Stress analysis: Excellent computer programs for stress analysis are commercially available. Successful use of stress analysis programs requires understanding of the limitations and requirements of the particular package, and practice in its use. Some mines have regular need for such analyses and on-mine analysis is then justified. However, when such analyses are only required occasionally, it is recommended that mines contract out this work to specialists with the necessary expertise.

Types of support: The purpose of this booklet is to provide guidelines rather than details of support requirements. There are many types of rockbolts and tendons, they may be tensioned or untensioned, they may be end anchored or fully grouted, they may have mechanical anchors or they may be friction anchors. It is beyond the scope of this booklet to deal with mechanisms of action of these different supports. Similarly there are other types of supports such as steel arches and timber sets, that have particular applications. The use of arches and straps are mentioned below with regard to drawpoints, but their application cannot be covered in any detail in this brief booklet.

4.3.2 Support of drawpoints

The size and spacing of drawpoints is dictated to a large extent by the rock mass quality, since this controls the stability of excavations and pillars, and the natural fragmentation of the rock. The *MRMR* rock mass classification system17is commonly used in massive mining situations, and a guide to fragmentation size and drawpoint spacing for block caving mines, using this method, is given in Figure 37. In sublevel caving mines, drawpoint spacing must ensure that pillars of adequate integrity are present, but must also allow for efficient drilling and blasting.

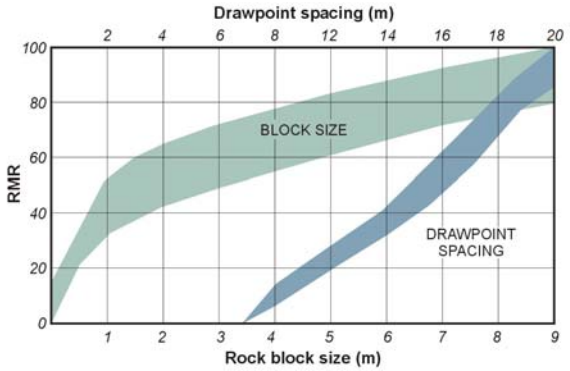

Figure 37 Guideline for fragmentation and drawpoint spacing (modified[19])

Support which is appropriate to maintain drawpoint stability for the full period of extraction, and not just to suit stability conditions at the time of development, must be installed. This is particularly important with block and panel caving methods, since drawpoints are subjected to a range of stress conditions during their life. Drawpoints must also withstand the effects of mechanical action during ore loading and secondary blasting. A general guide to the required level of support in drawpoints, corresponding with the mining environment stress and design rock mass strength *(DRMS),* is given in Figure 38[18]. This is particularly applicable to block caving operations. The mining environment stress covers the full range of stresses to which the drawpoint will be subjected.

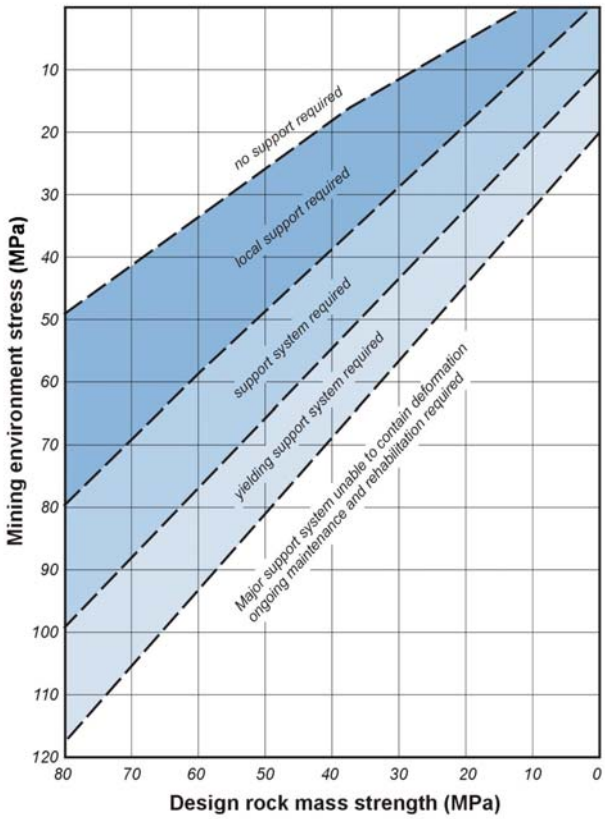

Figure 38 Guideline for drawpoint support[19]

The average support systems applicable for each of the support categories in Figure 38 are:

a) from local spot bolting to bolts at 1m spacings;
b) from bolts at 1m spacings with wire mesh or fibre reinforced shotcrete near the upper boundary, adding steel tendon straps, and then some cable bolts towards the lower boundary;
c) the full support in (b) plus yielding steel arches;
d) typically support (c) with planned rehabilitation and resupporting.

