Safety in Mines Research Advisory Committee

Final Report

Best practice rock engineering handbook for “other” mines

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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 (who know what excavations they would ideally like to have) 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 handbook provides the means of evaluating stability and support requirements simply and quickly. When the application of these simple and quick methods is considered to be insufficient, appropriate "best practice" methods are outlined. In addition, a process for identifying rock related hazards is provided. This process, and the evaluation of stability and support requirements, will serve as direct inputs to the Code of Practice to combat rockfalls. Example codes of practice, and a standardised system for auditing the implementation of a code of practice, are included.

The scope of application of this handbook includes "other mines" (both underground and surface) and officially excludes gold, platinum and coal mines. However, the material contained in the handbook is considered to be applicable, at least in part, to these mines as well. The consideration of open pit mines and quarries includes both rock slopes and slopes in weathered material.

The aim is to provide a means of ensuring that sound rock engineering practices, which will minimise rock related accidents, are being followed. Whilst the philosophy of preparation of the handbook has been to preserve the "best practice" concept, **it is not a textbook** - material has deliberately been summarised and simplified to make the handbook as user friendly as possible. Only material that is considered to be directly useful is included in the handbook. Background information such as basic rock mechanics principles, details of laboratory rock testing, rock material failure criteria, etc have not been included since they can easily be sourced from numerous rock mechanics text books. It is hoped that this handbook will be used by managers and others who have not specialised in the rock engineering field.

The approach to excavation design and stability evaluation follows a straight forward 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 handbook has been structured logically to deal firstly with rock mass behaviour. This is followed by stability evaluation (which takes into account a range of excavation geometries), and 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 areas or domains, which is an essential requirement in a Code of Practice to combat rockfalls.

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).

Advanced techniques for design and analysis, such as stress analysis, kinematic analysis and probabilistic approaches, are described briefly. In addition, factors of specific influence are also dealt with, such as the effect of blast damage on stability of underground stopes and pit slopes, and the effect of groundwater on stability of slopes.

2 Rock mass characterisation

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. This may be scale dependent - for example, a tunnel in a certain rock mass with widely spaced geological structure may be unaffected by that structure, but the behaviour of an open stope in the same rock mass may be influenced significantly by the structures;
- 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 must be carried out to determine which of these conditions is applicable. It will also determine which method of stability analysis or design is most appropriate.

Rock mass characterisation is therefore fundamental to the planning of any mining operation. It is the basis of the definition of geotechnical areas or domains, required for the preparation of a code of practice to combat rockfalls; for the evaluation of rock mass strength and deformation behaviour; for the identification of the most likely modes of potential rock mass failure; for the evaluation of stability of the rock mass; and for the evaluation of the requirement for support of the rock mass.

In this chapter, the following will be dealt with:

- **The Required Geotechnical Database**: this defines the geotechnical data which should be collected, and recommended methods of collecting the data. Site investigations, geotechnical logging of borehole core, mapping of joints, rock mass classification mapping including recording of joint properties, and requirements for laboratory testing of rock samples are dealt with.

- **The Boundary Conditions**: evaluation or estimation of in situ stress is important since the in situ stress is an essential boundary condition for the evaluation of stability, particularly if a method of stress analysis is used in this evaluation. Other boundary conditions may also be relevant such as, for example, water pressures, dynamic loading, etc.

- **Rock Mass Quality**: rock mass classification has developed as a means of quantifying the quality of a rock mass. This numerical value for the quality of the rock mass has been correlated with many rock mass properties, including stability, support requirements, strength and deformability. **Extensive use will be made of rock mass classification in this handbook.**

- **The Geotechnical Model**: the recording and presentation of the geotechnical data collected is important to the initial and ongoing use of the information. To be of most use, the information must be well presented and readily available. The basic and interpreted data may be recorded manually on plans, but the trend is towards computerised storage in a form that is compatible with mine planning. This can be termed a **geotechnical domain model**.

2.1 Collection of geotechnical data

2.1.1 Site investigations

To provide data for the satisfactory planning and mining of safe excavations, information must be obtained on which basis the rock mass will be characterised. Typically, 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

In many cases exploration boreholes will have been drilled to provide ore reserve data as part of pre-feasibility, feasibility and on-going mining stages. The core represents a source of geotechnical information, and the standard geological exploration logs may provide some geotechnical information of value. In particular, these logs will almost certainly define major structural features such as faults, which is critical information for structurally controlled stability situations. In most cases, however, it is necessary to relog the core specifically for geotechnical purposes. For this purpose, a rapid geotechnical core logging technique, which has been used successfully on large mining projects, is recommended.

In the rapid core logging technique (Dempers, 1994), 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 1facilitates the activity. The sequence of vertical columns in Figure 1 is as follows:

Drilling record

The driller's drill run intervals.

Recovery

This column is divided into three sub-columns, namely, the measured distance drilled in metres (m Drilled), the measured recovery per drill run (m Rec) and the percentage recovered, which is the metres recovered divided by the metres drilled expressed as a percentage (% Rec).

Rock Quality Designation (*RQD*)

The standard rock quality designation is recorded, both in metres and as a percentage, which is calculated as follows:

$$RQD\% = \frac{Total\ length\ of\ core \geq 100mm\, x\, 100}{Length\ of\ core\ run}$$

Geotechnical interval

The length of rock core which displays similar geotechnical characteristics with regard to expected excavation performance, measured in metres down the borehole, is recorded. The geotechnical interval can extend over many metres depending on the geological complexity of the rock types encountered.

Rock type

The rock type present in the geotechnical zone measured.

Rock competence

This column is divided into solid, matrix and matrix type. After the geotechnical interval and rock type have been recorded, the quantity of rock that is solid and that which is matrix within a geotechnical zone is measured. Matrix is identified as material which includes broken rock and material that derives from faulted, sheared or deformable horizons. Geotechnical parameters for solid and matrix are recorded separately excluding jointing distribution relative to the core axis which can normally only be determined for solid core. The category of matrix present is also recorded, the categories being:

M1 - Faults
M2 - Shears
M3 - Intense fracturing
M4 - Intense mineralisation
M5 - Deformable material

Weathering

The degree of weathering of the rock is rated on a scale from 1 – 5 as follows:

1 - Unweathered
2 - Slightly weathered
3 - Moderately weathered
4 - Highly weathered
5 - Completely weathered

Hardness

The rock material hardness is rated on a 5 class scale:

1 - Very soft
2 - Soft
3 - Hard
4 - Very hard
5 - Extremely hard

Jointing distribution relative to core axis

Jointing inclination relative to the core axis, as shown in Figure 2, is determined by counting the number of joints (complete separations) that occur within a geotechnical zone, for each range of joints i.e. 0° to 30°, 30° to 60°, 60° to 90°. The total number of joints is also recorded.

Joint surface condition

The joint surface condition is an assessment of the frictional properties of the joints (not man-made or drilling fractures) and is based on surface properties, alteration zones, joint infilling and water. The macro and micro conditions of the joints are recorded from a scale of 1 to 5 and 1 to 9 respectively as follows:

Macro Roughness	**Micro Roughness**
1 – Planar	1 – Polished
2 – Undulating	2 - Smooth Planar
3 – Curved	3 - Rough Planar
4 – Irregular	4 - Slickensided Undulating
5 – Multi Irregular	5 - Smooth Undulating
	6 - Rough Undulating
	7 - Slickensided Stepped
	8 - Smooth Stepped
	9 - Rough Stepped/Irregular

The last two columns relating to joint surface condition are for recording the infill found in the joint and the joint wall alteration.

Joint infill and joint wall alteration are recorded on scales of 1 to 8 and 1 to 3 respectively.

Joint Infill		**Joint Wall Alteration**
Non-softening	Coarse-8	Wall = Rock Hardness - 1
and sheared	Medium -7	Wall > Rock Hardness - 2
Material	Fine-6	Wall < Rock Hardness - 3
Soft sheared	Coarse-5	
material, e.g.	Medium -4	
talc, clay	Fine-3	
Gouge thickness		
< amplitude of irregularities-2		
Gouge thickness		
> amplitude of irregularities-1		

Input to rock mass classification

Data from the geotechnical log can be input directly into rock mass classification systems, which will be dealt with in Chapter 3.

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.

Joint mapping

A detailed description of methods of joint mapping is not appropriate for this handbook, and it is considered that such detailed mapping will not normally be necessary. 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 and their variability for each joint set:

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

From studies of jointing in different parts of the world, typical statistical distributions of these parameters have been reliably established. These distributions are:

- joint dip angle - normal distribution;
- joint dip direction - normal distribution;
- joint spacing - log normal;
- joint length - negative exponential.

Typical examples of these distributions are shown in Figure 3. Owing to this knowledge of the distributions, the amount of joint data that needs to be collected can be reduced. If the mean, minimum and maximum values can be defined by mapping, then they can be used with the established statistical distributions to create a satisfactory joint data set. This data set will then provide a sufficient basis for deterministic and probabilistic analysis of stability to pre-feasibility level accuracy. In some cases the accuracy might be much better than this.

Rock mass classification mapping

A less rigorous format than the systematic joint mapping described above, but equally effective for the experienced geomechanics practitioner, is the application of rock mass classification mapping. The important aspect of this approach is to ensure that the required input data for good quality rock mass classification is obtained. The recommended way in which this can be achieved is to use a standardised rock mass description sheet, an example of which is shown in Figure 4. 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. The example sheet in Figure 4 caters for three methods of rock mass classification, which are dealt with in Section 2.3 below.

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.

The following tests are recommended:

- Uniaxial compressive strength (*UCS*). The *UCS* test is probably the most common laboratory test carried out, and serves as the “characterisation” test;
- Point load tests. These are carried out directly on unprepared core, are very quick, and, since their results have been correlated with *UCS*, can effectively extend the *UCS* database;

- Brazilian tensile strength tests. These tests are quick and easy to perform and will provide the tensile to compressive strength ratio, an indication of the brittleness of the rock;
- Shear strength tests. These are only recommended in certain cases and could include shear tests on cut surfaces, to determine the base friction angle, and possibly shear tests on natural joint surfaces;
- If failure through intact rock material is a possibility, some triaxial compressive strength tests may be appropriate to determine the rock strength improvement with confinement and hence a triaxial strength failure criterion;
- If the rocks are suspected of being non-durable, slake durability testing should be carried out to determine their susceptibility. Rocks such as Kimberlite, shale, mudstone and basalt can be non-durable, and their behaviour will be an important consideration in evaluation of stability and the support required.

2.2 In situ stress conditions

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

- if the stress is low, instability may occur since rock blocks may have the freedom to fall out;
- if the stress is higher, the rock mass will be well confined and stability will be achieved;
- if the stress level is sufficiently high, 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 (Stacey and Wesseloo, 1998). Some of the information from this database is summarised in Figure 5 to Figure 8. In the first instance it is recommended that an indication of the in situ stress levels that might be applicable at the mine are obtained from this database. If it is found that stability is critically dependent on the confining stress (in the stability evaluation, dealt with in Chapter 3 below), then it may be necessary to carry out a programme of in situ stress measurements at the mine.

Under high stress conditions it is possible to obtain an indication of in situ stress magnitudes from observations of rock behaviour in boreholes and tunnels. Spalling and slabbing on the surfaces of these openings will indicate the orientations of the principal stresses and may allow an estimate to be made of the major stress magnitude. In most cases of "other mines", however, it is expected that there will be no observable stress effects.

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.

Quantitative classification of rock masses has become almost routine, since it provides a rapid means of quantifying the quality of a mass, comparing qualities, and assessing support requirements. Classification applied on a routine basis can have tremendous value in mines.

Two classification methods have stood out, the *Q* System developed by Barton et al (1974) and the Geomechanics Classification System developed by Bieniawski (1989). A system specifically for mining applications, based initially on Bieniawski's method, but now independent, has been developed by Laubscher and Taylor (1976) and refined by Laubscher (1994). A method refined by Potvin (1988), which is directly applicable to the evaluation of stability of open stopes, is a modification of the *Q* system. These four approaches are described in the following sections. A short cut approach is then given to enable very quick estimates to be obtained, and users of this short cut must recognise that it is just that – the recommended approach is to apply more than one classification method diligently in each case.

2.3.1 *Q* System

The *Q* System classification is based on three aspects:

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

Where: RQD is the rock quality designation

J_n is the joint set number

J_r is the joint roughness number

J_a is the joint alteration number

J_w is the joint water reduction factor

SRF is the stress reduction factor.

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

***RQD* – Rock Quality Designation**

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

$$RQD\% = \frac{Total\ \ length\, of\ core\ \geq 100mm\, x\, 100}{Length\, of\ core\, run}$$

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

RQD can be estimated from inspection of exposed rock surfaces by determining the number of unhealed joint planes per m^3 of rock. This may be done by counting the relevant number of joint planes (excluding blast fractures) which cross a 2 to 3m length of tape held against the excavated wall. The number of joint planes divided by the relevant sample length gives the number of joints per metre. This process is then to be repeated for 2 additional directions. The sum of these three values gives J_v, the number of joints per m^3, and hence *RQD* from the equation (Barton et al 1974):

$$RQD = 115 — 3.3\ J_v$$

Notes:

- Where *RQD* is reported or measured as less than 10, a nominal value of 10 is used to evaluate *Q*.
- *RQD* intervals of 5, giving 100, 95, 90 … 10 are sufficiently accurate.
- J_n - Joint Set Number
- A numerical value is allocated, corresponding with the number of joint sets present in the rock mass.

Table 1 Joint set number

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

Notes:

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

J_r - Joint Roughness Number

Distinction is made between the large scale nature of planes and the small scale roughness as well as between continuous and discontinuous joints. Table 2 gives joint roughness number (J_r) values.

Table 2 Joint Roughness Number

Description of Joint Surface Roughness	**Discontinuous**	**Undulating**	**Planar**
Rough	4.0	3.0	1.5
Smooth	3.0*	2.0	1.0
Slickensided	2.0*	1.5	0.5
Planes containing gouge thick enough to prevent rockwall contact	1.5*	1.0	1.0

* Data added to original sequence of Barton et al (1974).

Note:

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

J_a - Joint Alteration Number

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

Table 3 Joint Alteration Number

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

*Figures added to original data of Barton et al (1974) to complete the sequence

Notes:

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

J_W - Joint Water Reduction Factor

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

Table 4 Joint Water Reduction Factor

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

Notes:

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

SRF **- Stress Reduction Factor**

SRF values for weakness zones and competent rock are dealt with separately below.

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

Table 5 Stress Reduction Factor for weakness zones

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

Notes:

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

- Competent rock, rock stress problems

Table 6 Stress Reduction Factor for competent rock and rock stress problems

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

Calculation of *Q*

All selected values for the above six parameters, based on observed or estimated conditions are substituted into the equation to obtain the value of the rock quality index *Q*.

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

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

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

2.3.2 Modified stability number, *N′*

This system is a modification of the *Q* System. The parameter *SRF* (the stress factor) is not used, and three specific multiplying factors are applied to take particular account of rock strength to stress effect, joint orientation, and gravity. Initially *Q′* is calculated as in the *Q* System as:

$$Q' = RQD/J_n \; x \; J_r /J_a \; x \; J_w$$

Dry conditions are experienced in most underground hard rock environments and *Q′* then reverts to:

$$Q' = RQD/J_n \; x \; J_r /J_a$$

The modified stability number *N′* is calculated as:

$$N' = Q' x \; A \; x \; B \; x \; C$$

where:

A is a factor which allows for the strength to stress effect. *A* is given by:

$$A = 1.125R - 0.125 \qquad 1>A>0.1$$

where

R is the ratio of the uniaxial compressive strength of the rock material to the maximum induced compressive stress. The latter is determined by stress analyses.

B is a factor which allows for the ease of block fall out effect. *B* is given by the following equations:

$$B = 0.3 - 0.01 < \alpha < 10$$

$$B = 0.210 < \alpha < 30$$

$$B = 0.02\alpha - 0.430 < \alpha < 60$$

$$B = 0.0067\alpha + 0.460 < \alpha < 90$$

where α is the true angle between the "hanging" surface of the excavation and the joint plane. In the case of several joint planes, the smallest angle is applicable. The true angle can be determined using a stereonet.

C is the gravity adjustment factor. In the case of gravity falls and slabbing, in which no sliding on joints is involved, the factor is given by the following equation:

$$C = 8 - 6 \text{ x } \cos \text{ (dip of stope face)}$$

When sliding on joints is involved, the gravity adjustment factor is given by the following equations:

C = 8	Dip of critical joint < 30^o
C = 11 – 0.1 x Dip of critical joint	Dip of critical joint > 30^o

2.3.3 Geomechanics Classification

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

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

The descriptions and corresponding ratings for these parameters and the joint orientation adjustment are given in Table 7.

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

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

A relationship has been found between *RMR* and *Q* as follows (Bieniawski, 1989):

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

Table 7 Geomechanics Classification

	Parameter		Ranges of Values						
1	Strength of Intact rock material	Point load strength index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>250 MPa	100-250 MPa	50-100 MPa	25-50MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality *RQD*		90%-100%	75%-90%	50%-75%	25%-50%	<25%		
	Rating		20	17	13	8	3		
3	Spacing of joints		>2m	0.6-2m	200-600mm	60-200mm	<60mm		
	Rating		20	15	10	8	5		
4	Condition of joints		Very rough surface Not continuous No separation Weathered wall Rock	Slightly rough Surfaces Separation <1mm Slightly weathered Walls	Slightly rough Surfaces Separation <1mm Highly weathered Walls	Slickensided surfaces, or Gouge<5mm thick, or Separation 1-5mm continuous	Soft gouge >5mm thick, or Separation >5mm continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10m Tunnel length (l/min)	None	10	10-25	25-125	>125		
		Joint water press-ure/major principal stress	0	0.0 – 0.1	0.1-0.2	0.2-0.5	>0.5		
		General Conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		
6	Strike and dip orientations of joints*		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable		
	Rating		0	-2	-5	-10	-12		

*The Effect of Joint Strike and Dip Orientation in Tunnelling

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

2.3.4 Mining Rock Mass Classification

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

- rock material strength (*UCS*)
- *RQD*
- joint spacing
- joint condition and ground water.

Rating values for each of these parameters are given in Table 8 below. Adjustments for the joint condition and groundwater parameter in Table 9 are cumulative. For example, a dry straight joint with a smooth surface and fine soft-sheared joint filling would have a minimum rating of 40 x 70% x 60% x 60% = 10.1. The mining rock mass rating *MRMR* value is obtained by summing the four parameter ratings. The range of *MRMR* lies between zero and 100.

Correlation between *MRMR* and *Q* is adequately represented by the equation between *Q* and *RMR* given in Section 2.3.3.

The mining rock mass classification is better suited to real stability assessment since it is also concerned with cavability.

Table 8 Mining Rock Mass Classification

	Parameter	Range of Values										
1	*RQD*	100-97		96-84	83-71	70-56	55-44	43-31	30-17	16-4	3-0	
	Rating (= *RQD* x 15/100)	15		14	12	10	8	6	4	2	0	
2	*UCS* (MPa)	185	184-165	184-165	164-145	144-125	124-105	104-85	84-65	44-25	24-5	4-0
	Rating	20	18	16	14	12	10	8	6	4	2	0
3	Joint Spacing	Refer Figure 9										
	Rating	25						0				
4	Joint Condition Including Groundwater	Refer Table 9										
	Rating	40						0				

__Table 9 Adjustments for Joint Condition and Groundwater__

Parameter	Description		Dry Condition	Wet Conditions: Moist	Wet Conditions: Moderate pressure 25-125 1/mm	Wet Conditions: Severe Pressure >125 1/min
A Joint Expression (large scale irregularities)	Wavy	Multi-Directional	100	100	100	95
	Wavy	Uni-Directional	95 90	95 90	90 85	80 75
	Curved		89 80	85 75	80 70	70 60
	Straight		79 70	74 65	60	40
B Joint Expression (small scale irregularities or roughness	Very rough		100	100	95	90
	Striated or rough		99 85	99 85	80	70
	Smooth		84 60	80 55	60	50
	Polished		59 50	50 40	30	20
C Joint Wall Alteration Zone	Stronger than wall rock		100	100	100	100
	No alteration		100	100	100	100
	Weaker than wall rock		75	70	65	60
D Joint Filling	No fill – surface staining only		100	100	100	100
	Non softening and sheared material (clay or talc free	Coarse Sheared	95	90	70	50
		Medium Sheared	90	85	65	45
		Fine Sheared	85	80	60	40
	Soft sheared material (eg talc)	Coarse Sheared	70	65	40	20
		Medium Sheared	65	60	35	15
		Fine Sheared	60	55	30	10
	Gouge thickness <amplitude of irregularity		40	30	10	
	Gouge thickness <amplitude of irregularity		20	10	Flowing material 5	

Adjustments are applied to the *MRMR* value to take account of weathering of the rock mass, joint orientation relative to the excavation, mining-induced stresses and blasting effects. The magnitudes of these adjustments are described below.

A weathering adjustment is relevant when rock types occur which are susceptible to deterioration over time. The adjustment percentages are given in Table 10 below (Laubscher, 1993).

Table 10 Weathering Adjustment

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

In explanation of the adjustments given in this table, if the rock will weather to a residual soil in 6 months, the adjustment is 30%; if it will be only slightly weathered after 3 years, the adjustment is 94%.

The orientations of the joints, and whether the bases of blocks formed by the joints are exposed, have a significant effect on excavation stability. The joint orientation adjustment depends on the orientations of the joints with respect to the vertical axis of the block. The adjustment percentages are given in Table 11 below (Laubscher, 1993).

Table 11 Adjustments 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

A shear zone or a fault can have a significant influence on stability. Adjustments applicable, with respect to the dip of the feature relative to the development, are 0° to 15° – 76%; 15° to 45° – 84%; 45° to 75° – 92%.

The stresses acting around the mining excavations have an effect on the stability of those excavations as described in Section 2.2 above. The adjustments for mining induced stresses are essentially based on judgement. 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%.

The quality of blasting has an influence on the fracturing and loosening of the rock mass. Adjustments for blasting effects are given in Table 12 below (Laubscher, 1993).

Table 12 Adjustments for Blasting Effects

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

The above 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

2.3.4.1 Rock mass strength (*RMS*) and design rock mass strength (*DRMS*)

As part of the mining rock mass classification system, a rock mass strength (*RMS*) and design rock mass strength (*DRMS*) can be calculated (Laubscher, 1990). The *RMS* is determined from the *UCS* of the rock and the *MRMR* of the rock mass, using the relationship:

$$RMS = UCS \times (MRMR - R_{UCS})/100$$

Where R_{UCS} is the rating value in Table 8 corresponding with the rock *UCS*.

The *DRMS* is the rock mass strength in a specific mining environment. It is obtained by applying adjustments to the *RMS* to take into account the effects of weathering, joint orientation and method of excavation. These adjustments are described in Section 2.3.4 above.

2.3.5 Shortcut to rock mass classification

To obtain a very quick indication of rock mass conditions, Figure 10 can be used to estimate a value of *RMR* based on visual assessment of the rock mass. The adjustment for strike and dip orientations of joints (Item 6 of Table 7) should be applied to this value. A *Q* value can then be calculated from the correlation equation between *RMR* and *Q* given in Section 2.3.3. An *MRMR* equivalent will be between 5 and 10 points lower than the *RMR* determined from Figure 10.

In spite of the suggested use of a shortcut method, the recommended approach remains that the rock mass classification procedures described above 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.

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 13.

Table 13 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

ORGANISATION : **PROJECT :** **HOLE NO :** **Page: of**
SITE : **INCLINATION :**
GEOTECHNICAL BOREHOLE LOG **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			0°-30°	30°-60°	60°-90°	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
Unweathered
Slightly
Moderately
Highly
Completely

HARDNESS
Very soft
Soft
Hard
Very hard
Extremely hard

JOINT SURFACE
MICRO ROUGHNESS
Polished
Smooth planar
Rough planar
Slickensided undultating
Smooth undulating
Rough undulating
Slickensided Steppped
Smooth stepped
Rough stepped/irregular