In severely deforming situations, long cables, wrapping of drawpoint mouths (noses) with tensioned wire ropes, wire rope lacing, and massive concrete arches may all be used. With mechanised loading of ore, shotcrete will normally be required to a height of about 2m to protect the support against mechanical damage by the loaders.

Similar considerations are required for mining methods such as sublevel open stoping and shrinkage stoping, in which drawpoints, located at the base of the stopes, must serve their purpose throughout the life of the stope. In mining methods such as sublevel caving and blasthole open stoping, the drilling drives also serve as drawpoints or loading drives. Compared with block caving conditions, these drawpoints are not subjected to the same range in stress conditions, nor do they require the same life. Since the mining face is retreating, the drawpoint brow is also retreating, and the support is therefore required to be effective for a shorter period.

4.3.3 Support of shafts

The method of evaluation of the stability of shafts dealt with earlier was based on the Q rock mass classification system. As indicated there, for vertical excavations $Q_{wall} = 5 \times Q_{roof}$ when $Q > 10$, $Q_{wall} = 2.5 \times Q_{roof}$ when $0.1 < Q < 10$ and $Q_{wall} = Q_{roof}$ when $Q < 0.1$. Using the value of Q_{wall} determined for the rock mass through which the shaft is passing, the charts in Figures 35 and 36 can be used to estimate support requirements. In this case the diameter, or largest dimension of the shaft, should be considered as the span on the ordinate axis~ The implication from this is that, as a guide, a 6m shaft will not require systematic support if the rock mass quality exceeds a Q value of 3 (equivalent to a Q_{wall} of 7.5 and an *RMR* of about 60). Some shaft support may be required for safety purposes, however.

It is appropriate to check for structural instability in shafts as well, since unstable blocks and wedges may require specific support.

4.3.4 Support of orepasses

Support is not usually installed in passes. However, if the evaluation of stability indicates that there may be a significant risk of pass instability, then support may be required. The requirement for support will depend on:

- geotechnical factors: rock mass quality, geological structure, in situ stresses, stress changes, rock material strength;
- construction factors: method of excavation, size, shape, and inclination;
- planning factors: desired life, tonnage to be handled, strategic importance, time between excavation and usage.

Rockbolt reinforcement has been used in orepasses, but with limited success. In blocky rock and scaling rock environments, rock tends to fall out between the bolts. Conventional rigid rockbolts are usually inappropriate, since impact from rock being passed causes vibrations in the bolt which destroys the bonding. This is not the case with fibreglass bolts and wire rope reinforcement. Rock support should be installed in upwards inclined holes so that any impact from material flowing down the pass does not contact the support at an acute angle.

In weak, fissile, scaling and closely jointed rock, a lining may be the only suitable support method. Special types of lining have been used to combat wear, including corundum and andesite lava based concretes and steel fibre reinforced shotcrete. In passes which are not sub-vertical, a greater thickness of lining on the footwall of the pass, to accommodate wear, increases the life and stability of the pass.

Precast concrete pipes, steel "tubes", and steel rails set in concrete have been used as pass liners. Any support, and steel items in particular, are "foreign material" which, when worn and loosened, can be the cause of hang-ups.

4.3.5 Support of shallow tabular stopes

Narrow tabular stopes in shallow mines in South Africa are typically in a low stress environment, with competent hangingwalls and little closure. Falls of ground that

occur are usually due to the interaction of joints and a lack of confinement. The following support elements are commonly used in the industry:

Permanent support

Mine poles: unturned timber mine poles are commonly used, with diameters ranging from 100mm to 200mm. The most commonly used diameter is about 160mm, but actual diameters of unturned mine poles vary considerably. The diameter of the mine pole affects the strength of the mine pole considerably - Figure 39 shows the results of laboratory tests, and indicates a linear relationship between peak load capacity and diameter.