MACRO ROUGHNESS
Planar
Undulating
Curved
Irregular
Multi irregular

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

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

Figure 1 Geotechnical core logging sheet

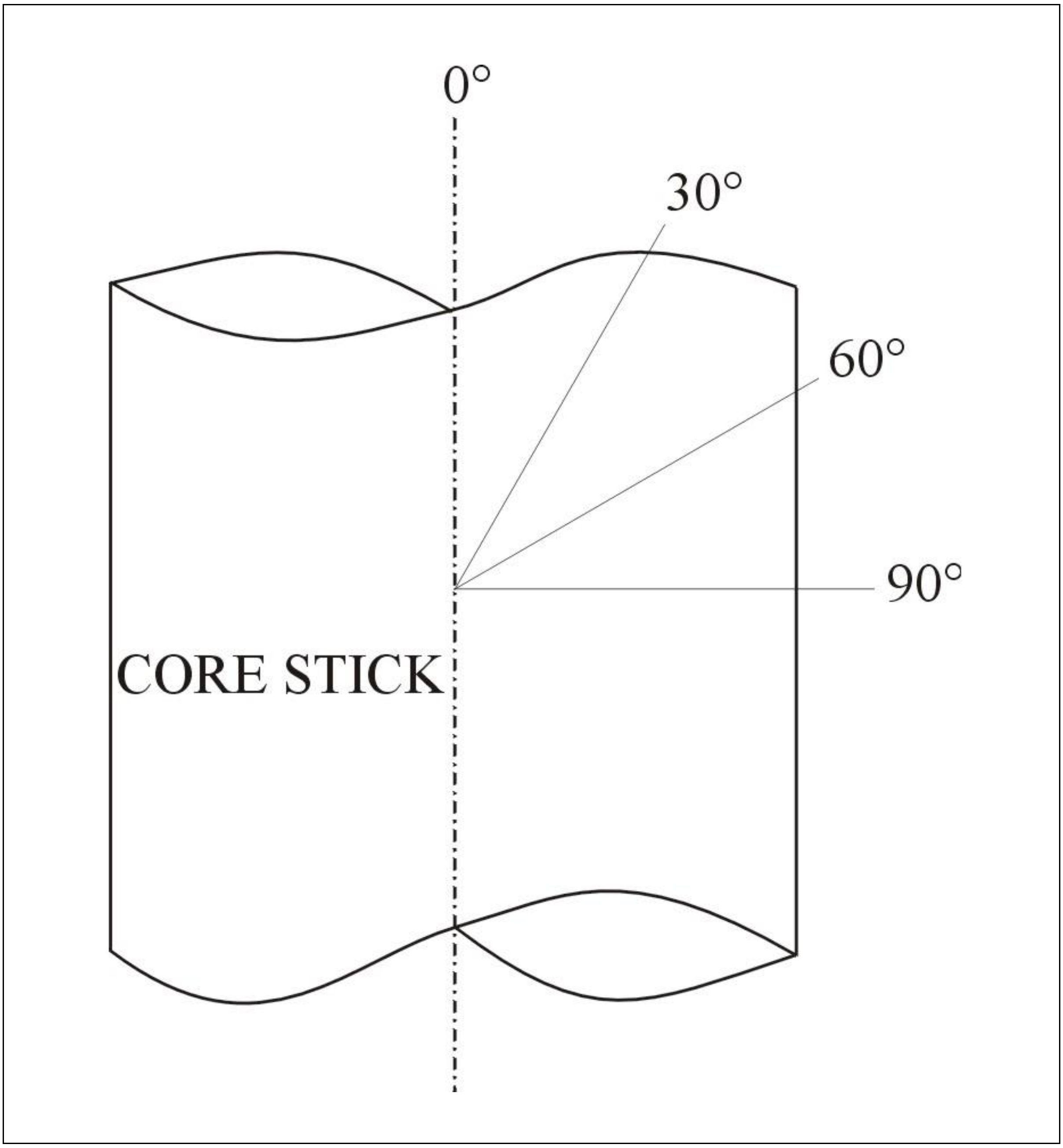


Figure 2 Joint inclination relative to core axis

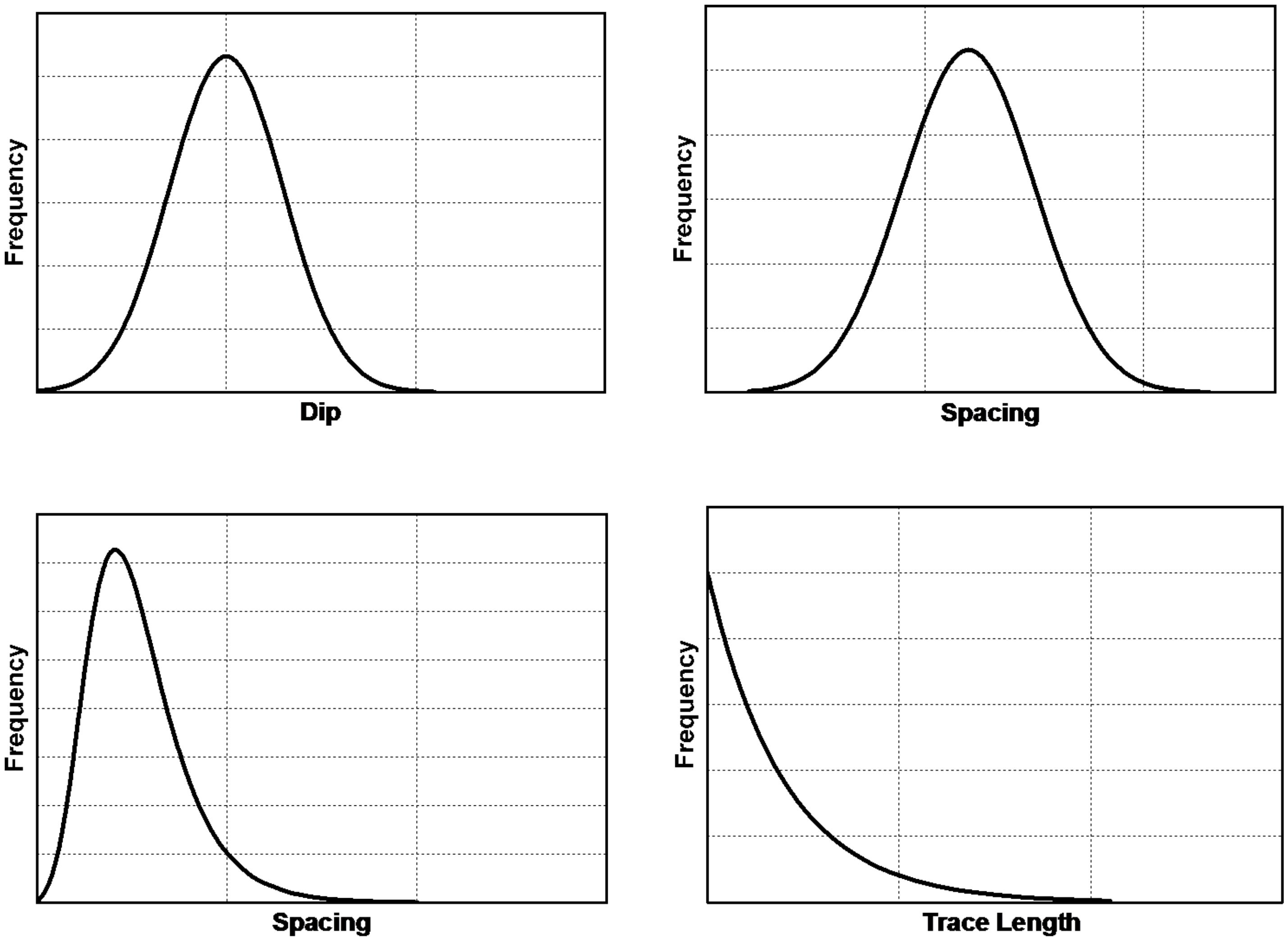


Figure 3 Typical Statistical distributions of joint characteristics

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

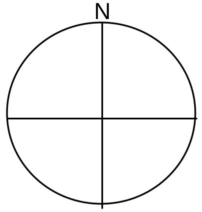
Jc =

RQD equivalent

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

RELATIVE JOINTING ORIENTATIONS

N



COMMENTS

Figure 4 Rock mass classification field mapping sheet

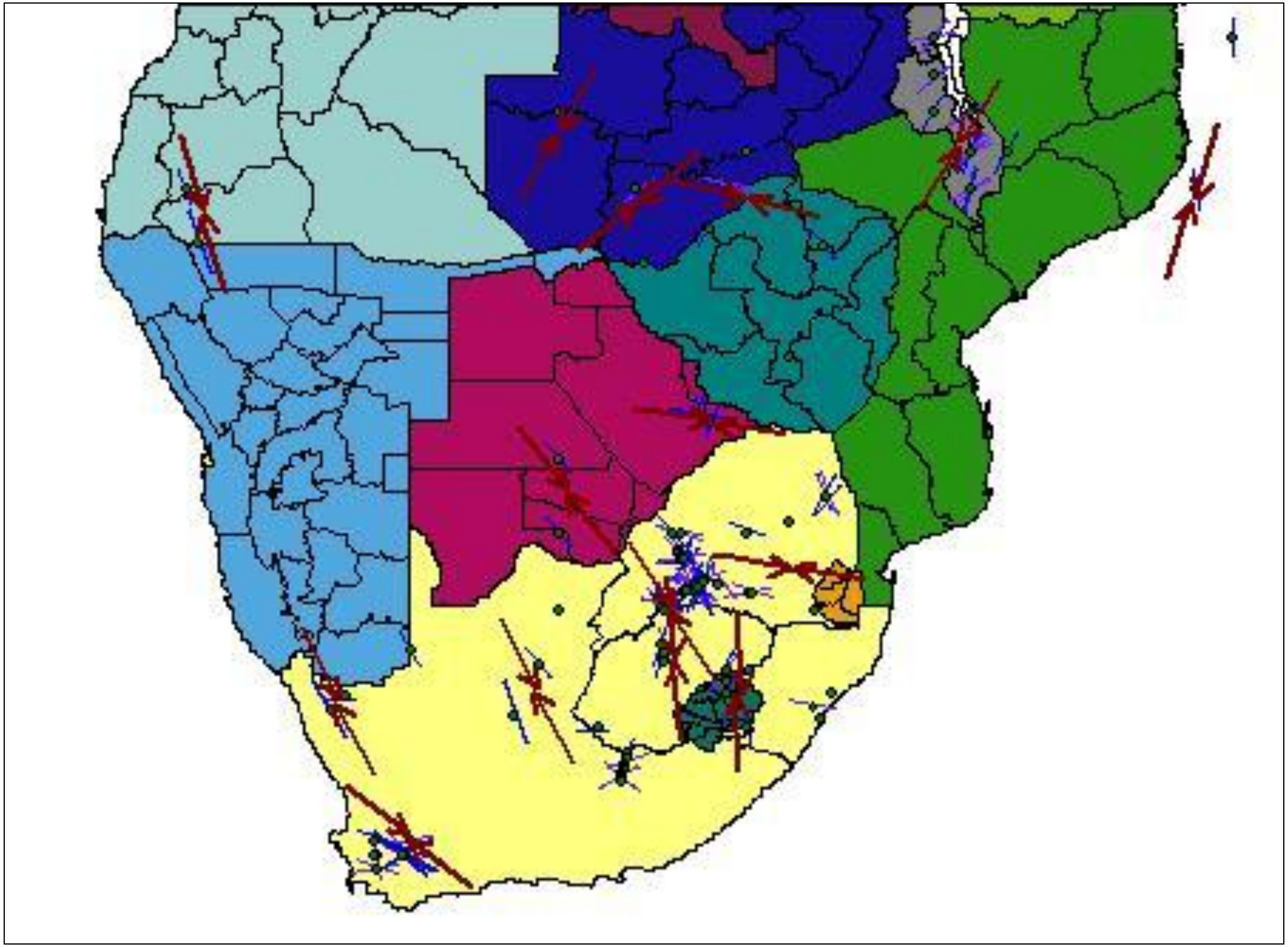


Figure 5 Orientation of horizontal stresses

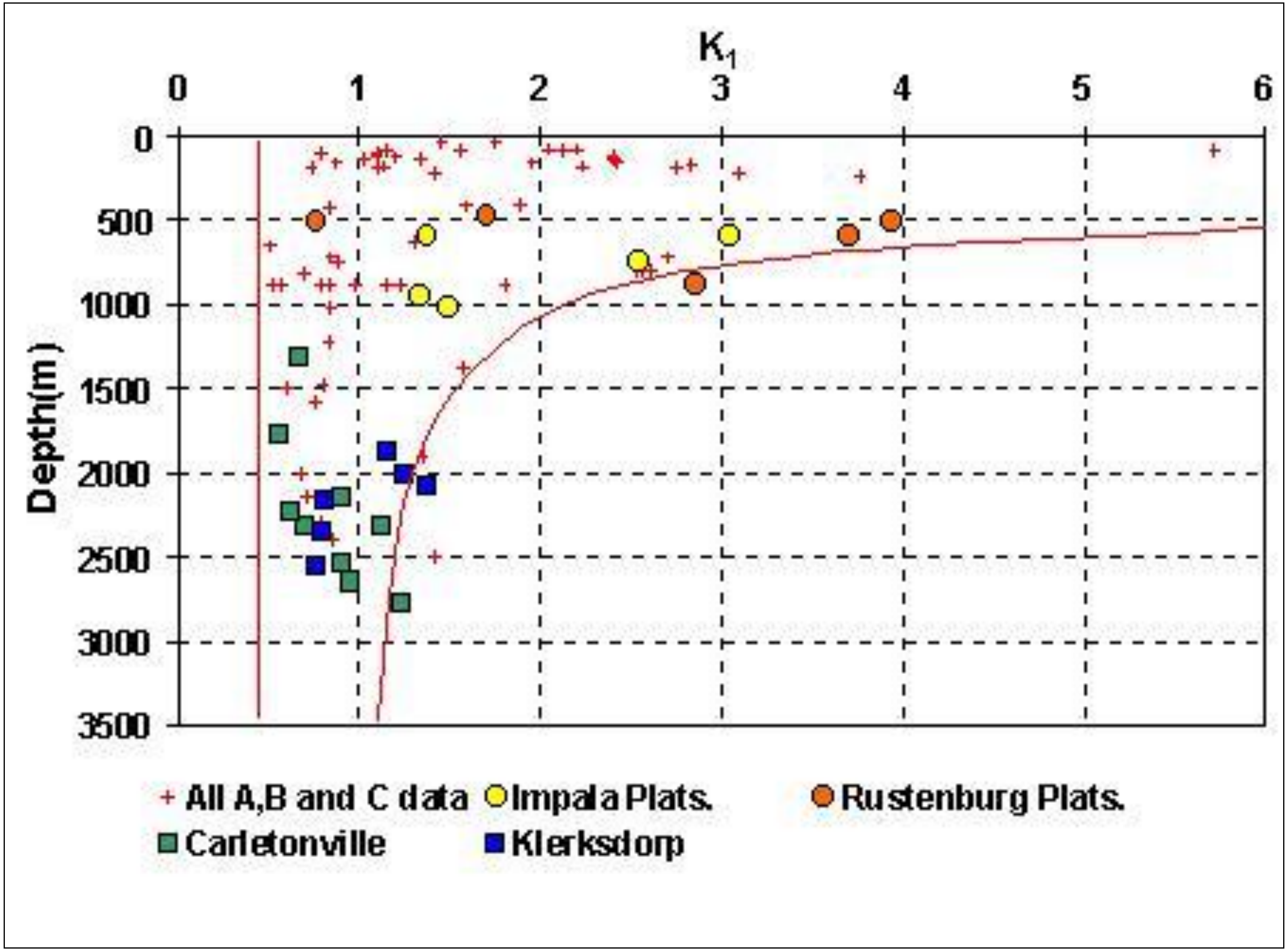


Figure 6 Ratio K_1 of major horizontal in situ stress to vertical stress

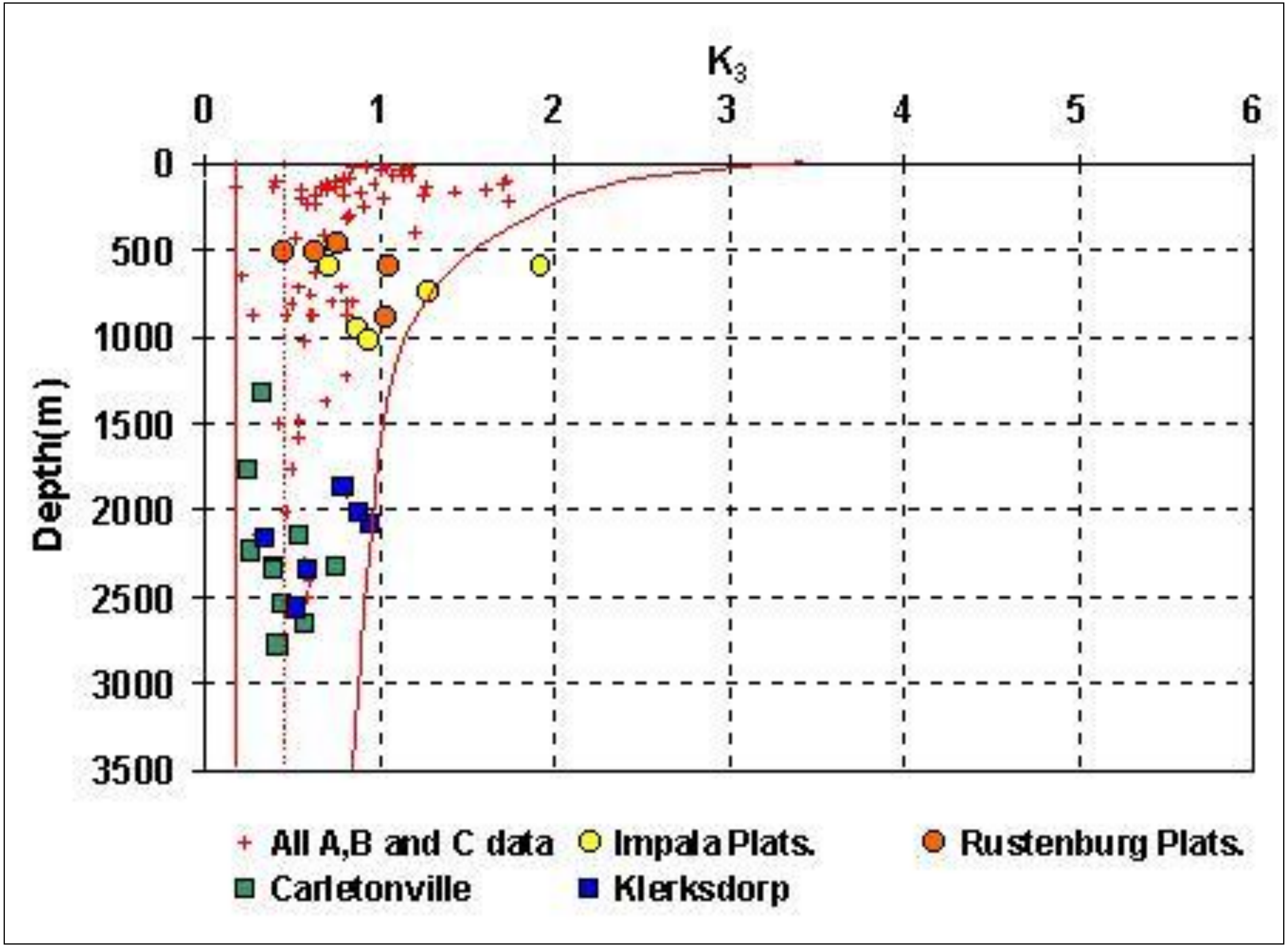


Figure 7 Ratio K_3 of minor horizontal in situ stress to vertical stress

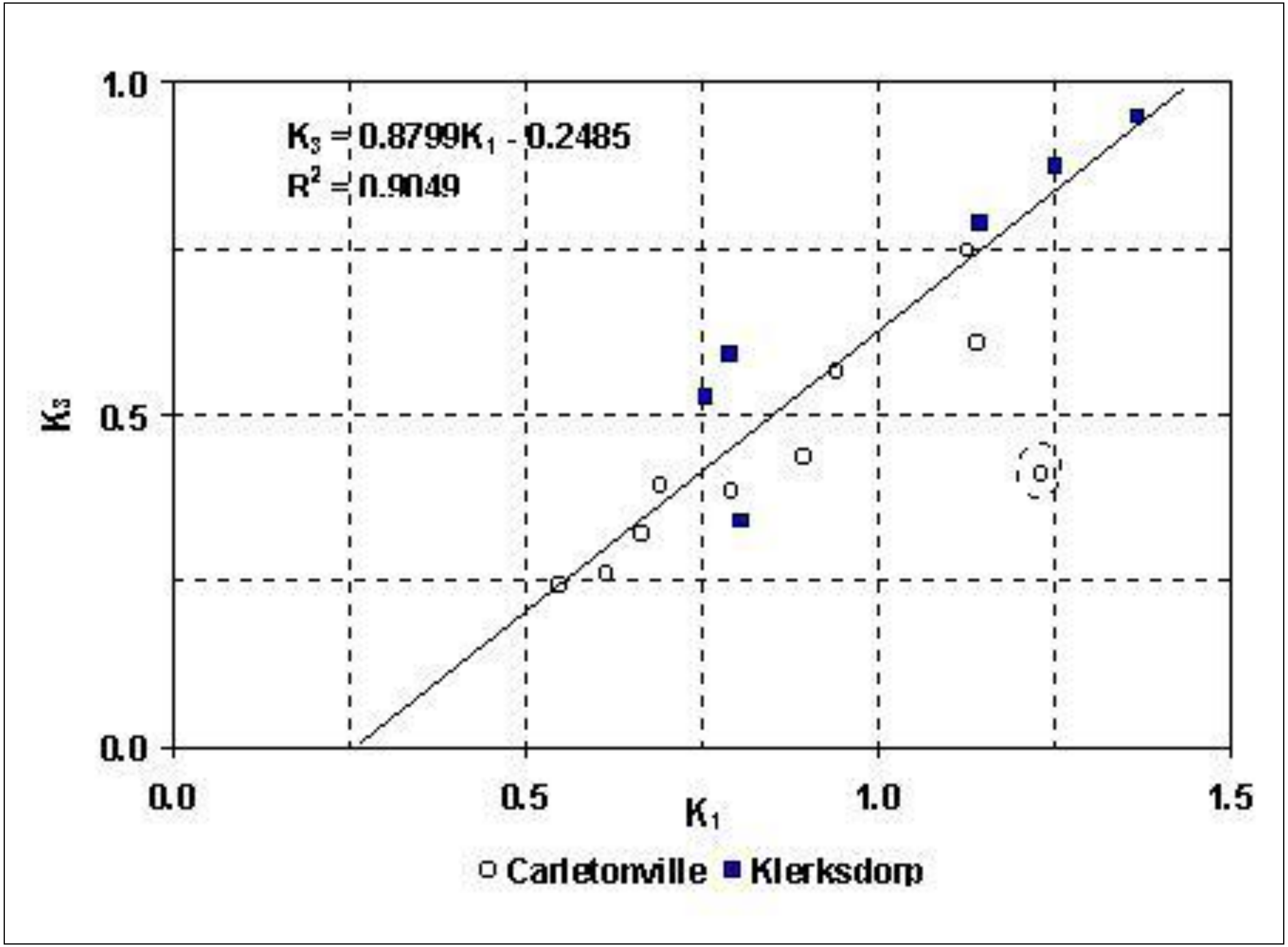


Figure 8 Relationship between K_1 and K^3

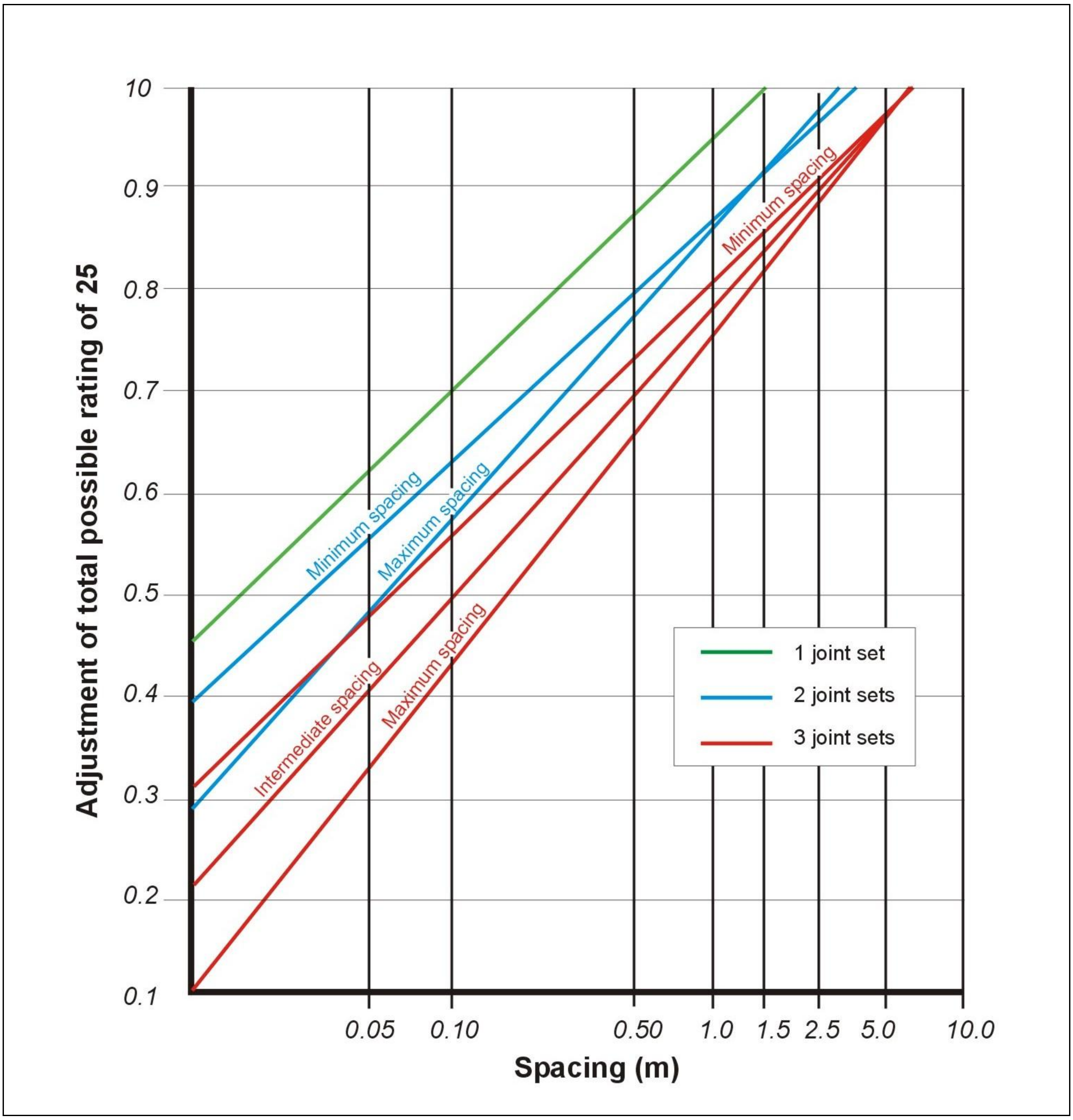
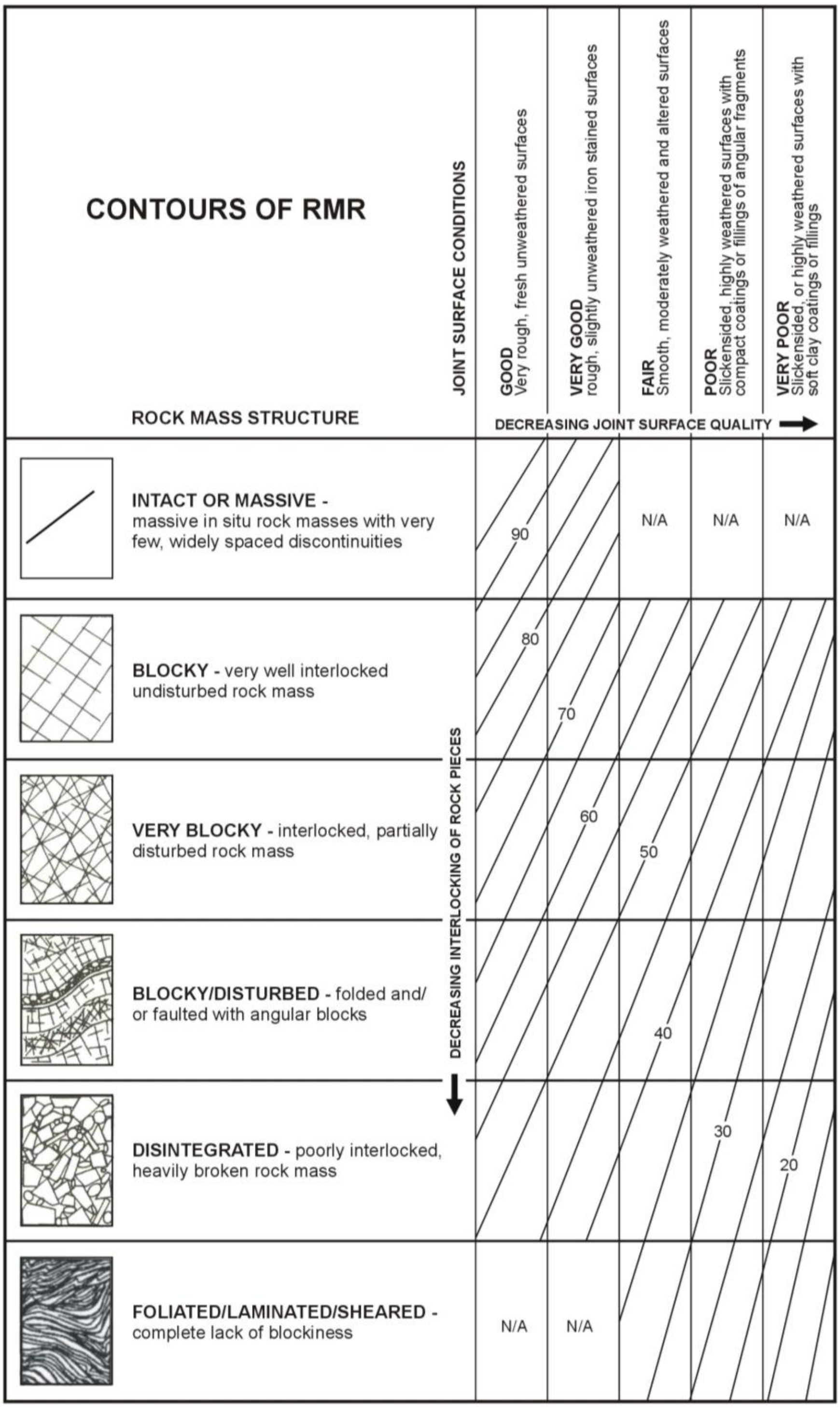


Figure 9 Graph for determination of joint spacing rating (after Laubscher, 1993)



The adjustment for strike and dip orientation of joints (Item 6 in Table 7) should be applied to the RMR obtained above. An MRMR equivalent will be between 5 and 6 points lower than the RMR determined from the above figure.

Figure 10 Estimation of RMR from visual assessment of the rock mass

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.

The types of instability, which are considered to be applicable, are intact rock failure, structural instability, failure of beams, and rock mass structural instability. The following sequence is recommended in carrying out stability evaluations:

i. **Rock failure**: 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 material. Failure through intact rock material is expected to be uncommon on most of the mines for which this handbook 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. It is recommended that the potential for this type of failure be determined using numerical analyses, and there are numerous computer programs available which are suitable for this purpose. It is very important that the failure criterion which corresponds with the actual mechanism of rock failure is taken into account. In this regard, differentiation must be made between brittle rock behaviour and yielding rock behaviour. In the former case instability occurs in the form of slabbing and spalling from excavation surfaces. For non-brittle massive rocks a shear failure mechanism is likely to be more applicable. Available computer programs usually include Mohr-Coulomb and Hoek-Brown failure criteria as part of their standard capabilities. Owing to the ready availability of suitable numerical analysis programs, evaluation of rock failure will not be dealt with further in this handbook;

ii. **Structural instability**: test for structural instability of blocks. Again this may be little more than a formality. Faults, dykes and joints may combine to form unstable blocks which can be specifically identified. A simply operated computer program has been included in this handbook for assessment of the stability of multi-sided rock wedges and blocks. Evaluation of structural instability is dealt with in Section 3.1.1

iii. **Stability of rock beams**: in stratified and bedded rocks, and other appropriate geometries, test for stability of natural rock beams. Stable unsupported spans can be estimated as a function of the thickness of the beam. Evaluation of the stability of rock beams is dealt with in Section 3.1.2 below.

iv. **Rock mass instability**: classify the rock mass and test for rock mass instability. For jointed rock masses with no identifiable major weak planes, rock mass classification techniques have been proved to be very successful in evaluating stability of underground openings. Regular, formalised classification is strongly recommended since it improves understanding and communication between technical and operating personnel, is a mechanism for control of mining operations, and ultimately results in safer conditions. Classification should be carried out in almost every situation, even when the mode of instability is stress-induced failure. The application of rock mass classification techniques to the evaluation of stability of different classes of excavations is dealt with in some detail in Section 3.1.3 below.

v. Optimise the opening with respect to location, orientation, shape and size. This is dealt with in sub-section 3.1.4. Return to the beginning of the stability evaluation procedure if any geometrical adjustment is made.

vi. Assess the influence of external factors, such as flowing water forces, flowing rock (orepasses), mechanical damage etc.

In the evaluation of stability of underground excavations, it is recommended that more than one method of assessment be applied in each case if possible. This will provide greater confidence in the assessment, and the results should be compared before arriving at a final design or assessment.

3.1.1 Evaluation of structural instability

The rock mass characterisation exercises carried out as described in Chapter 2 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 is included with this handbook to determine whether these blocks and wedges can fall out (i.e., are kinematically unstable). The use of this program is as follows:

- from the rock mass characterisation data it will be possible to visualise one or more geometries of blocks that might occur;
- the program allows for the stability of five different block shapes to be analysed, with from 3 to 5 planes of weakness being taken into account. These alternative block shapes are shown in Figure 11, which is a plot of one of the windows from the program;
- click on the appropriate block type based on the above visualisation, and a second window will appear, as shown in Figure 12;
- input data should be entered as indicated, and clicking on “calculate” produces the result;
- visualisation is not essential since the block shape in the Block View window in Figure 12 changes dynamically as the joint orientations are changed. The block in this view may also be rotated to improve the visualisation;
- a “help” function is available, which, if on, provides information on any item selected by the cursor. The explanation appears in the block at the bottom of the input data window, as shown in Figure 12.

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 frictional shear strength of 30^{o}.

3.1.2 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, for example, and the 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. The graphs in Figure 13 and Figure 14 provide a means of estimating stability in such cases. 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^{o} fall into the flat category and those greater than 60^{o} into the steep category. Interpolation should be used for intermediate rock strengths and inclinations. It must be emphasized that the stability of beams is critically dependent on the continuity of the beam. The graphs in Figure 13 and Figure 14 provide guideline values regarding stability, and if they indicate that stability might be critical, then more comprehensive analyses should be carried out.

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

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.

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

The use of rock mass classification is appropriate for the evaluation of stability of long term service excavations. The two methods applicable are the *Q* System and the *RMR* System described previously. 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 the graphs in Figure 15 and Figure 16.

Figure 15 (after Hutchinson and Diederichs, 1996) is based on Bieniawski's *RMR* system. Figure 16 (Houghton and Stacey, 1980) 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. This is dealt with in Chapter 4.

The degree of conservatism inherent in these graphs is illustrated by the following. For a factor a safety of 1.0 the corresponding unsupported span from Figure 16 is about 9m, which corresponds with a stand up time of about 6 months. Immediate collapse according to the Bieniawski approach would occur with a span of 18m. From experience with the application of Figure 16, 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.

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 $Q_{wall} = 5 \times Q_{roof}$ when $Q > 10$, that $Q_{wall} = 2.5 \times Q_{roof}$ 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 was developed by McCracken and Stacey (1989). It 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 Table 14 and Table 15 below.

Table 14 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 15 Adjustments 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:

$Q_{wall} = 2.5 \times Q$ when $Q > 1$ 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 17. 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.

3.1.3.2 Production oriented 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 Figure 15 and Figure 16 as in Section 3.1.3.1 above. 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 almost certain that, even in good quality rock masses, rock support will be required. 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 this is dealt with in Chapter 4.

Stability can be evaluated using Figure 16 with a factor of safety of 1.0. It should be noted that, in the evaluation of *Q*, the value of *Q* is reduced at intersections (see Section 2.3.1). Further, the stress reduction factor will change owing to variations in the mining induced stress. For example, at a block cave drawpoint, which represents an intersection, the *Q* value could be many times less than it would be at the same location for a normal tunnel.

Bieniawski's method becomes increasingly conservative at lower values of *RMR*, and its use for production oriented excavations is not recommended. An alternative approach is to use Laubscher's *MRMR* method, which is a more complicated process since it involves the evaluation of the rock mass strength and design rock mass strength. This process is closely associated with the evaluation of support requirements and will be dealt with in Chapter 4.

Stability evaluations for **stope excavations** in more jointed rock masses can be based on rock mass classifications. The degree of stability required will depend on the exposure of personnel in the stope. If personnel work in the stopes then a conservative approach is required. If there is no exposure of personnel in the stopes, then a more marginal stability condition can be tolerated. Three approaches are suggested and it is recommended that at least two of these are applied in each case. The approaches are:

- The use of the modified stability number *N′* (see Section 2.3.2). An empirical database has been developed, correlating *N′* with hydraulic radius. This relationship is shown in Figure 18, which can be used directly for empirical evaluation of stability;
- The use of Laubscher's adjusted *MRMR*, which he has correlated empirically with the hydraulic radius. This correlation is shown in Figure 19, which can also be used directly for empirical evaluation of stability. The evaluation of *MRMR* is described in Section 2.3.4. It

should be noted that Laubscher has updated this relationship regularly as new empirical data have become available. Figure 19 represents the most recently published version;

- The use of Figure 16, with a factor of safety of 1.0 or less, depending on exposure of personnel.

In this section dealing with the stability of stope excavations, 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. The possibility of occurrence, and the potential effects of an air blast, should be taken into account as part of the consideration of stability of the stope excavations.

3.1.3.3 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. It is surprising therefore that little attention is paid to the design and operation of passes.

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. It is particularly important to evaluate the possibility of occurrence of geometrically unstable blocks. However, 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 from this document are summarised in the following.

Design of passes includes the definition of pass location, orientation, size, shape, length, method of excavation, support, system geometry and operating principles. The following are considered to be important design aspects:

- 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 hangups. Sizes of branches and main passes should be compatible;

- the recommended pass inclination is between 55 ^{o} and 70^o, to allow free flow and to minimise impact damage and compaction of ore;
- the length of the pass should be minimised to avoid hangups and the difficulty of clearing hangups in long passes. Passes with leg lengths of less than 50m have rarely had problems;
- bends in passes should have an included angle of greater than 120^o;
- water in passes should be avoided, since it can lead to compaction, sticky ore, hangups and hazards due to mudrushes.

Rock mass characterisation, as described in Chapter 2 above, will provide the information required for the first two points.

3.1.3.4 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 two empirical relationships, based on rock mass classifications, in Figure 18 and Figure 19. The breadths of the transitional zones in both of these graphs demonstrate the uncertainty regarding prediction. In the database from which Figure 18 was developed, there are many unstable cases which plot in the stable area and stable cases which plot in the unstable area. To ensure cavability, the planned hydraulic radius should therefore plot well into the designated caving area.

The graph in Figure 16 may also be useful for giving a quick indication of cavability, provided that a factor of safety of about 0.7 is used.

3.1.4 Improvement of stability by changes in geometry

If the evaluation of stability, carried out using any of the methods described above, indicates that stability might be adverse, and that improvement in stability as a result of the installation of support may not be possible nor desirable, then it may be preferable to avoid the instability problem by altering the geometry. The following should be considered:

- altering the location of the excavation, if appropriate and if possible, so that it is placed in stronger rock and is hence more stable. This implies careful consideration at the planning stage;
- altering the orientation of the excavation such that:

- geometric instability of potential wedges and blocks is minimised, as illustrated in Figure 20;
- the orientations of long sidewalls are not parallel or sub-parallel to significant joints;
- the shape of the excavation corresponds with the naturally stable shape. For example, arch shapes will not be appropriate in horizontally layered strata. Examples are illustrated in Figure 21;

- altering the geometry of the excavation to maximise the stability required. For example, if the span of the excavation is likely to be unstable, it might be possible to reduce the span and increase the height of the excavation to obtain the required cross sectional area. It should be noted that the larger the size of an excavation, the less likely it is to be stable. Therefore, if stability is a requirement, the minimum appropriate size of excavation should be created;
- when stress induced failure is a possibility, the orientation, shape and size of the excavation have important controls on stability. The following should be taken into account:
  - long excavations should be oriented parallel to the major stress. For example, in high horizontal stress situations, it is often observed that tunnels in one direction show distress, but are completely stable in directions normal to this direction. In the latter case, the tunnels are sub-parallel to the major stress;
  - for intermediate stress situations, it may be possible to provide stability by optimising the stress distribution around the excavation. In such a case, the long cross sectional axis of the excavation will be in the direction of the major stress. If stress induced rock failure is impossible to contain, however, it is better to orient the long cross sectional axis of the excavation normal to the major stress direction. Stress failure will then occur locally, where it can be easily contained. These two alternatives are illustrated in Figure 22;
  - the smaller the size of the excavation, the less likely it is that uncontrolled stress induced instability will occur.

3.1.5 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 as far as this handbook is concerned. In contrast, there are numerous examples of discontinuous subsidence, such as that which occurred at Impala Platinum Limited's Bafokeng South Mine, in which the mine hospital was damaged (Kotze, 1986); 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 area of damage due to subsidence is defined by the angle of break, which is the angle to the horizontal drawn from the limit of mining to the outer limit of surface damage. Within this limit is a steeper angle defining the angle of draw or, in a caving operation, the cave angle. Within this limit major movement of the surface occurs, the maximum situation being the formation of a cave crater. From the angle of break limit to the cave angle limit, the rock mass will be progressively disturbed, and the surface correspondingly so.

The questions which need 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 as described in Section 3.1.3.2 above, 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, and these are dealt with in Chapter 4 below.

It is appropriate to include here the standard restrictions on mining (or the restrictions on surface development) imposed by the Department of Minerals and Energy. Implementation of these guidelines will give a conservative result. Relaxation of the restrictions can be motivated by means of thorough technical justification. The standard restrictions for seam type deposits are given in Figure 23. In the Minerals Act there are also statutory restrictions on the proximity of mining – no mining may be carried out within a horizontal distance of 100m of any surface which requires protection without the permission of the Inspector of Mines.

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 subsidence prediction which appear in the literature are mostly applicable to the continuous subsidence which occurs as a result of coal mining. These methods 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. These include:

- empirical prediction based on previous experience and observations at the particular mine, experience at other similar operations, or general precedent experience;
- empirical prediction of cave crack angles based on rock mass classification (Laubscher, 1993);
- informed prediction based on inspection and interpretation of rock mass characteristics and significant geological structures such as faults, dykes, shear zones and major joints;
- numerical analyses to model potential failure, subsidence and further development of subsidence (validity can be improved if a case of subsidence or failure can be back analysed).

Each of these will be dealt with briefly in turn.

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. It is therefore recommended that mines collect this information as a matter of course.

Although it is probable that this type of data collection has been carried out in South Africa, and that on-mine data exist, there are no published examples of such cases. Examples for two caving mines in the USA are given by Brumleve and Maier (1981) and Thomas (1971).

It is apparent that 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. From published data regarding cave angles in several caving mines in various countries around the world, minimum cave crack angles observed in a massive hard rock environment are about 50^{o}, 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

Laubscher (1993) has correlated adjusted mining rock mass rating values with estimated cave angles for caving situations. The presence of caved material will provide lateral restraint or confinement to the country rock, which will inhibit the flattening of the cave angle. Laubscher's estimated cave angles for confined and unconfined conditions are given in Table 16.

Table 16 Cave angles corresponding with rock mass classification (Laubscher, 1993)

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, as indicated in the next sub-section.

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 which are developed from interpretation of how the rock mass will deform in relation to its structure and geometry. For example:

- in dipping strata, the angle of break on the up-dip side will be shallower than on the down-dip side. This is taken into account in the restrictions imposed by the Department of Minerals and Energy, as illustrated in Figure 23;
- fault and dyke contact planes are surfaces of weakness which can inhibit or extend the area of subsidence, depending on their orientations. This is illustrated in Figure 24. To take such planes into account, it is necessary for the mine to have good information on the structural geology. Faults and faulted dyke contacts have had a significant influence on subsidence behaviour due to shallow gold mining on the Central Witwatersrand (Stacey, 1986; Brink, 1979);
- the plan shape of the mining will also influence the extent of caving that can occur. Piteau and Jennings (1970) investigated this effect in the mining of diamond pipes and observed slope angles of about 40^o for a 60m plan radius of curvature, compared with about 27^o for a 300m radius of curvature. Based on the data for open pit and quarry slopes (see Section

3.2.4.4 below) it is suggested that, 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^o 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^o shallower.

There are no rules for the implementation of the above effects into subsidence prediction, and it must be done on an intuitive or judgement basis.

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. However, the power of such theoretical analyses may lead to unfounded confidence in the quality of the predictions. It should be understood that 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. This problem is illustrated very well by the work described by Kay et al (1991) in the prediction of subsidence due to coal mining, the “new situation” results of which are shown in Figure 25. One of the conclusions of this work was “mathematical modelling of subsidence can result in spurious predictions and should be undertaken with an appreciation of empirically predicted subsidence and the importance of the various [computer] program inputs.”

In experienced hands, numerical modelling can be very useful for determining potential subsidence. It allows sensitivity analyses to be carried out quickly (a recommended approach) and in this way the problem can be bracketted. It is particularly advantageous if some observed behaviour can be back analysed to “calibrate” the numerical model. This will give greater confidence in the predictions made.

In addition to the above analyses, limit equilibrium analyses have been developed for chimney caving, and hangingwall caving as a result of mining of a dipping tabular orebody. These are summarised by Brady and Brown (1985). These methods are essentially mathematical formulations of observed behaviour. They have application in specific situations, but are not considered to have general merit.

3.1.6 Advanced techniques for stability evaluation

The methods of stability evaluation described above are mostly empirically based. In contrast with these, there are what can be termed “advanced techniques” available for analysis and prediction of stability. These can be divided into three categories:

- techniques for the specific kinematic analysis of the stability of wedges and blocks. This has been dealt with briefly in Section 3.1.2 above, and a simple computer program has been included for evaluation of stability of multi-sided blocks. If there is a frequent requirement to carry out kinematic analyses, there are good commercial packages available for such stability analyses, and one of these should be purchased for the purpose;
- numerical stress and deformation analysis approaches, including both continuum and discontinuum programs. Again, there is a variety of excellent software available commercially for these types of analyses. These types of analyses have been referred to above in analysing rock failure and subsidence;
- probabilistic analyses, in which individual stability evaluations may include the use of the above two approaches, or which are based on geometric freedom of occurrence of blocks, or on a variety of other approaches.

The success which can be achieved in the prediction of instability using the first two techniques is dependent on the skill of the user, the correct interpretation of the input data necessary, and the use of the “correct” failure criterion (it is very important that the potential mechanism of failure is correctly taken into account). This usually implies that some form of calibration against observed behaviour is necessary. As a result, the record of good absolute prediction of behaviour using these techniques has not proved to be very satisfactory. This is illustrated very well by the subsidence prediction exercise that was carried out in Australia (Kay et al, 1991), the results of which have been described briefly in Section 3.1.5 above. The results of this exercise showed that very different results could be obtained by different, competent analysts, using sophisticated software. The absolute prediction of behaviour was not satisfactory. Nevertheless, the use of numerical stress and deformation analyses in sensitivity and comparative analyses is strongly recommended.

The application of all of these techniques falls into the specialist area, and frequent use of them is necessary to ensure satisfactory familiarity and expertise.

The use of a probabilistic approach to the evaluation of stability is strongly recommended, since it provides data which can be used by management for decision making on a risk basis. Table 17 below puts values of probability into context by providing corresponding descriptive terminology (Kirsten, 1999).

A probabilistic approach allows variability to be taken into account and therefore the absolute accuracy of the modelling and prediction of behaviour is of less importance. A simple example of the application of such an approach to the evaluation of stability of a circular shaft is illustrated below. This application includes the following:

- the use of rock mass classification to determine the rock mass deformation and strength parameters. For this example, the *Q* classification system has been used;
- the chosen indicator of instability for the example is the depth of the potential failure zone. The calculation of the depth of the potential failure zone is carried out using a closed form solution;
- variability is taken into account as follows:
  - variations in each of the input parameters to the rock mass classification are used. Typical observed statistical distributions for these parameters, which were used in the analyses, are shown in Figure ;
  - variation in the in situ horizontal principal stress magnitude and in the ratio between the two horizontal principal stresses. Assumed distributions, based on the in situ stress database for South Africa (Stacey and Wesseloo, 1998), are given in Figure 27. These distributions have been derived from data obtained at a mine in the Rustenburg district at a depth of 900m. They are site dependent and are not representative for other mines.

Table 17 Acceptable life-time probabilities of occurrence

Degree of risk	**Attitude to reliability**		**Probability of occurrence (%) with regard to human safety**		
	Voluntary	**Involuntary**	**Total loss**	**Partial loss**	**Inconvenience**
Very risky	Very concerned	Totally unacceptable	70(-) (deep sea diving or rock climbing)	700	7000
Risky	Concerned	Not acceptable	7 (1.6) (deep sea diving or rock climbing	70	700
Some risk	Circumspect	Very concerned	0.7 (2.5) (deep underground mine, high rise construction)	7	70
Slight chance	Of little concern	Concerned	0.07 (3.2) (car, aeroplane or home accident)	0.7	7
Unlikely	Of no concern	Circumspect	0.007 (3.8) (public transport accident)	0.07	0.7
Very unlikely	Of no concern	Of little concern	0.0007 (4.4) (fatality in public place)	0.007	0.07
Practically impossible	Of no concern	Of no concern	0.00007 (4.8) (failure of nuclear power plant)	0.0007	0.007

Note : values given in parenthesis after the probabilities are corresponding factors of safety.

The analysis process involves the following:

- random sampling of the rock mass classification parameters from the statistical distributions, and their use to calculate *Q*;
- calculation of *RMR* from *Q* and hence calculation of the rock mass modulus, and the Hoek-Brown rock mass strength parameters *m* and *s* (in this case it is assumed that the Hoek-Brown failure criterion is applicable to the rock mass surrounding the shaft);
- random sampling of the in situ stress values from the statistical distributions;
- using the sampled and calculated input data, calculation of the depth of the potential failure zone by means of the closed form solution.

This process is repeated many times and the resulting distribution of the depth of the failure zone is shown in Figure 28. This distribution is expressed in the form of a "probability of excedence" in Figure 29, which is a particularly useful format. For example, if it is contemplated that conventional 1.8m long rockbolts will be used for support, this curve gives, directly, the probability that the depth of the potential failure zone will exceed the effective bolt length (say 1.5m, allowing for a 0.3m anchor length) and therefore render the support ineffective. From Figure 29 it can be seen that this probability is 5%. The acceptability of this probability will be a management decision, and Table 17 above can be used as the basis for this decision.

Risk is the product of the probability of occurrence and the consequences of the occurrence. The consequence can usually be expressed in financial terms, which facilitates decision making by management, who will have to make a conscious decision as to the magnitude of the loss or risk that they can accept as part of their mining activities.

3.2 Evaluation of stability of open pit and quarry slopes

Slopes in open pits and quarries can represent a safety hazard for the following reasons:

- large scale failure of slopes is a hazard for workers operating within the pit or quarry;
- bench failures are a local hazard for workers within the vicinity;
- localised failure can lead to subsequent bouncing of boulders down into the pit or quarry;
- failure of surface soil horizons or weathered material may disturb other areas and may lead to mud flows and the accumulation of mud in the bottom of the pit or quarry.

All of these can be a direct threat to personal safety, for example whilst drilling and charging, and loading from below newly blasted benches, and, indirectly, as a result of deterioration of access ways within the open excavation. In addition, large scale failures may affect the surface surrounding the open excavation, which may involve structures and infrastructure.

With regard to the safety aspects of slopes in quarries and open pits it is relevant to acknowledge the beneficial contribution that customising the slope architecture to the particular geotechnical environment can have on the operational performance of the mine. The use of the term geotechnical implies all of the geological, structural and hydrogeological in situ conditions which combine to determine the ultimate rock mass characteristics. The term architecture refers to the spatial relationship, face inclinations and geometrical dimensions of each of the individual elements of a particular sector of the mine shell. These individual elements are as follows;

- 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.

In determining the optimum design parameters for each sector of the mine shell it is first necessary to evaluate all geotechnical materials separately before reconstructing them in their correct spatial relationship with each other. This final “sandwich” of individual materials can then be modelled to determine its performance with regard to magnitude, rate and direction of slope displacement during the mining of the deposit. Adjustments can be made to various elements of the slope architecture, at this stage, in order to control the resulting slope displacements to within acceptable levels. These levels will be in accordance with the required safety factors or probabilities of failure, which have been established for each particular project.

Of relevance to observe during this design phase are the frequency and magnitude of likely bench or slope failures, and whether they are occurring in critical sectors of the mine shell, such as in the vicinity of an access ramp or above a zone of high grade ore. These types of potential failure may still be accommodated provided that monitoring of the actual conditions is carried out on a regular basis and that suitable remedial measures can be implemented without delay. Of importance is a comparison of the predicted conditions with the actual performance of the resulting mine shell. It is therefore essential that slopes in open pits and quarries are designed adequately. In the following sections, simple and quick techniques for this purpose are described.

It is important to note that the optimum design of pit and quarry slopes also involves economic issues, for example:

- selectivity of mining ie. high, medium and low grade ore may be mined at differing bench heights due to the distribution of the metal grades;
- selection of mining equipment to suit the scale of mining ie. daily production requirements may demand 15 to 30m bench heights in order to produce the tonnage.

These economic issues are not dealt with in this handbook.

There are three categories of stability evaluation, which follow directly from the rock mass characterisation approaches described in Section 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;
- situations in which the behaviour will be dominated by the geological structure. In such cases, kinematic stability analyses, taking the structural planes into account, will be required.

Toppling instability is a special case of kinematic instability.

These three categories will be dealt with in turn in the following sections. The methods to be described are simple and will be adequate in most cases. They will also be appropriate for identifying those cases in which stability is critical, or which are particularly sensitive, and for which more sophisticated analyses will be necessary.

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

The stability of the side slopes of clay pits, sand and gravel quarries, saprolites and other similar mining operations will be dependent on the behaviour of the "soil" material. Also, most open pits and rock quarries will have some overburden cover of soil or weathered rock, that will have to be taken into account in the assessment of the stability of the overall slopes.

In soil slopes the formation of a failure surface is little affected by structure (it might be in weathered rock in which the residual rock structure is still present and where the resulting geometry and volume of the failure will be controlled by the relic structures. This situation can be dealt with as for rock slopes which are dealt with in later sections). In this section, the consideration of stability is restricted to soil and completely to highly weathered slopes. In such materials it has been observed that the failure surface is commonly circular or near circular, although there will be some variation on this in non-homogeneous slopes. During the site inspection of soil slopes, it is important to check for subvertical tension cracks, subhorizontal shear slips, water seepage lines, zones of differing consistency or grading (this changes the permeability profile) and the alteration or weathering profile. All of these factors may influence the geometry of the failure surface.

Methods of stability analysis for soil slopes are well established, and many commercial software packages for this purpose are readily available. For the purpose of this handbook, however, it is considered that the use of slope stability charts will be most appropriate. These will enable any user to obtain a good estimate of the stability quickly, without the requirement for any sophisticated analysis techniques. Critical cases can be identified for more sophisticated analysis.

Soil slope stability charts have been produced by numerous researchers (Bishop and Morgenstern, 1960; Spencer, 1969; O'Connor and Mitchell, 1977; Chandler and Peiris, 1989; Barnes, 1991). However, the most convenient for the purposes of this handbook are those developed by Hoek and Bray (1981). They produced a set of charts corresponding with five different groundwater conditions. Only three of these have been included here:

- dry conditions: the presence of a pit or quarry will result in groundwater drawdown conditions, with the consequence that the soil adjacent to the excavation is likely to be in a dry or only partially saturated condition. It is to be borne in mind that South Africa is a semi-arid country;
- saturated conditions: this condition is included to take account of a worst case situation which could occur after sustained rainfall;
- a partially saturated condition intermediate between the dry and saturated conditions.

These conditions are included on the slope stability chart in Figure 30. 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 (kg/m3)
ϕ	angle of internal friction of the soil mass (°)
H	height of the slope (m)

The parameters for the soil mass (cohesion, angle of friction and density) should be determined by means of laboratory and field testing. It must be ensured that these values are relevant to, and representative of, the condition of the slope. As a guide, however, Table 18 gives descriptions of the consistency of soils and corresponding typical values for the parameters. These values do not take account of any soil structure that may be present in the soil mass.

Table 18 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

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

i. Calculate the value of $c/(\gamma H.tan\phi)$ 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 $Tan\phi\ /F$ (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. It can also be used for back analysis of slopes, as well as to determine, for example, the value of cohesion necessary for stability. It can also be used to investigate the effect of variability in soil properties on the stability. This requires that the stability be expressed as a probability of failure, which will be dealt with in Section 3.2.5.

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

In this section the use of rock mass classification methods for evaluation of the stability of rock slopes will be described. The use of rock mass classifications is valid in situations in which the control on global or overall stability is not dominated by major structural features.

Slope stability considerations were incorporated into Bieniawski's original classification method (Bieniawski, 1973). However, Bieniawski's rating adjustments for slopes represented possible over-riding effects on the *RMR* value, and, to overcome this, Romana (1995) introduced an alternative modification system. Similarly, Hack and Price (1993) introduced modification factors in Laubscher's system (Laubscher, 1990) to cater for rock slopes, and applied the approach to 140 road cuttings ranging in height from 5m to 50m. Hawley et al (1994) describe the development of classification methods, specific to each site, for the purpose of open pit slope design. Whilst this is individually satisfactory for each open pit, it does not allow direct comparison of values for different pits. Haines and Terbrugge (1991) worked with Laubscher to develop a correlation between his mining rock mass rating (*MRMR*) values and rock slope angles. The advantage of this approach is that it makes use of Laubscher's standard classification, which includes adjustments for weathering, structural orientation, stress influences and blasting practice. It is therefore the same classification method that would be used for underground excavations. Haines and Terbrugge (1991) applied the method to rock slopes in South Africa, Chile, Turkey, Namibia, and Zambia. This involved a range of rock types and structures, and a range in slope heights from 10m to 200m. In almost all cases there was very close agreement between the actual successfully excavated slope angle or the designed angle determined using more rigorous analytical techniques.

The Haines and Terbrugge approach has been selected for inclusion in this handbook since it makes use of the standard rock mass classification system, and its validity has been proved in application to a variety of mining rock slopes. Laubscher's classification method has been described in the section on rock mass classifications in Chapter 2. The rock mass characterisation approaches, also described in Chapter 2, are the bases for determining the inputs for the rock mass classification, and a field mapping sheet for classification methods was presented in Section 2.1.3 above. In this case, however, it is probably appropriate to use a mapping sheet that is specifically for Laubscher's system, and such a sheet is included in Figure 31. It is essential that the input parameters are recorded by an experienced user of classification systems who fully appreciates their significance in engineering design.

The correlation between recommended overall rock slope angles and adjusted *MRMR* values is given in Table 19.

Table 19 Correlation between adjusted mining rock mass rating (MRMR) values and overall rock slope angles

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

This table is best suited to the preliminary determination of overall slope angles. The design chart developed by Haines and Terbrugge (1991) is better suited to the selection of inter-ramp or bench stack slope angles.

It is necessary to take the slope height into account as well and, based on case history data, Haines and Terbrugge developed an empirically based chart correlating slope height, slope angle and adjusted *MRMR* values. This is reproduced as Figure 32, and provides a very convenient means of determining an appropriate slope angle using the rock mass classification approach.

This chart is useful in that it can provide the user with a recommended slope angle for the proposed height of the slope. In addition, however, it indicates to the user the limitations of the approach, showing those situations in which further analysis is required.

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. Individual bench slope angle-slope height relationships are also best determined from a structural evaluation of the influence of intersecting structures. The potential failure volumes can then be used to determine the dimensions of the required spill berms, which then provides input into the optimisation of the overall architectural configuration of each design sector. In the following sections, the evaluation of structurally defined stability will be dealt with - potential plane failure, wedge failure and toppling failure will be covered.

The continuity and spacing of geological structures will determine their applicability to the stability of benches, bench stacks or overall slopes. Therefore, the nature of the rock mass fabric must be well understood at this stage, and its variability with regard to the various sectors of design around the perimeter of the final pit or quarry. These design sectors should then accommodate the location and influence of the major structures as well as reflect the impact of the rock mass fabric. All of this information will be provided by the rock mass characterisation work described in Chapter 2 above.

3.2.3.1 Evaluation of stability of rock slopes exposed to potential planar failure

It will usually be obvious when there is a potential for planar failure to occur in a rock slope. Such situations are, for example, when bedding planes strike sub parallel to the slope and dip out of the slope. Even in such cases, there must be a lack of lateral confinement for pure planar failure to occur. Such conditions can occur in slopes which are convex in plan, or which are long in the direction parallel to the strike of the weak planes. In practice therefore, planar failures are likely to be restricted to benches and areas of the pit or quarry which have adverse geometry, or where there are release structures which strike perpendicular to the slope face and thus allow a planar slide to occur. If there is potential for planar failure on a larger scale, then it is strongly recommended that detailed evaluation is carried out. This is beyond the scope of this handbook.

The stability of a planar wedge or block, such as that shown in Figure 33, can be determined using the following equation:

$$F = \frac{cA + (W\cos\theta - U - V\sin\theta - T\cos\varphi)\tan\phi}{W\sin\theta + V\cos\theta - T\sin\varphi}$$

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
- θ 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
- φ is the angle of inclination between the anchor and the normal to the failure plane
- ϕ is the friction angle on the failure plane.

In this equation, the negative effects of groundwater on stability are obvious. Also, the contribution of cohesive strength on the failure plane to stability is direct. A small change in cohesion can have a major effect on stability.

In practice, since planar conditions will mostly be localised and small scale, the slope is likely to be drained. In addition, owing to the creation of the slope by blasting, which will have loosened the rock locally, the contribution of cohesion can be assumed to be zero. The problem of stability then reduces to the simple case of dependence on frictional strength. In this case stability is independent of wedge size, since the weight of the wedge appears in both numerator and denominator in the equation. Stability can therefore not be improved by flattening the slope.

Should planar conditions be present and the dip of the potential failure planes be steeper than their angle of friction, it is recommended that the slope be regarded as unstable and precautions implemented to ensure safety. It should further be noted that stability will deteriorate when temporary groundwater pressures develop during periods of rainfall.

It is recommended that, if at all possible, the slope design process should aim to avoid the creation of slopes in which there is planar failure potential.

3.2.3.2 Evaluation of stability of rock slopes exposed to potential wedge failure

The rock mass characterisation which has been carried out will define the planes, combinations of which will delineate potential wedge failure geometries. These will include major planes that define very large wedges, which could affect the overall slope, and joint planes, which could combine to form smaller wedges, which could affect bench stability or perhaps even ramp stability.

The stability analysis of wedges is one of the main components in the design optimisation of suitable bench face angle-bench height configurations. The required spill berm width follows from this process.

In this section a simple method for wedge stability evaluation will be described. When the application of this method indicates that stability might be critical, it will be necessary to carry out more rigorous wedge stability analyses. There are computer programs available commercially for these sophisticated analyses, and it is recommended that they should be used in critical cases.

The set of stability charts developed by Hoek and Bray (1981) 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 is shown in Figure 34. This has two planes on which sliding may take place, with movement being in the direction of the line of intersection of the two planes.

The wedge stability charts are presented in Figure 35 to Figure 38. In these charts the A plane is always the plane with the flatter dip angle. The use of the charts is straightforward. 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:

$$F = A \cdot Tan\phi_A + B \cdot Tan\phi_B$$

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

For intermediate dip angles and dip directions, interpolation of results from two charts should be carried out.

Since cohesion and water pressure have been omitted from the analyses, conservatism is necessary in the interpretation of the results as indicated in Table 20 below.

Table 20 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

The charts can also be used to examine the effects of variability in the orientations of the planes and in the friction angles of the planes. A probabilistic approach for this purpose will be described in Section 3.2.5 below.

3.2.3.3 Evaluation of rock slopes exposed to potential toppling failure

Large scale toppling failures of rock slopes involve a combination of mechanisms, in particular rotation, with associated shear on one or more structural surfaces. They also require time to develop to the ultimate failure state. Such failures are unlikely to occur in open pits or quarries and are therefore of little significance. In the unlikely event that major toppling instability should 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. This will provide sufficient time in which to implement stabilising measures, to design avoidance measures or to vacate the area.

Of more significance for the purposes of this handbook is the potential for small scale toppling failures which could occur suddenly from bench faces. Such failures represent hazards during the drilling, charging and mucking operations, and to geologists who may be required to undertake mapping of the exposed surfaces. Boulders which topple from upper bench faces may bounce off and over benches to pose a hazard at lower levels.

The theoretical analyses which are available for the assessment of toppling instability are not considered to be of practical value. Instead, it is recommended that toppling instability is treated on a recognition basis. The potential for toppling depends on the following:

- occurrence of sets of continuous joints;
- strikes of these joints being sub parallel to bench faces;
- dip angles of the joints being steep.

As shown in Figure 39, if the dip of the joints is between 60^{o} and 95^{o}, then there is toppling potential. Special cases in which toppling potential may be present are, for example, dolerite dykes and sills, in which columnar jointing occurs.

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. Loose material on the bench surface will also absorb impact energy and therefore reduce the bouncing energy. Visual monitoring of benches in such areas is recommended to check that there is no build up of spoil near the toe of the bench which could increase the potential for boulder bouncing. These mechanisms are illustrated in Figure 40.

3.2.4 Influence of various factors on slope stability

From the above consideration of methods of evaluation of the stability of slopes it will be clear that there are numerous factors, which have significant influence on stability. Some of these are taken into account in the methods of analysis described, and some are not. The most important factors of influence are dealt with in the following sub-sections so that the user of this handbook can include their effects on a qualitative judgement basis.

3.2.4.1 Geology and geological structure

The importance of geology is obvious and it has been taken into account in the stability evaluations described in the previous sections. It is included here to emphasise its importance - it is by far the most important factor to be considered. 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. It is strongly recommended that operators obtain this input from appropriately qualified and experienced geotechnical personnel. Ignorance of, or the wrong approach to, geological conditions in design and operations can have severe consequences. For example, convenience may indicate that haulroads should be in particular locations of the pit, but the geology may dictate that the convenient locations are not satisfactory from a stability point of view. Adopting the convenient choice due to ignorance could be disastrous for the mine.

3.2.4.2 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. Pressure due to water in open cracks causes active forces on blocks and wedges. Water pressures on potential failure planes cause uplift or reduced normal stresses across these planes, which reduce the frictional strength on these planes. Water inflow into slopes should therefore be minimised, and surface water should be drained away from the slope crest areas.

Drainage of slopes can be used as a means of improving their stability. This can be in the form of boreholes (sub-horizontal, drilled into the slope, or vertical, behind the slope, with extraction of water by pumping), dewatering adits into the slope, or dewatering ring tunnels with associated adits or pumping. Ring tunnels have been used in diamond mines at Kimberley, and adits, boreholes and pumps have been used at the Nchanga open pit in Zambia.

If instability develops in a slope, drainage is one of the first measures that should be considered for stabilisation.

3.2.4.3 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. This rock then has much less cohesion, and much greater potential for ingress of water and for further loosening. Good quality blasting will limit the damage to the rock mass and therefore enhance stability. In particularly sensitive cases, the use of presplit blasting to form the faces of the slope may be necessary.

Based on the adjustments for blasting given in Laubscher's rock mass classification system, good conventional blasting decreases the rock mass quality by only a few percent, whereas poor conventional blasting has an effect of the order of 20%. Very poor blasting, which can often be observed in mining operations, is likely to decrease rock mass quality by 30-40%. Using the chart in Figure 32, it can be estimated that the stable slope angle for a good quality rock mass could, in some cases, be reduced by about 15% due to very poor blasting.

The implications of poor blasting practice are increased hazard in operations, increased potential for rockfalls and bouncing boulders, and decreased stability of benches and ramps. The safety and economic implications can therefore be very significant.

3.2.4.4 Slope geometry

In the consideration of planar failure above it was mentioned that this type of failure is more likely when slopes are convex in plan. 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.

The rock mass classification method of Barton et al (1974) indicates that the *Q* value of a tightly confined rock mass can be up to 10 times greater than the quality of the same mass in an unconfined condition.

This corresponds with a decrease in quality in Laubscher's classification value of about 33%. Based on the chart in Figure 32, for a 100m high mine slope, this would result in a decrease in slope angle from 67^o to 56^o. This is a rough indication of the influence of a convex slope profile in plan, in which the slope is laterally unconfined.

Hoek and Bray (1981) suggest that, when the radius of curvature of a concave slope is less than the height of the slope, the slope angle can be 10^o steeper than determined by conventional slope stability analysis. Conversely, for a convex slope, with the radius of curvature less than the slope height, the slope angle should be 10^o flatter than conventionally determined. These values appear to be optimistic compared with those from the chart in Figure 32, which indicate an angle change of 5^o rather than 10^o.

Use of the above data must be made with some circumspection since other factors, in particular the presence of major geological planes of weakness, may be of overriding importance.

3.2.4.5 Earthquake loading

South Africa is not a seismically active country. It is therefore not considered necessary to take earthquake loading into account in the routine evaluation of slope stability in open pit mines and quarries. In certain locations such as the Western Cape there is a higher probability of significant earthquakes occurring, and it is recommended that the stability under these conditions should be checked. Conventional computerised stability analysis methods, which are commercially available, are easily able to carry out these analyses.

3.2.5 Advanced stability evaluation techniques

The methods of stability evaluation that have been described above are simple routine methods. In special cases of high slopes, sensitive slopes, or in cases when optimisation is necessary, the application of more sophisticated methods of stability analysis may be required. Some of these methods are dealt with briefly in the following sections. However, if the application of such methods is necessary, it is recommended that they be carried out by geotechnical engineers experienced in their application.

3.2.5.1 Stress analysis of slopes

Although the application of numerical stress analyses to slopes was first carried out about 30 years ago (Duncan and Goodman, 1968; Mahtab and Goodman, 1970; Stacey, 1970; Stacey, 1973), there has been relatively little application in this area. The advantages of the approach are that the following can be modelled:

- the excavation sequence;
- the inclusion of water pressure and flow;
- the progressive development of yield and failure zones (and their possible return to stability as excavation proceeds);
- the modelling of deformations, and the identification of progressive (accelerating or unstable deformations) or regressive (decelerating of stabilising deformations) conditions. The outputs of the analyses are not in the form of factors of safety, and therefore the interpretation of "stability" requires engineering judgement and experience. Stress analysis approaches, however, give insights into possible slope behaviours that cannot be gained from conventional stability analyses. They do provide a means of comparison with actual slope behaviour, calibration with such behaviour, and subsequent updating of the numerical model to improve its validity for prediction of instability. The modelling results achieved from these types of stability analyses are often very useful indicators of the rate and magnitude of displacement that can be anticipated for the slopes. The modelled time-displacement

graphs are useful for comparison with the actual results obtained during mining. It is likely that the use of stress analyses for slope stability will become more routine in the future.

Numerical stress analyses have been used, with good benefit, for slope stability evaluations for numerous open pits in southern Africa and Chile. Typical results of analyses of this type are shown in Figure 41 and Figure 42.

3.2.5.2 Probabilistic analysis of stability

The methods of stability analysis described in Section 3.2.1 to 3.2.3 above can be referred to as deterministic methods. Input parameters are specified, and the output is a factor of safety. In reality it must be understood that the input parameters for slope stability analysis are subject to considerable variation. For example:

- slopes, more often than not, are non-homogeneous. Within a slope there may be several different rock or material types, with different deformation and strength properties; rock, material and discontinuity properties are not constant, but have some variability associated with them. For example, cohesions and angles of friction may be normally or log-normally distributed about mean values, joint spacings are log-normally distributed, and joint lengths follow a negative exponential distribution;
- stability is usually determined along defined "failure" surfaces. These surfaces are assumed, and not actually known, since the jointing or weakness paths, which are the paths along which failure will take place, are not known within the rock mass; groundwater conditions, which can have a major effect on stability, are usually not known, but are assumed or calculated;
- the method of analysis used will result in some variation - different methods of analysis may give somewhat different results.

From this it will be understood that there is no unique value for the factor of safety of a slope. The value determined will depend on the values of the input parameters. In cases of such variability, it is often appropriate to determine the probability of failure rather than the factor of safety (it must be recognised that a factor of safety of 1.0 is equivalent to a probability of failure of 50%).

The probability of failure can be expressed as the probability that the capacity is exceeded by the demand:

$$P(f) = P(C - D) < Zero$$

where $P\ (f)$ is the probability of failure

C is the capacity (the sum of all the forces promoting stability)

D is the demand (the sum of all the forces promoting instability)

In the context of the methods of analysis described, this is equivalent to the probability that the factor of safety is less than 1.0, that is:

$$P(f) = P(FOS < 1)$$

To determine the probability of failure it is necessary to know (or assume with some confidence) the statistical distributions defining the variability of each of the parameters involved in the analysis. To carry out a probabilistic analysis, each of the input parameters is sampled, and a stability analysis carried out using a suitable method of analysis (this could be one of the methods described above) to obtain a result (eg, (C - D) or *FOS*). In jointed rock slopes, in which the potential failure surface is not known, part of the process will involve the generation of possible failure surfaces from the sampling of joint statistical distributions.

The whole sampling and stability analysis process is repeated a large number of times to determine a distribution of the results (the probability density function). The probability of failure is then determined by the area under the probability density function to the left of the criterion value, as shown in Figure 43. This process is known as a Monte Carlo simulation approach, and requires appropriate computer software, which is commercially available.

A simplified approach, which is applicable when only a small number of variables is involved, is to use the point estimate method (Harr, 1987). The use of this approach is feasible with the stability charts presented above. For example, consider the slope in weathered material shown in Figure 44. Assume that the groundwater conditions are as shown in Figure 44, and that the variabilities of the cohesion and friction angle of the weathered material are defined by the means and standard deviations given in Figure 44. Using the chart in Figure 30, stability evaluations can be carried out. To apply the point estimate approach, it is necessary to carry out four evaluations as shown below.

For the purpose of this example, the two input parameters c and ϕ are assumed to be normally distributed independent variables. To apply the pint estimate method, one needs to perform four separate analyses, one analysis for each combination of what are called the point estimates of the variables. The two point estimates for each of the variables are defined as follows:

$c_{-} = c_{mean} - c_{std}$

$c_{+} = c_{mean} + c_{std}$

$\phi_{-} = \phi_{mean} - \phi_{std}$

$\phi_{+} = \phi_{mean} + \phi_{std}$

In these equations the subscript "mean" refers to the mean value, and "std" refers to the standard deviation. The factor of safety for each of the following combinations of the four point estimates can be determined from the stability chart in Figure 30:

FOS_{--} using c_{-} and ϕ_{-}

FOS_{-+} using c_{-} and ϕ_{-}

FOS_{+-} using c_{-} and ϕ_{-}

FOS_{++} using c_{-} and ϕ_{-}

The mean and standard deviation of the factor of safety of the slope is given by the mean and standard deviation determined from the four calculated *FOS* values. The values for the mean and standard deviation of the *FOS* can be used to calculate the probability of failure of the slope.

Using the point estimate approach, the probability of failure of the slope shown in Figure 44 is determined as 25%. This should be compared with the factor of safety determined for the slope, using mean values and the chart in Figure 30, of 1.1. If the slope is drained, repeating the evaluations gives a probability of failure of 7% and a factor of safety of 1.4. This demonstrates the beneficial effect to slope stability provided by drainage.

The significance of the probability of failure of a slope is not commonly understood. The information contained in Table 21 below is based on the publication of Kirsten (1982) and indicates the probabilities of failure for which open pit mine slopes should be designed.

Table 21 Comparative significance of probability of failure

Probability of failure (%)	**Design criteria on basis of which probability of failure is established**		
	Serviceable life	Minimum surveillance required	Frequency of evident slope failure
50 – 100	Effectively zero	Serves no purpose (excessive probability tantamount to failure)	Slope failures generally evident
20 – 50	Very short term (temporary open-pit mines – untenable risk of failure in temporary civil works)	Continuous intensive monitoring with sophisticated instruments	Significant number of unstable slopes
10 – 20	Very short term (quasi-temporary slopes in open pit mines - undesirable risk of failure in quasi-temporary civil works)	Continuous monitoring with sophisticated instruments	Some unstable slopes evident
5 – 10	Short term (semi temporary slopes in open-pit mines, quarries of civil works)	Continuous monitoring with simple/rudimentary instruments	Occasional unstable slope evident
0 – 5	Medium term (semi-permanent slopes)	Conscious superficial monitoring	No ready evidence of unstable slopes

One of the advantages of a probability of failure approach is that it can be used to quantify risk, risk being the product of the probability of failure and the consequence of the failure. For example, if failure in a particular section of an open pit mine was to occur, it might result in the closure of a haul road by the spoil from the failure. The removal of this spoil to re-open the road would have a cost associated with it. In addition, however, production would be lost, with corresponding costs, and there could be associated costs with crushing, process and smelting plants. If the total potential cost is X and the probability of failure of the slope is Y then the risk to the operation is X.Y, which is a cost figure that management will understand. Based on this, the decision may be taken to design that section of the open pit slope to a lower probability of failure. It is common that benches are designed for a higher probability of failure than ramps which in turn have a higher probability of failure than the overall open pit slope.

It is recommended that bench geometries are designed to a probability of failure of 40% (corresponding with a factor of safety of about 1.05 to 1.10) and that the corresponding potential failure volume from the bench scale is used to determine the spill berm width. When these benches and spill berms are stacked, the bench stack geometries can be defined. It is appropriate then to check these geometries against the recommendations derived from the chart in Figure 32. They should compare well, and if they do not, the whole design process should be checked thoroughly. It is recommended that probabilities of failure for bench stacks should be < 5% below major access ramps or areas containing mining infrastructure, < 10% above major access ramps, and < 15% for slopes with no access ramps or infrastucture. It is then possible to design the slope geometries containing all of the required architectural elements ie. ramps, benches, special geotechnical berms etc.

3.2.6 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. The use of cable anchors or shear keys may be required locally for the support of a critically important ramp, for example. However, this would be a temporary situation and the longer term solution would be to relocate the haul road to a more stable area. An exception to this is when an open pit operation is specifically designed on the basis of installed support. A particular example is vertical pit mining (Redford et al, 1998). The pit is excavated with vertical sidewalls, whose stability is ensured by the installation of cables, rockbolts, wire mesh and shotcrete. Removal of ore and waste is by means of a crane or a Blondin cable system. The installation of support in open excavations is dealt with briefly in Section 4.3.6.

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. Monitoring of the slopes is an integral part of the management process (see Table 21). In smaller operations this may be a visual monitoring record, but in larger operations, monitoring by survey methods, installed instruments, seismic or microseismic systems, or even satellite based global positioning systems, are appropriate. It must be emphasised that 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, subsequently requiring the clearing of the unstable, failed material.

To be successful, monitoring must be carried out regularly and routinely, and the resulting observations and measurements documented and interpreted. Only if this is done will it be possible to identify changes or trends in the pattern of behaviour at the earliest opportunity.

As has been mentioned previously, besides the pit geometry, there are two controllable factors which have a significant contribution to slope stability. These are blasting and groundwater. In many cases of unstable slopes, therefore, stability can be ensured, or at least improved, by improving the quality of the blasting, and by dewatering of the slopes.

Finally, it must be emphasised that the aim in mining is to carry out a safe, efficient and profitable operation. This should not be interpreted as a requirement that all slopes should be completely stable. If this is the case, the mining will probably not have been optimised. **Some slope failures should occur**, which will indicate that mining is being carried out close to the limit and stripping of waste is being kept to the minimum. The important point, however, is that **failures should not be allowed to occur unexpectedly nor inconveniently!** It has been said that the most efficient open pit mine is one in which the slopes fail the day after mining has been completed.

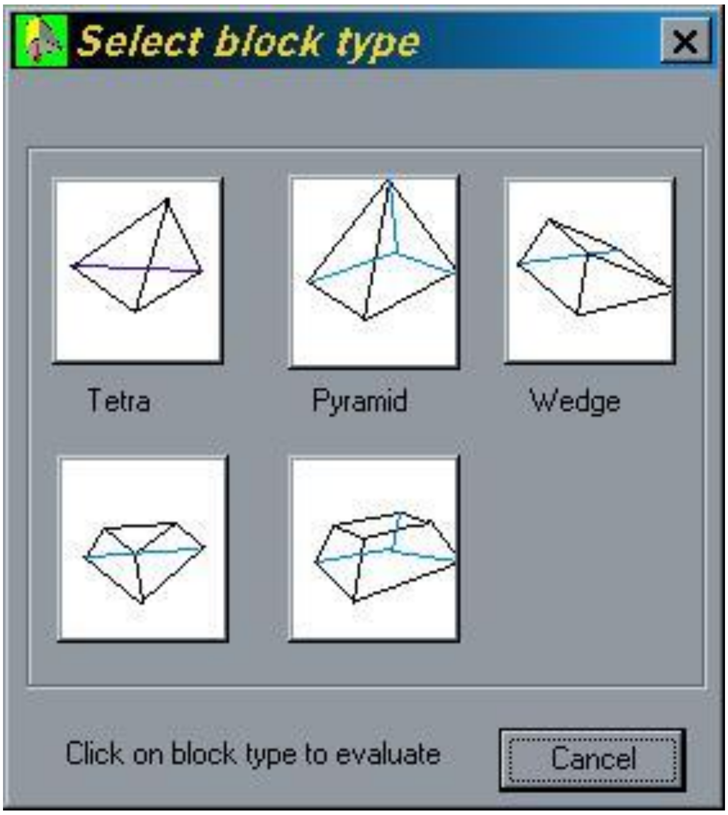


Figure 11 Block shapes catered for in BlockEval

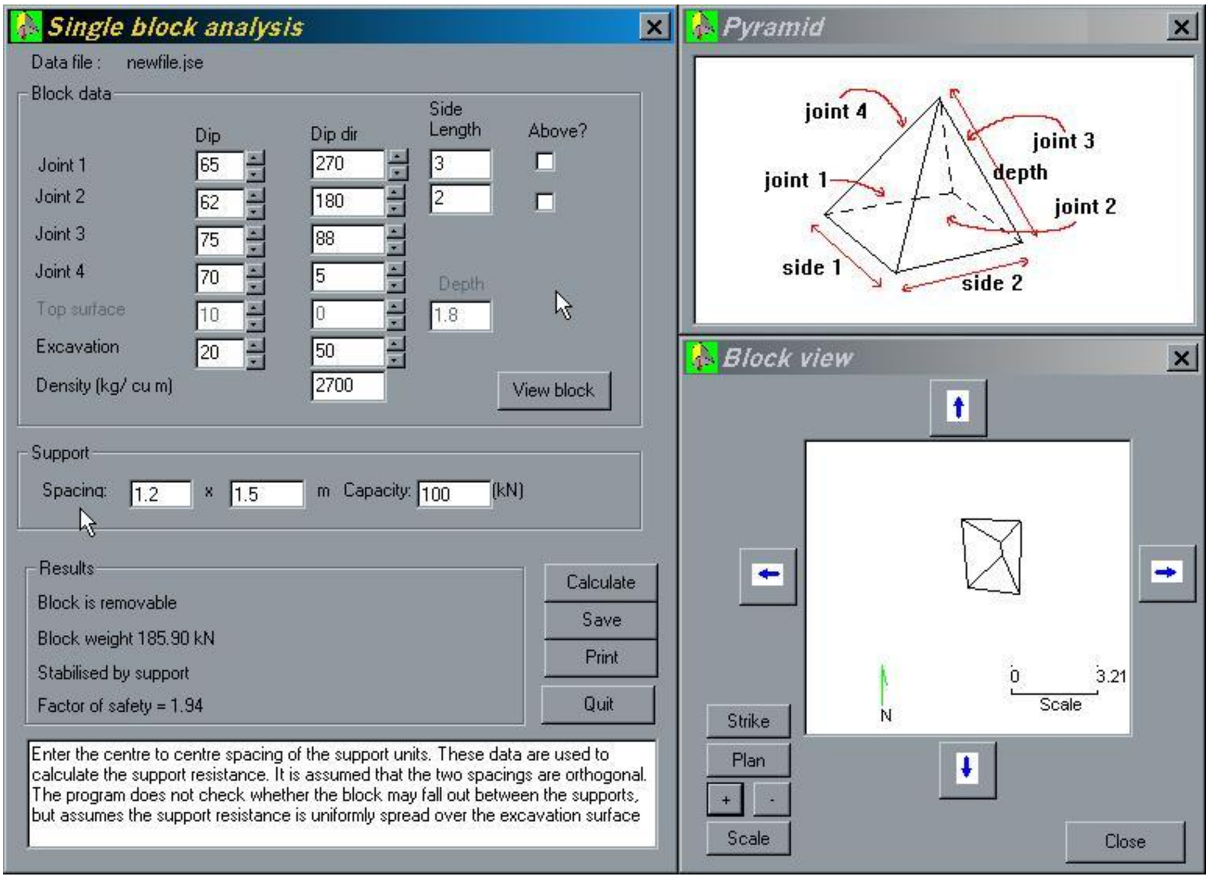


Figure 12 BlockEval windows for input data, output results and block visualisation

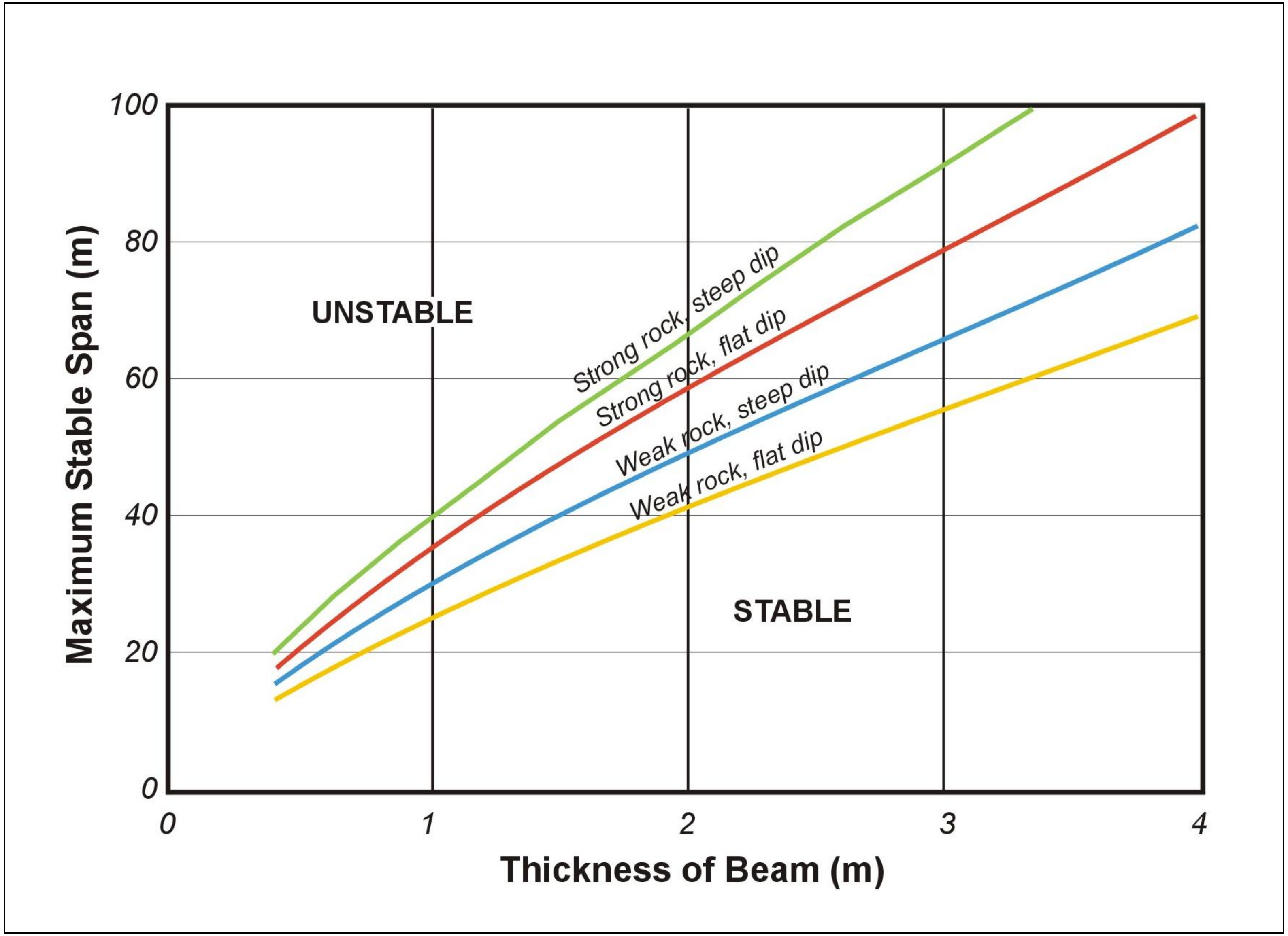


Figure 13 Stability of beams - long excavation

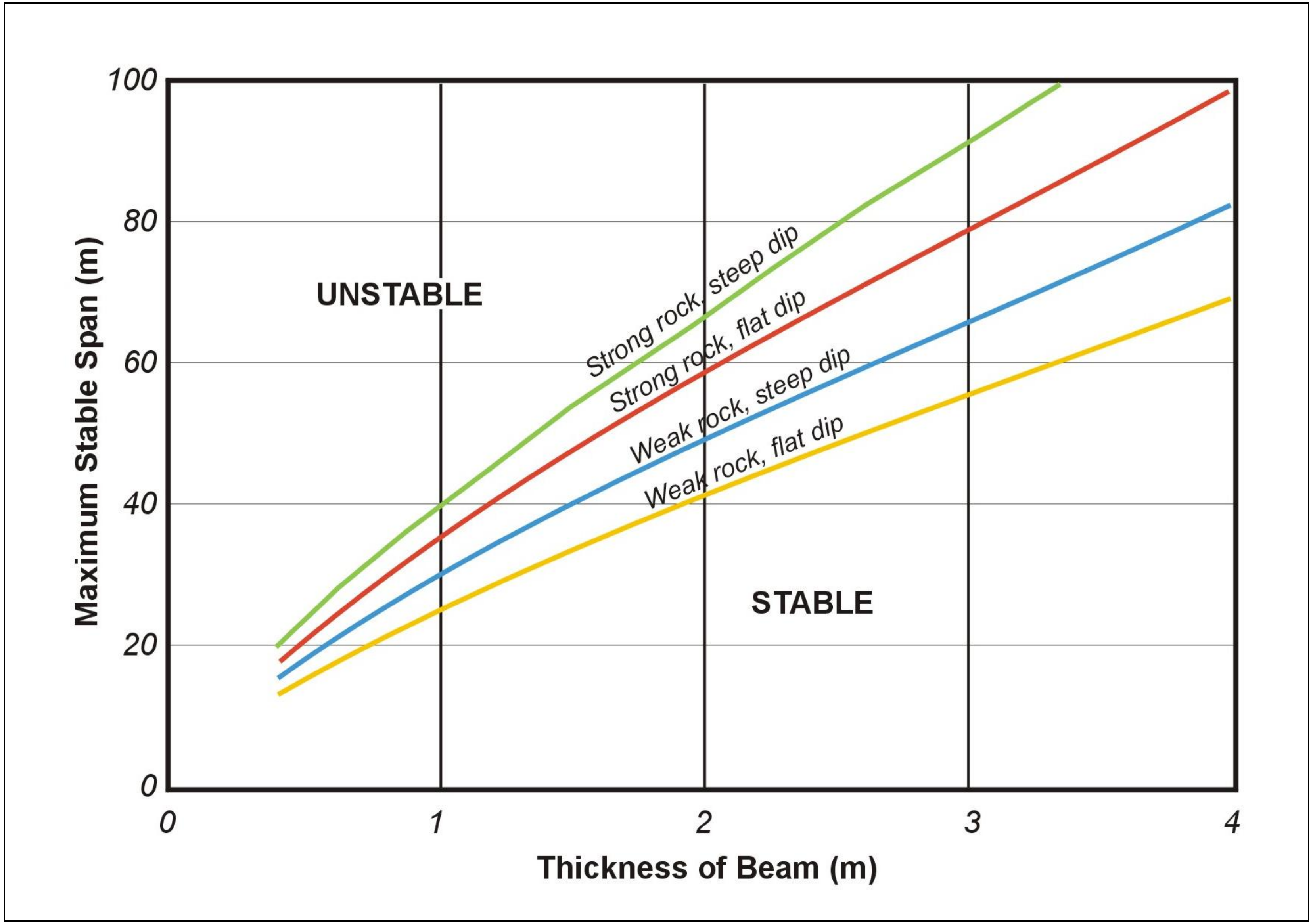


Figure 14 Stability of beams - square excavation

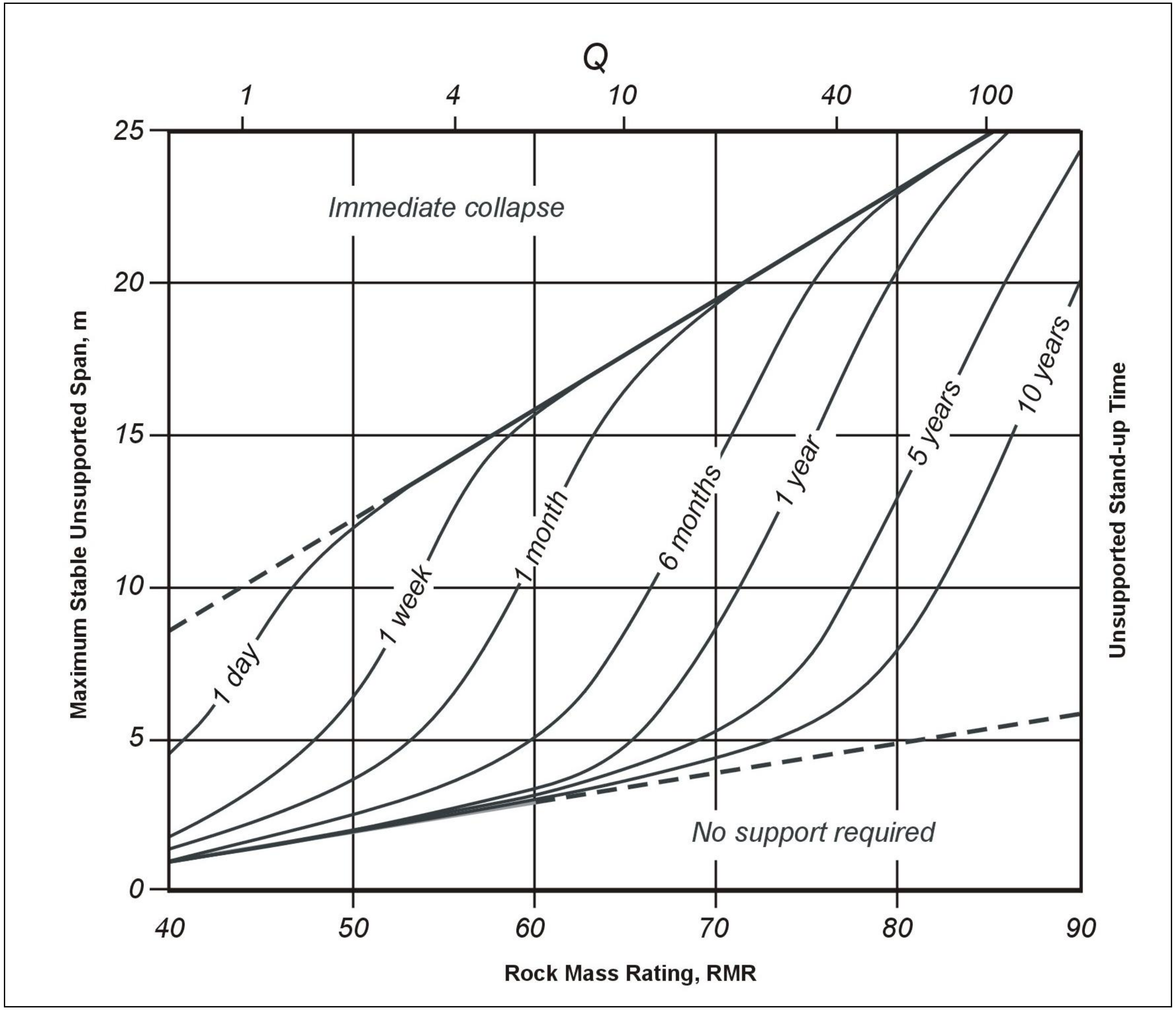


Figure 15 Unsupported span vs. RMR relationship

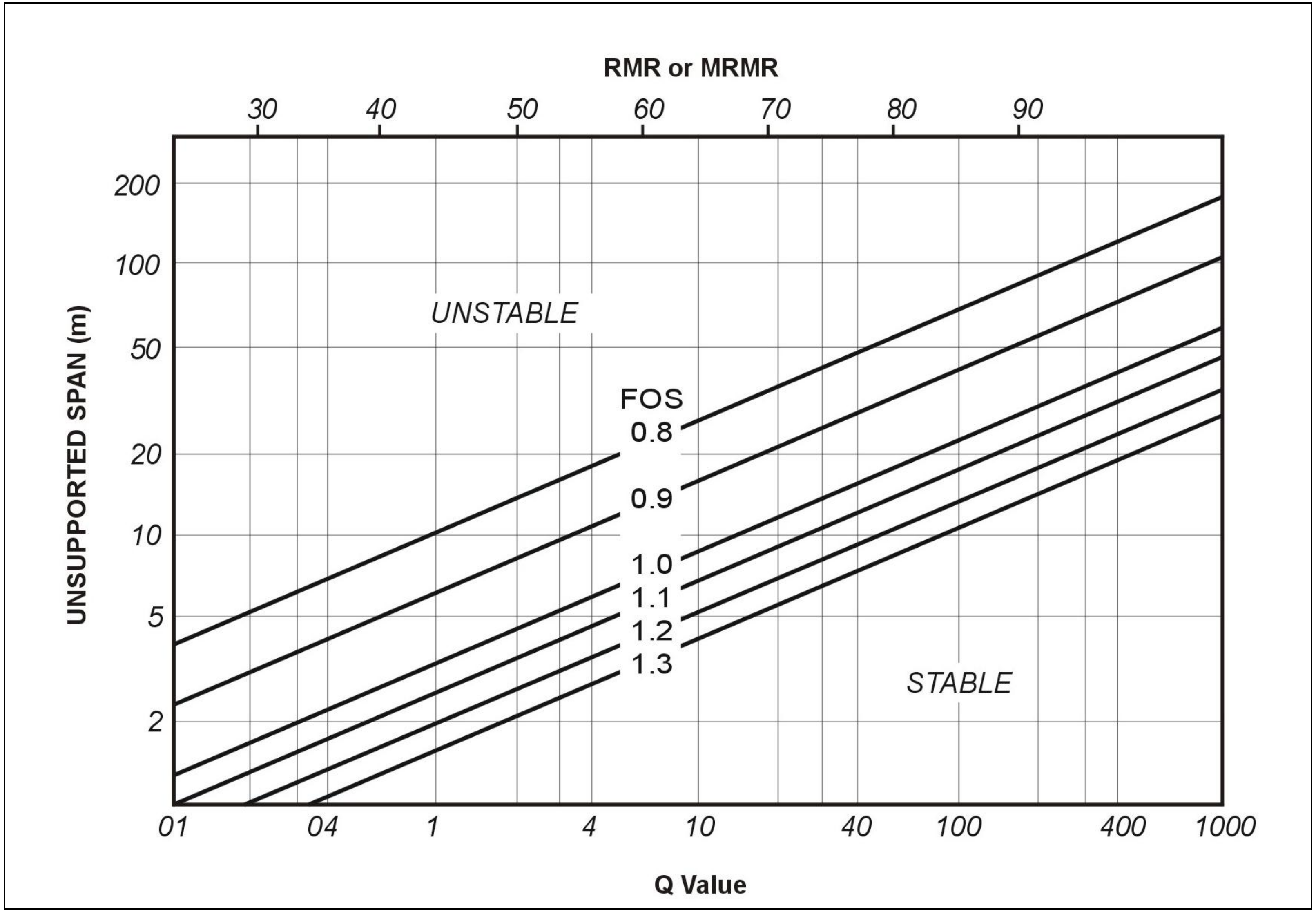


Figure 16 Unsupported span vs. Q relationship

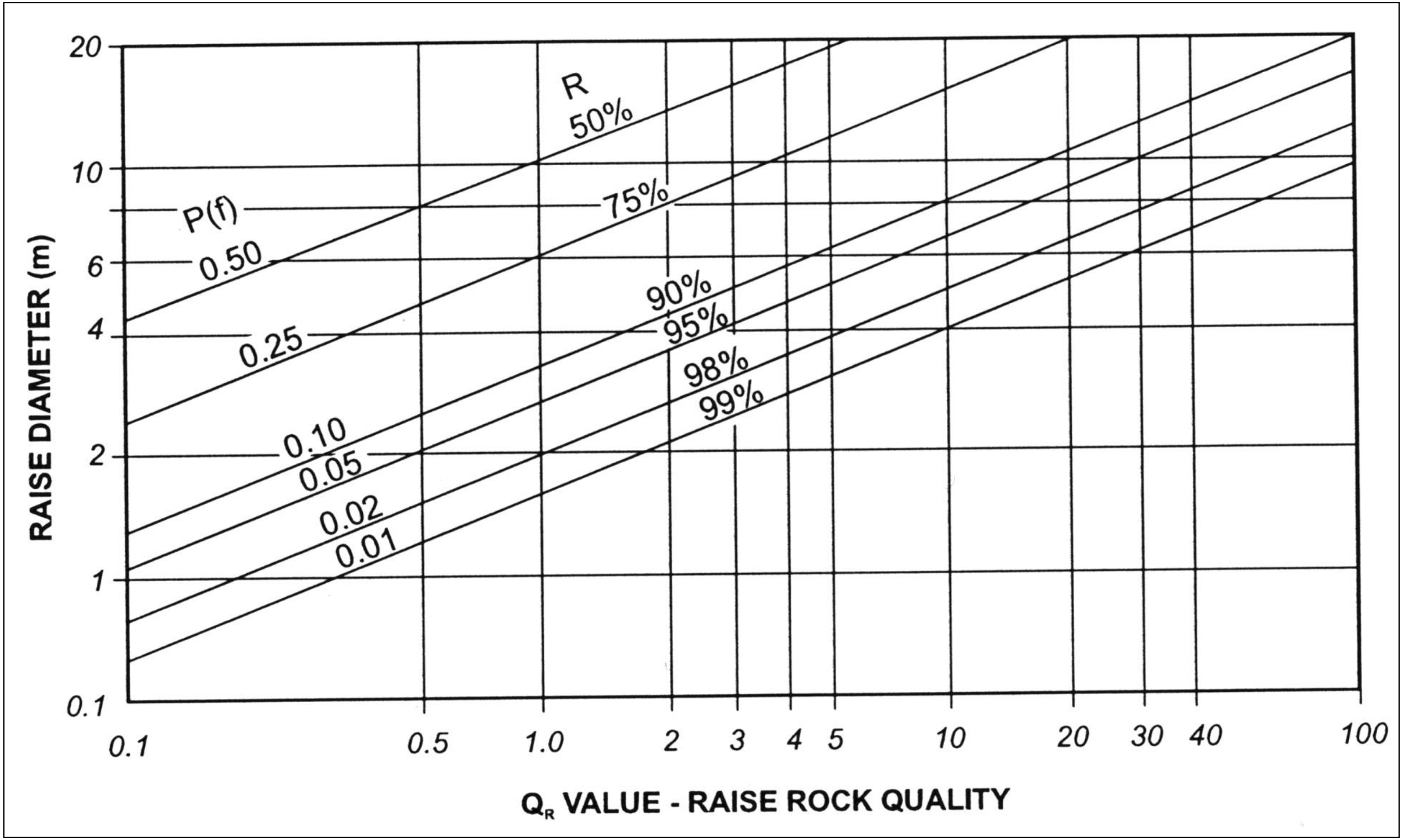


Figure 17 Raisebore reliability chart

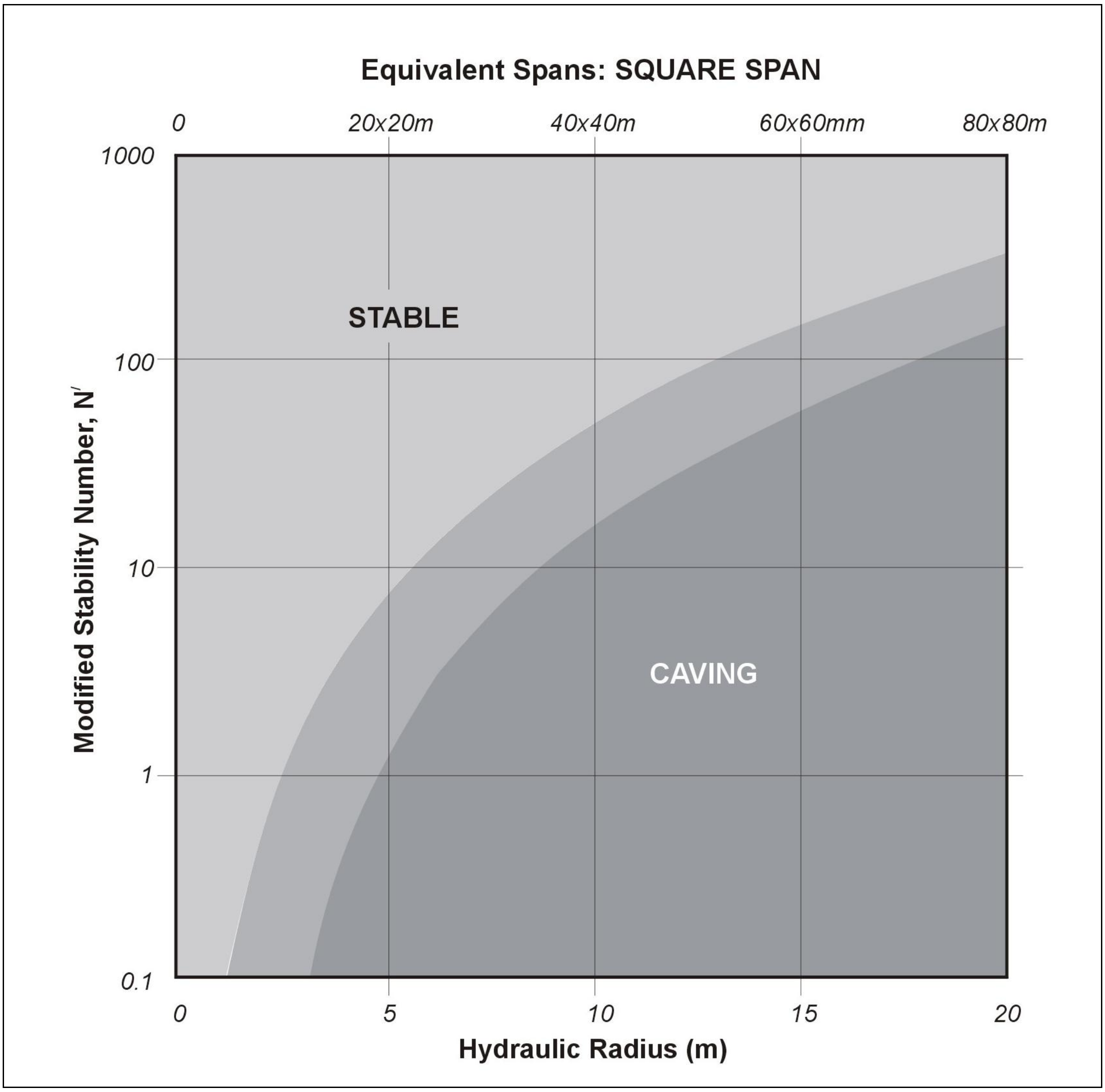


Figure 18 Stability diagram - modified stability number N' (after Hutchinson and Diederichs,1996)

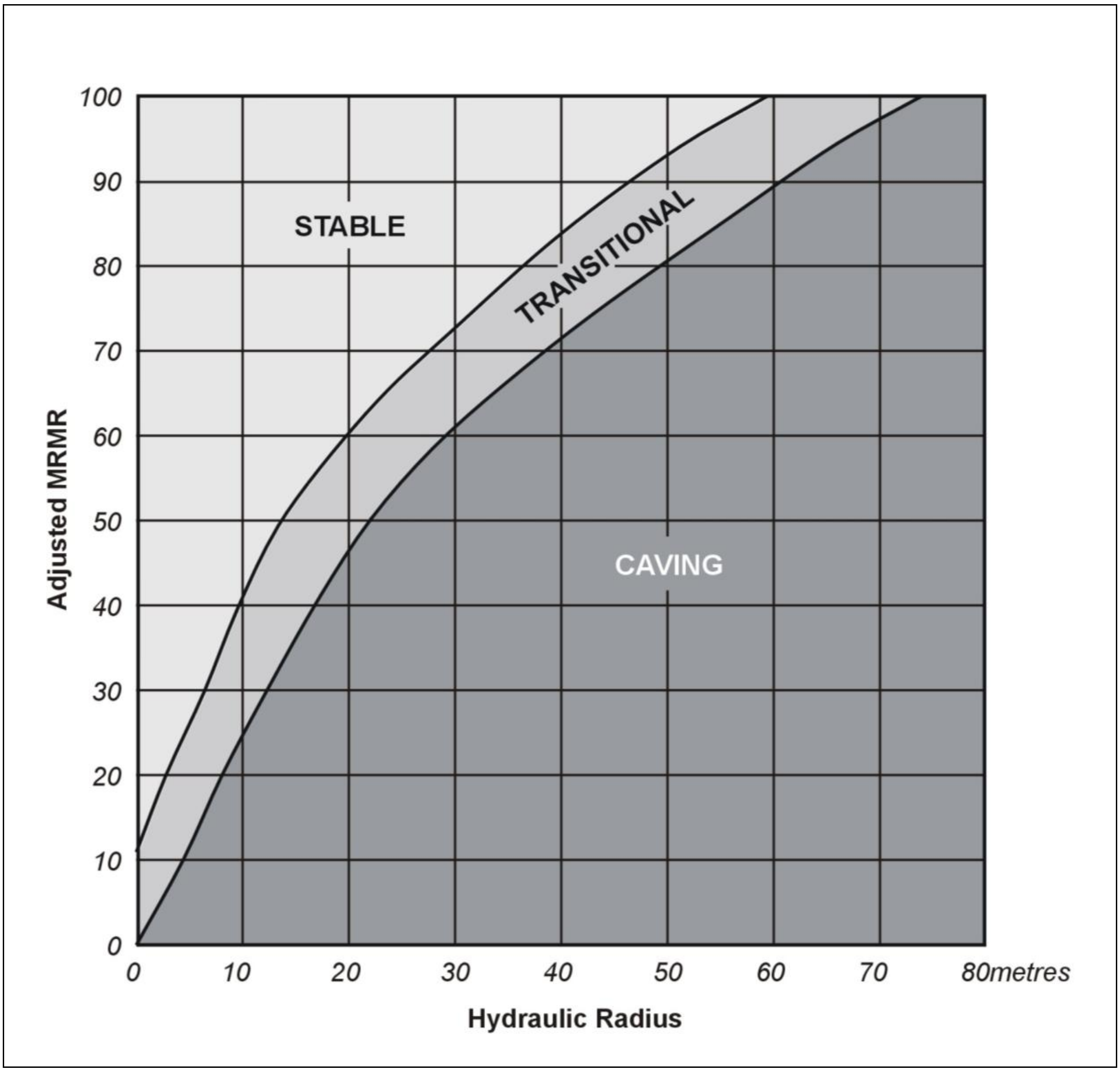


Figure 19 Stability diagram (after Laubscher, 1994)