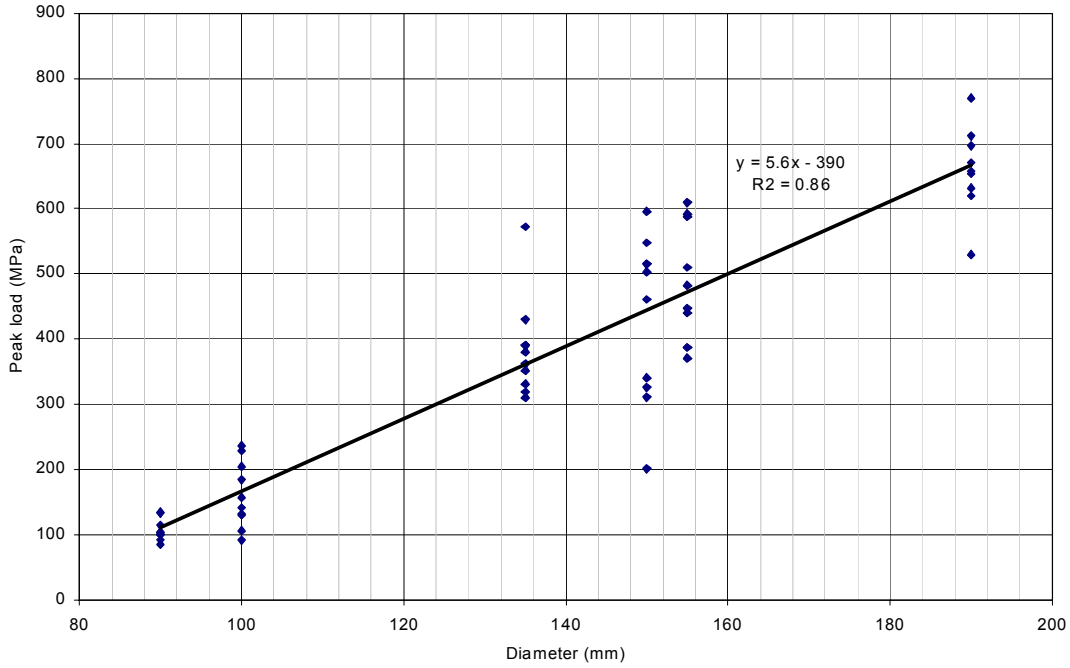

Figure 39 Peak load capacities of mine poles

Underground tests indicate that the peak load capacities in the underground situation may be as low as 40% of laboratory values[25], largely caused by timber creep and the slow loading rate underground.

Mine poles are often blasted out when installed close to the face. Pre-stressing devices can be used to prevent mine poles from being blasted out, which allows them to be installed within 3m of the face before the blast.

Timber packs are not commonly used in shallow, narrow tabular orebodies since they are costly and require large deformations to generate the required support loads.

Rockbolts and cable anchors: these support elements are becoming more popular as they can be installed closer to the face than other types of support. In narrow stopes, coupled rockbolts or cable anchors can be installed using coupled drill steels and modified airlegs to drill the holes.

Temporary support

Mechanical props can be pre-stressed and provide effective active support at the mining face.

Evaluation of support requirements for stopes

Saplings: although small diameter (<100mm) timber saplings are commonly used. The slenderness of saplings makes them very prone to buckling failure at low loads.

Demand: systematic stope support systems should be designed to carry the deadweight of a potential rockfall:

$$\text{Demand} = \rho t g$$

where ρ is the density of the rock
t is the thickness of the potential rockfall, and
g is the gravitational acceleration (assume $10 m/s^2$).

"t" should be greater than or equal to the thickness of 95% of the cumulative frequency of occurrence of rockfalls, determined from observations of past falls. If there is not enough data to form a cumulative frequency distribution, observations of brow thickness and height of wedge failures should be used to estimate "*t"*. Should the demand determined in this way prove to be too onerous, then the mining span should be reduced. This will increase the self-supporting capacity of the hangingwall and reduce the probability of large rockfalls occurring.

Capacity: the capacity of a support unit is the peak load that can be carried by the support unit. If underground performance data are not available, the peak capacity for support design purposes should be 40% of the laboratory determined strengths. Stiff support types are required, since rock displacement must be minimised. Should excessive displacements occur, mine poles may topple (fail be rotation) before they reach their compressive capacity, which is particularly the case in non-horizontal tabular stoping environments. Pre-stressing of mine poles can reduce this risk, but, in a low closure environment, mine poles can lose 50% of the pre-stressed load within 24 hours due to timber creep[3].

For consideration of the capacity of support elements, the stope should be divided into the working area and back area. The working area is the area in which routine mining operations take place and is the area between the face and a defined distance back from the face. Within this area, failure of support units should not occur. The face area is part of the working area and is the first three metres from the face, where most of the activity takes place. Beyond the working area is the back area, where entry of workers should not be permitted.

The mean underground peak load capacity of a support unit is divided by the tributary area applicable to that unit to determine its support resistance. When more than one support type is used, an area which is representative of the support system pattern should be defined. The peak underground load capacities of each unit within the representative area should be added together to determine the total load capacity in the area.

A typical *mine pole support* system layout, with pre-stressed mine poles as permanent support and mechanical props as temporary support is indicated in Figure 40. The support resistance for this support system is as follows:

Support area	Support type	Tributary Area (m^2)	Peak load (kN)	Support resistance (kN/m^2)
Working area (permanent support)	160-180mm Mine poles	4	220	55
Face area	Mechanical props	6	150	25

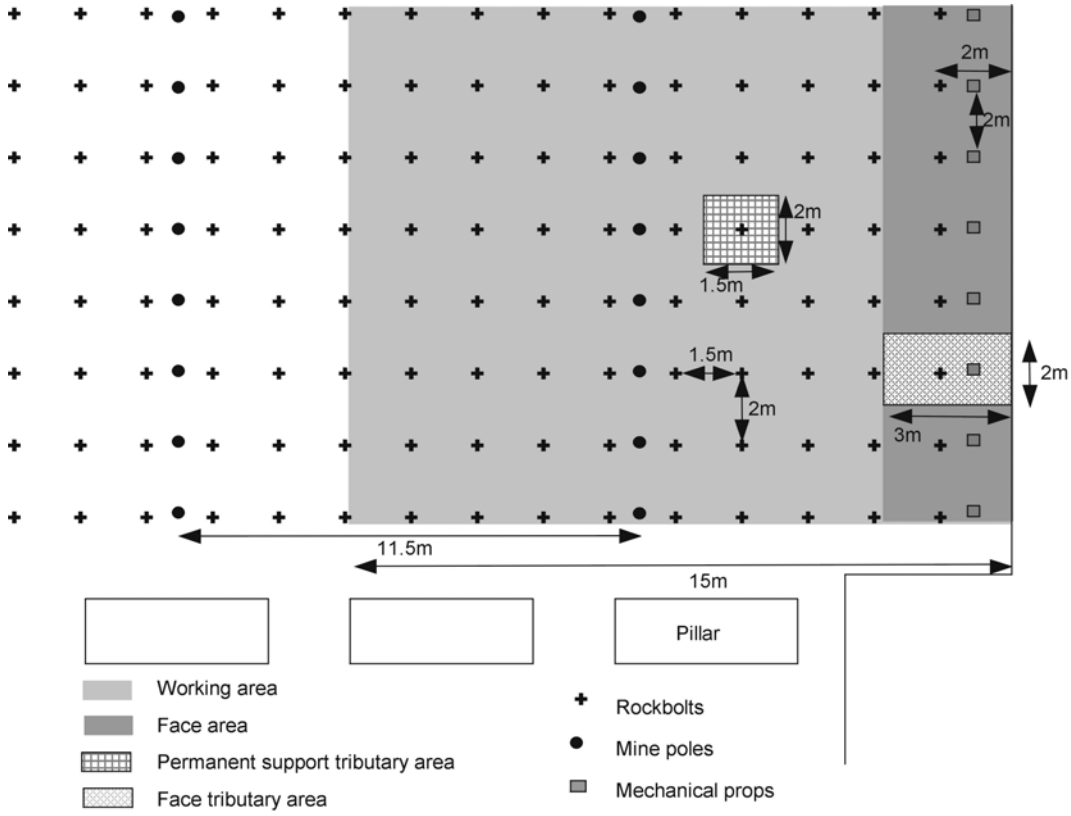

Figure 40 Typical support layout using mine poles

Assuming a rock density of $3000kg/m^3$ and rockfall thickness of 1,0m the demand is $30kN/m^2$. The permanent support provides the relatively high safety factor of 1.83. However, the face area support resistance does not meet the demand. This situation can be addressed by increasing the temporary support density at the face or by reducing the permanent support distance from the face.