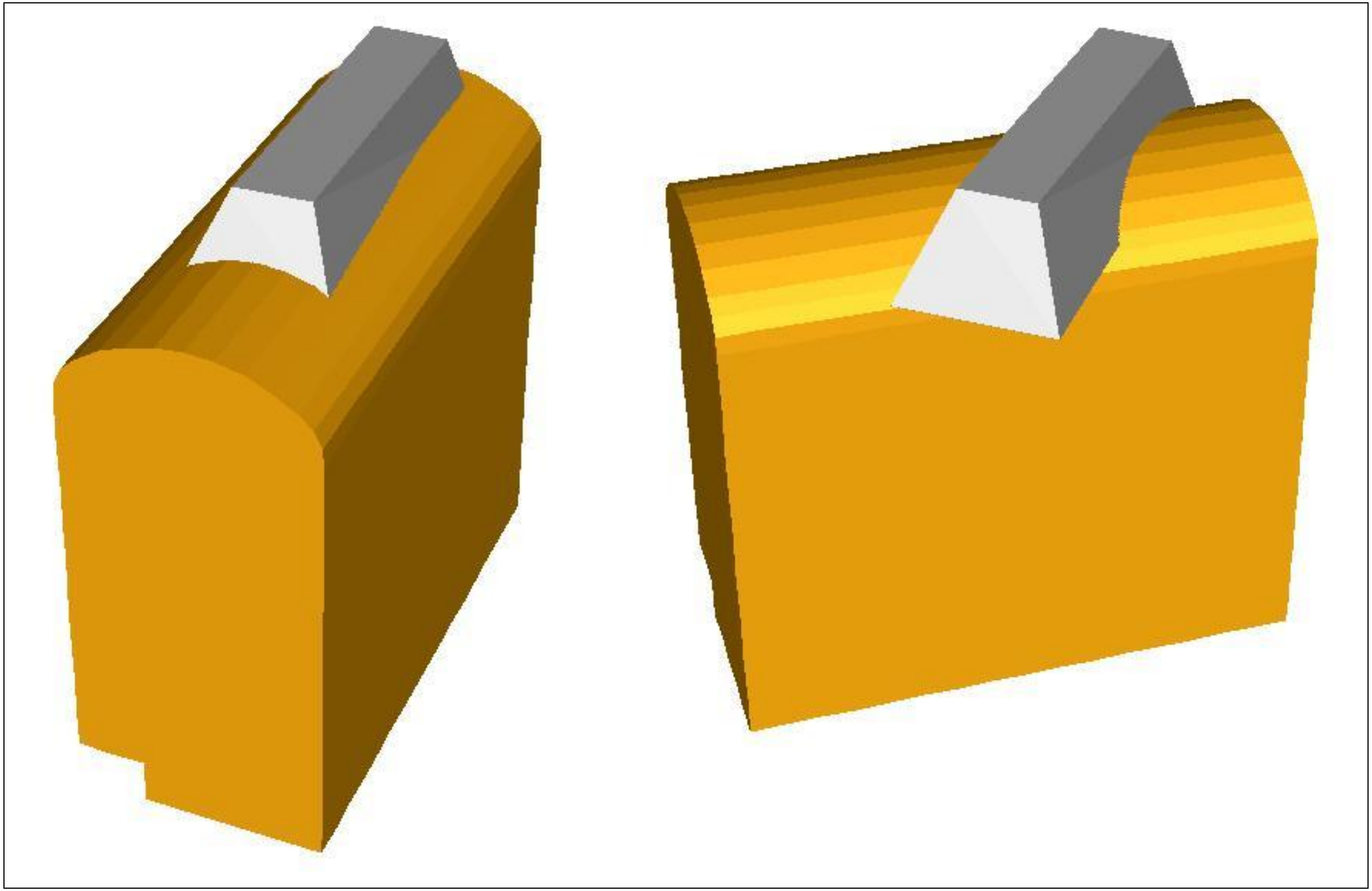


Figure 20 Minimising instability with respect to major geological structures

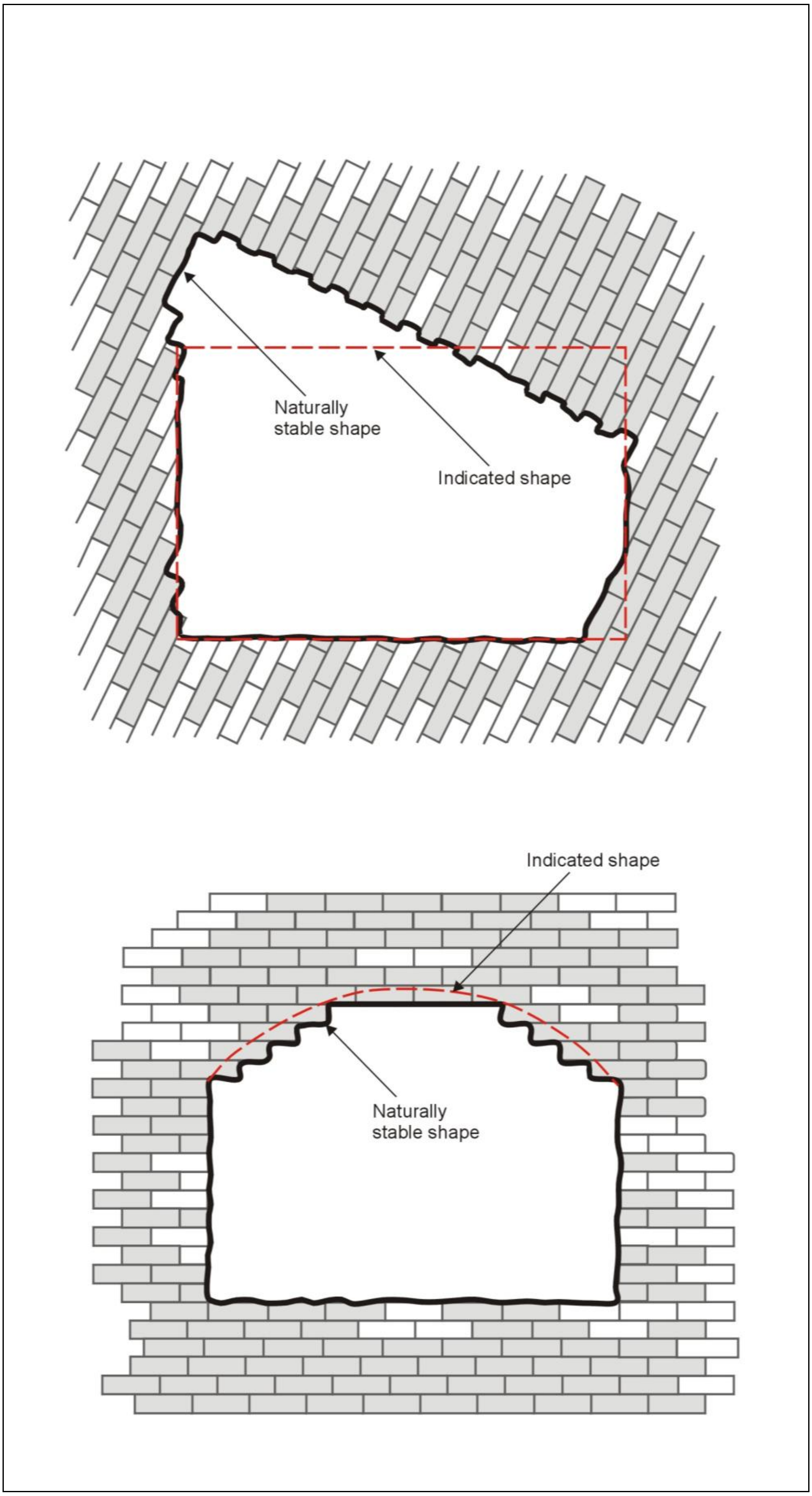


Figure 21 Minimising instability with respect to local geological structure

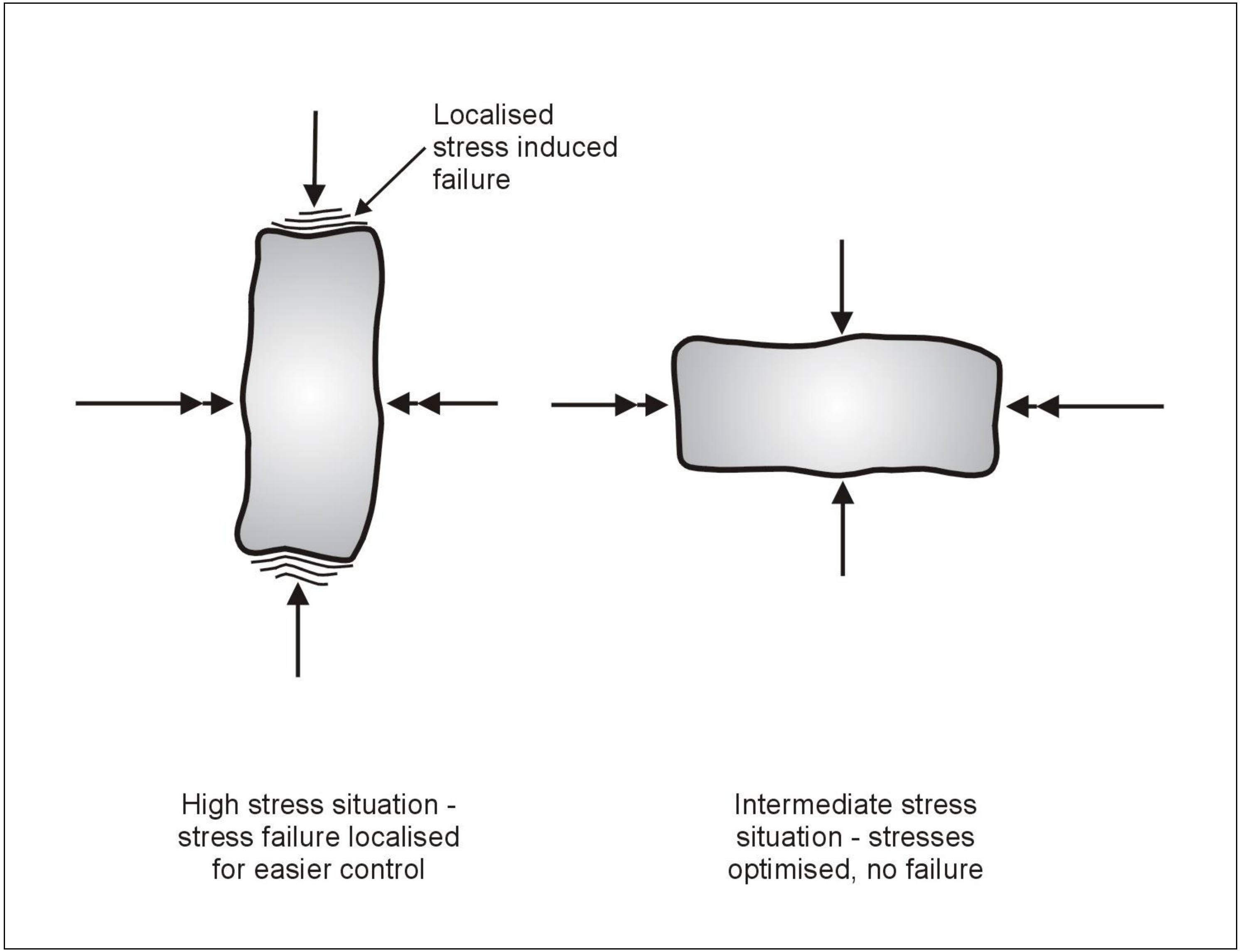


Figure 22 Minimising instability with respect to in situ stress

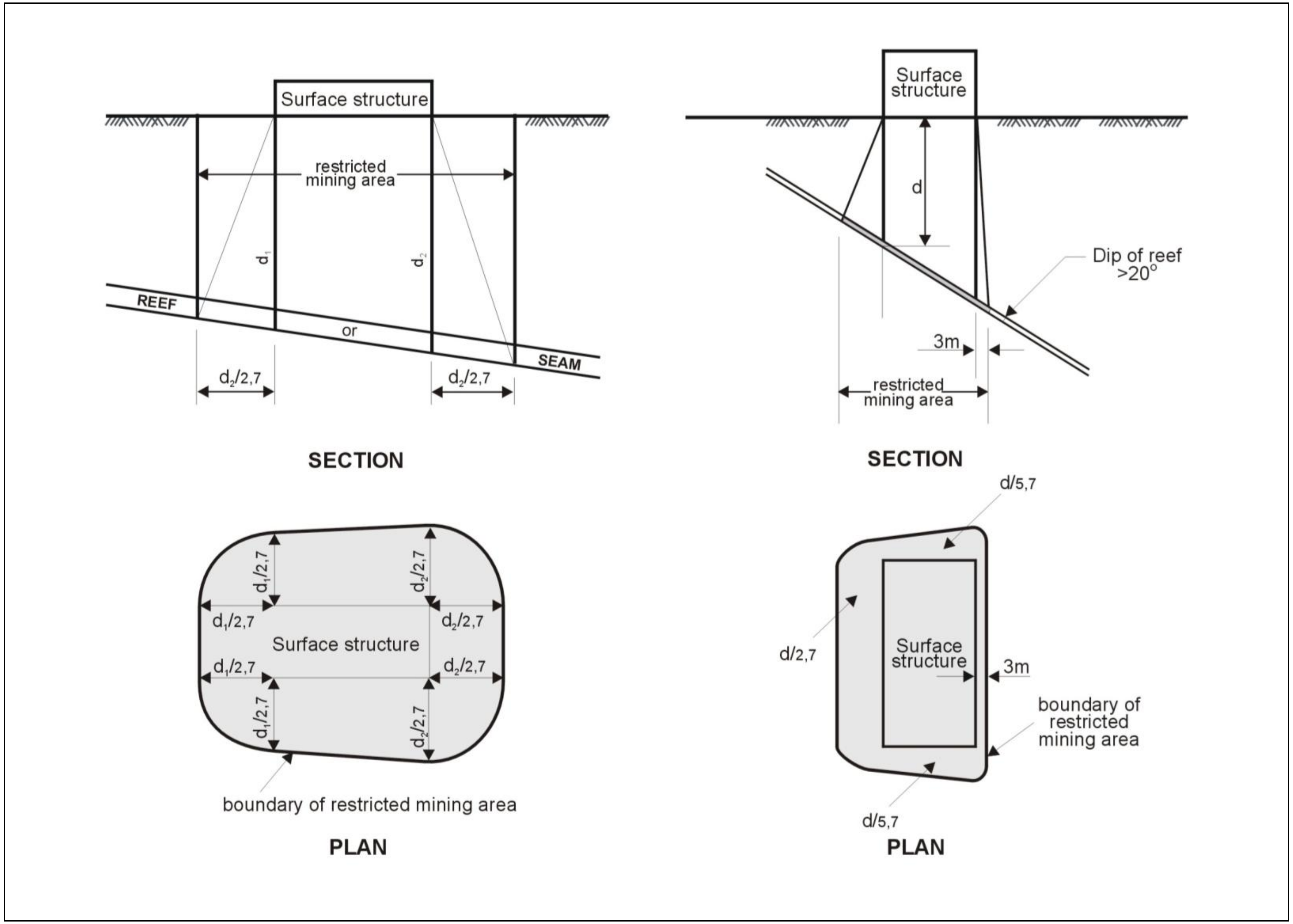


Figure 23 Restrictions on undermining of structures

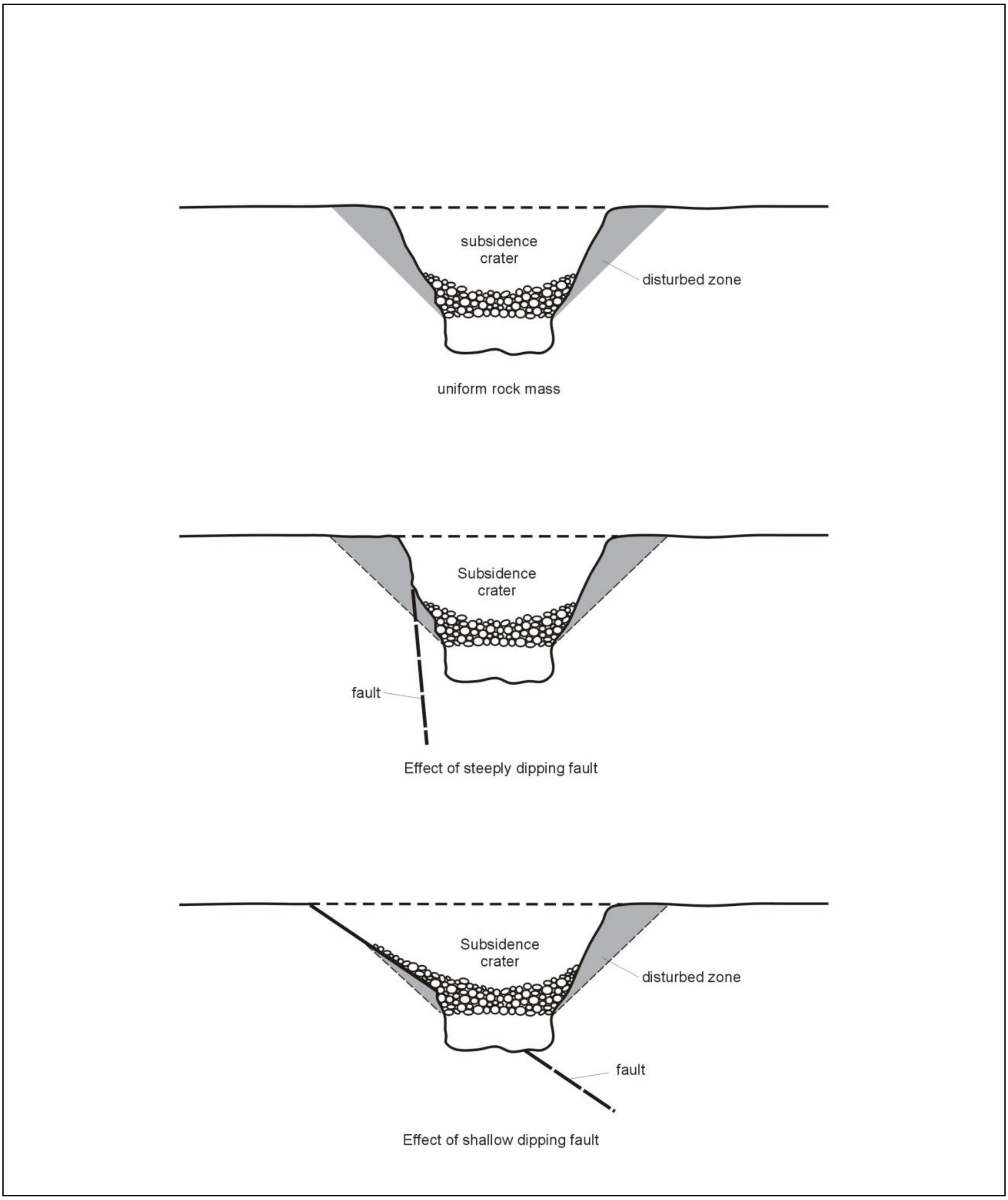


Figure 24 Influence of major geological structural planes on subsidence

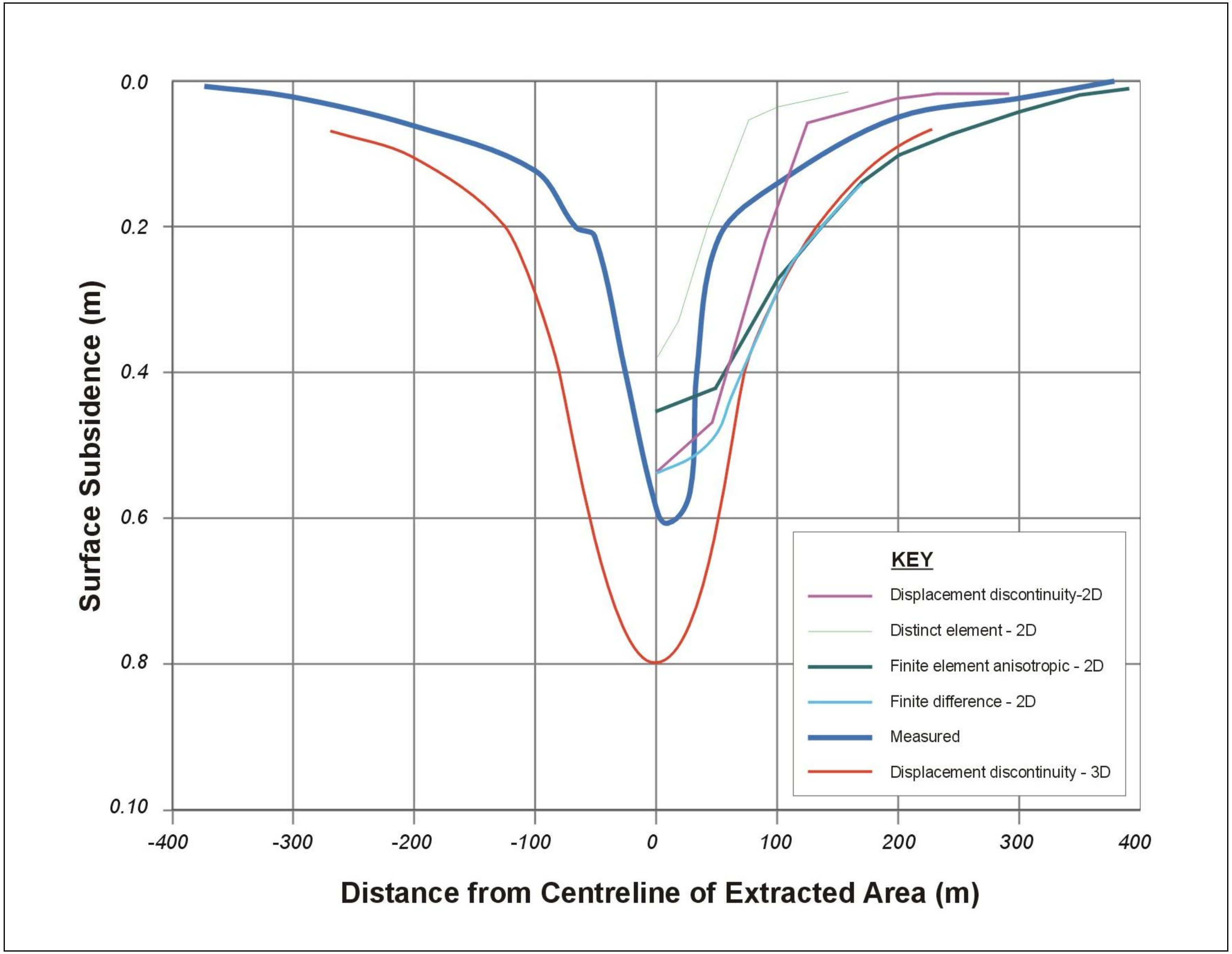


Figure 25 Results of prediction of subsidence by numerical analyses (after Kay et al, 1991)

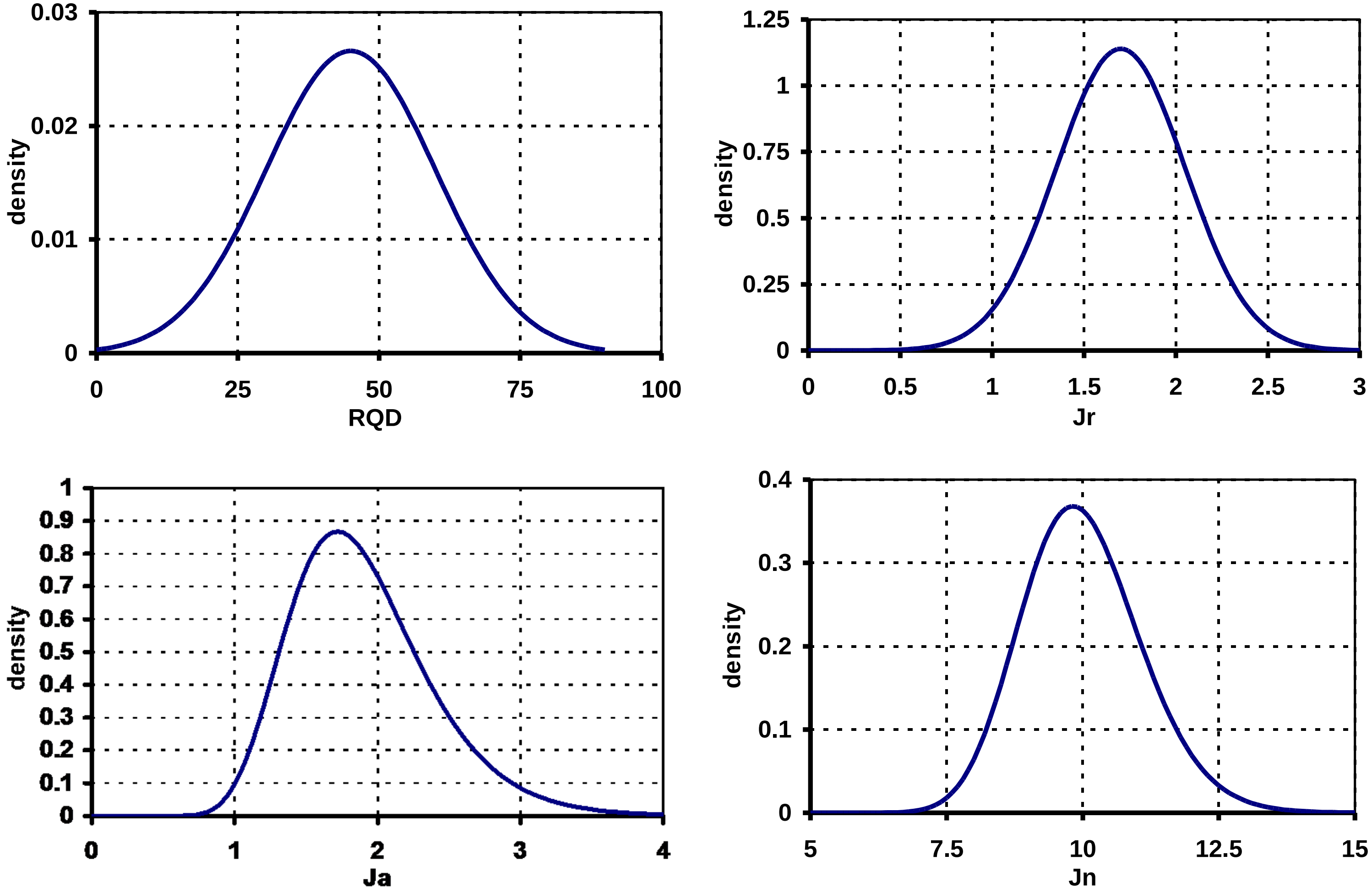
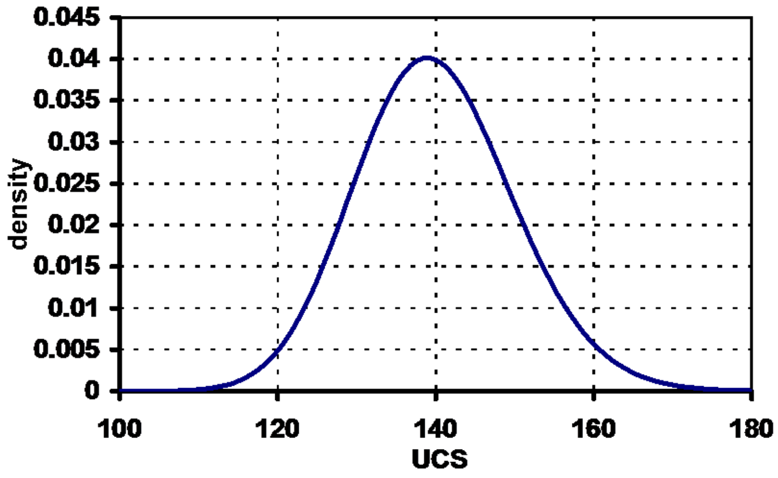


Figure 26 Variation in rock mass classification input parameters



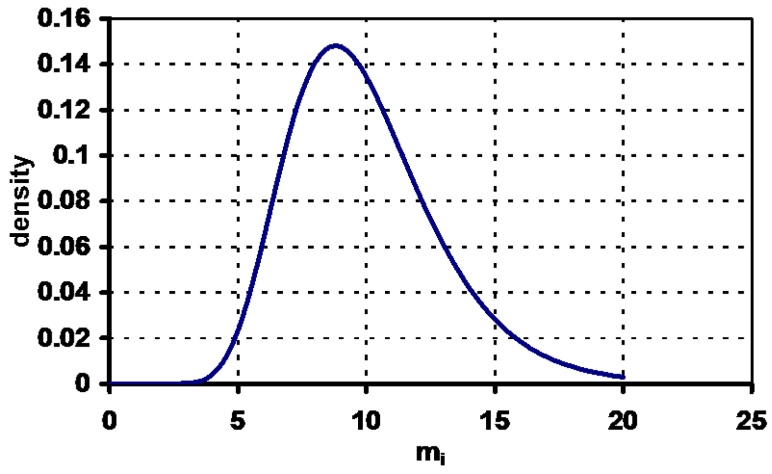


Figure 27 (Cont.) Variation in rock mass classification input parameters

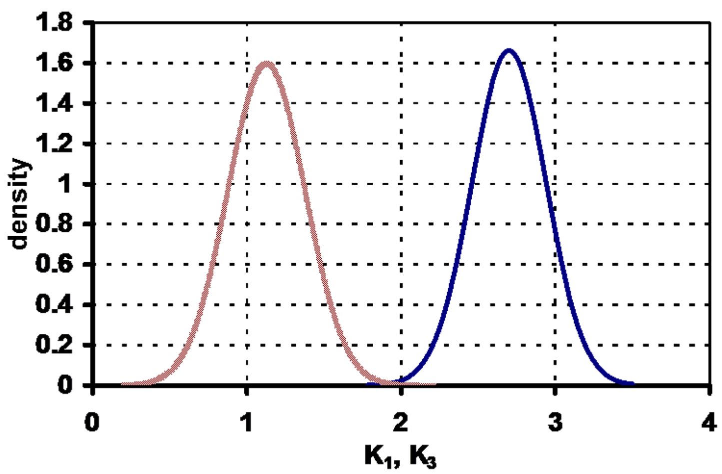


Figure 27 Variation in in situ stress

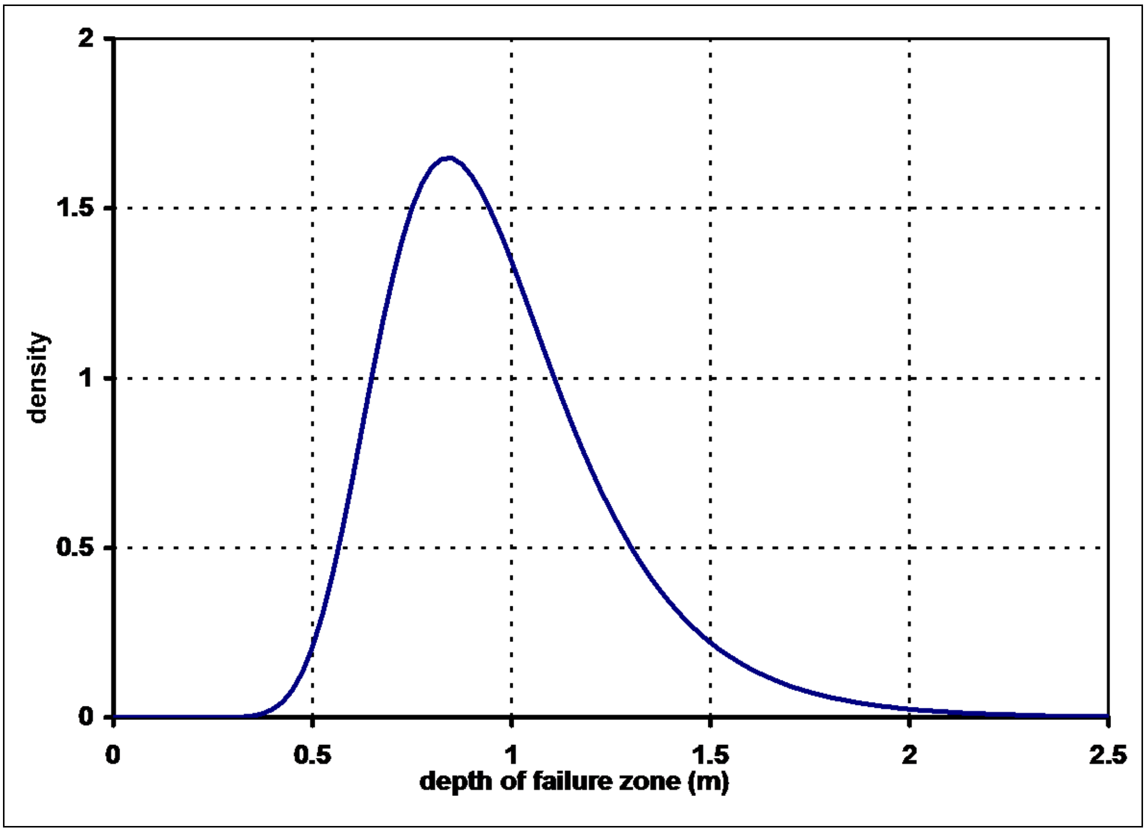


Figure 28 Distribution of depth of potential failure zone

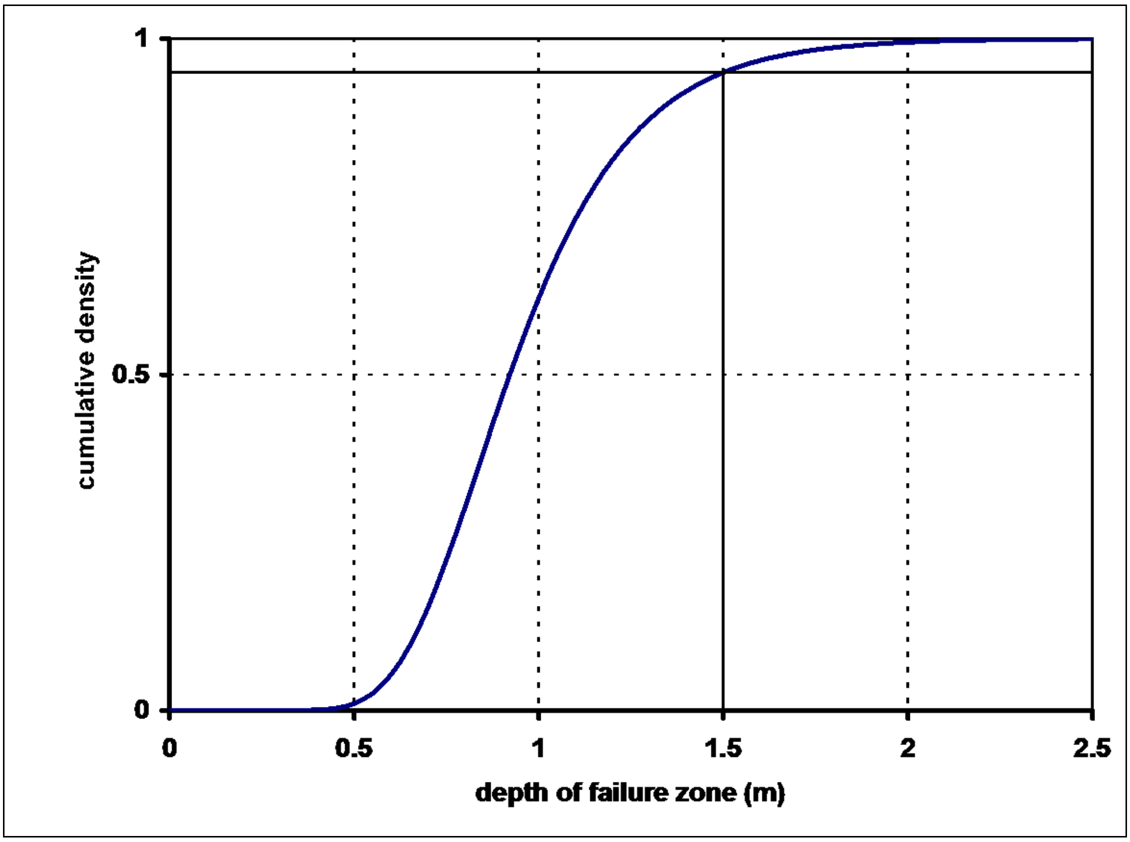


Figure 29 Probability of failure zone exceeding length of rock bolt support

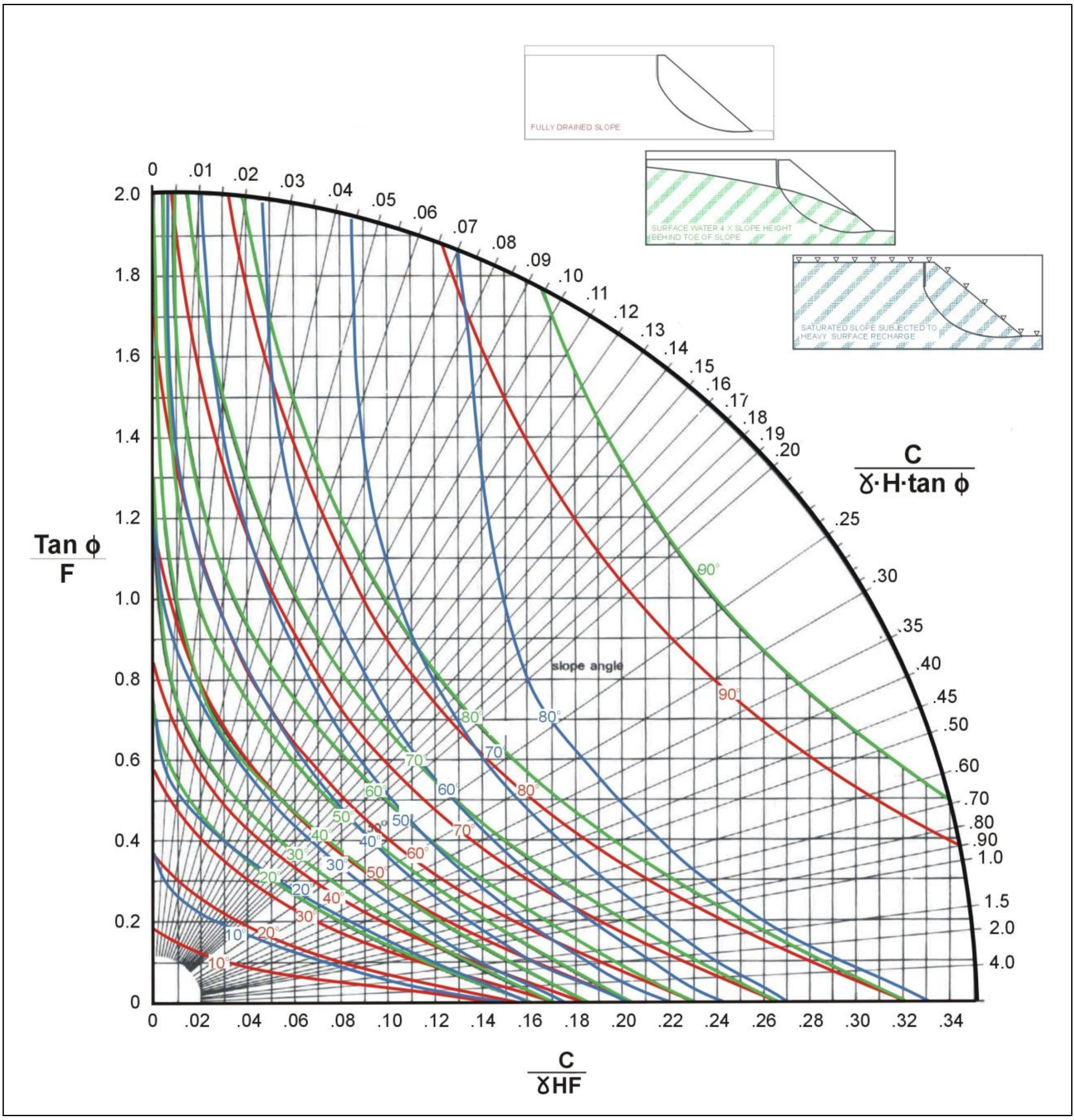


Figure 30 Stability and deisgn chart for soil and weathered rock slopes

Field mapping sheet for the mining rock mass classification

DATE : RECORDED BY: PROJECT (NO): *MRMR* LOCATION:					ROCK TYPE:	SHEET NO:
IN SITU CONDITIONS PARAMETER (CATEGORY) VALUE RATING (RANGE)				ADJUSTED FOR PARAMETER (CATEGORY) VALUE PERCENTAGE (RANGE)		
UCS		(0-20)		WEATHERING		(30 – 100%)
RQD and		(0-15)		ORIENTATION		(63 – 100%)
JOINT SPACING		(0-25)		INDUCED STRESS		(60 – 120%)
Or FRACTURE FREQUENCY		(0-40)		(BORING) BLASTING (SMOOTH) (GOOD CONV) (POOR CONV)		(100%) (97%) (94%) (80%)
(LARGE) JOINT CONDITION (SMALL) (ALTER) (FILLING)		(0-40)				
		TOTAL RATING			TOTAL ADJUSTMENT%:	
ADJUSTED *MRMR* = TOTAL RATING x TOTAL ADJUSTMENT =				COMMENTS :		

ADJUSTED *MRMR*	100-81	80-61	61-41	40-21	20-0
ADJUSTED CLASS	A1 B	A2 B	A3 B	A4 B	A5 B
SLOPE ANGLE	±75°	±65°	±55°	±45°	±35°
RECOMMENDED SLOPE ANGLE :					

Figure 31 Field mapping sheet for the Mining Rock Mass classification

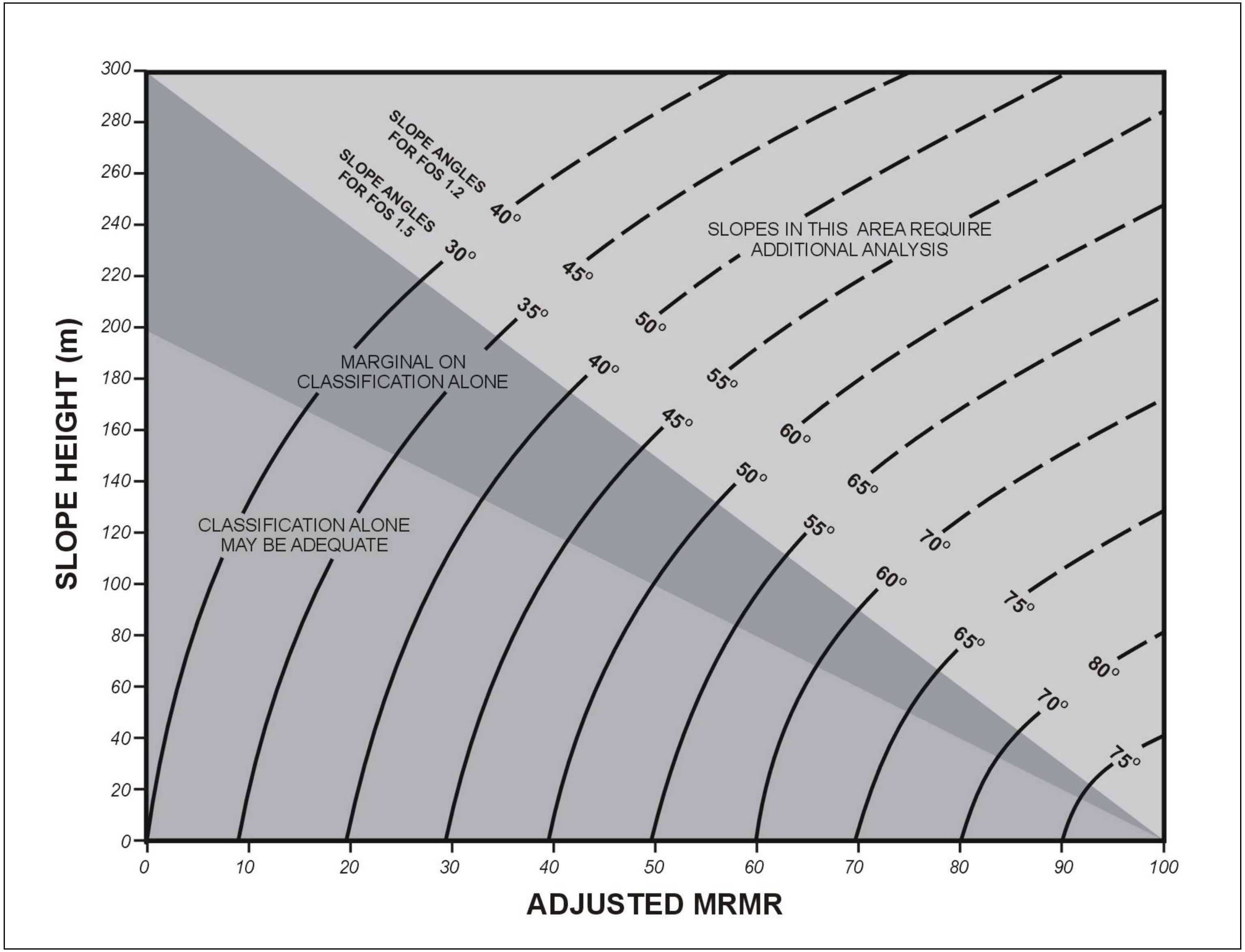


Figure 32 Empirical slope design chart (after Haines and Terbrugge, 1991)

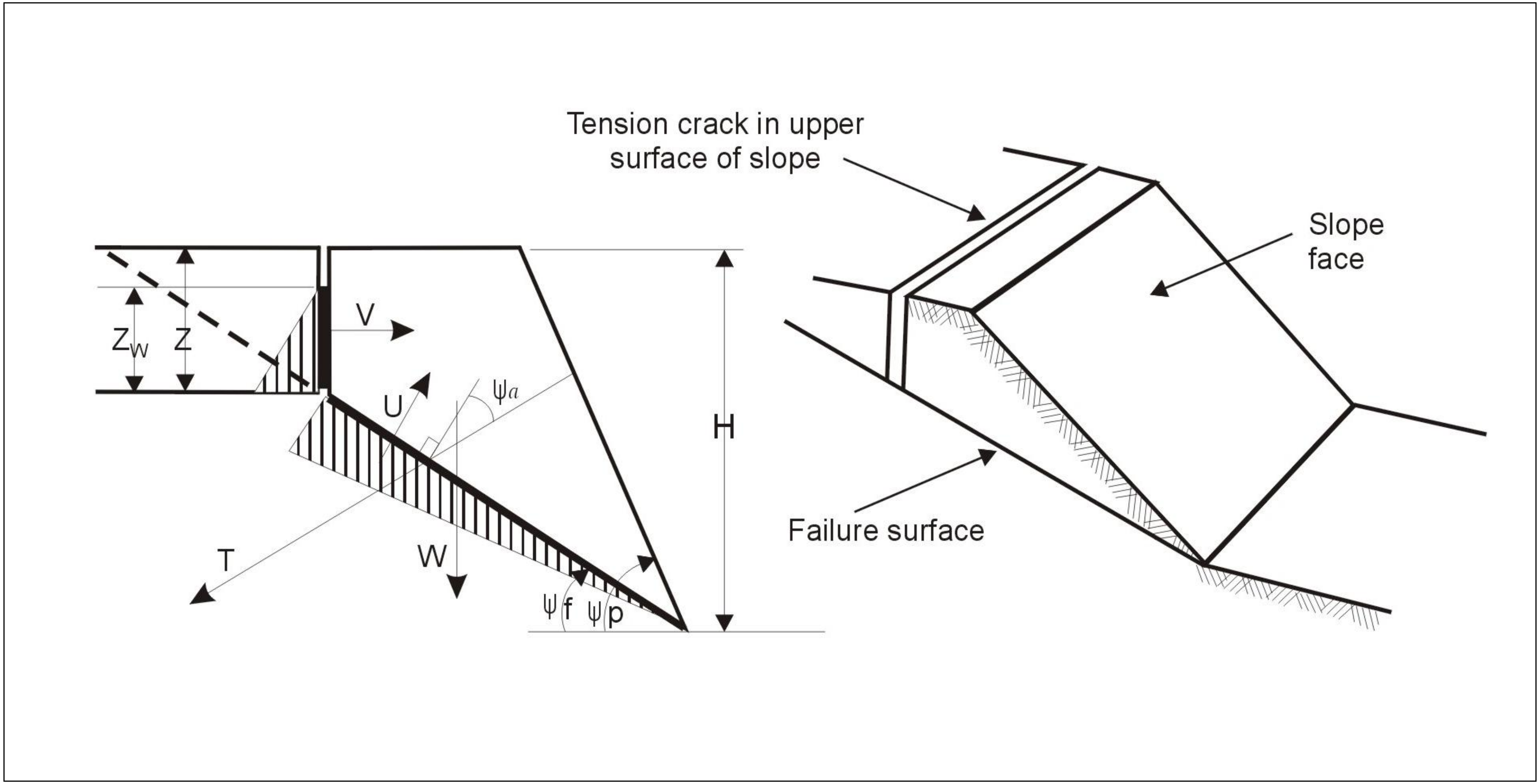


Figure 33 Planar wedge mode of slope instability

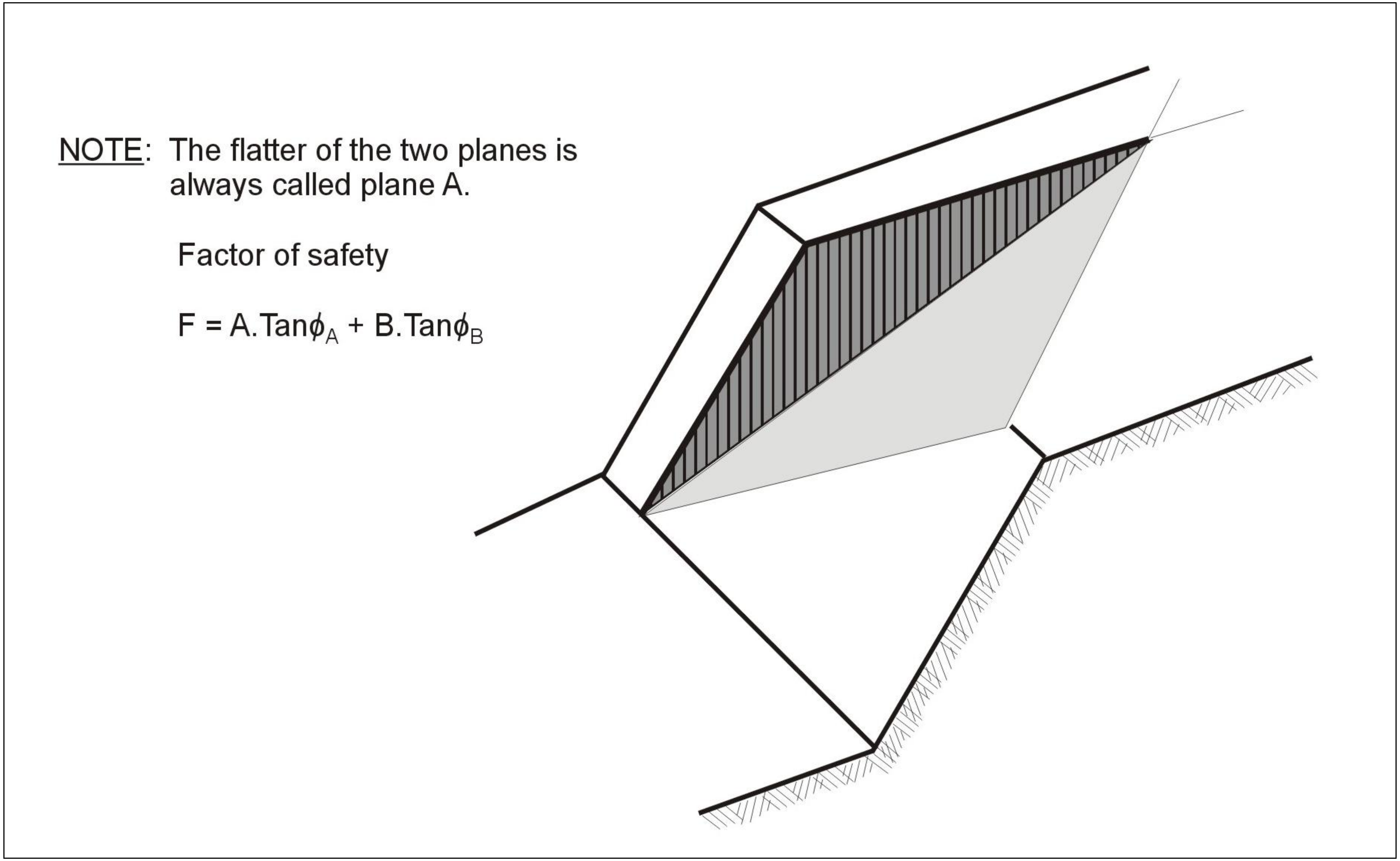


Figure 34 Three dimensional wedge mode of slope instability

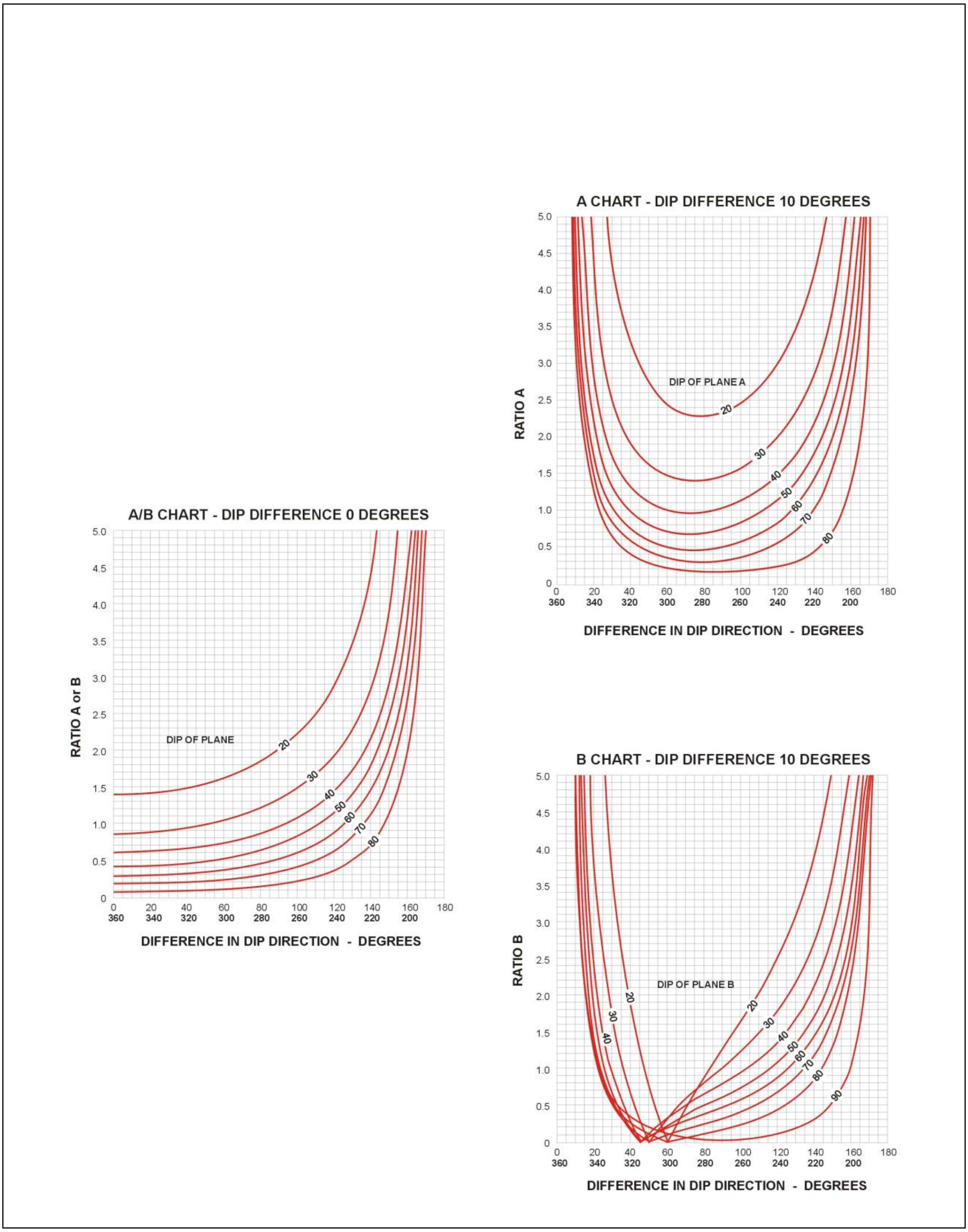


Figure 35 Stability and design charts for 3D wedges - dip difference 0° and 10°

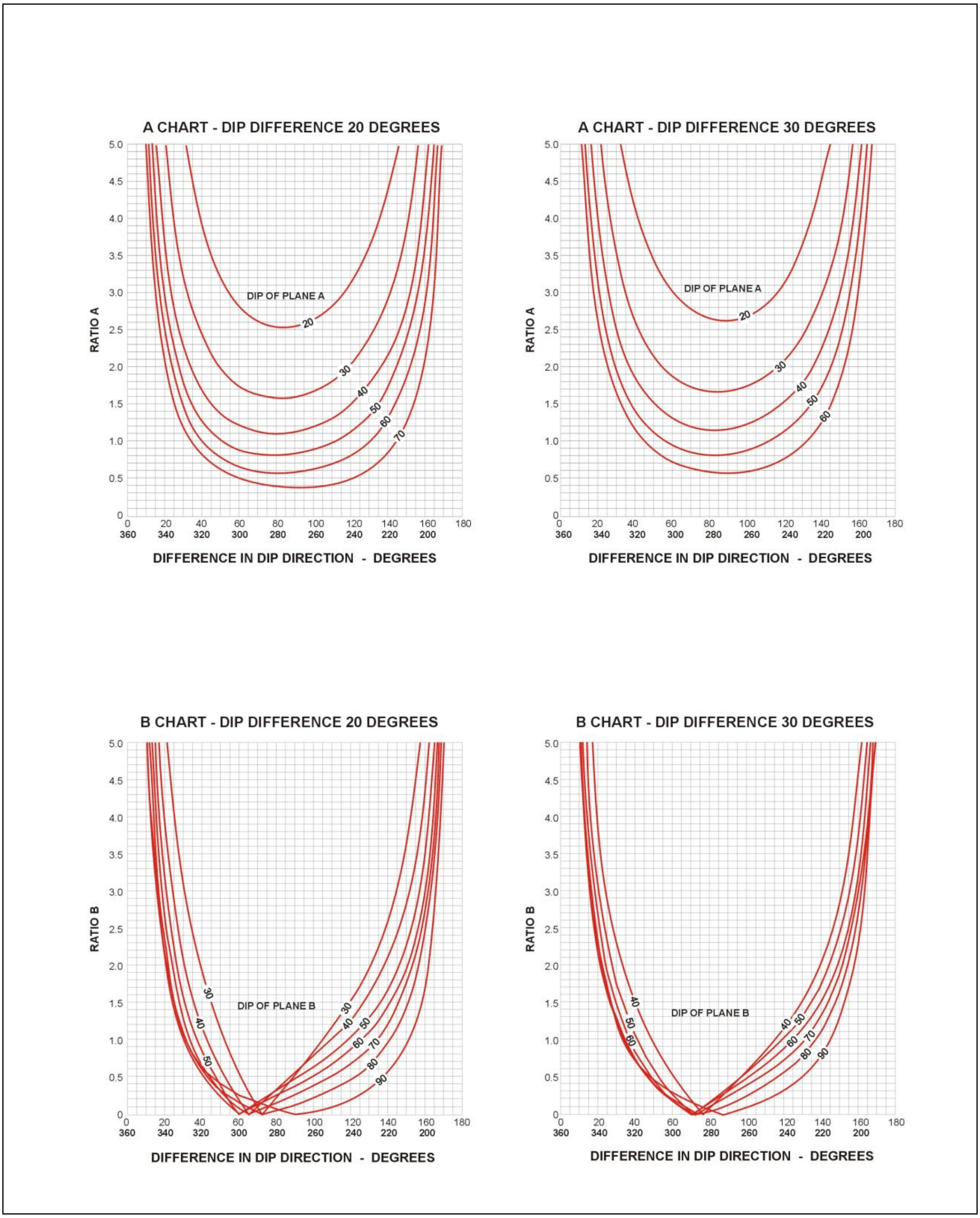


Figure 36 Stability and design charts for 3D wedges - dip difference 20° and 30°

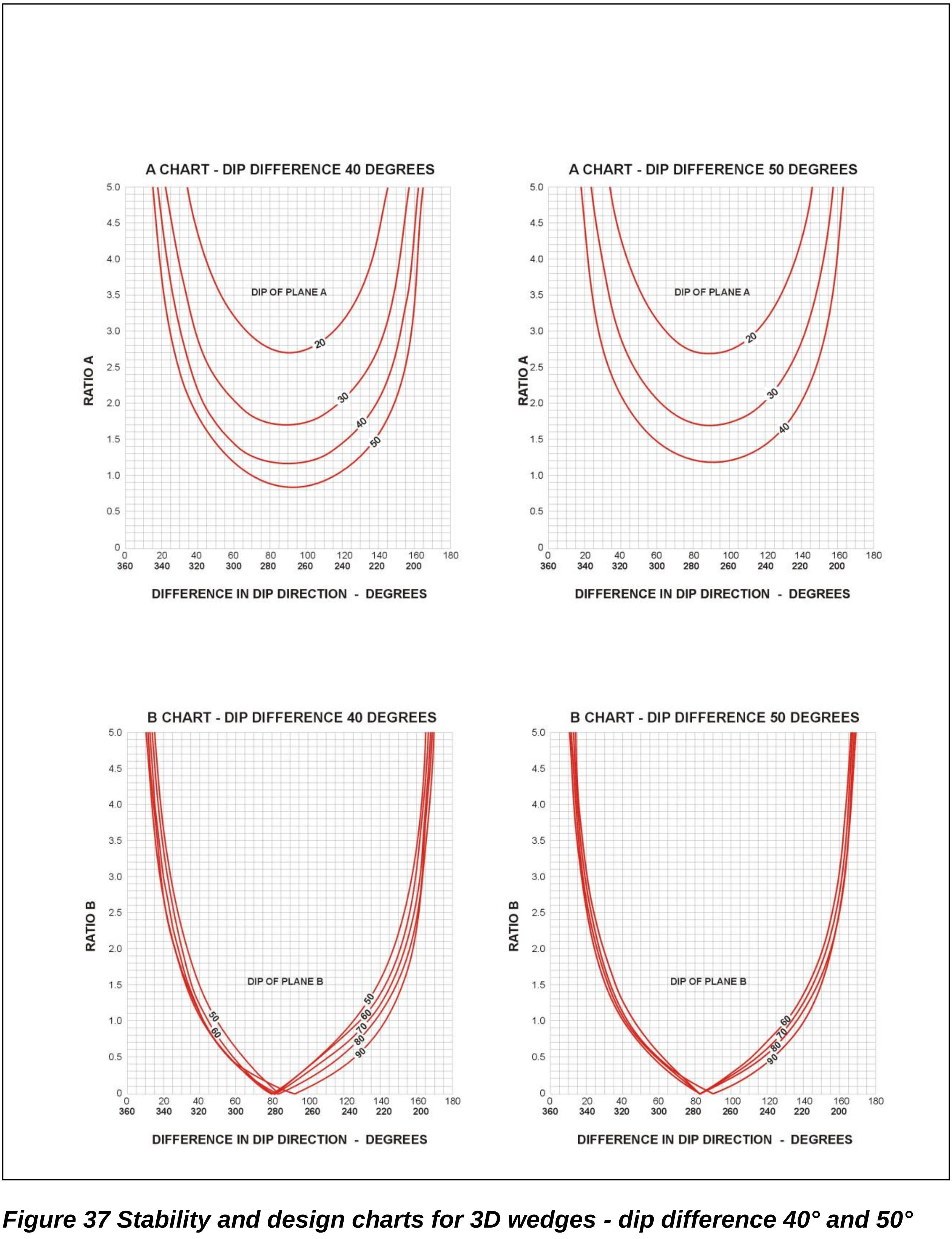


Figure 37 Stability and design charts for 3D wedges - dip difference 40° and 50°

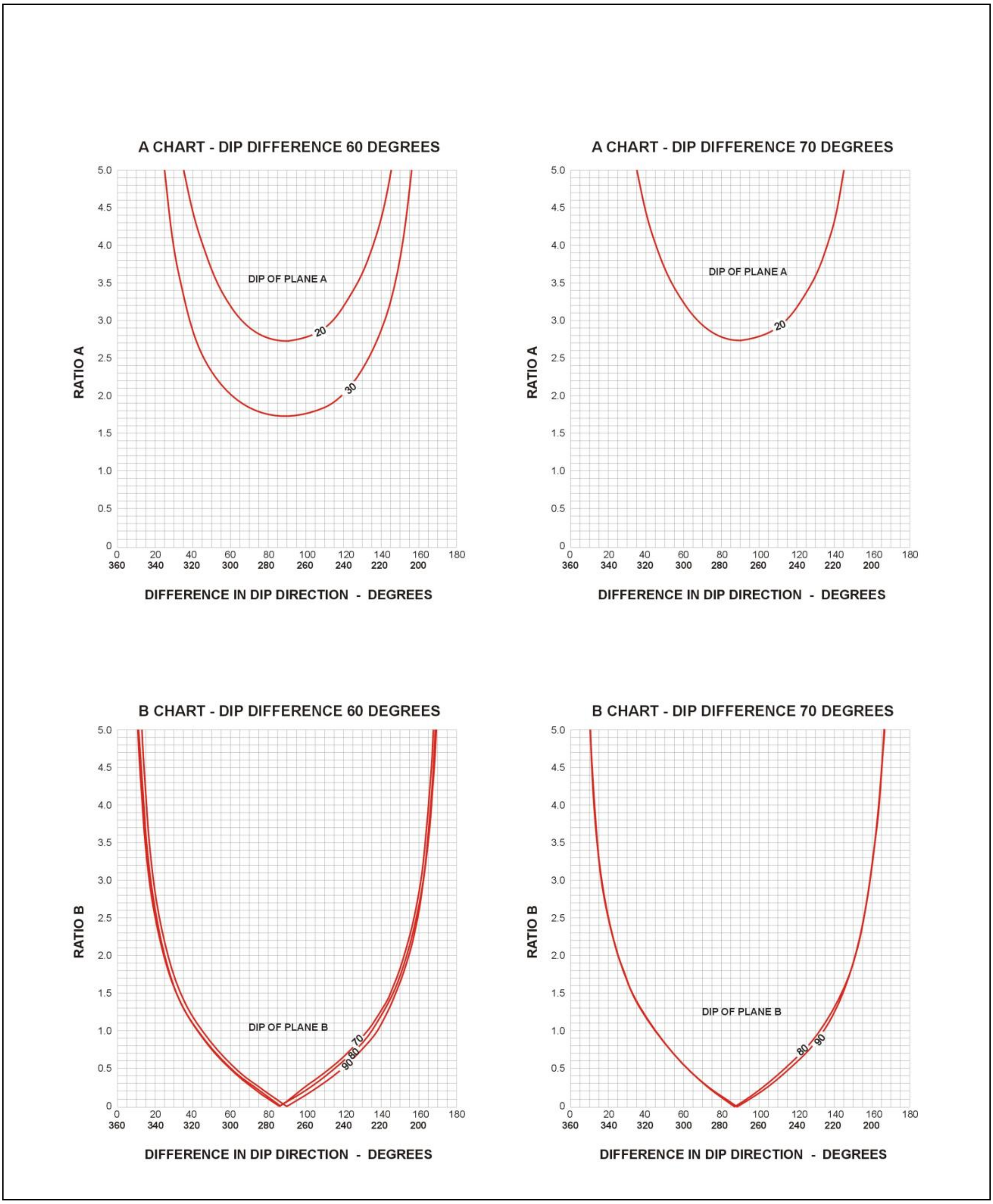


Figure 38 Stability and design charts for 3D wedges - dip difference 60° and 70°

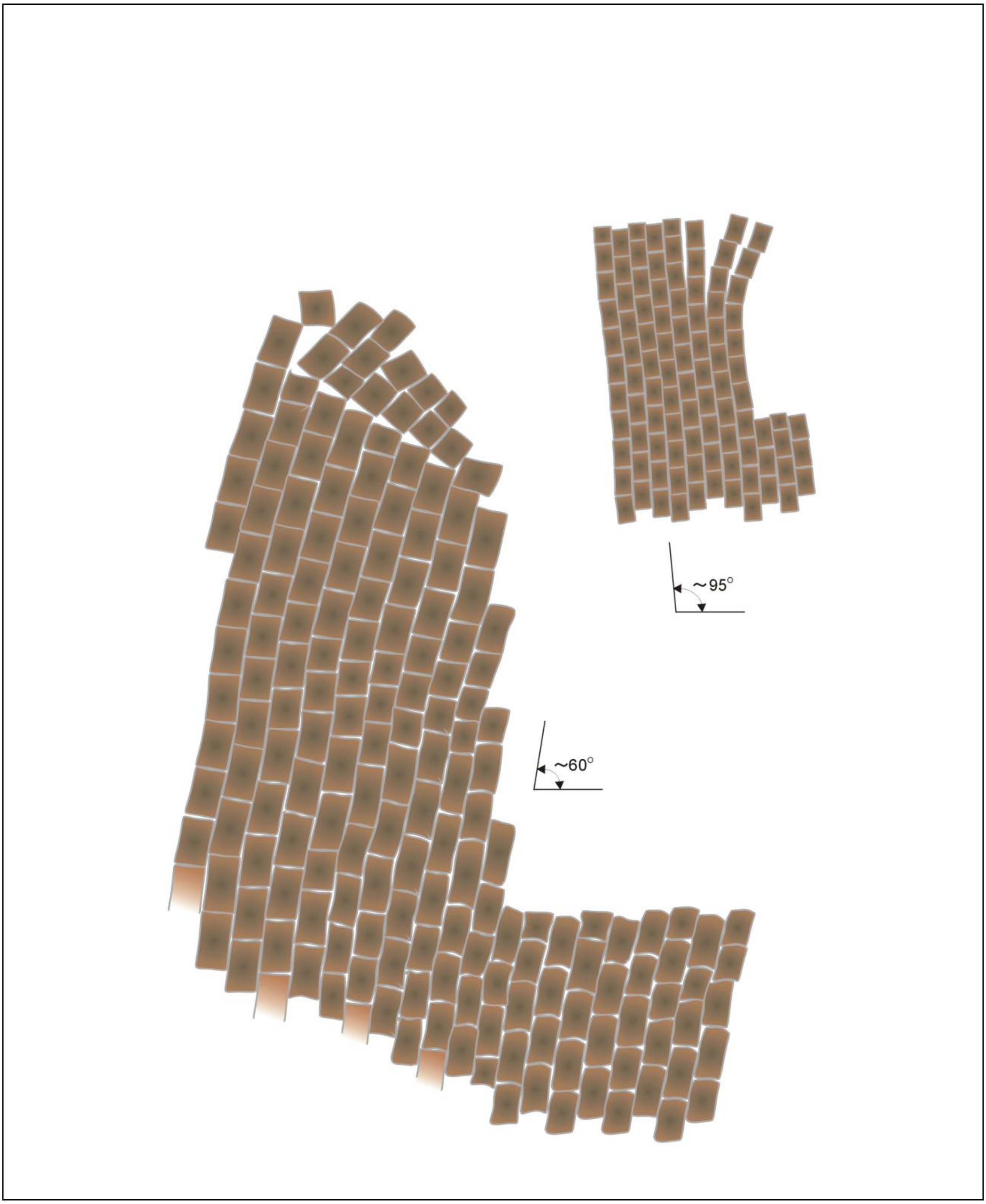


Figure 39 Potential for toppling failure

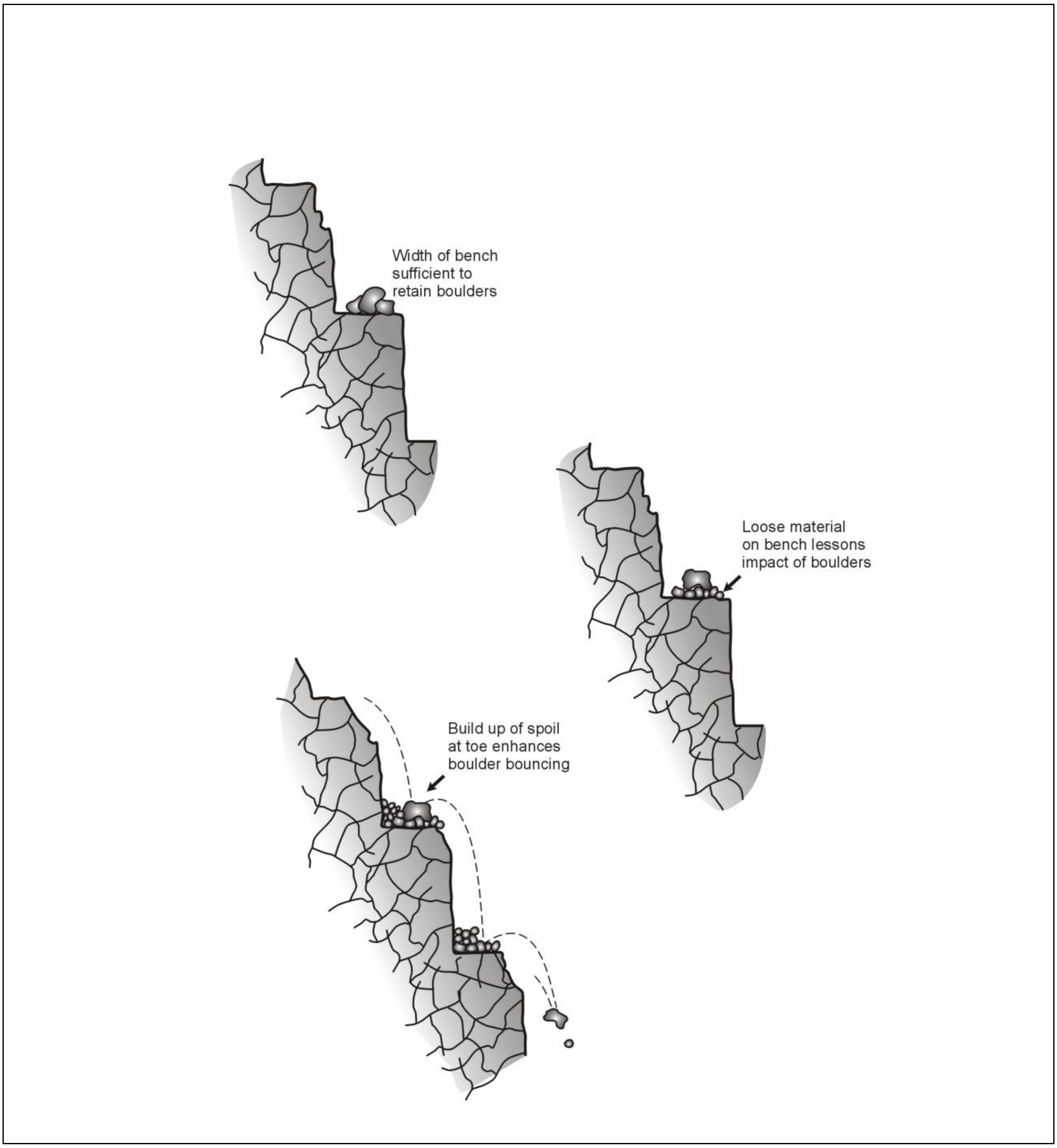


Figure 40 Accumulation of spoil on benches

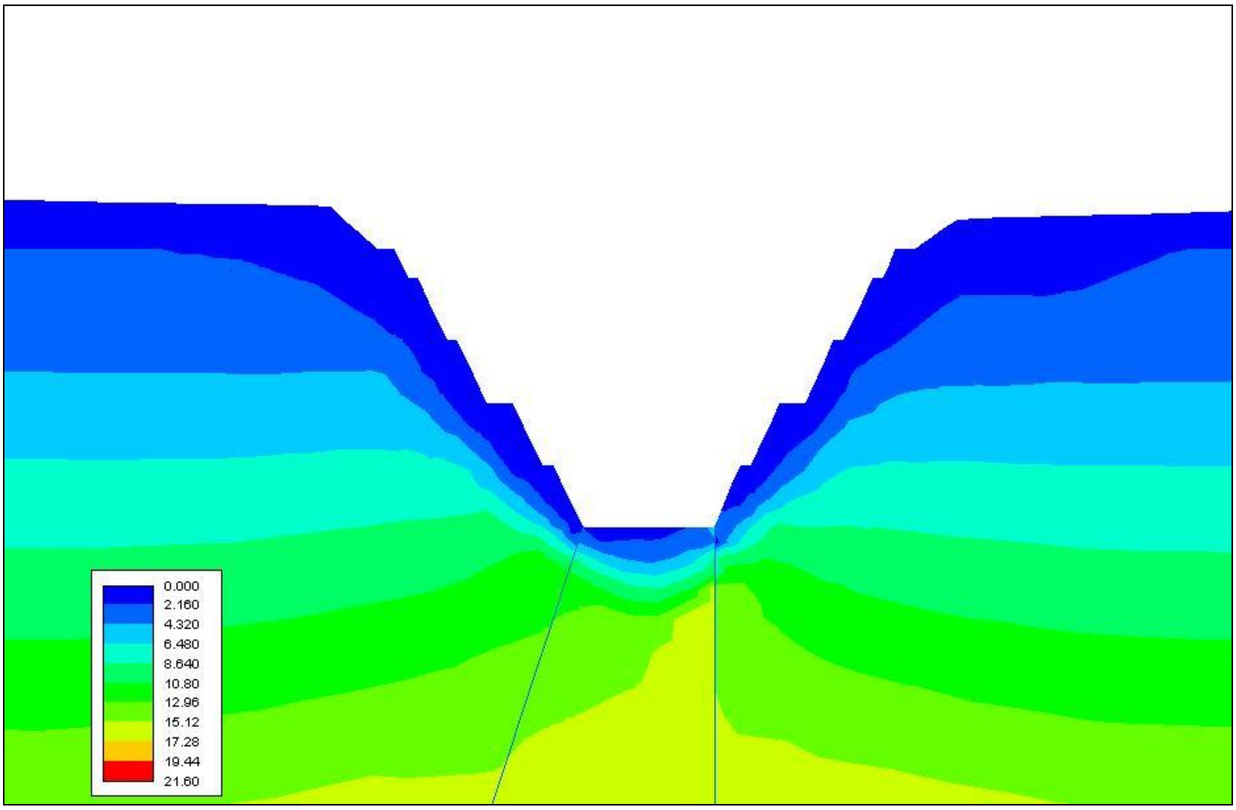


Figure 41 Stress distribution in open pit mine slopes

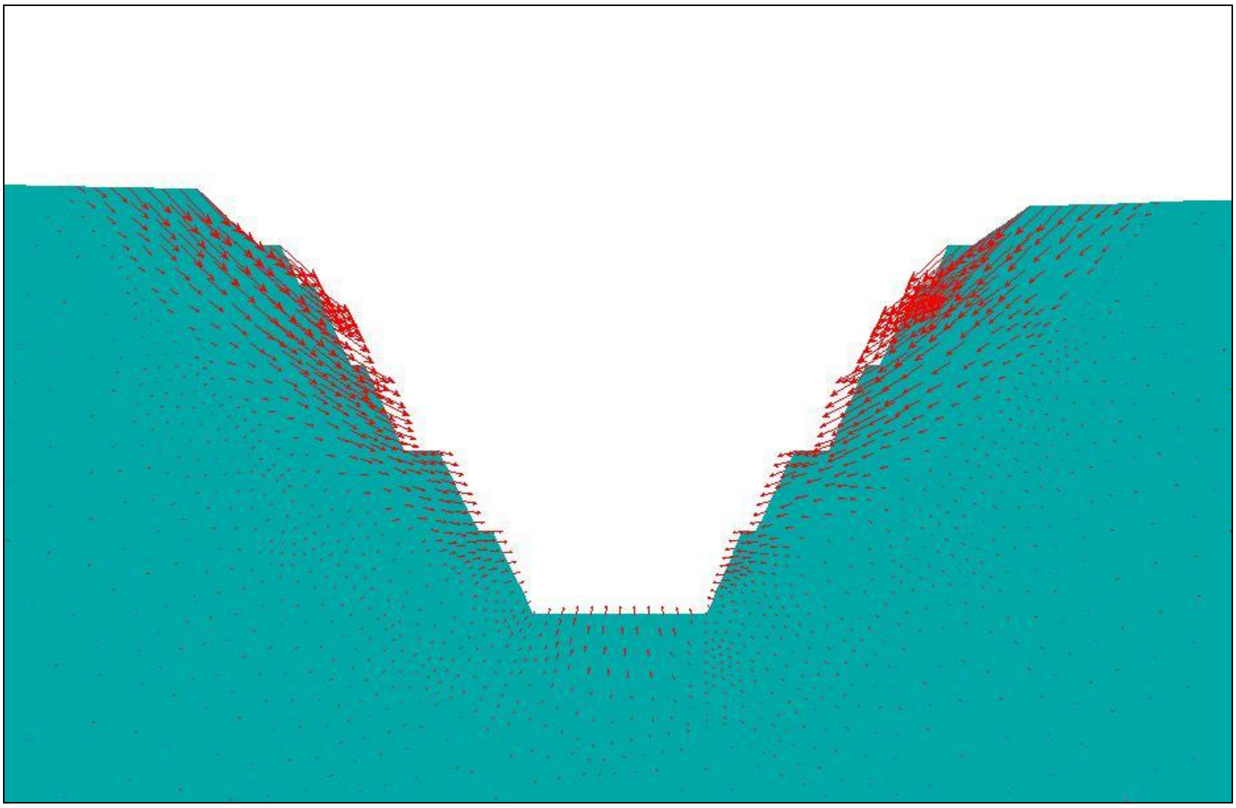


Figure 42 Deformation of open pit mine slopes

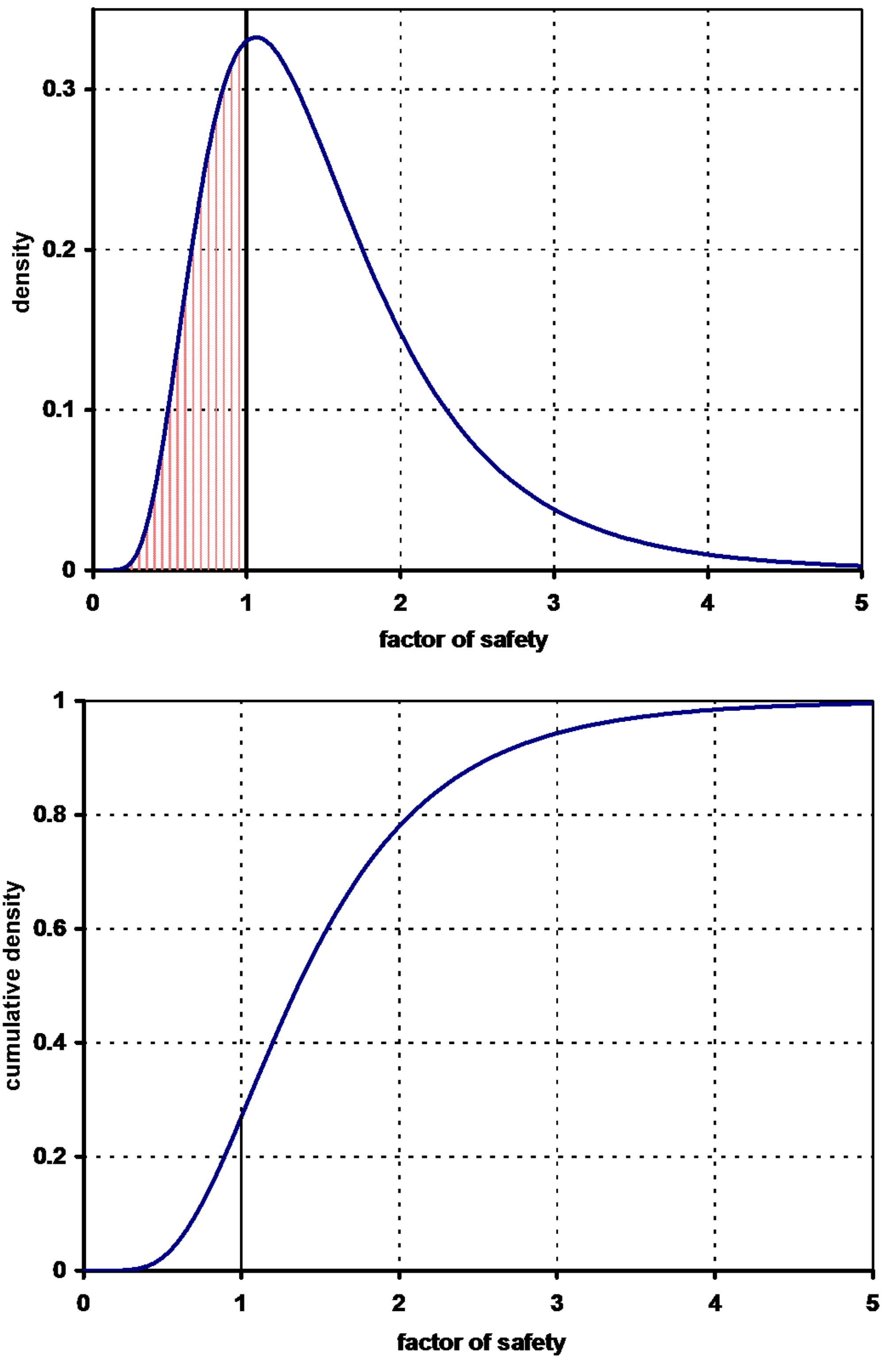


Figure 43 Distribution of safety factor

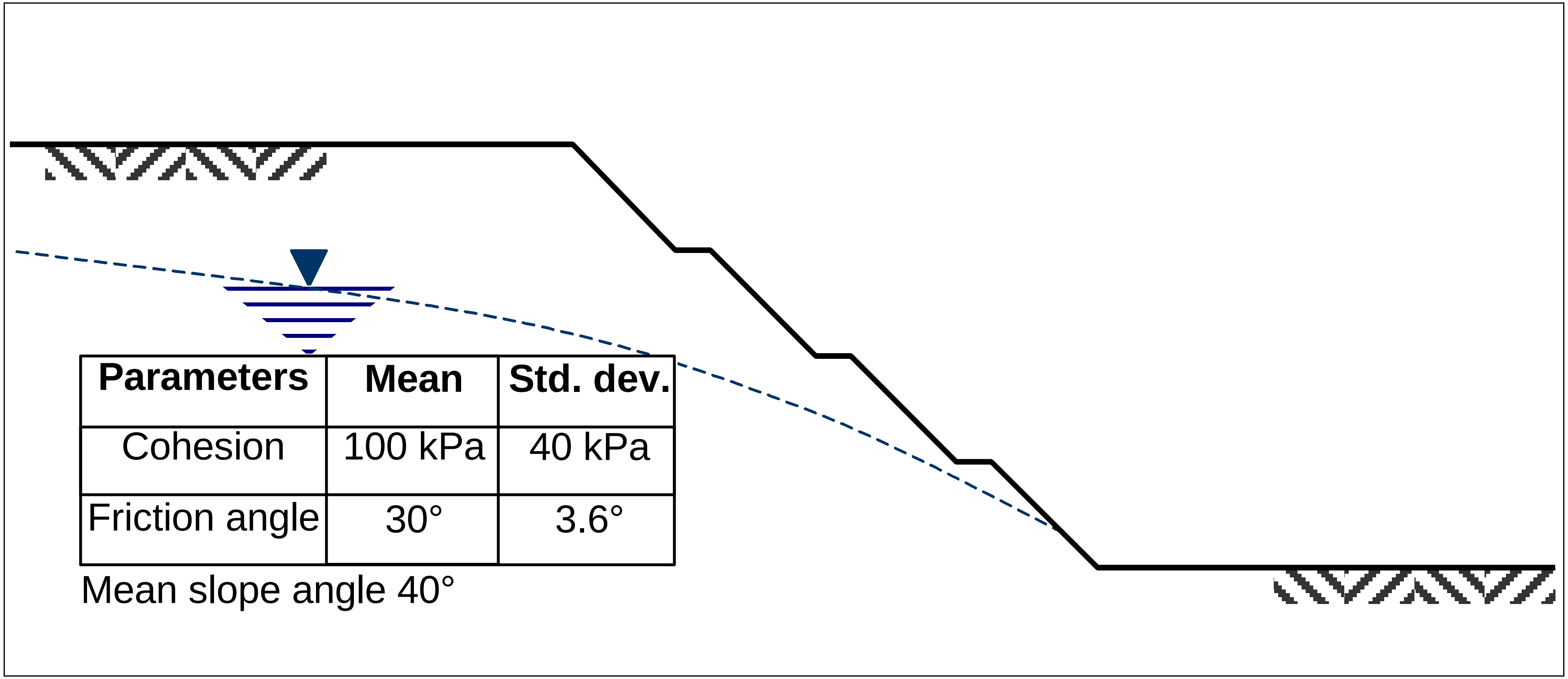


Figure 44 Slope in weathered material

4 Support

Support of various mining openings is dealt with in this chapter. The consideration of support is split into 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. Methods for determining suitable dimensions for such pillars will be dealt with
in the following sections.

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.

Once these two aspects are known, individual pillars and pillar layouts can be designed, according to the degree of stability required. Pillar strength, pillar stress and the design procedure are dealt with below.

Pillar strength

It is generally accepted that the strength of pillars is 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.

Several pillar strength formulae have been developed to take into account these factors. One such formula is the following:

$$P_{strength} = K.\ W_{eff}^{\alpha}/H^{\beta}$$

Where

$P_{strength}$ is the pillar strength

K is the "large" scale strength of the rock mass, say of $1m^3$

W_{eff} is the effective pillar width

α and β are constants

Since pillars are often irregular in shape, in particular in massive mining and in situations in which orebody thickness is variable, pillar widths and shapes may vary considerably. The effective pillar width, which has been included in the above formula, takes these variations into account.

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

The values of the constants in the above equation will vary for each mine, and will be best determined by back analysis of the observed behaviour of pillars on the mine. However, such back analysis is often not possible, and design of pillars must be carried out without this information. In such cases it is recommended that the α and β exponents in the formula are those derived for hard rock pillars by Hedley and Grant (1972). It is also recommended that the value used for K is Laubscher's design rock mass strength (*DRMS*), the derivation of which is described in Section 2.2.4.1. The pillar strength formula recommended for use in design and stability evaluation therefore becomes:

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

It must be noted that this is a general formula, and its applicability to particular mining cases needs to be verified by observation of pillar behaviour and correlation of this behaviour with the expected behaviour determined from the application of the formula.

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 45 provides a guideline relationship between *DRMS* and *RMR* for different values of *UCS*. This guideline can be used to obtain a quick estimate of *DRMS* or to provide a *DRMS* value when very limited data are available. It does not replace the required detailed evaluation of *DRMS* which should be carried out when sufficient data are available.

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.

The presence of jointing in a pillar will decrease its strength since the joints represent defects. This is taken into account in a general sense by the use of the *DRMS* which is based on rock mass classification. However, if the joints are inclined, they may have a significant weakening effect on the pillar, and this will be accentuated for smaller width to height ratios. Since slender pillars are more common in massive mining, it is particularly important that the influence of inclined jointing is taken into account in the design of pillars for such situations. Adjustments to the pillar strength (as determined from the above pillar strength formula) for the effect of jointing may be determined from Figure 46 and Figure 47. These figures are based on the work of Esterhuizen (1998) and take into account the orientation of the jointing, the intensity of the jointing, and the pillar width to height ratio.

Recent research work carried out into the design of pillars in areas of variable topography (Swart et al, 2000) has shown that dimensions determined for pillars at shallow depth using the above formula can be unconservative 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 48. 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 48, this extraction should be reduced to 82.5%.