An advantage of using a mine pole support system is that, when backbreaks occur, mine poles provide an early warning system in that fibres in the poles start to snap loudly, usually several hours before the backbreak occurs.

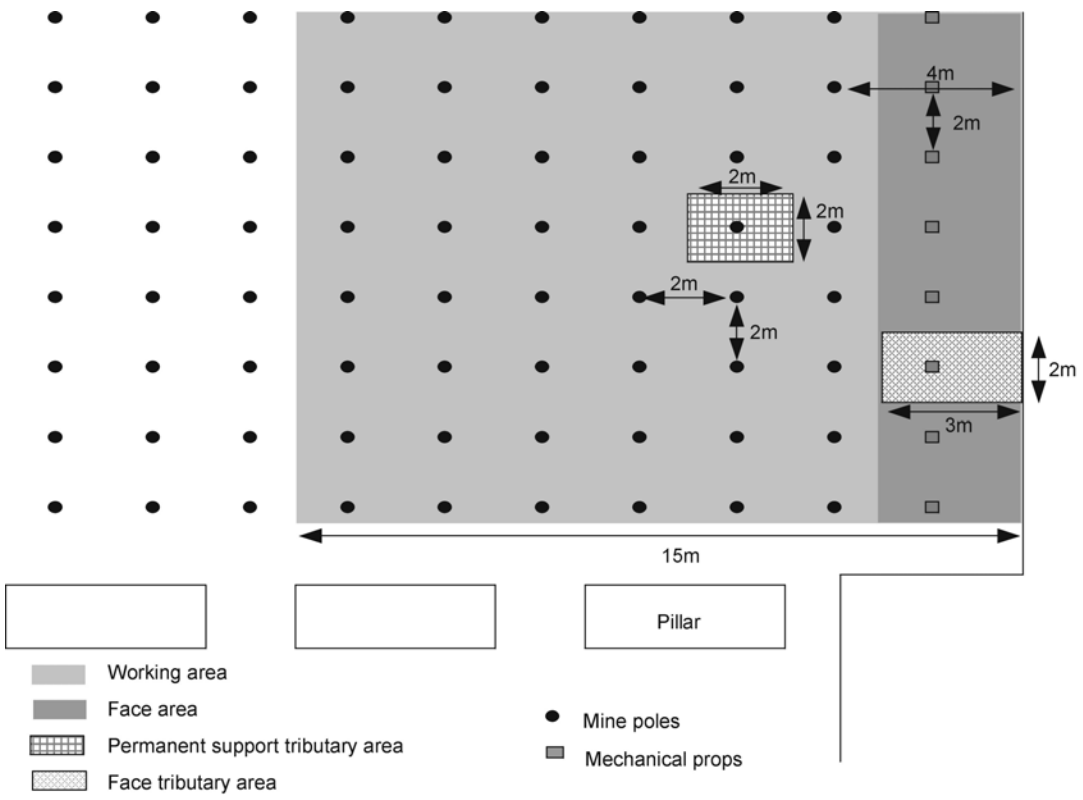

Figure 41 Typical support layout using rockbolts

A typical *rockbolt support* system support layout with rockbolts as permanent support and mechanical props as temporary support is indicated in Figure 41. Due to the lower capacity of the individual support units compared with the mine poles, a denser support layout is required. In the face area there are both mechanical props and rockbolts. The combined load can be calculated as follows:

Support type	Number of units	Peak load (kN)	Combined load (kN)
20mm, 1.2m Rockbolts	1	140	140
Mechanical props	1	150	150
Total			**290**

With reference to Figure 41, both support systems work in the face area and the combined tributary area is $6m^2$. In the working area, the tributary area is $3m^2$. The support resistance for this system is calculated below:

Support area	Support type	Tributary Area (m^2)	Peak load (kN)	Support resistance (kN/m^2)
Working area (permanent support)	20mm, 1.2m Rockbolts	3	140	46.7
Face area	Mechanical props and rockbolts	6	290	48.3

Assuming that the demand is 30kN/m^2 as above, the permanent support provides a safety factor of 1.56, which although lower than that for mine poles, is still acceptable. In the face area, the safety factor is 1.61. This support system provides high support resistance in the face area where it is needed most. In addition, support is provided on the face prior to the installation of temporary support.