Pillar stress

The stresses which will act on a pillar will depend on the in situ stress conditions and on the extent of both local and regional mining extraction. In regular mining layouts such as room and pillar mining, pillar stresses are commonly determined using the tributary area theory. This theory simply assumes that the pre-mining stresses are evenly distributed on the pillars and that the average pillar stress is the pre-mining stress pro-rated up to take account of the extent of mining extraction. Therefore, if σ_v is 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*)

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

$$P_{stress} = (\sigma_v.cos2\theta + \sigma_h.sin2\theta)/(1 - e)$$

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 horzon

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 three dimensional numerical stress analyses. In some cases, simplification to two dimensions may be possible, but this will not always be possible.

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:

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

An *FOS* of unity is equivalent to a probability of failure of 50%. 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. Special pillars are dealt with in the following section.

- 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. For example, if failure of pillars within a stope leads to a pillar run, then the barrier pillar must arrest this run. Therefore, the *FOS* of the pillar must be sufficiently high to ensure that the pillar does not fail under these conditions. Barrier pillars also provide regional support for the mining layout. 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. The requirement for stability is therefore not as critical as that for barrier pillars, and occasional instability and failure of pillars is acceptable provided that it does not compromise safety. An indication of some instability is when spalling begins to occur from the pillar sidewalls. 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. They must therefore not be stable pillars, but must yield consistently as the load on them, or deformation across them, increases. To achieve this they must be in a state of failure immediately they have been

cut. Their probability of failure must be high, and the width to height ratio will generally fall into the range of 1.5 to 2.5. A recommended *FOS* is 0.7. Safe mining with crush pillars requires that the stiffness of the pillars is less than the stiffness of the surrounding rock mass. In the depth range of 200m to 400m, the indications are that this condition may not exist (Ozbay and Roberts, 1988).

- 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 (Ozbay et al, 1995). A recommended *FOS* is 1.0. As for crush pillars, the stiffness of the pillars should be less than that of the surrounding rock mass.

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. The stability of pillars is substantially enhanced if they are confined by the placement of backfill. The fill prevents the spalling of material from the pillar sides, and therefore prevents a decrease in the pillar's load carrying capability.

4.1.2 Special pillars

The pillars dealt with in the section above are part of the routine stoping operations. There are numerous other pillars which can be considered to be special cases and whose design needs to be considered individually. The pillar design technique described above may be applicable to some special pillars.

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. It is essential that failure of the pillars is not possible, and conservatively large pillars are therefore specified. 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 Figure 46 and Figure 47. The stability of the pillar in beam action must be checked. The techniques described in Section 3.1.2 can be used for this purpose. The potential for blocks and wedges to detach from the under side of the pillar must be checked. The simple computer program BlockEval described in Section 3.1.1 can be used for this purpose.

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 (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 non-yield pillars, 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 Figure 49 to Figure 53. 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 22. 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 (Ortlepp, 1989).***

Table 22 Damage criteria for shafts (modified from McKinnon, 1989)

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)

The required shaft pillar radius for a given depth and rock elastic modulus can easily be determined from Figure 49. 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. Figure 50 to Figure 52 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 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. This cost would need to be offset against the additional revenue gained from reducing the pillar size.

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. These options are not widely used.

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 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 53. When the shaft is sited below the orebody, larger pillars will 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 guide, 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.

4.1.4 Probability and risk considerations

Probability considerations in the evaluation of stability of underground openings and slopes have been described in Sections 3.1.6 and 3.2.5.2 above. Similar approaches can be used in the design of pillars to take into account the variability and uncertainty in strength of the rock and rock mass, and in the loads imposed on the pillars. The application of these approaches will result in the determination of distributions for both pillar strength and pillar stress. The former is the capacity of the system and the latter the demand. As shown in Section 3.2.5.2, the probability of failure can then be determined as:

$$P(f) = P(P_{strength} - P_{stress}) < Zero$$

4.2 Backfill

Consideration of backfill in this handbook involves only the technical aspects of backfill – its function, types of backfill and their properties, support and trafficability. Backfill plant issues, material transport issues, environmental issues and backfill placement issues are either not dealt with, or only dealt with very superficially.

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. Quantification of its contribution to pillar strength has been dealt with in Section 4.1 above;
- 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.

4.2.1 Types of backfill

Five alternative backfill systems that can be considered for mining operations are:

- crushed waste fill;
- classified cycloned tailings (*CCT*) fill (with or without binder addition);
- cemented full plant tailings;
- dewatered or paste fill;
- slurry fill.

Additional information is given on some of the systems below, and the advantages and disadvantages of the systems are then summarized briefly in Table 23. It should be noted that imported fill can also be used when such material is economically available, an example of such a case being Black Mountain Mine. This is rather uncommon, however.

Crushed waste backfill

This is a low technology type of fill, dumped under gravity, and spread with suitable equipment if necessary. It will usually be a "loose" fill unless some cementitious binder is added. In its loose form it must be contained by pillars or barriers. Failure of such containing pillars or barriers could be hazardous and lead to dilution of ore at deeper levels.

Classified cycloned tailings

Classified cycloned tailings (*CCT*), sometimes called sandfill, is prepared from concentrator tailings by hydrocyclone treatment to remove the slimes or clay-sized fraction. In most cases, the highest proportion of the fill product lies in the 40-150μm range of particle size and the proportion of -10μm material is less than 4%. The -10μm fraction is most critical in terms of percolation and must be removed by hydrocycloning.

Slurry backfill

Slurry backfill, with gravity as the prime mover, offers almost all the desired features of paste fill. The secret of this system is chemistry, whereby cementitious additives are added to the fill material in order to achieve the desired performance underground. Backfill additives have been developed that:

- reduce water run-off to a minimum (to less than 5%);
- achieve high early strength with minimal shrinkage;
- contain a relatively low total additive addition (typically less than 6% by dry mass);
- have a relatively high angle of repose (slightly less than achieved by paste filling);
- can be transported over kilometres using gravity alone.

Binder in the form of a cement/pozzalan blend and a silicate activator is added to the full plant tailings (can be used with deslimed tailings as well). The activator gels the water in a stable form and hydration takes place. Slurry fill systems can thus be designed to match a specific paste system and tend to be far simpler to operate. In South Africa, where the required binders are readily available, this system is less costly than paste fill systems.

Paste fill

Paste backfill is a high density backfill using full plant tailings. Full plant tailings are preferred since well-graded tailings are generally easier to pump in paste form than uniform-graded tailings. Paste fill systems can be regarded as the "Rolls Royce" of backfill systems and their popularity is increasing. They are high-capital and operating cost systems which require high-tech paste pumps, disc or belt filters and other capital intensive equipment.

Paste fill systems normally use a 20cm slump product. Moisture content influences not only the slump of the product, but also the pressure gradient, and must be well controlled. For example, a 2% reduction in moisture content from 24% to 22% by mass can result in an increase in friction loss in the pipeline from 10kPa to 20kPa per metre. Control of the moisture content is thus the most critical aspect of the backfilling operation.

If a full plant tailings product is essential in order to prevent a shortfall of backfill material, paste fill is attractive.

Table 23 Advantages and disadvantages of fill systems

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

4.2.2 Backfill material properties

The desired backfill characteristics depend upon the type of support required. The fill properties described in the following will affect the backfill characteristics and must be considered when designing the backfill system. A clear understanding of their influence on the fill behaviour is also important:

Slurry relative density

$$Slurry\ relative\ density = \frac{Mass\ of\ slurry}{Volume\ of\ slurry}$$

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.

$$Porosity = \frac{volume\ voids}{total\ volume}$$

Porosity

As porosity increases, the backfill stress-strain response becomes less stiff. Typical porosities are:

deslimed tailings	- 52%
full plant tailings	- 48%
belt-filtered tailings	- 45%
crushed waste fill	- 42%

Moisture content

$$Water\ Content = \frac{mass\,water}{total\ mass}$$

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 is the opposite of moisture content and can be defined as:

$$Pulp\,Density = \frac{Mass\ of\ Solids}{Total\ Mass}\ x\,100$$

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

The optimum pulp density is the maximum density at which the fill can be transported without line blockages being caused. 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%. However, this 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 1.7 tons/m^3.

Particle size grading

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. There is also a direct relationship between the particle size distribution of a backfill material and its porosity and thus also its load bearing capabilities. 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.

Compression characteristics

The shear strength τ of backfill can be described by the Mohr-Coulomb shear strength criterion:

$$\tau = c + (\sigma - u) \tan \phi$$

where: C is the cohesion (MPa)

ϕ is the angle of internal friction

σ is the normal stress (MPa)

u is the pore water pressure (MPa)

Backfill can be described as a low cohesion, granular medium whose shear strength is a function of the pore-water pressure. Therefore, great care must be exercised in fill design, and in mining practice, to ensure that significant pore pressure does not develop in a body of backfill. The particular problem is the potential for catastrophic flow of fill should high pore pressure lead to complete loss of shear resistance and subsequent liquefaction of the medium. The practical requirement is to ensure that any in situ loading of backfill occurs under drained conditions.

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.

If the backfill is required to be self-supporting, it must clearly have sufficient shear strength to support its own weight. Although this might not be required in the cut and fill mining, free standing capability will be required in mining in which pillars are removed adjacent to previously placed fill.

The following are the most pertinent factors which affect the load-bearing characteristics of fills:

- increasing water content reduces the strength. This effect is more pronounced for total tailings fill than for deslimed tailings fill due to the lack of permeability;
- the greater the porosity at placement, the weaker the fill;

- the addition of flocculants weakens a fill;
- in shallow mines and in stopes 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.

Permeability

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. However, this will depend on the length of the drainage path as well as the effectiveness of the drainage system in removing the water. If total tailings is used, artificial dewatering will be required.

Backfill for cut and fill stopes, which is required both as a bulk fill and as a working surface, must have rapid setting properties and high percolation rates. High moisture contents result in high pore pressures, with instability occurring readily. The main cause of poor percolation is the presence of -10μm material, which must be removed by hydrocycloning.

4.2.3 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, for example, will require fill that is trafficable soon after placement. Waste rock fill, or a fill capping approximately 0.5m thick, with a higher 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
- slurry fill 80
- cemented full plant tailings 85
- uncemented CCT 70
- crushed wastefill 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 mat be subjected, whether they have access for mine workers, and so on. Consequently, support of various types of openings will be dealt with separately in the sub-sections below. The support of tunnels and underground chambers, shafts, orepasses, drawpoints and stopes will be considered specifically. Finally, methods of support of open excavations will be considered briefly.

4.3.1 Support of tunnels and underground chambers

Before considering the estimation of support requirements in these excavations, it is appropriate to review the types of support elements commonly used in the industry. Comments on these elements are as follows:

Mechanical, end-anchored rockbolt: quick to install. Prone to anchor slip, particularly due to blast vibrations and in weaker rocks. Allows shear movements on block surfaces to take place. This, with anchor slip and hence loss of tension, allows rock mass to loosen.

Grout (resin or cement) end-anchored rockbolt: same as above except that anchor slip is less likely.

Fully-grouted rockbolt: stiff support. Well-grouted deformed (such as rebar) bar bolts will have limited extensibility in tension and shear if movement takes place on joints. Smooth bar has much improved yield performance, but tensile load capacity is not well defined. Usually untensioned, but bolts may be tensioned if required. Tensioning will reduce the yield capacity.

Friction rockbolts: usually quickly installed. Performance dependent on hole size. Owing to thinness of steel section, can be prone to corrosion. Connecting arrangements are weak.

Yielding rockbolt: for example Conebolt or Durabar. Fully grouted, provides good performance in both tension and shear, and has defined tensile load capacity. Maintains load capacity for significant yield (say 300mm).

Tensioned cables: used when significant support pressures are required and when special support measures are appropriate.

Diamond mesh (chain link mesh): flexible, surface support. Prone to unravel when one strand is broken. Not easy to install due to its floppiness. Because of this, easy to shape to a variable rock surface, but conversely, difficult to keep against the rock surface. Floppiness and the fact that it often is not up against the rock inhibit shotcreting quality (vibration of mesh, airpockets behind mesh), particularly for aperture less than 100mm. Good reinforcement for shotcrete (tough and yielding).

Weldmesh: reasonably easily handled, but due to its stiffness, difficult to mould to the rock surface. Conversely, does not flop away from rock surface. Good containment capability, but strands and welds are prone to "brittle" failure. Allows higher quality shotcreting, but owing to welded geometry, does not allow yielding, therefore not as good a shotcrete reinforcement.

Tendon straps: flexible, easily installed surface support, with good yield capability. Additional straps can be added as required if suitable rockbolts have been installed.

Wire rope lacing: over wire mesh, lacing is a proven surface support capable of containing major yielding of rock mass.

Shotcrete: good surface support when used with mesh or fibre reinforcement. Shotcrete will crack after small deformation, and further performance is dependent on the reinforcement. Diamond mesh performs better than weldmesh as reinforcement. Steel and monofilament polypropylene fibres can both perform well – the longer the fibres, the greater the yield capability. The performance of steel fibres in corrosive conditions is suspect. Shotcrete provides protection to other support elements against mechanical damage.

Steel arches: passive support applicable in particular circumstances such as rehabilitation or with spiling. Allows rock mass to loosen, particularly if arches are not well blocked. Quality of arch installation is often poor, with arch legs not being satisfactorily founded or supported.

Evaluation of support requirements for tunnels and underground chambers

Four approaches to the evaluation of support requirements will be given below. 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. It is important that the following points be noted in this regard, however:

- rock mass classification techniques are applicable to rock mass behaviour. In their standard form, as described in Chapter 2, 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. This is satisfactory for civil engineering applications, but not for mining applications;
- since they are applicable to general rock mass behaviour, they do not take into account particular mechanisms of rock instability. It is therefore important that possible instability mechanisms are well understood to ensure that the support indicated is appropriate.

Notwithstanding these reservations, rock mass classification provides 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 Figure 54 and Figure 55, which show the required rockbolt density and the shotcrete thickness. 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). In these charts, the conservatism of the standard civil engineering approach has been removed. For poor rock conditions, and for particularly important excavations, more specific support design calculations should be carried out. This could involve the use of stress analysis methods, which will be described in a later section.

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). 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, determined in Section 3.1.2 above, 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 Figure 54 and Figure 55 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 54 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 described in Section 3.1.1. The simple computer program described in that section can be used to evaluate the stability when rockbolt support has been installed, and hence to determine the required rockbolt support to ensure stability. The following example illustrates the use of the method.

A fault with a dip of 85^o and dip direction of 110^o intersects a 4.5m span tunnel whose axis runs in the 140^o direction. Two joint sets are present, with dip angles of 60^o and 40^o, and dip directions of 5^o and 250^o respectively. The block formed by these planes of weakness is shown in Figure 56,and, as can also be seen on this figure, the analysis indicates that it can be stabilised with rockbolts on a 2m x 2m spacing, with a factor of safety of 1.15.

Stress analysis: Excellent computer programs for stress analysis are commercially available. Two that are very commonly used for geotechnical problems are FLAC (Itasca, 1999) and Phase^2 (Rocscience, 1998), and both packages can handle the modelling of rock support. Phase^2 is particularly easy to use and quick to run. An example of its application to the analysis of rockbolt and shotcrete support is given below for illustration purposes.

The example involves a D-shaped tunnel with a span and height of 4.5m, which is typical of the size used in many mechanised mines. The analysis of the support has been carried out for two classes of rock mass – a good quality rock mass with an *RMR/MRMR* of 60, and a poorer rock mass in the same rock type, with an *RMR/MRMR* of 35. In situ stresses of 12 MPa vertical and 18 MPa horizontal, typical of expected values at a depth of about 500m have been assumed. It was further assumed that the Hoek-Brown rock mass failure criterion was applicable, with m and s values of 5.2 and 1.6, and 0.015 and 0.0004 respectively for the good and poor rock masses. The tunnels were modelled both with and without rockbolt and shotcrete support, and the in situ stresses were applied to the excavated and supported tunnels. The results are shown in Figure 57. The total set up and run time for this example, using Phase^2 , was about 30 minutes.

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.

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. Laubscher's rock mass classification system is 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 58. In sublevel caving mines, drawpoint spacing must ensure that pillars of adequate integrity are present, but must also allow for efficient drilling and blasting. A typical sublevel cave layout is shown in Figure 59.

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 Laubscher's design rock mass strength (*DRMS*), is given in Figure 60 (after Laubscher, 1993). 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.

The average support systems applicable for each of the support categories in Figure 60 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 in Section 3.1.3.1 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

Figure 54 and Figure 55 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 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).

It is appropriate to check for structural instability in shafts as well, since unstable blocks and wedges may require specific support. The simple computer program for evaluation of the stability of blocks, described in Section 3.1.1, can also be used to determine the support requirements for wedges in shafts. Such an application is illustrated in Figure 62.

The types of support appropriate for shafts are the same as those used in tunnels – rockbolts, cables, wire mesh, wire rope lacing, straps, shotcrete, and concrete. Mention must be made of the use of expanded metal as a support during shaft sinking in poor rock conditions, where support needs to be carried to the shaft bottom. Expanded metal is resilient to blasting and can be quickly bent or beaten downwards after the blast.

It may be practical to install a concrete or shotcrete lining in the shaft during the sinking process. In most cases such a lining will be adequate. However, this adequacy should be checked by the methods outlined above.

4.3.4 Support of orepasses

Support is not usually installed in passes. However, if the evaluation of stability dealt with in Section 3.1.3.3 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.

Design considerations, which will benefit the stability of passes without the requirement of support, are noted in Chapter 3. 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 hangups.

4.3.5 Support of shallow tabular stopes

This section deals with installed support in narrow tabular stopes in shallow mines. These stopes are typically in a low stress environment, with competent hangingwalls and very 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 minepoles 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. Laboratory peak loads for mine poles of various diameters, obtained from tests carried out by Mondi, are provided in Table 24. The diameter of the mine pole affects the strength of the mine pole considerably. Figure 61 shows the variability in the peak load capacities of mine poles, and indicates a linear relationship between peak load capacity and diameter, based on the Mondi information.

Table 24 Laboratory peak loads for mine poles

Diameter (mm)	**Mean peak load (kN)**
80-100	107
100	161
130-140	384
150	431
150-160	481
160-180	550
180-200	660

Underground tests indicate that the peak load capacities in the underground situation may be as low as 40% of those determined in laboratory tests (Roberts et al, 1987). This is largely caused by timber creep and the slow loading rate underground.

Mine poles are often blasted out when installed close to the face. As a result, they are typically not installed closer than 6m from the face before the blast, which may often be unacceptable from the support requirement point of view. 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: these are not commonly used in shallow, narrow tabular orebodies since packs require large deformations to generate the required support loads. In addition, the cost of pack support systems is generally prohibitive.

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: these can be pre-stressed and provide effective active support at the mining face. Typically these props fail at approximately 150kN. Mechanical props are removed and re-used and are therefore cost effective. In this regard a system for remote release of mechanical props is required to prevent injuries during removal of temporary support.

Saplings: although small diameter (<100mm) timber saplings are commonly used, the more effective mechanical props are recommended. The slenderness of the saplings makes them very prone to buckling failure at low loads. Higher support densities are required to achieve the required support resistance with saplings.

Evaluation of support requirements for stopes

To evaluate the support requirements the demand and capacity of the support system need to be established. In this case the demand represents the load imposed on the support system, while the capacity represents the support resistance of the support system. It is recommended that the demand should not exceed 1.5 times the capacity. Capacity divide by demand represents the factor of safety.

Demand: systematic stope support systems are designed to support key blocks in the immediate hangingwall. The support system should be designed to carry the deadweight of a potential rockfall. The demand for a support system can be calculated as follows:

$$Demand = \rho t g$$

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

Back analyses of rockfall occurrences should be carried out to determine “t” for each geotechnical area. Where sufficient data are available and a cumulative frequency distribution of thickness of rockfalls can be determined, “t” should be greater than or equal to the thickness of 95% of the rockfalls. Invariably on small mines, there is not enough data to form a cumulative frequency distribution, and, in such cases, observations of brow thickness and height of wedge failures should be used to estimate “t”. Existing weak parting planes and low angle joints in the hangingwall should also be noted. 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 individual support units is the peak load that can be carried by the support unit. If underground trials have not been carried out, 40% of the laboratory average peak load should be used as the peak capacity for support design purposes. Stiff support types are required, since rock displacement must be minimised. Should excessive displacements occur, mine poles may topple before they reach their compressive capacity. This is particularly true in non-horizontal tabular stoping environments. Pre-stressing of mine poles can reduce this risk, but it should be noted that, in a low closure environment, mine poles could lose 50% of the pre-stressed load within 24hours due to timber creep (Bristow and Ortlepp, 1980).

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 or the capacity of the system. 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 to determine the total load capacity in the area. Similarly, an area representing the support pattern in the face area should be defined to calculate the face area support load resistance. Examples of these representative areas are indicated on Figures 63 and 64. Examples of the calculation of support resistance for mine pole and rockbolt support systems are given in the following sub-sections.

Mine pole support system: a typical support layout, with pre-stressed minepoles as permanent support and mechanical props as temporary support is indicated in Figure 63. The support resistance for this support system is calculated as follows.

Table 25 Mine pole support calculation

	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

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.

Rockbolt support system: a typical support layout with rockbolts as permanent support and mechanical anchors as temporary support is indicated in Figure 64. 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 anchors and rockbolts. The combined load can be calculated as follows:

Table 26 Mechanical anchor/rockbolt combination load calculation

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 64, 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:

Table 27 Mechanical anchor/rockbolt combination support calculation

	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, if rockbolts are not long enough, when 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 re-tensioned after the blast, and rockbolts in the working area should be checked and re-tensioned 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 ledging, 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. 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. As indicated in Section 3.2.6, slope management is preferred, 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, as described in Section 3.2. For cable anchor support of a slope subject to plane failure, the equation in Section 3.2.3.1 is applicable. For example, consider the 40m high vertical slope in Figure 65, which contains a weak cohesionless plane, with a friction angle of 30^o, dipping at 40^o. For simplicity, the slope is considered to be dry. The density of the rock mass is 2500 kg/m^3. For a factor of safety of 1.5, the required total cable anchor capacities for cables installed at various angles are given in Table 28.

Table 28 Support of slopes with cable anchors

Angle of inclination of cable (φ^o)	**Required cable anchor capacity (kN)**
15	12700
30	9600
45	8200
55	7800

For wedges with the potential to slide, the simple computer program described in Section 3.1.1 can be used to give a quick indication of stability and cable anchor support requirements for vertical slopes. For example, the "windows" in Figure 66 and Figure 67 show the input data, the geometry of the wedge, applied support forces normal to the slope face and the results in terms of the corresponding factors of safety. The line of intersection of the two planes, on which the block slides, dips at 39^o, which is the same dip as the plane used in the planar failure example above. The support requirements for the wedge, in terms of mean anchor force per unit face area, are about half those required for the plane failure. This confirms that stability in a three dimensional situation is usually better than for a two dimensional consideration.

Localised surface instability, leading to occasional falls of loose rock blocks, can be stabilised using shotcrete, or wire mesh draped over the slope face.

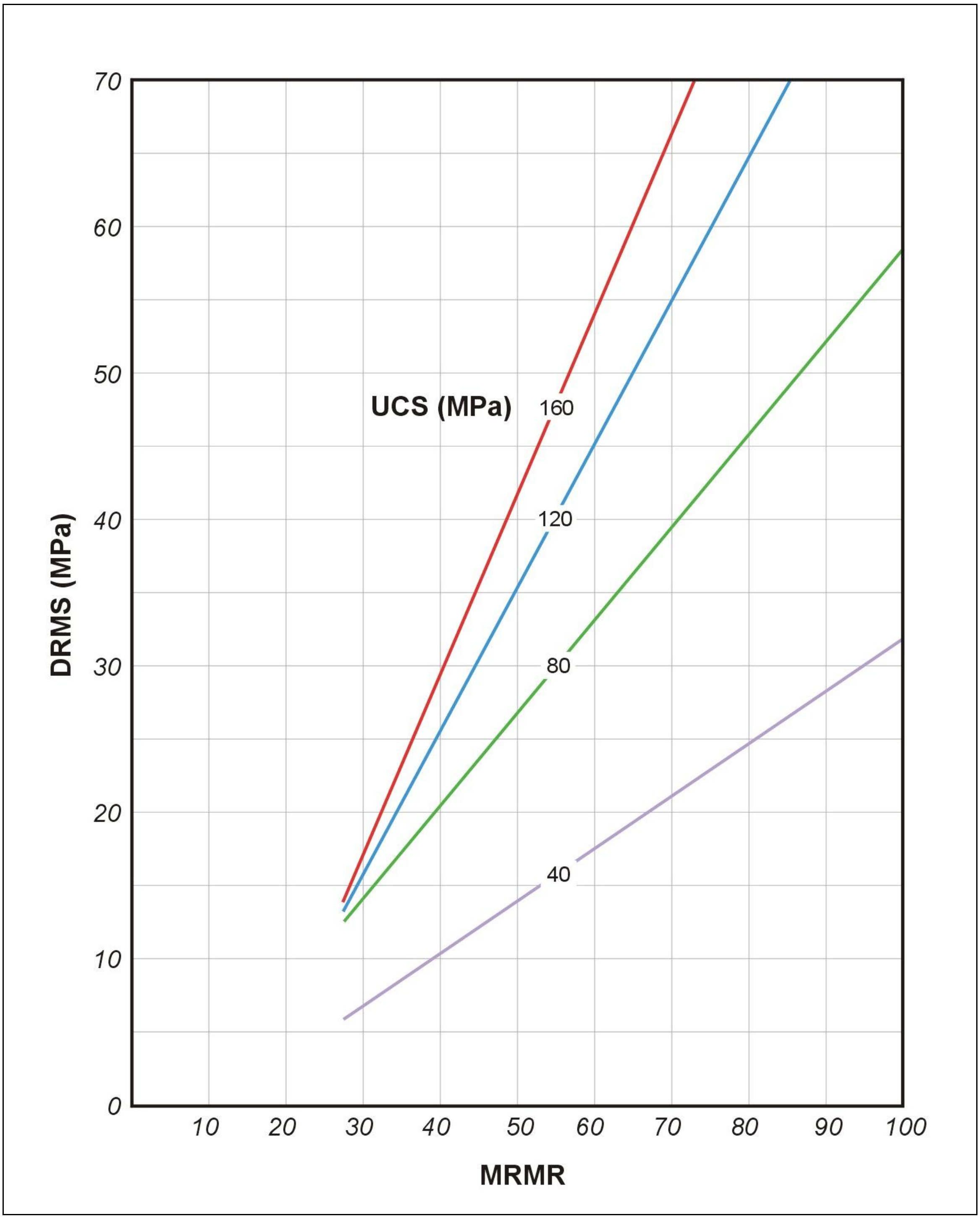


Figure 45 Guideline relationship between DRMS and MRMR

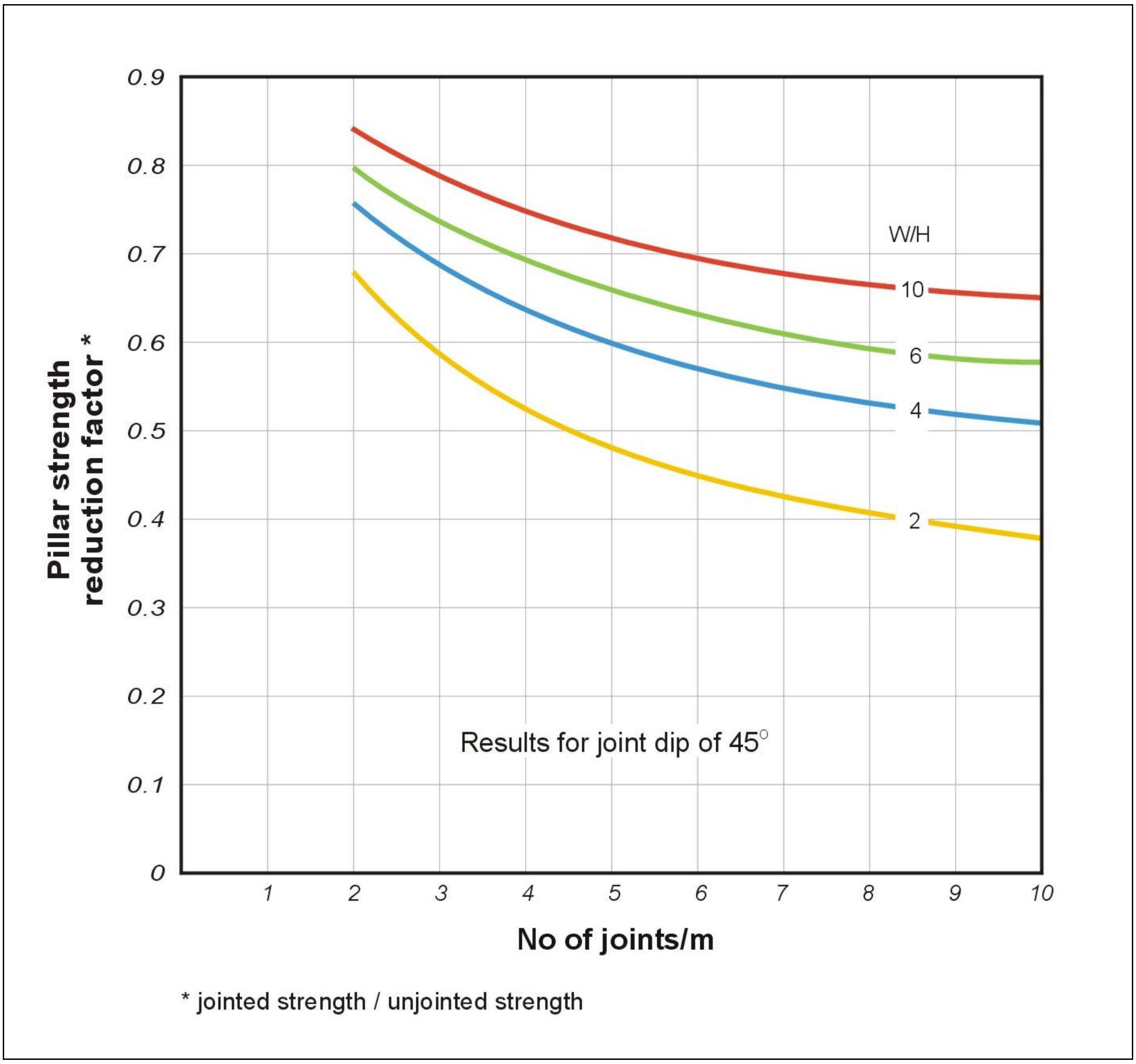


Figure 46 Pillar strength reduction factors for various pillar width to height ratios

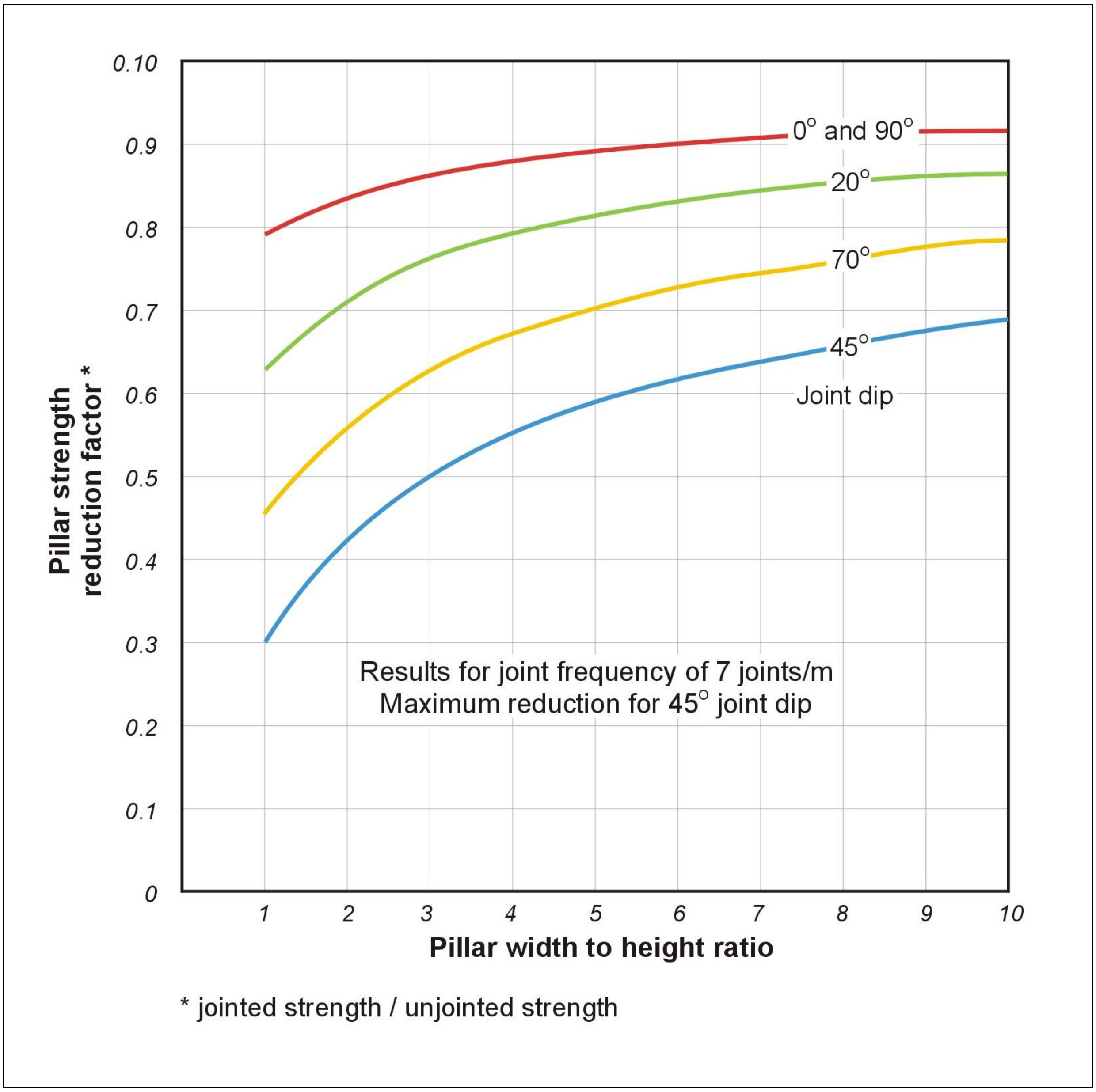


Figure 47 Pillar strength reduction factors for various joint dip angles

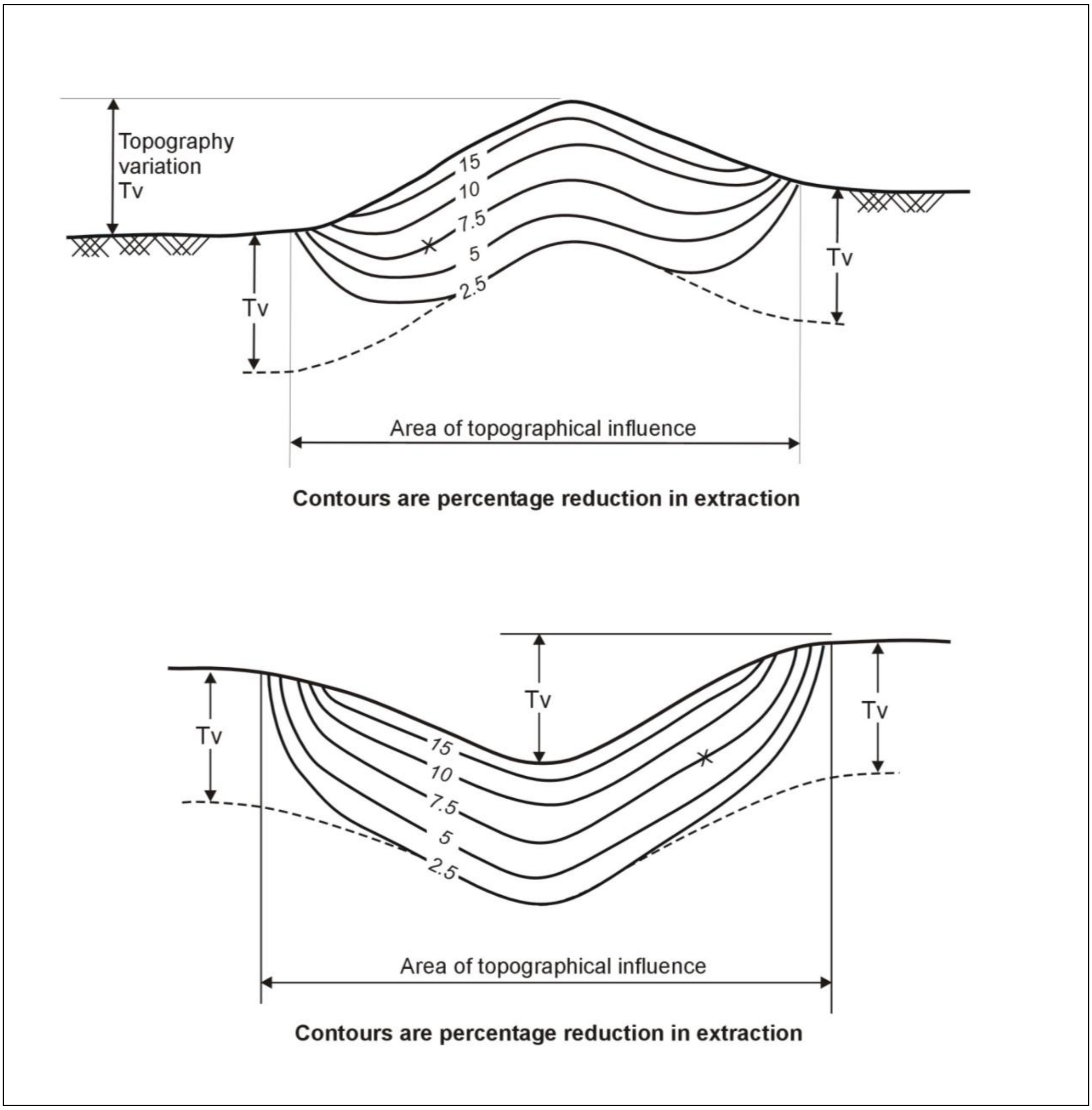


Figure 48 Guideline for reduction of extraction in areas of topographical variation

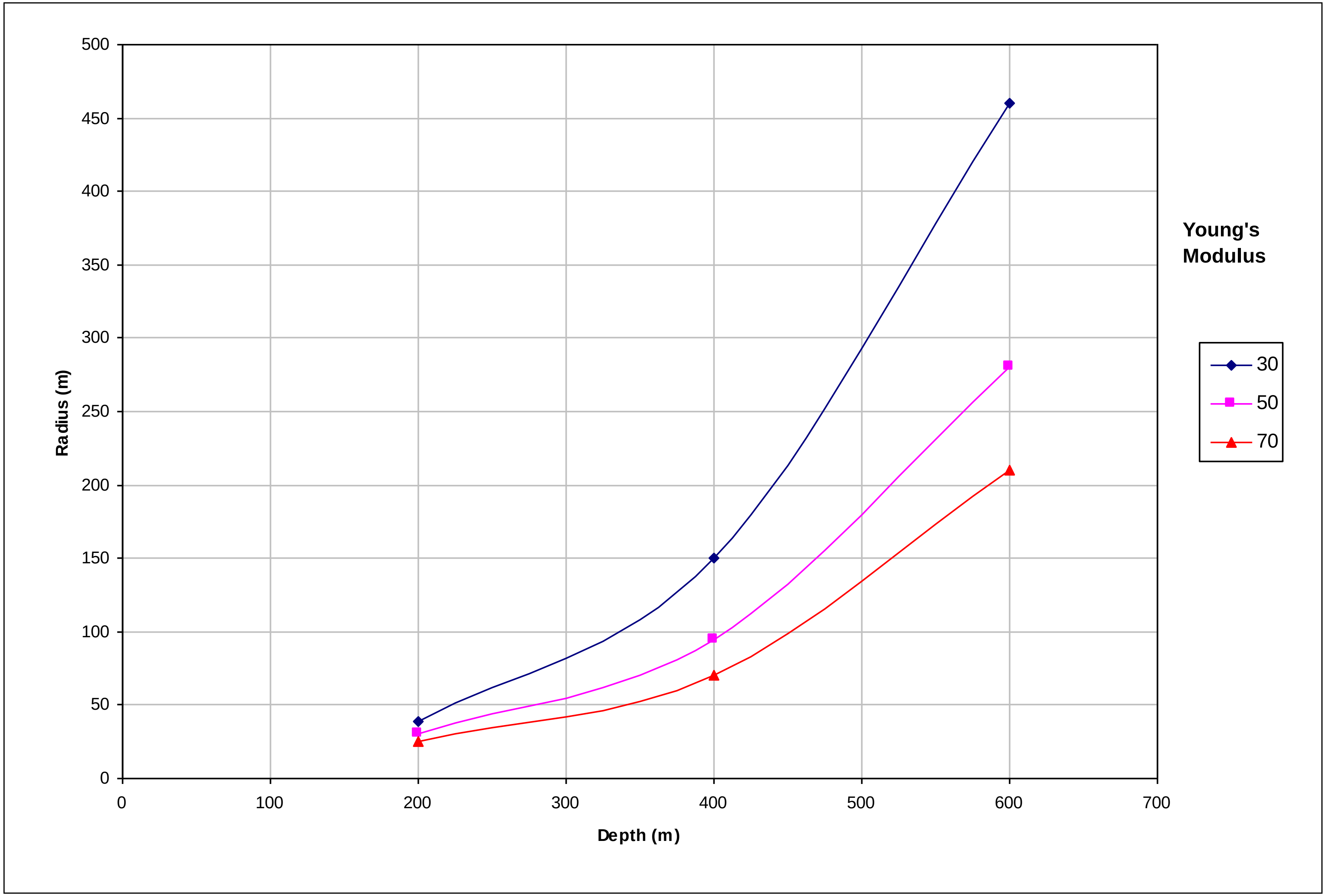


Figure 49 Vertical shaft pillar vs. depth

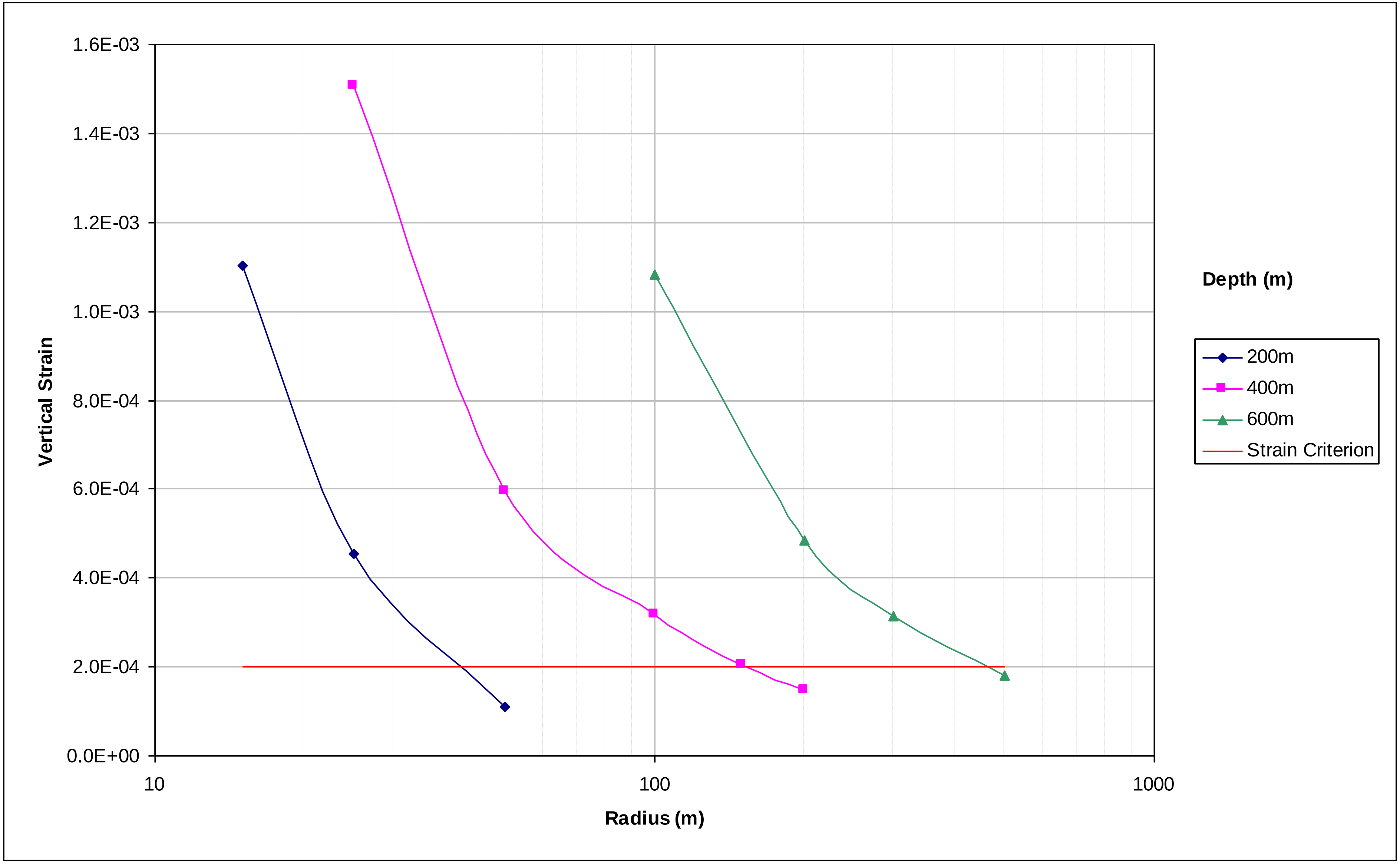


Figure 50 Strain in pillar vs. radius - E = 30 GPa

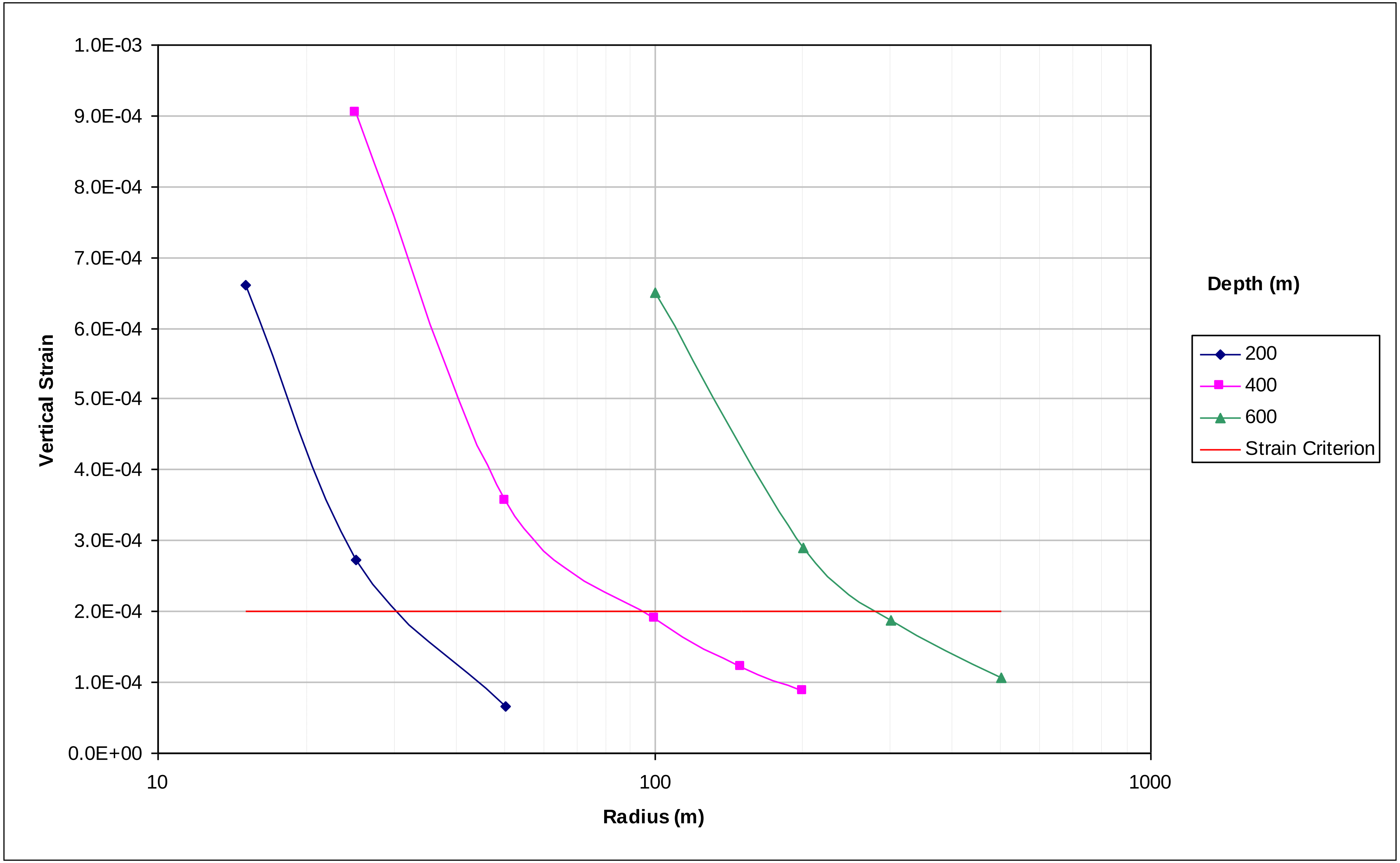


Figure 51 Strain in pillar vs. Radius - E = 50 GPa

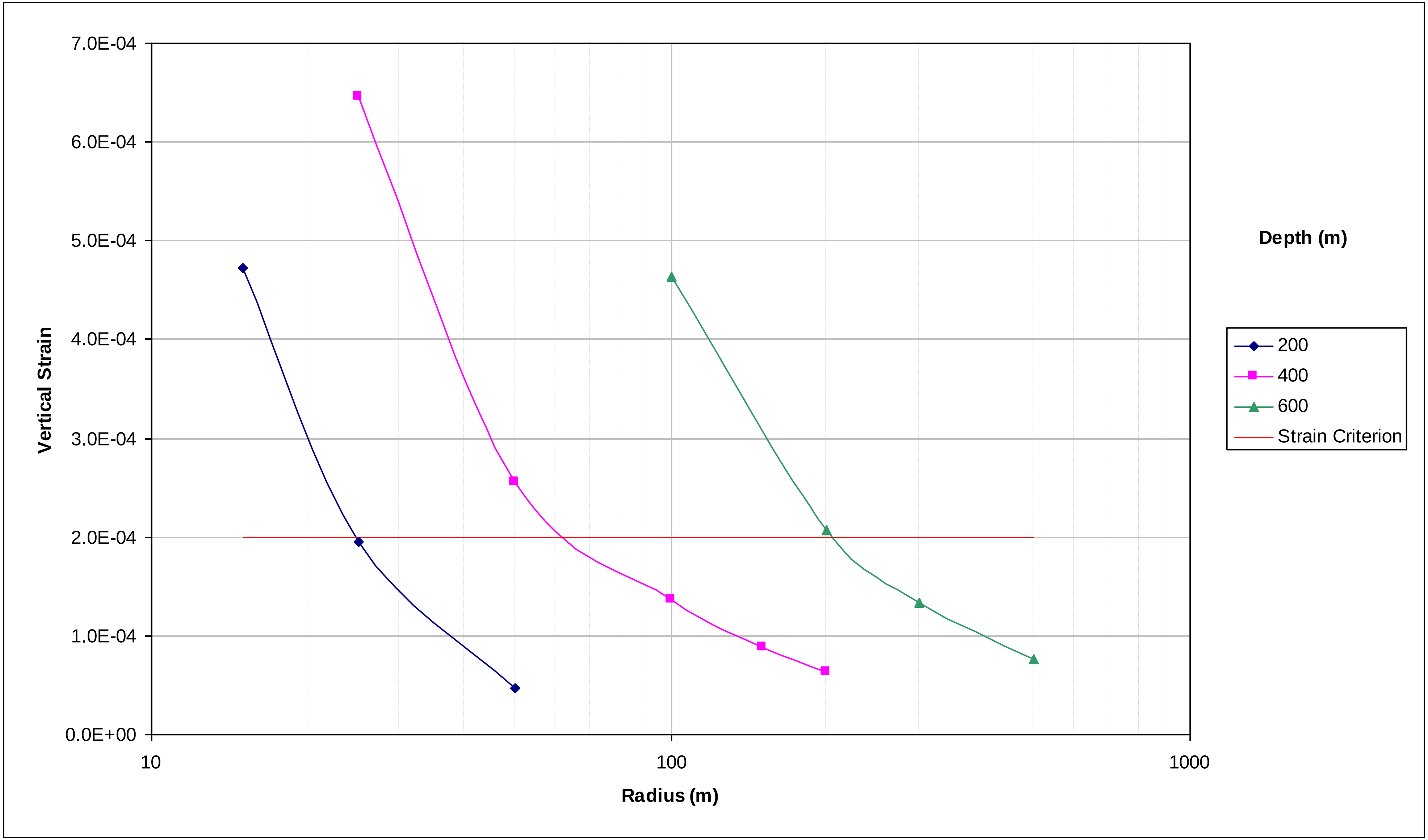


Figure 52 Strain in pillar vs. radius - E = 70 GPa

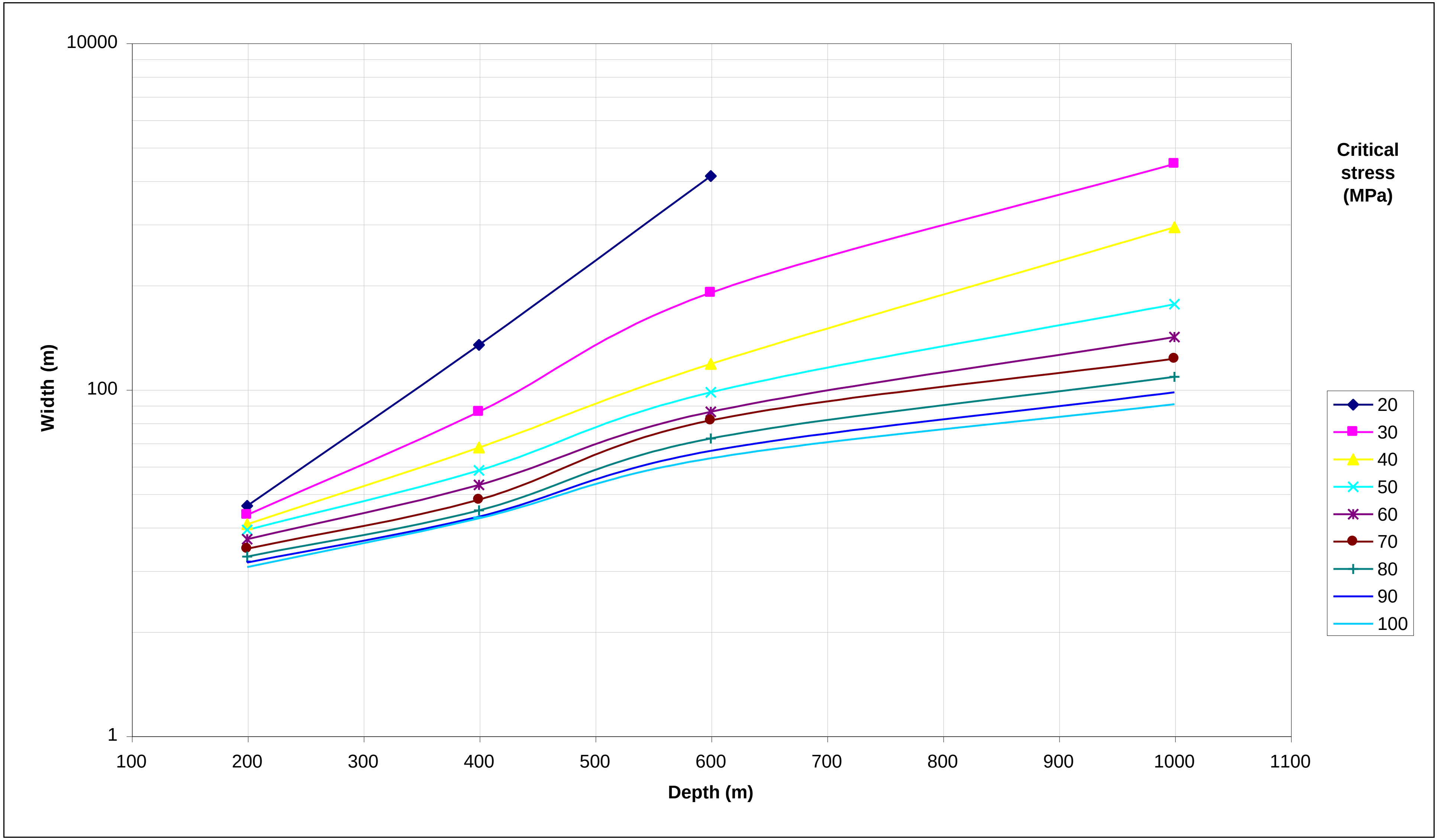


Figure 53 Inclined shaft pillar width vs. depth

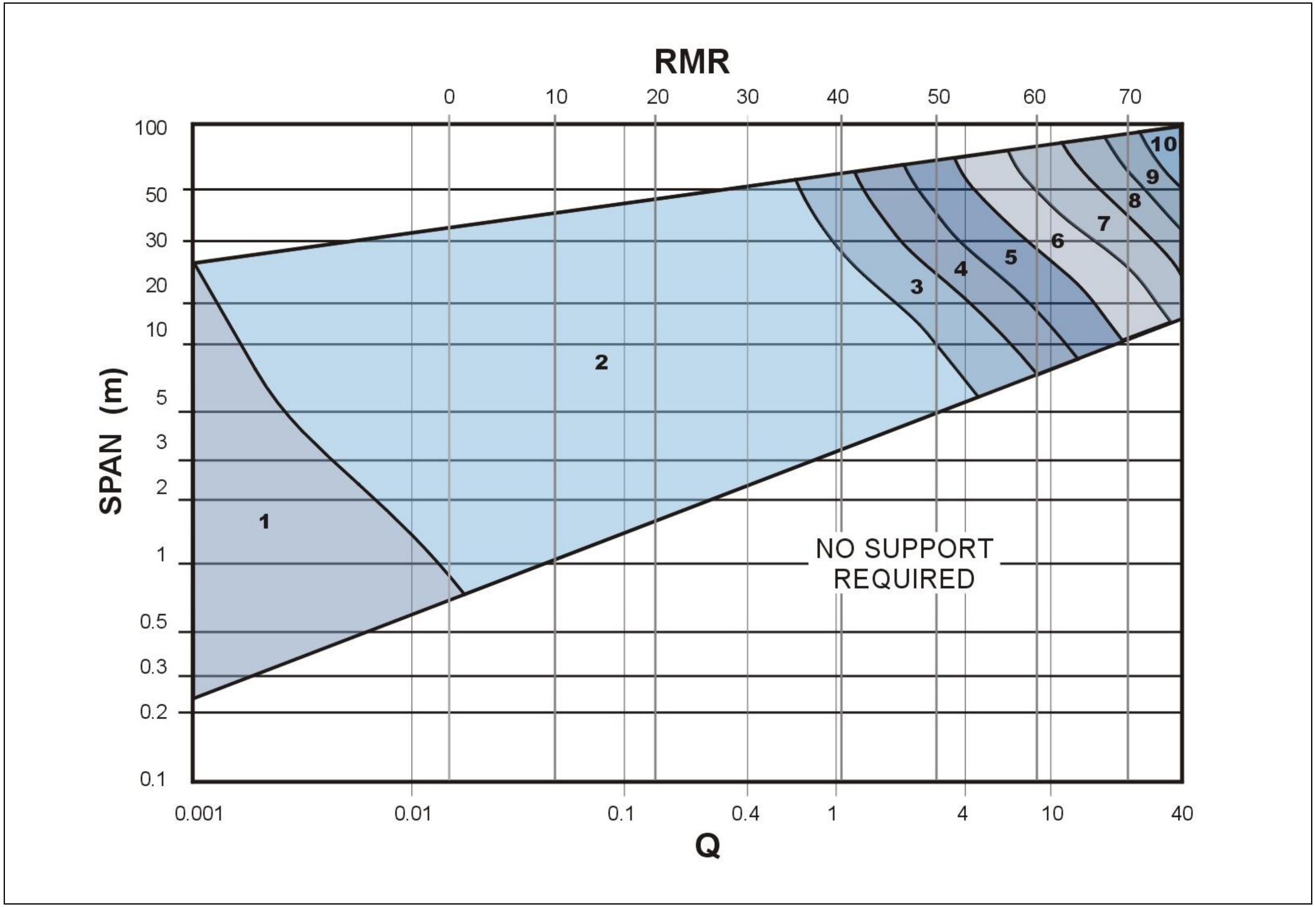


Figure 54 Rockbolt support design chart (Q system)

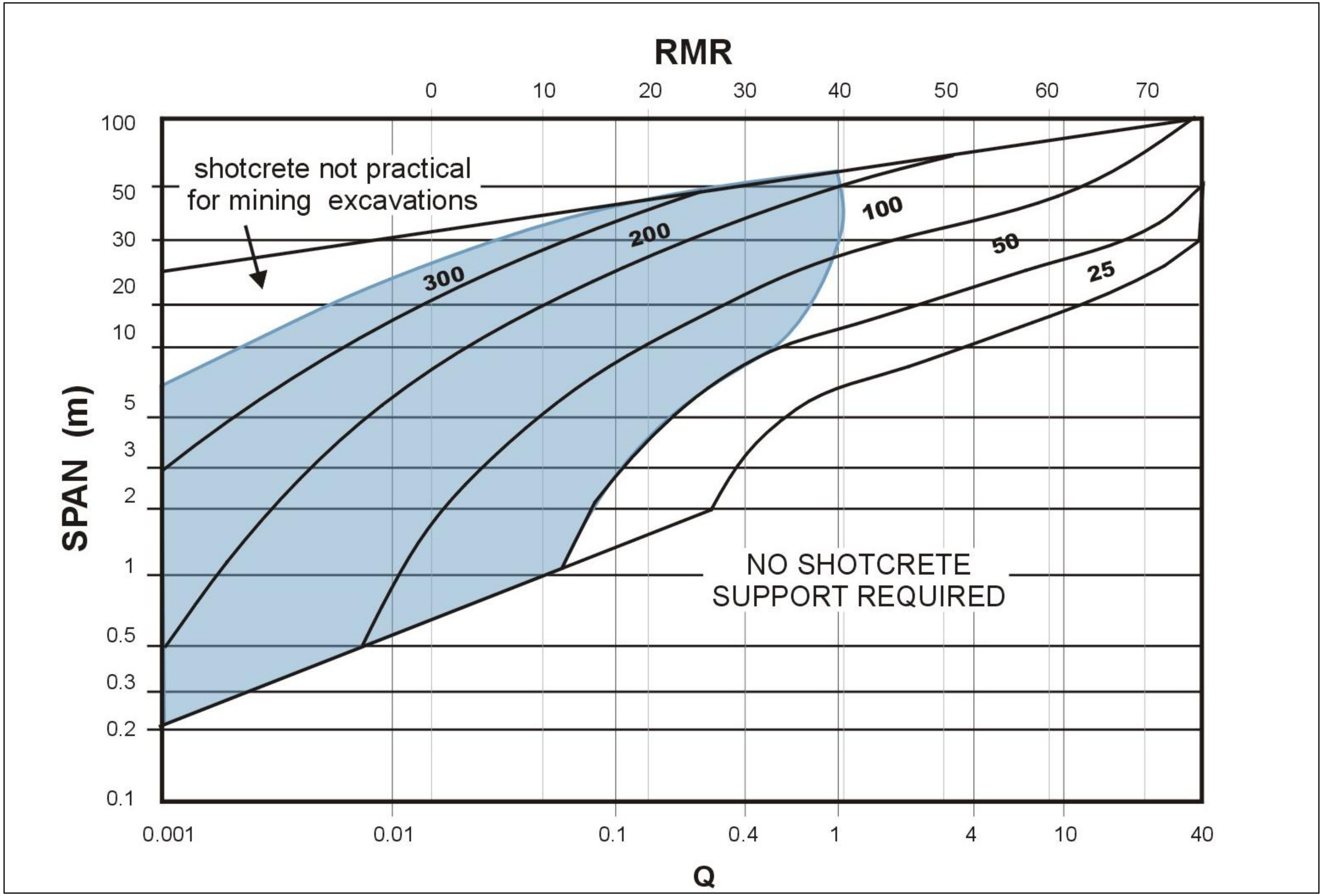


Figure 55 Shotcrete support design chart (Q system)

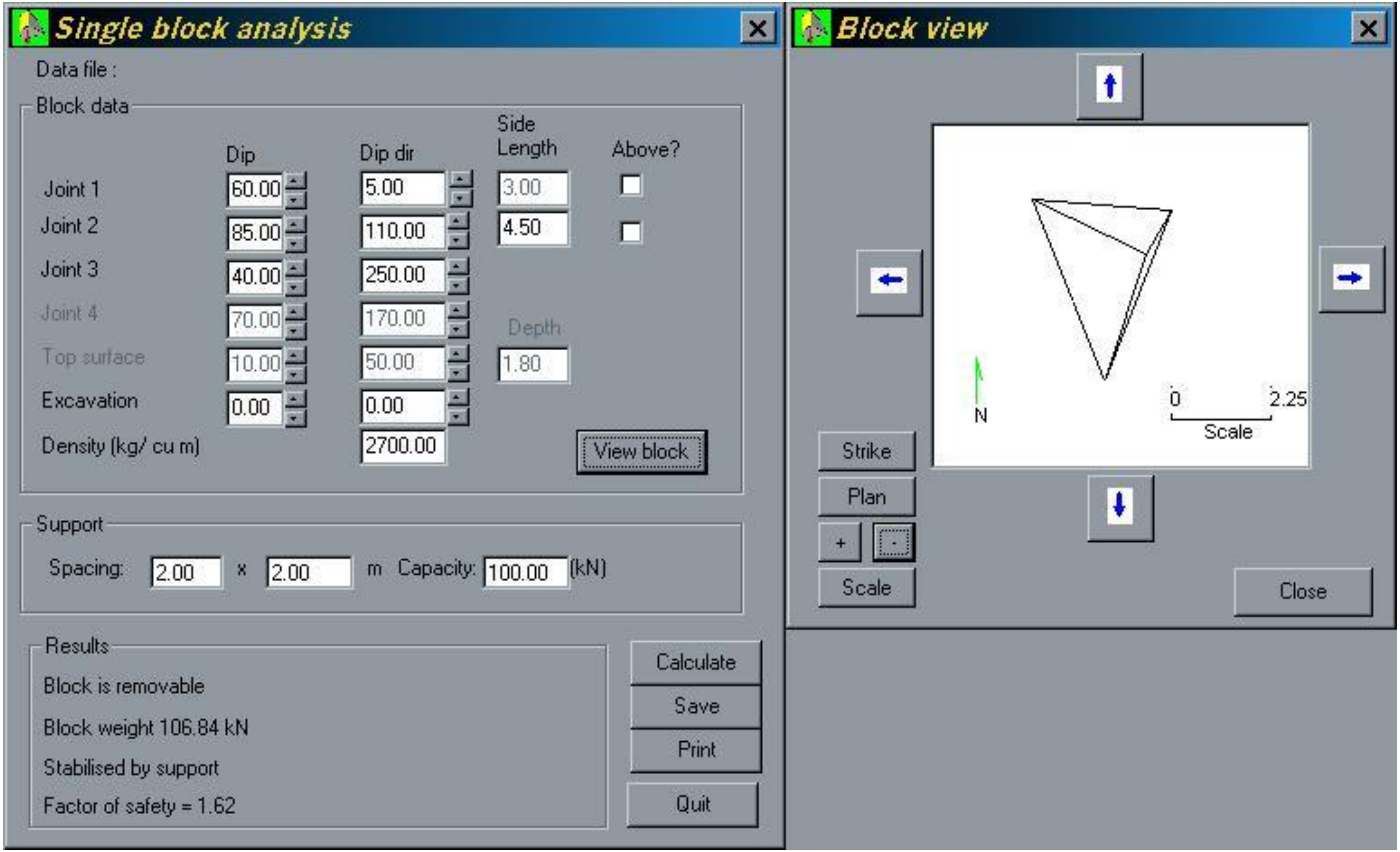
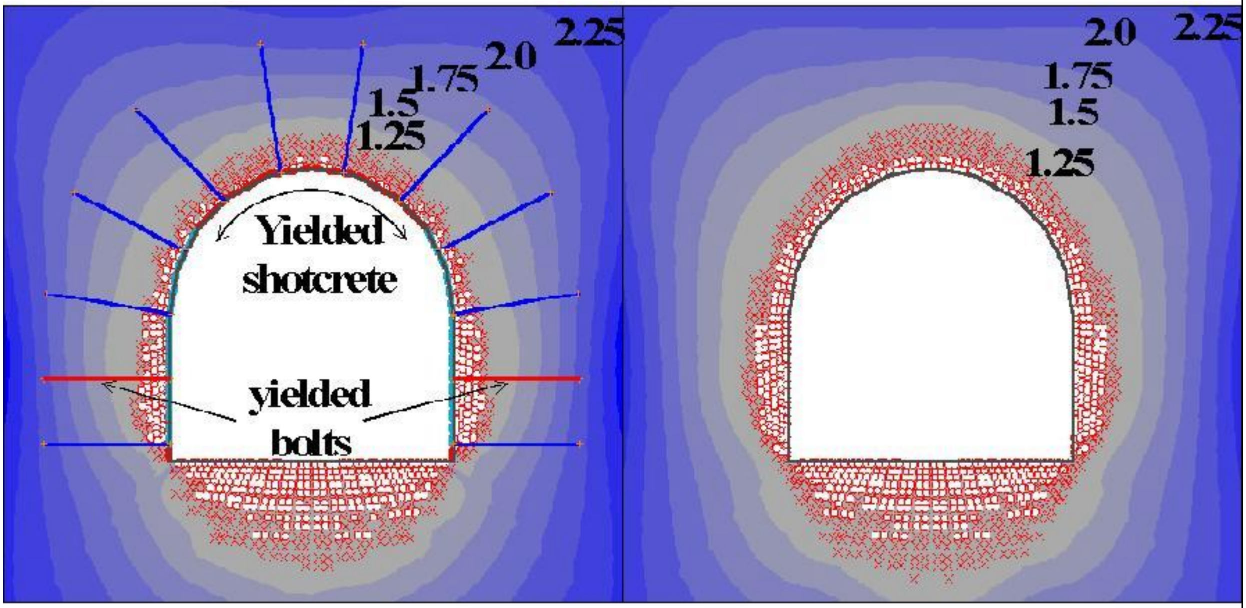
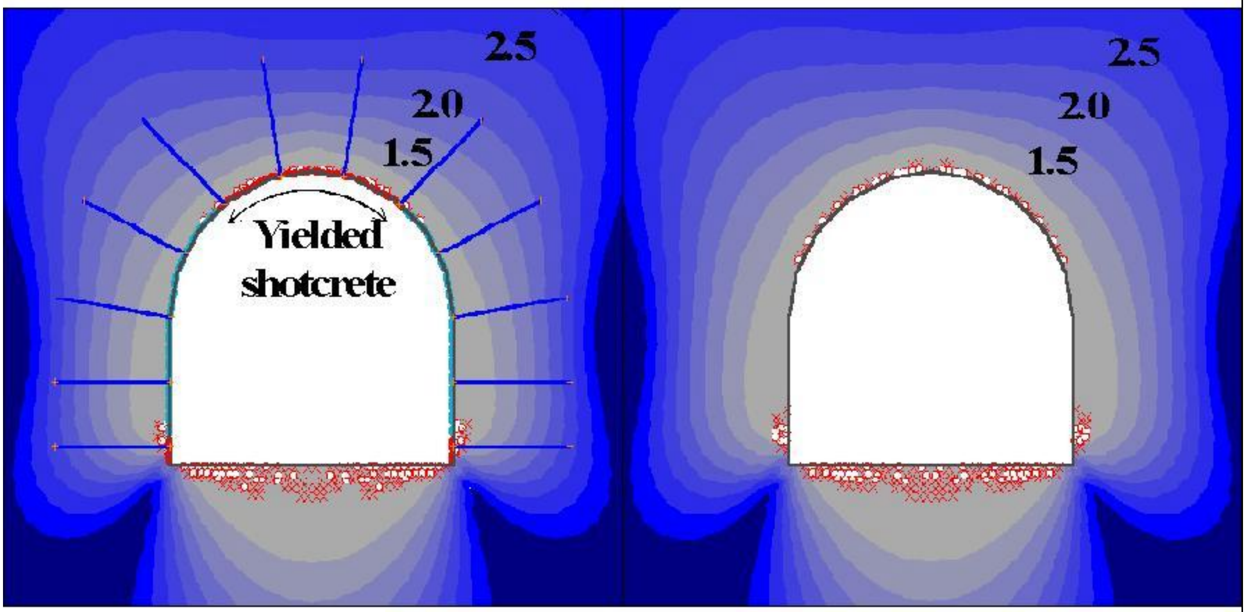


Figure 56 Kinematic stability of block



Weaker Rock Mass



Stronger Rock Mass

O – Yielding of rock mass in tension
X – Yielding of rock mass in shear

Figure 57 Result of support analysis using stress analysis approach

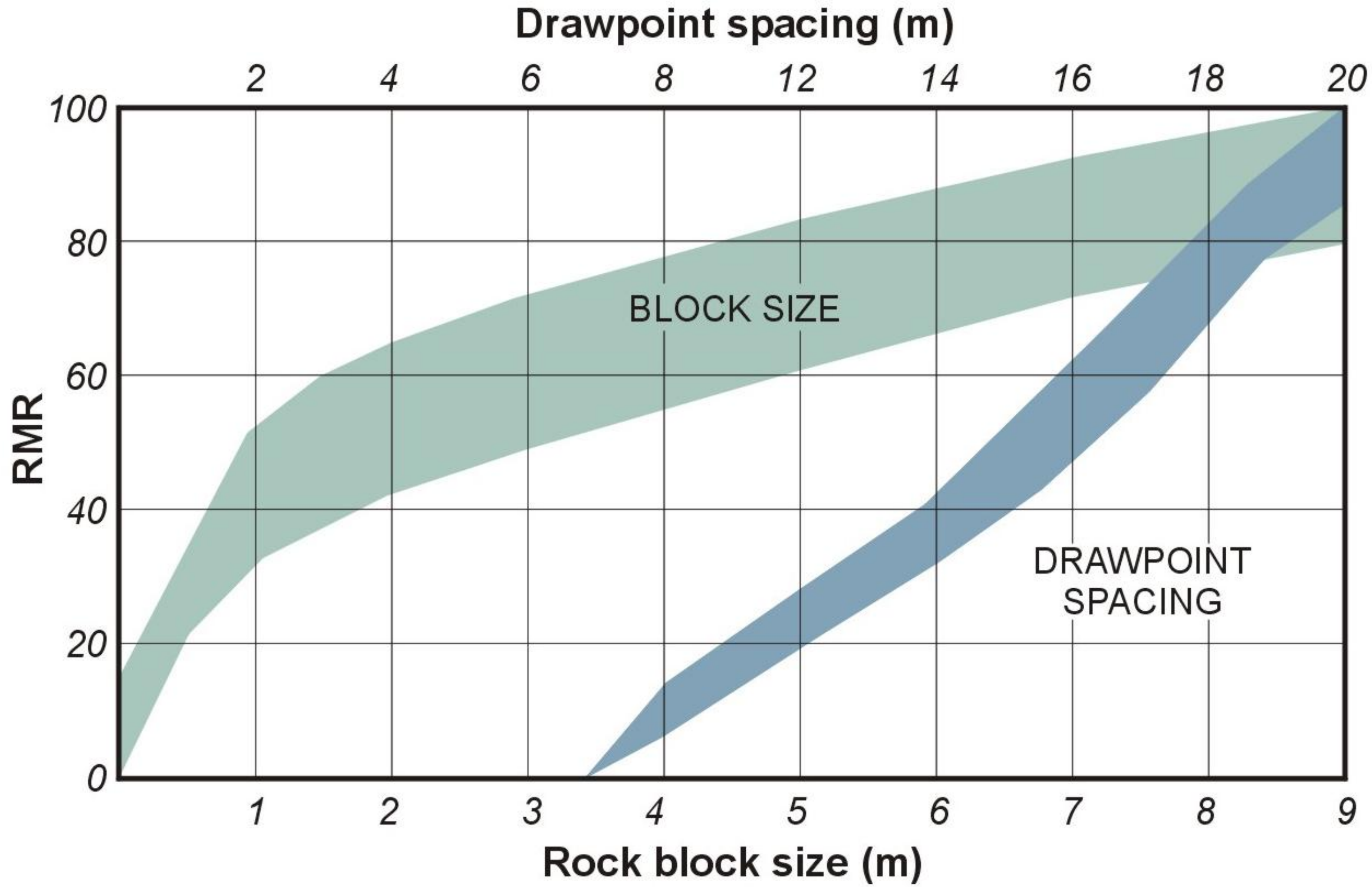


Figure 58 Guideline for fragmentation and drawpoint spacing

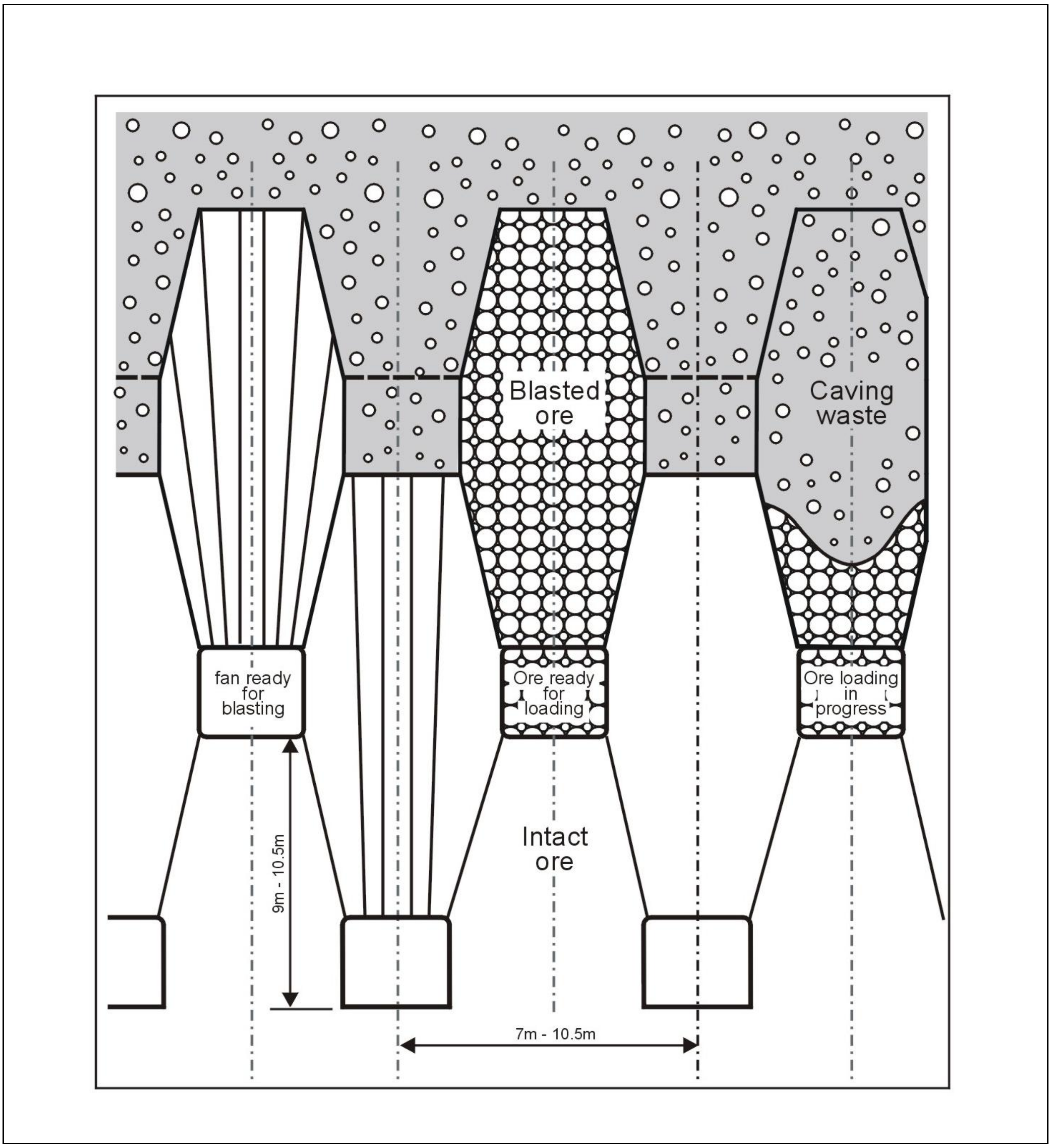


Figure 59 Typical sub-level caving layout

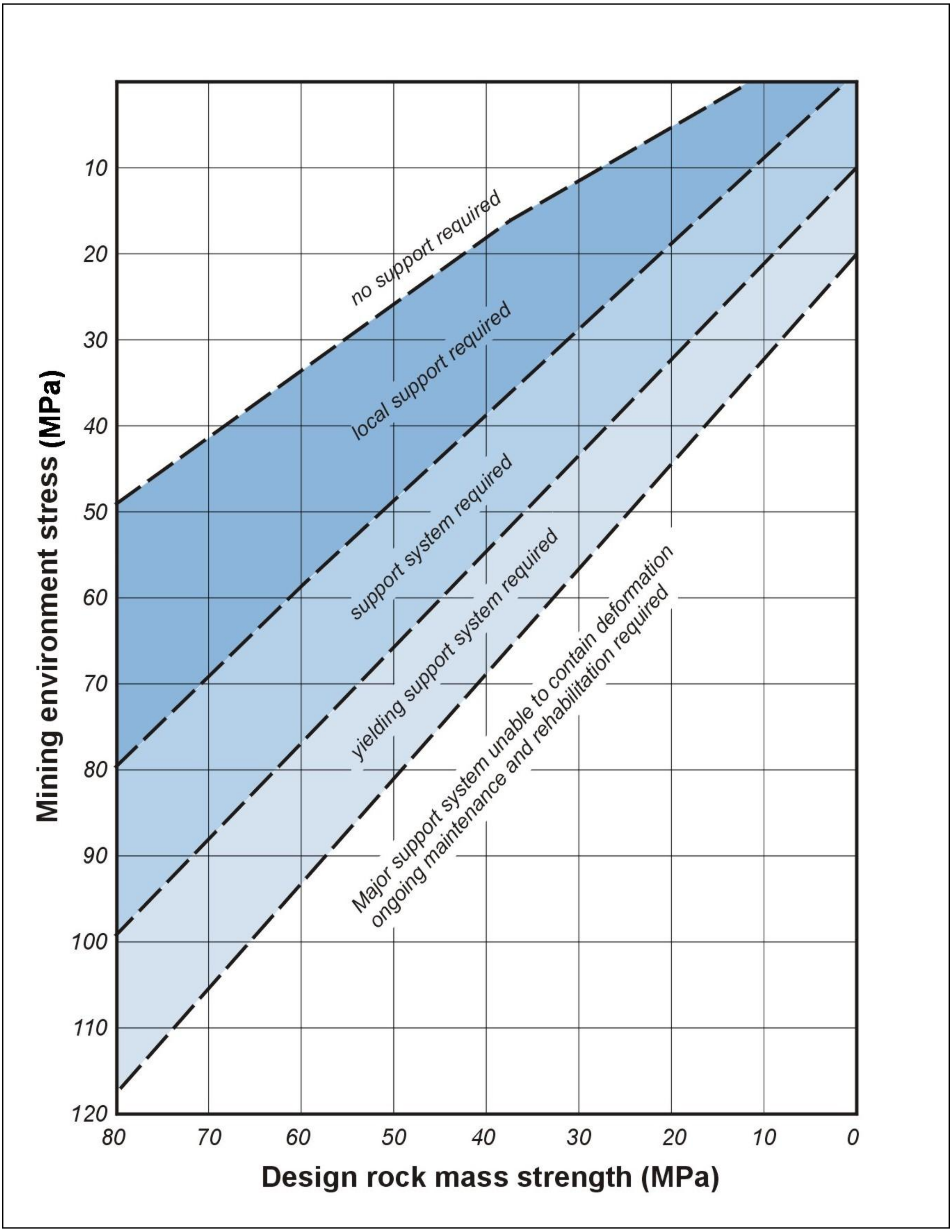


Figure 60 Guideline for drawpoint support (after Laubsher, 1993)