The rockbolts must be at least 200mm longer than *"t"* in order to anchor above the parting plane or potential weakness. Rockbolt holes must be drilled vertically in a systematic support system to maximise use of the bolt length. Unlike the mine pole system, when rockbolts are not long enough, if backbreaks occur on a parting plane above the rockbolt anchor, the rockbolts will not provide any warning. Installing a line of pre-stressed mine poles at 10m or 12m intervals, will overcome this problem. These mine poles will also act as a breaker for small rockfalls.

The first line of rockbolts will need to be retensioned after the blast, and rockbolts in the working area should be checked and retensioned regularly if required.

Geologically disturbed areas: in addition to a systematic support system, special support will be required in geologically disturbed areas. Brows, flat dipping joints, wedges, faults, dykes, frequent joints, and shear zones all require additional support. Support should be installed close to the edge of brows, and faults and prominent joints must be supported on the weak side.

Back areas: in back areas some failure of support units and small falls of ground can be tolerated. However, should a large proportion of the support units in the back area fail, there is a risk that hangingwall failure could propagate into the working area.

Raise and Gully support: rockbolts are usually installed in gully and raise hangingwalls. During 1edging, several blasts are required before mine poles are installed, leaving a large unsupported span. It is therefore recommended that rockbolts be considered for hangingwall support during ledging.

Monitoring: when testing a new support system it is important that a risk assessment should be carried out and the performance of the support system should be monitored. Test certificates from an approved laboratory should be obtained for any new support unit. Observations of the behaviour of support units are crucial. Support units should not fail within the working area and failure in the back area should be closely monitored. Any rockfalls will need to be investigated.

4.3.6 Support of open excavations

It is not often practical to provide artificial support in open mining excavations and likely options are to flatten the pit slopes, or to introduce slope drainage measures. However, in certain cases artificial support may be essential - for example, to protect surface infrastructure adjacent to the excavation, to stabilise key components of the mining geometry such as a critical access ramp, to prevent loss of ore reserves, and to provide temporary stability during "over" deepening of an open pit. Artificial support in such situations could include the following:

- buttresses across the pit, either consisting of in situ rock left unmined, or constructed from waste rock, or, temporarily, ore;

- loading of the toe of the slope with placed rockfill;
- shear keys to increase the cohesion on actual and potential failure planes;
- tensioned cable anchors to increase the normal stress across actual or potential failure planes, and to reduce the disturbing forces.

The evaluation of the required amount and capacity of artificial support can be carried out with the techniques used for evaluating slope stability, described previously. Localised surface instability, leading to occasional falls of loose rock blocks, can be stabilised using shotcrete, or wire mesh draped over the slope face.

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APPENDIX

ROCK ENGINEERING RISK ASSESSMENT

APPENDIX

ROCK ENGINEERING RISK ASSESSMENT

The Mine Health and Safety Act requires that a rock engineering risk assessment be carried out to identify significant risks, one of which is the occurrence of rockfalls. Risk assessment can appear to be a complex, involved and confusing process. To facilitate the carrying out of a systematic and logical risk assessment, particularly by small mine operators who do not have appropriate expertise readily at hand, a simple interactive computer based assessment system (GRA - Geotechnical Risk Assessment) has been developed and is on the CD included with this booklet. The system makes use of fault-event tree methodology.

The GRA program records risk assessments that are carried out for gravity falls of ground and induced falls of ground. Gravity falls of ground can occur at any time and their occurrence is dependent on the local ground conditions, the effectiveness of the design or strategy to prevent falls of ground and the implementation of the design or strategy. Induced falls of ground occur directly as a result of carrying out specific tasks or functions. Their occurrence is dependent on the effectiveness of strategies to prevent this type of fall of ground and on the implementation of these strategies. In both cases, the exposure of workers to these falls of ground is taken into account.

The program is Windows based and has Help files available in each window. The Help files explain how to use the program and describe the information that is required in order to complete the risk assessment. Minimum hardware and software requirements are a 486 processor, with 16Mb RAM and Windows 95 software. To install the program:

- open the zip file GRA.zip with WinZip;
- run the file setup.exe and follow the instructions;
- unless otherwise specified, the file will be installed into the directory: c:\Program Files\GRA;
- click on Start, then on Programs and then on the GRA icon.

The program is simple to operate, and has Help files to provided explanations and indicate what information is required. The final output of the risk assessment is the probability of occurrence of fall of ground and of injuries. These are displayed in the final Window of the GRA. If the probabilities of occurrence are unacceptable, these numbers are red, if acceptable they are green.

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