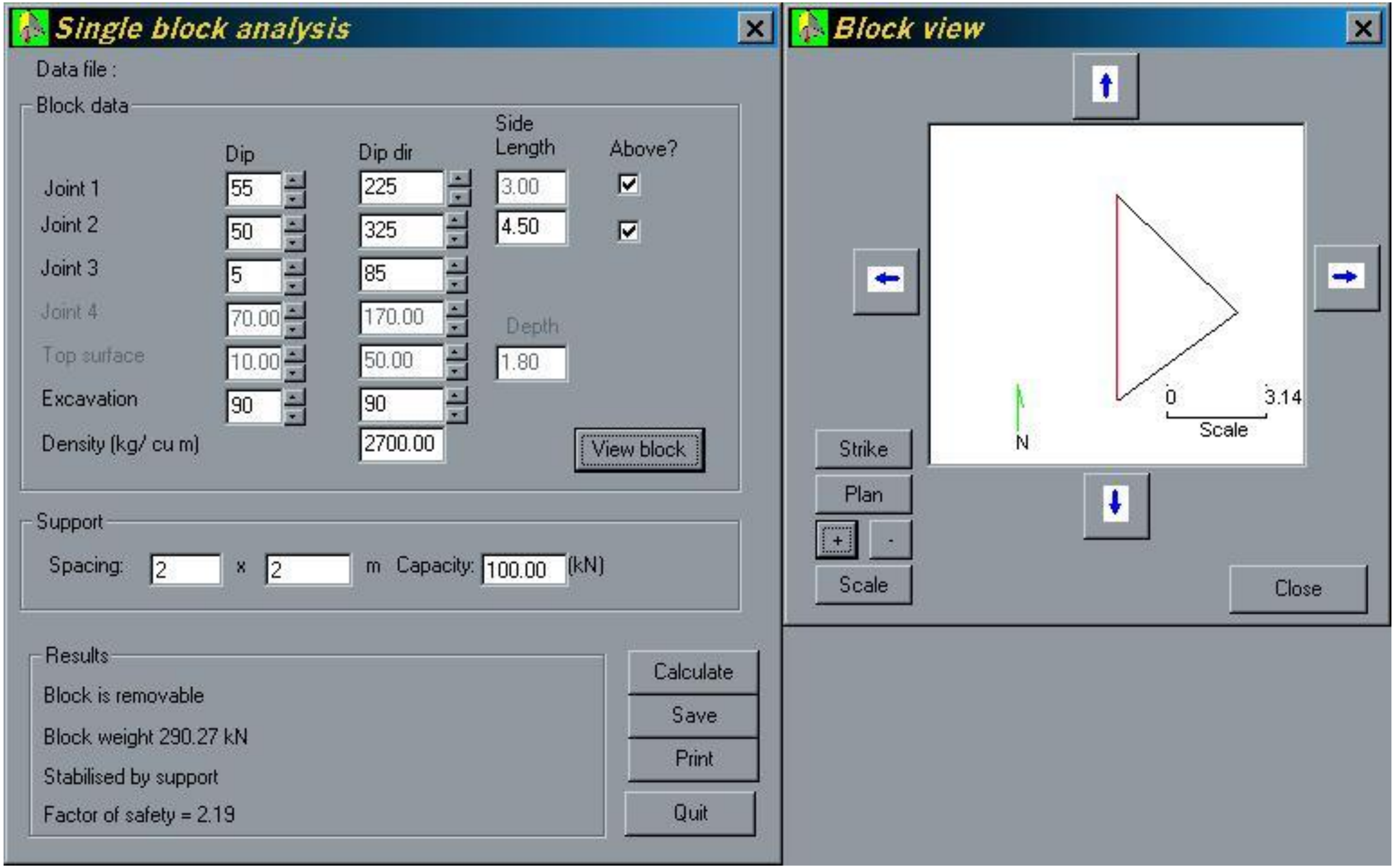


Figure 61 Evaluation of shaft stability using BlockEval

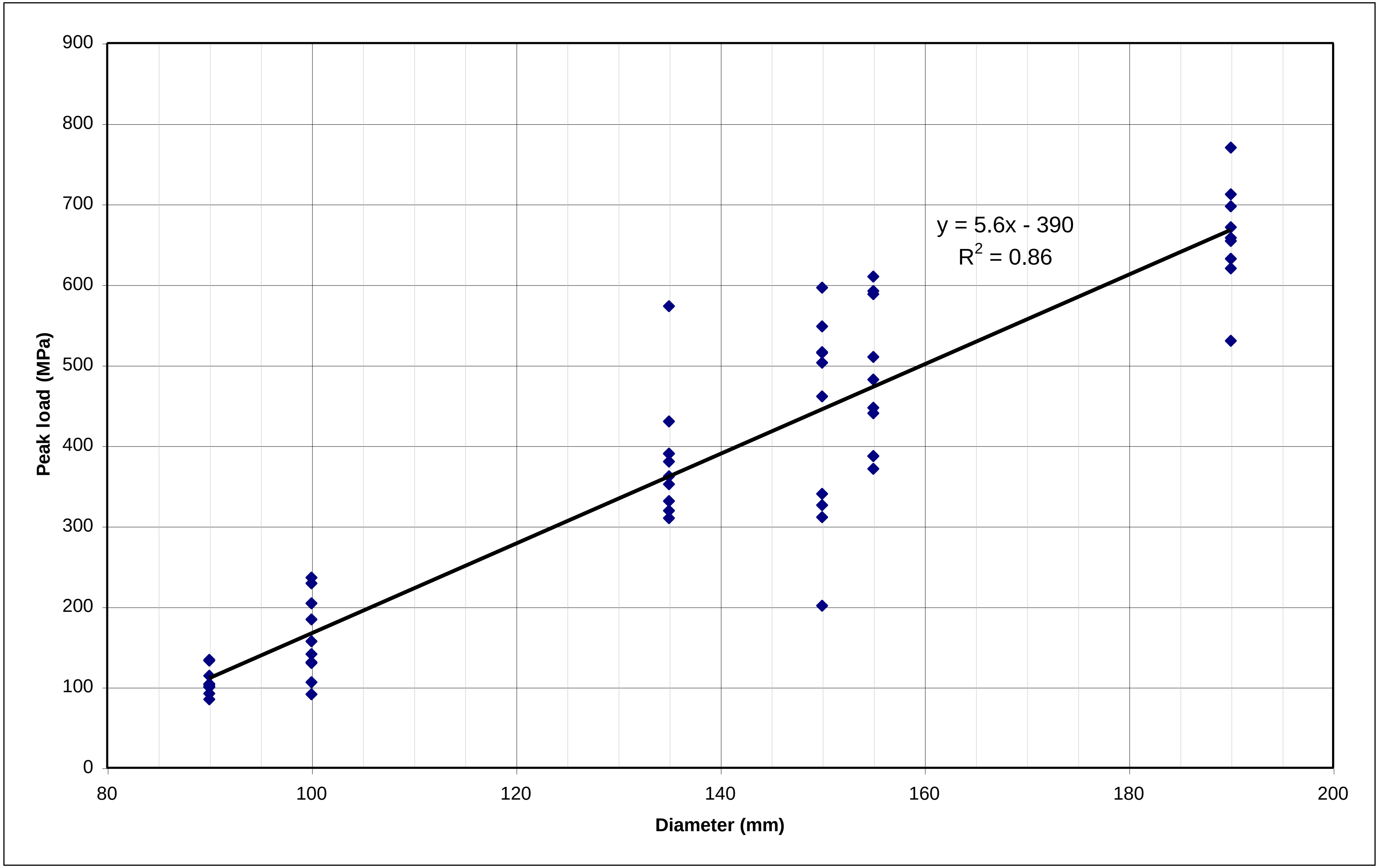


Figure 62 Peak load capacities of mine poles

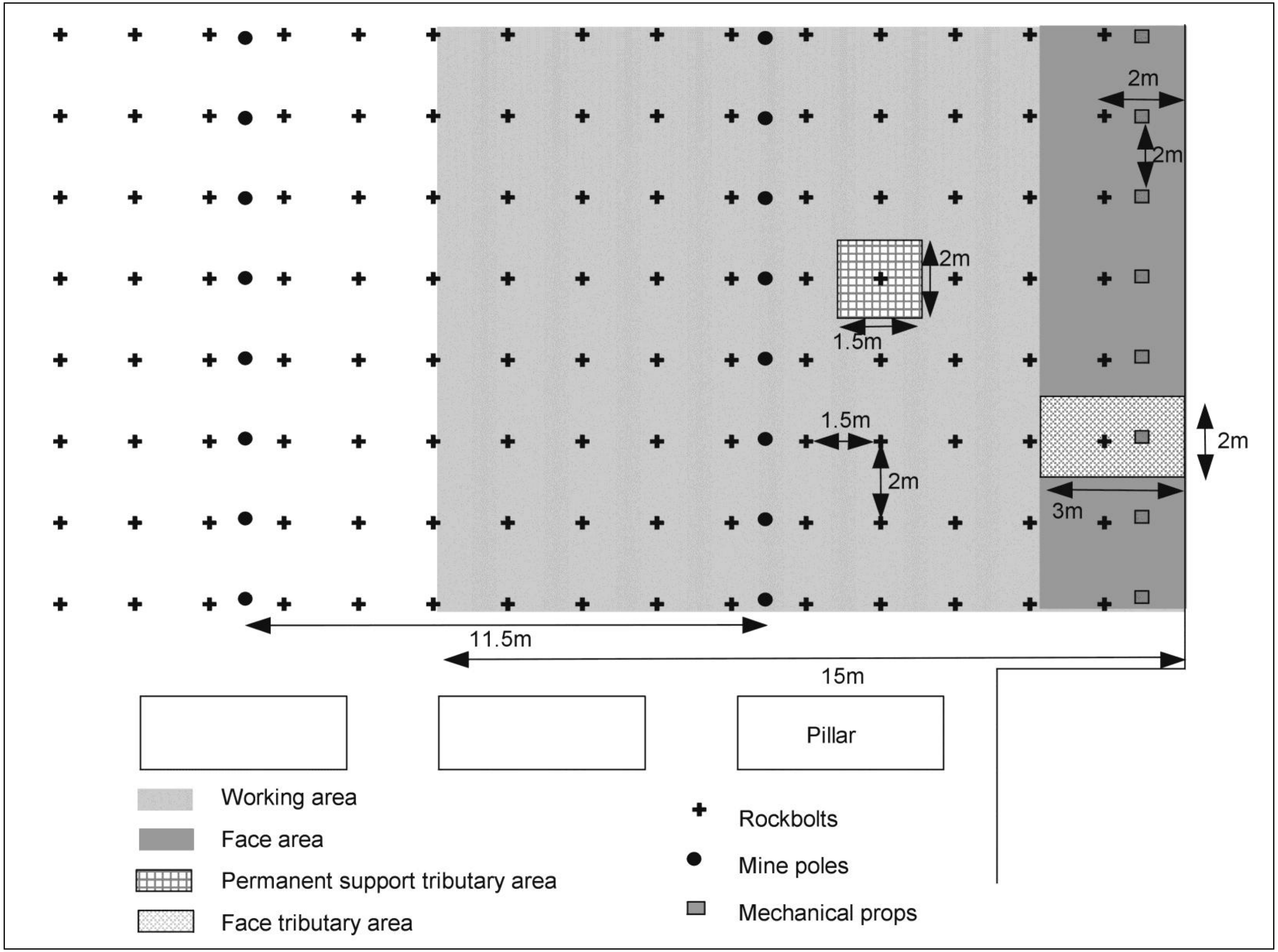


Figure 63 Typical support layout using mine poles

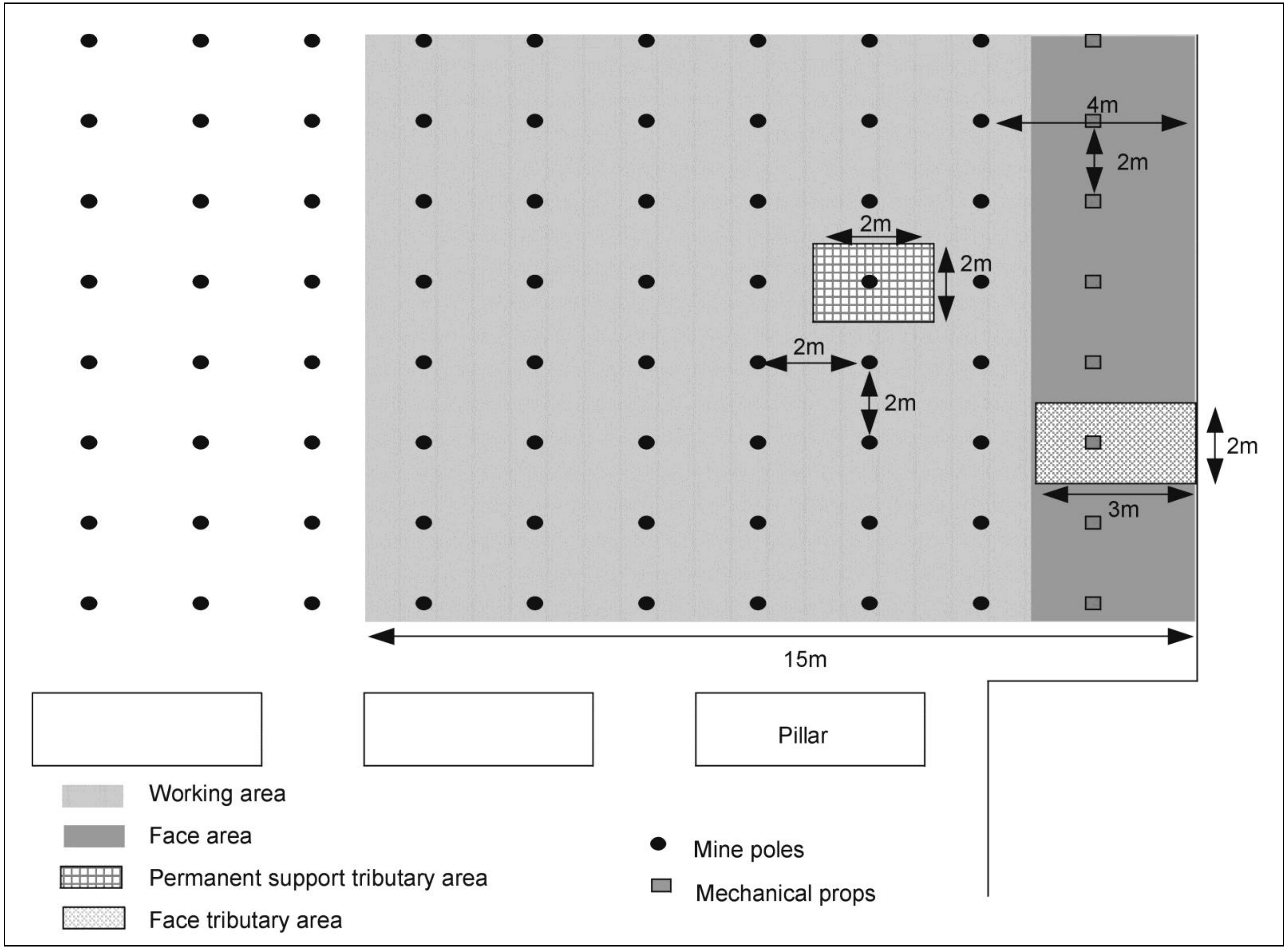


Figure 64 Typical support layout using rockbolts

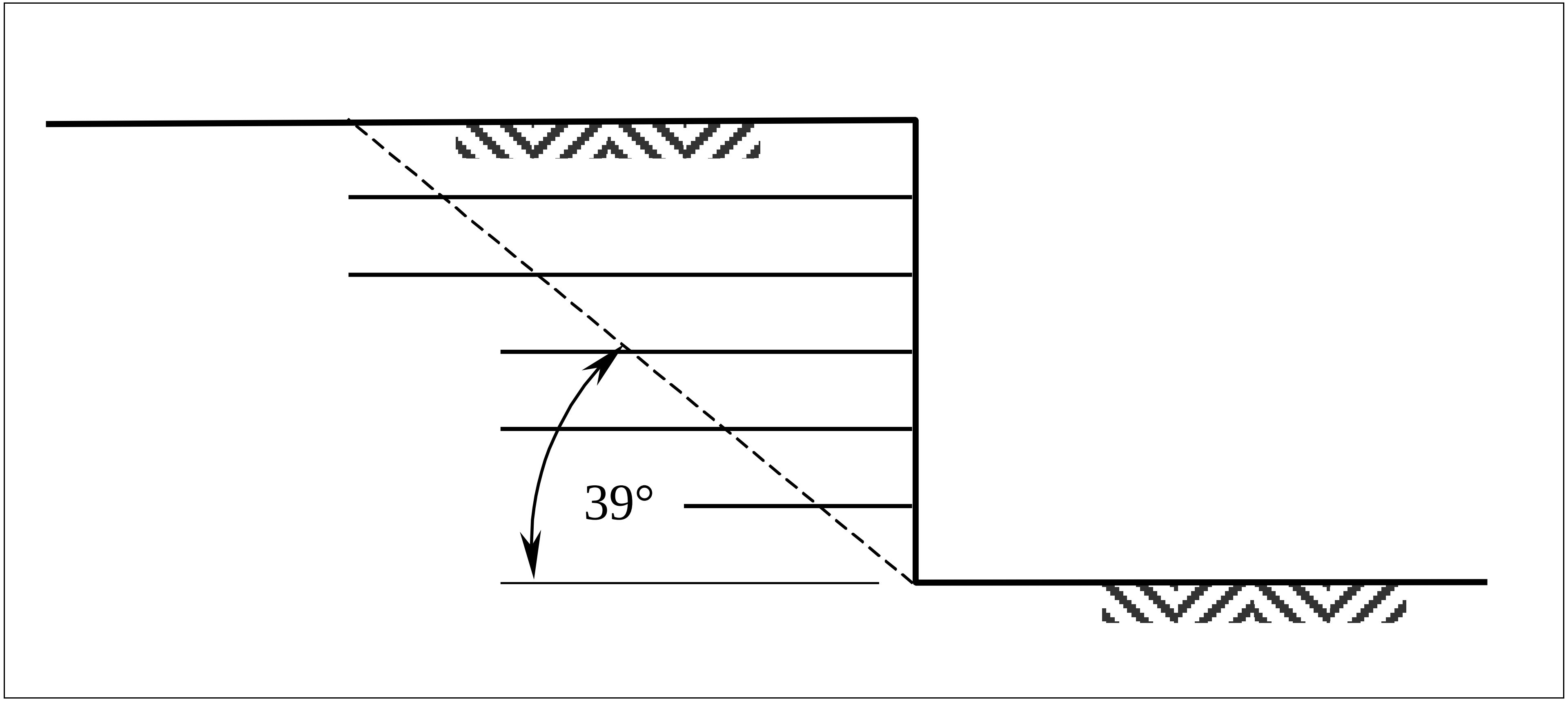


Figure 65 Support of rock slopes with cables

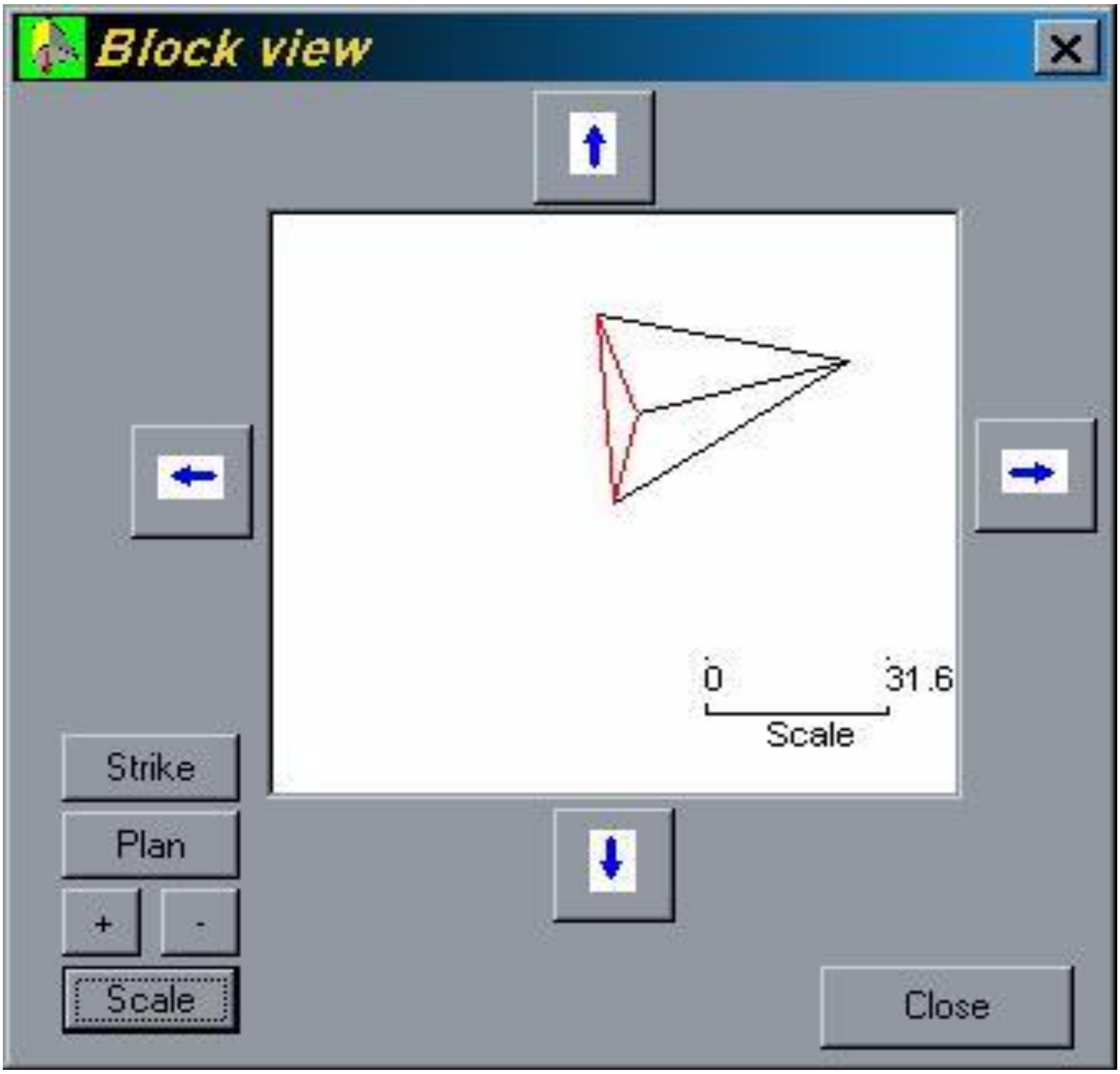


Figure 66 BlockEval wedge geometry window

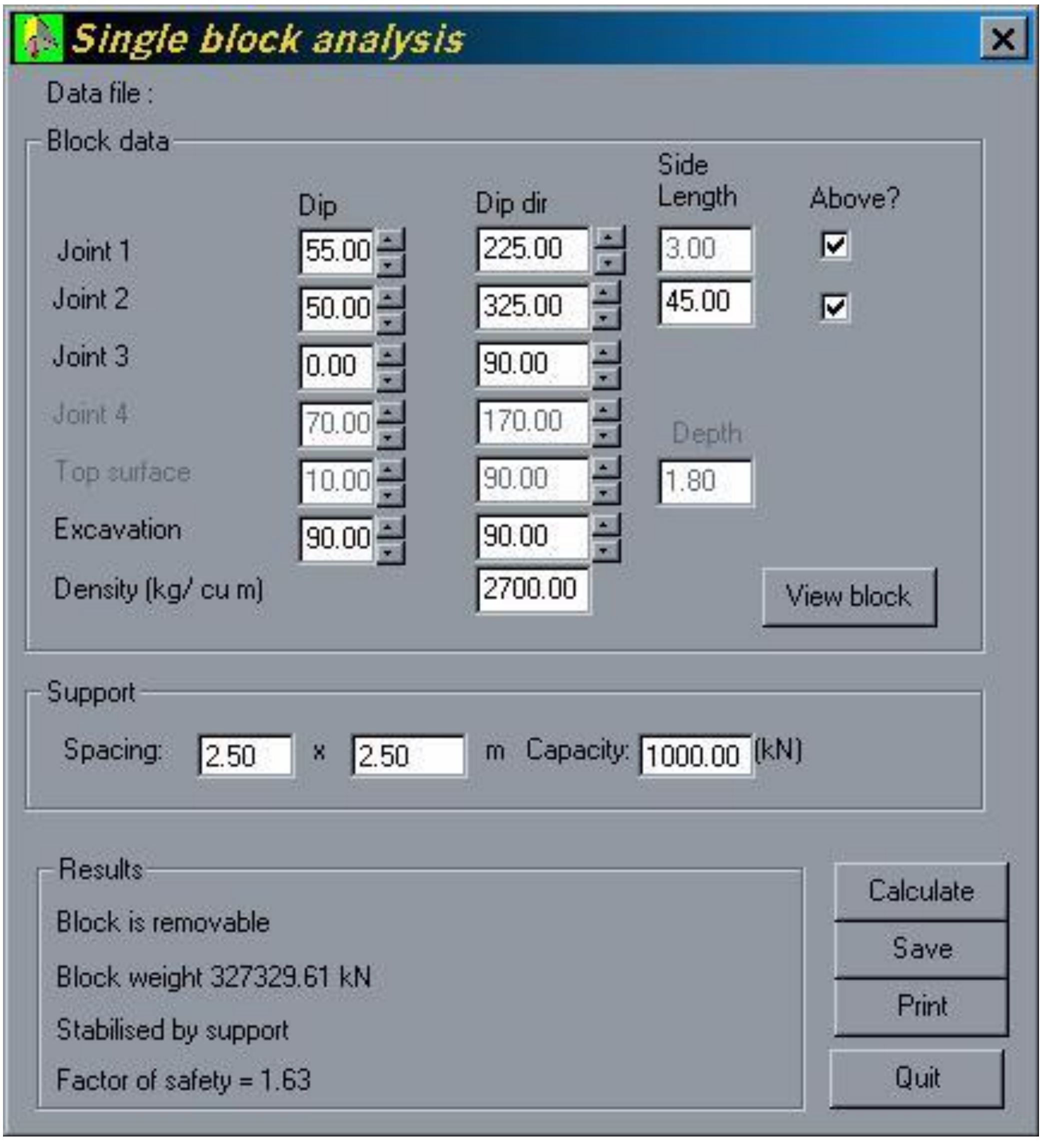


Figure 67 Input and results window for BlockEval

5 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 appropriate expertise readily at hand, a simple interactive computer based assessment system (GRA – Geotechnical Risk Assessment) has been developed and is included with this handbook. The system makes use of fault–event tree methodology, and greater detail on this methodology is included in Appendix A.

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 GRA Windows are shown in Figure 68. 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 (see Figure 68). If the probabilities of occurrence are unacceptable, these numbers are red, if acceptable they are green.

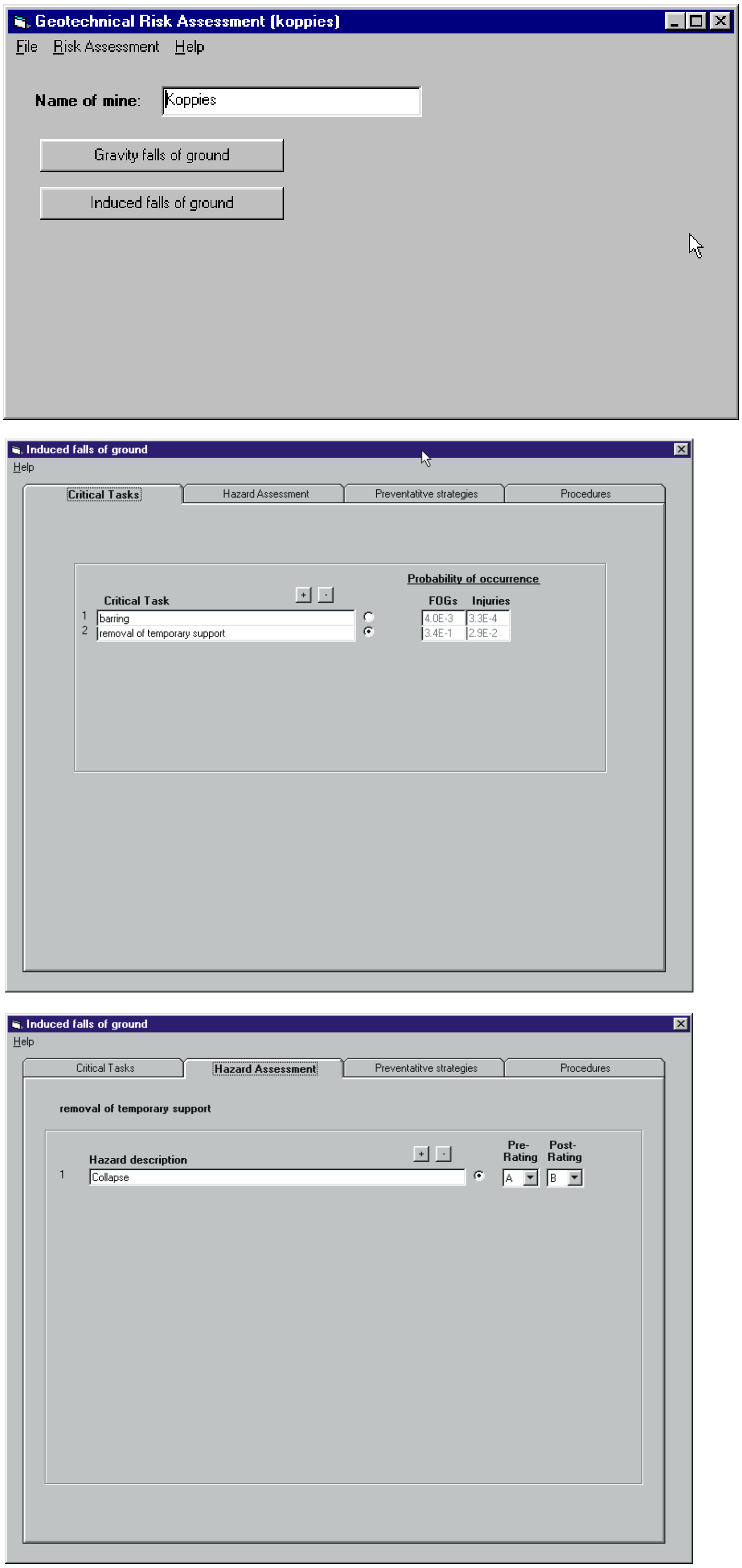
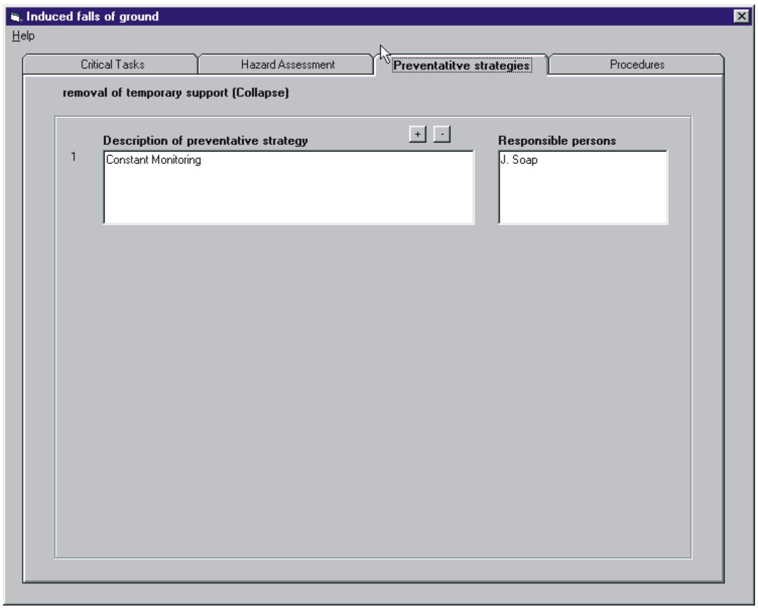
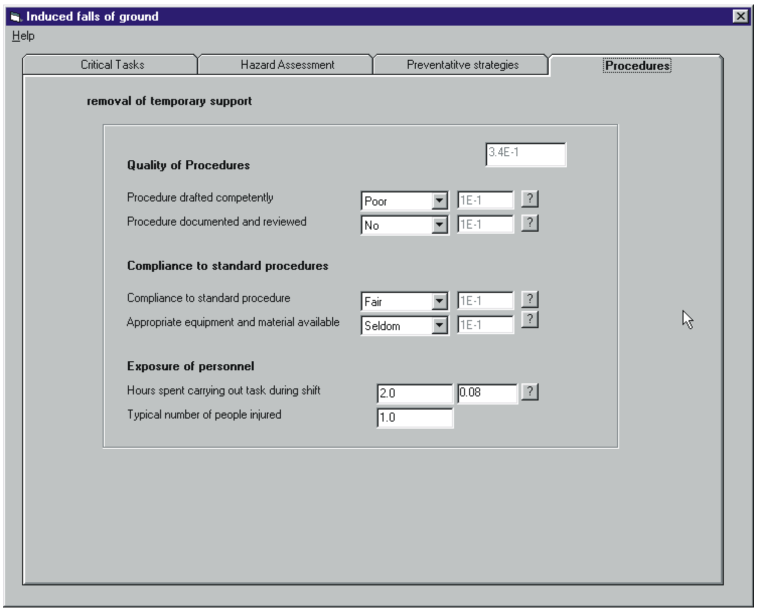


Figure 68 Windows for the geotechnical risk assessment program





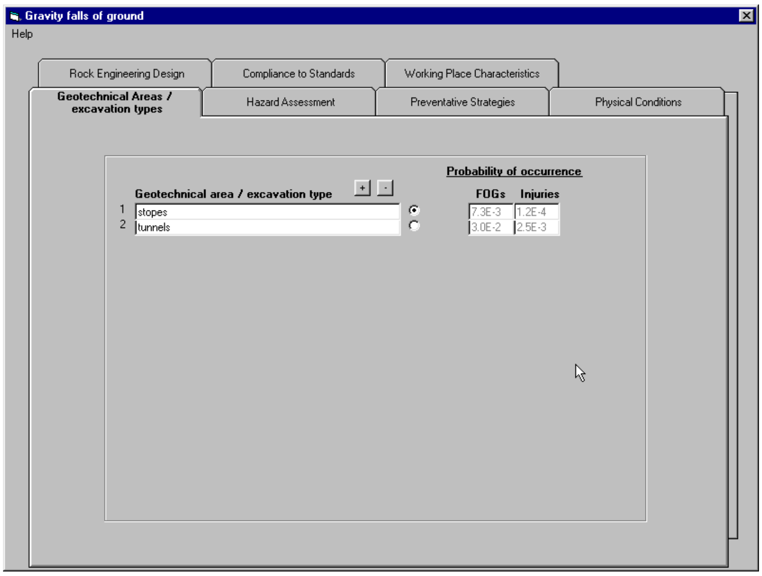
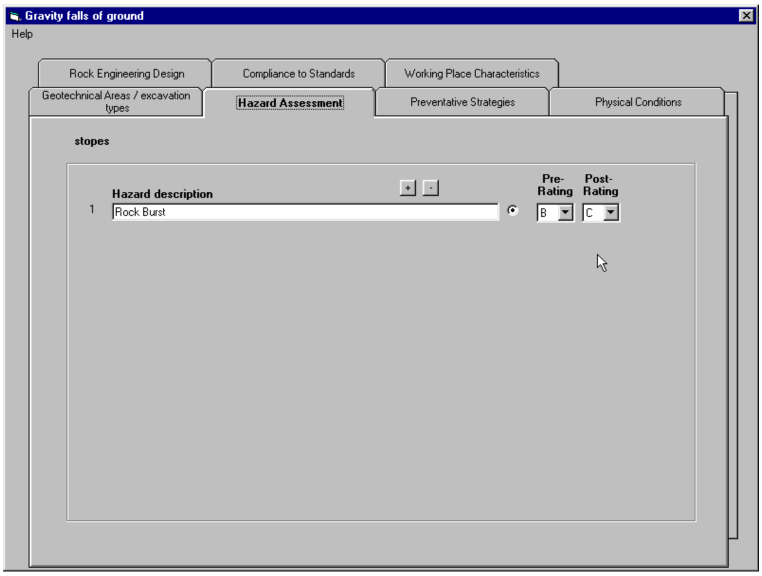
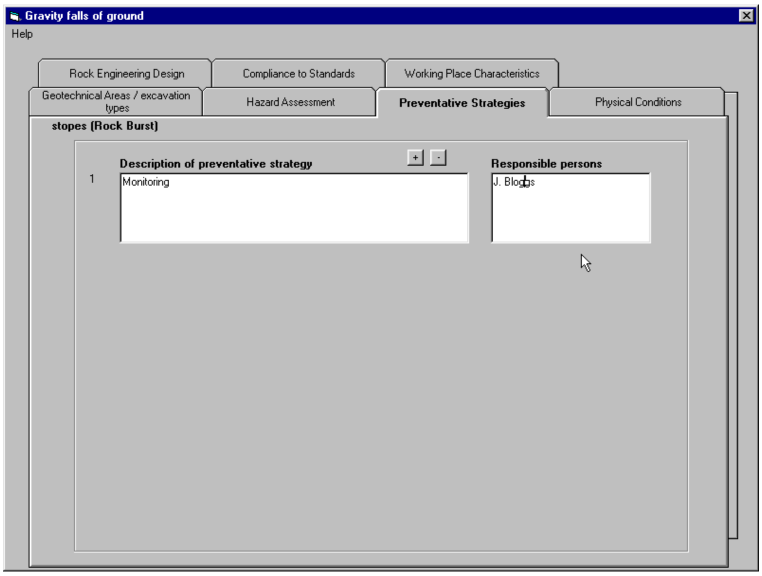


Figure 68 (cont.) Windows for the geotechnical risk assessment program





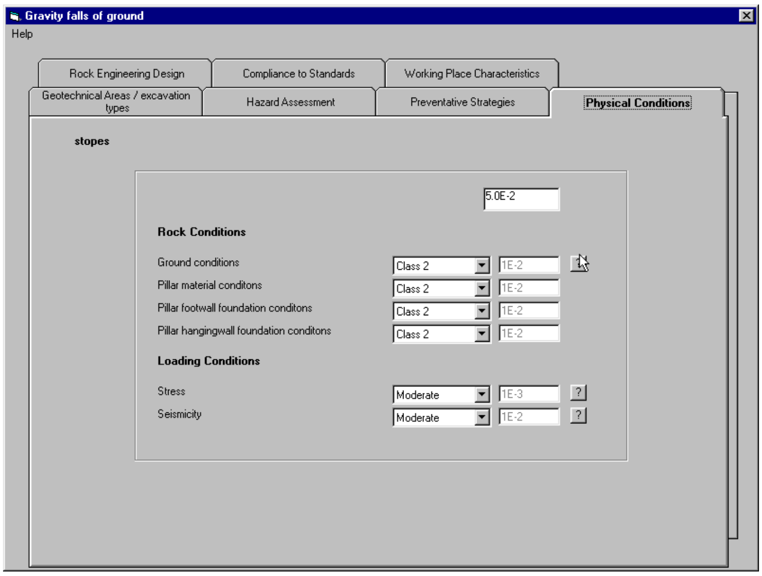
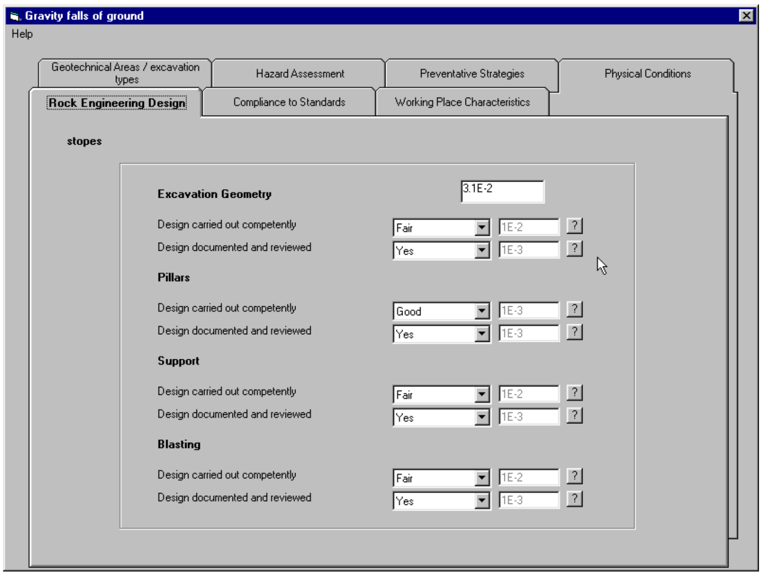
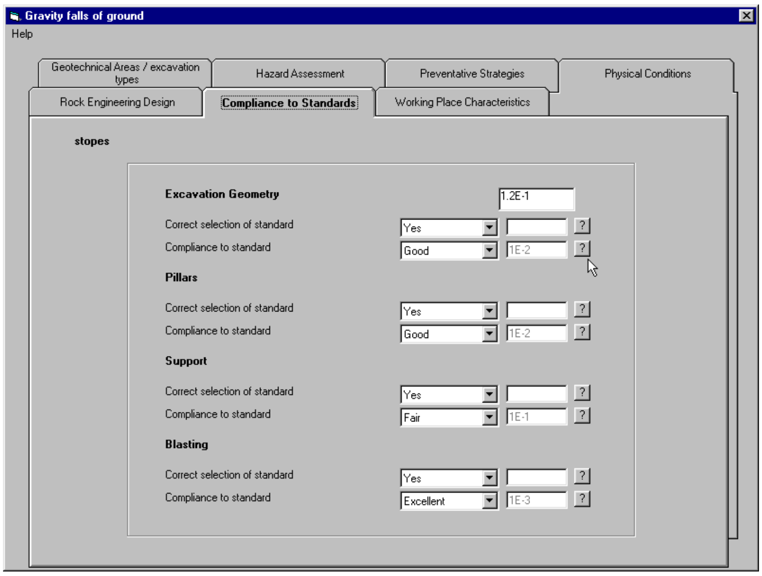


Figure 68 (cont.) Windows for the geotechnical risk assessment program





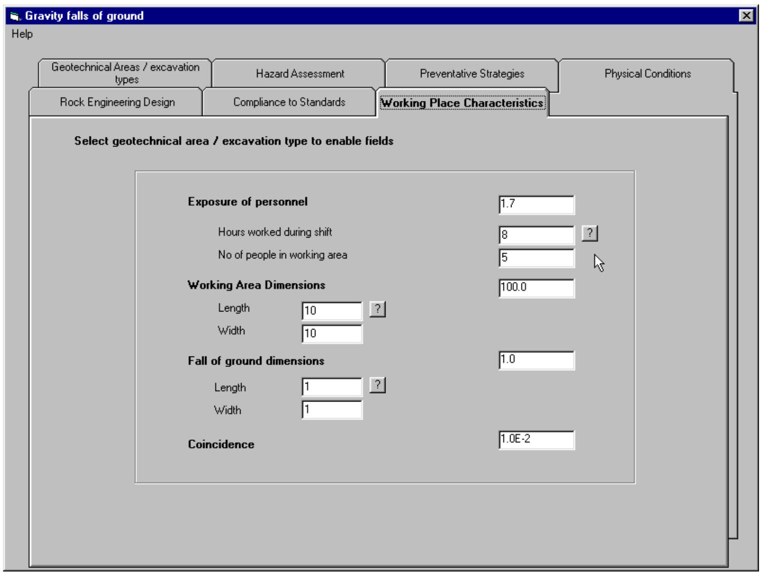


Figure 68 (cont.) Windows for the geotechnical risk assessment program

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Appendix A

Fault-event tree analysis

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Fault-event tree analysis

A1 Introduction

The failure of any system, e.g. a mine tunnel, is seldom the result of a single cause, or fault. Failure usually results after a combination of faults occurs in such a way that the factor of safety of the system falls to below unity. A disciplined and systematic approach is required to:

- determine the logic involved in the failure of the system, and
- analyse the potential consequences of failure.

A satisfactory approach is to use a **Fault-Event Tree Analysis**, which is described briefly in the following. A glossary of terms is given at the end of the description.

A2 Fault tree analysis

Fault Tree Analysis (FTA) is a quantitative technique by which conditions and factors that can contribute to a specified undesired incident (called the **top fault**) are deductively identified, organised in a logical manner, and presented pictorially.

Starting with the top fault, the possible causes or failure modes (**primary faults**) on the next lower level are identified. Following the step-by-step identification of undesirable events to successively lower levels will lead to the root causes.

FTA is a disciplined approach that is systematic, but at the same time flexible enough to allow analysis of a variety of factors. The application of the top-down approach focuses attention on those effects of failure which are directly related to the top fault. FTA is especially useful for analysing systems with many interfaces and interactions. The pictorial representation leads to an easy understanding of the system behaviour and the factors included Figure A69 gives an example of a fault tree.

In order to determine the logic involved in the failure of the system, faults are not initially given probabilities of occurrence. In this form the “tree” is referred to as a “**cause tree**”. Once the “cause tree” is considered to correctly reflect the combinations of faults necessary to result in failure, probabilities are either calculated or assigned to the faults. In this form the “tree” is referred to as a “**fault tree**”.

A “fault tree” therefore represents a quantitative evaluation of the probabilities of various faults leading to the calculation of the **top faults**, which result in failure of the system. The objective of the “fault-tree” is to identify and model the various system conditions that can result in the top fault (e.g. collapse of a tunnel).

The systematic nature of the fault-event tree enables the sensitivities of potentially adverse consequences to any causative hazards to be evaluated. This enables the most threatening causative hazards to be identified and eliminatory measures to be defined.

A3 Probability evaluation in fault tree

Probability evaluations within the “fault tree” are computed by means of “**AND**” gates or “**OR**” gates.

AND gates are used where faults are statistically dependent. If it is necessary for *n* secondary faults to occur in order for a primary fault to result, then the probability of occurrence is represented by:

p[primary fault] = p[secondary fault 1] * p[secondary fault 2] * …* p[secondary fault n]

OR gates are used where faults are statistically independent. If a primary fault can result as a consequence of the occurrence of any *n* secondary faults, then the probability of occurrence is determined from the calculation as follows:

p[primary fault] = 1 - (1 – p[secondary fault 1])(1 – p[secondary fault 2]) …
(1 – p[secondary fault n])

A4 Event tree analysis

Not every failure may lead to the consequence that is under investigation (e.g. loss of life). In order to facilitate modelling and computation of the possible consequences (or **events**) of a failure, an “**event tree**” is used. The “event tree” is developed by assigning probabilities of positive answers to a series of questions in such a way that lesser consequence events are eliminated and the probability of the top event can be calculated. The final result, or probability of occurrence of a consequence arising from failure can therefore be determined using a combined “fault-event tree”.

A5 Allocation of probabilities of occurrence

It is important to note that probabilities of occurrence may not have unique or discrete values. It is possible for a probability of a particular fault (or event) to change in sympathy with another probability that it is coupled with. This is best illustrated by means of an example such as “wrong support installation procedure” being used in an underground excavation. The probability of an incorrect support installation procedure being used depends upon the probability that:

- the knowledge about the correct support installation procedure is lacking, or;
- the equipment being used for support installations is out of order, or;
- the discipline and supervision are poor.

The probability that the knowledge about the correct support installation procedure is lacking in turn depends on the probability that:

- the support installation procedure is not defined by the mine standards, or;
- the support installation procedure is not communicated to the workers, or;
- the workers are incompetent.

The probability that the workers are incompetent depends on the probability that:

- inadequate training is provided, or;
- the workers are untrainable.

The probability of an incorrect support installation procedure being used could be different for different parts or sections of the mine. For example, the equipment being used for support installation in one section could be more reliable than the equipment being used in another section. Table A29 gives a correlation between quantitative probabilities of occurrence and qualitative descriptions of the likelihood of occurrence.

Table A29 Uncertainty and probability of occurrence

Qualitative likelihood of occurrence	Probability of occurrence
Certain	1.0
Very high	10^{-1}
High	10^{-2}
Medium	10^{-3}
Low	10^{-4}
Very low	10^{-5}
Extremely low	10^{-6}
Practically zero	10^{-7}

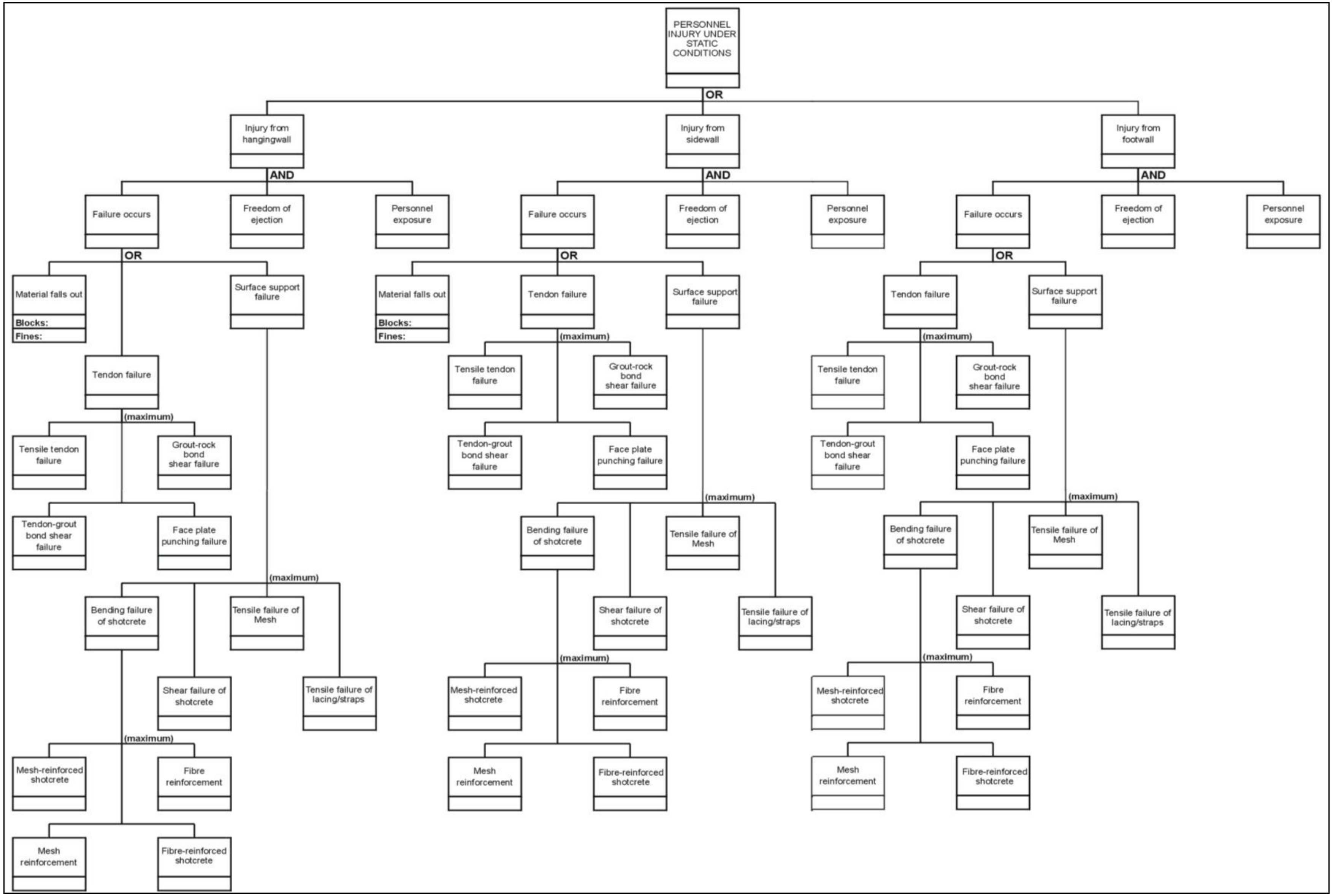

Figure A69 Fault tree example

A6 GLOSSARY OF TERMS

Hazard, cause, fault	Something that is a threat or has the potential to cause harm
Event, effect, consequence	Potentially damaging consequences. It denotes the effects of the causative hazards, for example, injury to people or damage to machines and equipment.
Fault Tree	The systematic display of causative hazards
Event Tree	The systematic display of the events
Primary Faults	The primary categories in which the hazards are considered
Secondary Faults	The component hazards that can be identified in each of the primary categories of hazards/faults
Tertiary Faults	The component hazards that can be identified in each of the secondary categories of hazards/faults.
Risk	Is the product of the probability of occurrence of a hazard and the consequence of the hazard (severity of the damage of an event).
Probability or likelihood	The number of times that a particular condition or situation of occurrence, chance can occur out of a total number of accounts.