SECTIONS 7 AND 8

EXCAVATION SUPPORT ANALYSIS

Mike Turner
Australian Mining Consultants

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7.1 DEVELOPMENT

7.1.1 Support and Reinforcement

The vast majority of underground excavation in mines is supported with one or more support elements, where in general terms a support element is an individual component such as a rockbolt, plate, mesh, cable etc. A support system includes one of more of these elements and the main function of these systems is to keep the excavation open and to prevent fall of ground accidents. There are usually one or more types of support elements and support systems which will outperform others with regards to site specific ground conditions, working environment etc. The objective of this section is to attempt to outline the available support and reinforcing elements and the methods for determining a suitable support system.

All geotechnical information for the orebody or mine should be incorporated into a geotechnical model. This model should be included in a Ground Control Plan for reference of data, assumptions, implementation of support designs and monitoring of performance. Documentation of all aspects of this design procedure should be maintained for audit purposes, including plans indicating locations of different ground conditions and support systems.

The major references for ground support design are included in the reference section. The ‘Support of Underground Excavations in Hard Rock’ and ‘Cablebolting in Underground Mines’ are the two main sources of information, with reams of additional information of rockbursts also available in the Canadian Rockburst Research Program. The three programs frequently referred to in the ‘Support of Underground Excavations in Hard Rock’ - *DIPS*, *UNWEDGE* and *PHASE2* are also recommended.

7.2.1 Why?

Support is installed in mines to enable orebodies to be extracted efficiently and safely. This involves obtaining permission to mine, physically keeping the access excavations open and preventing fall of ground accidents.

7.2.1.1 Legislative reasons

Applications to mine orebodies (Notice of Intent) submitted to the DME generally include a section on the geotechnical aspects of mining. This should indicate how the orebodies are planned to be extracted efficiently and safely. There is every likelihood that the DME would reject applications which did not satisfy the safe mining objective. To satisfy the fall of ground accident prevention aspect, the application should include a geotechnical model, including indications of rock mass properties, the response of the rock mass to mining and the support and reinforcement methods planned to reduce falls of ground, collapses and subsidence.

7.2.2.1 Access

A primary reason for installing for support is to establish and maintain access to enable extraction of economically mineable orebodies. If the support requirements are too extensive and expensive the orebodies might become uneconomic to mine. If support is insufficient and/or ineffective accidents could occur resulting in temporary or permanent closure of the mine.

It is therefore in everybody’s interest to design, develop and install the most cost effective system for the prevention of falls.

7.2.3.1 Safety and productivity

In this day and age, a stated industry objective has been to make underground workings as safe as surface working environments. Supported rock is safer than unsupported rock and the benefits of support, especially steel bolts and mesh, have been significant in accident reduction. The cost of fall of ground accidents is considerable – figures quoted for a fatal accident range up to the cost of one month’s production. Increased safety, however, results in improved workforce morale and hence productivity.

Safety is a major factor in support design and increased requirements for the prevention of, even minor, fall of ground occurrences can dominate support systems. Whilst support can be designed according to methods described later, recent movements towards ever safer mining practices have led to a more holistic approach being required.

It is also important to bear in mind the cost of rehabilitation and remedial support. Excavations not supported for the required life could require re-supporting, in effect the excavation will have been supported twice. Correctly designed support installed to high standards during the excavation stage should remove the future requirement for remedial support. Correctly designed and installed support also reduces delays due to additional inspection and scaling.

7.3.1 Support Types

Many different types of support are available and when the combinations are considered there are hundreds of support systems possible. Support has been installed in mines ever since mining commenced many thousands of years ago, consisting of timber poles, stone/brick packing and pillars. The objective of support system design and installation was, and still is, to establish and maintain safe economic access to the orebody.

The attached list includes many of the currently available elements for both development and stoping excavations. This list is by no means exhaustive and there are also many more variations to support elements on the list. Descriptions for a selection of these elements are also included in 'Support of Underground Excavations in Hard Rock', and more detailed descriptions on cable bolts in the 'Cablebolting in Underground Mines' book by Hutchinson and Diederichs.

Each of these support elements has specific properties and load displacement characteristics which will provide one or more of the 3 primary support functions, i.e. to reinforce the rockmass, to retain broken rock and to securely hold rock, loose rock or to tie back the retaining elements. Typical properties are outlined in the table below.

Characteristics of Typical Support Elements

Support Characteristic	**Support Function**		
	Reinforcing	**Retaining**	**Holding**
stiff	grouted rebar	shotcrete	grouted rebar
soft	-	mesh	long mech. bolt
strong	cable bolt	reinforced shotcrete	cable bolt
weak	thin rebar	#9 gauge mesh	split set
brittle	grouted rebar	plain shotcrete	grouted rebar
yielding	cone bolt	chain link mesh	yielding Swellex

(after CRRP 1996)

In addition, the load displacement performance of support elements should be considered – especially in relation to rock mass deformation. High expected rock mass deformations need to be matched to high displacement limits.

Load Displacement Parameters of Support Elements

Description	**Peak Load (kN)**	**Displacement Limit (mm)**	**Energy Absorption (kJ)**
19mm resin grouted rebar	120-170	10-30	1-4
16 mm cable bolt	160-240	20-40	2-6
16mm 2m mechanical bolt	70-120	20-50	2-4
16mm 4m debonded cable bolt	160-240	30-50	4-8
16mm grouted smooth bar	70-120	50-100	4-10
split set bolt	50-100	80-200	5-15
Yielding Swellex	80-90	100-150	8-12
Yielding Super Swellex	180-190	100-150	18-25
!6mm cone bolt	90-140	100-200	10-25
#6 gauge weld mesh	24-28	125-200	2-4/m^2
#4 gauge weld mesh	34-42	150-225	3-6/m^2
#9 gauge chain link mesh	32-38	350-450	3-10/m^2
shotcrete and weld mesh	2xmesh	$<$ mesh	3-5xmesh

(after CRRP 1996)

Examples of Support Types

Type	Examples and Variations	Advantages	Disadvantages
Mechanical Rockbolts	Different mechanical shells available. Forged or nut type heads.	Immediate active support	Shells only perform within a limited range of hole diameter. Susceptible to corrosion.
Friction Stabilisers	Split Sets, mild steel, galvanised, Stainless steel	Immediate support and reinforcing	Hole diameter critical to effectiveness. Susceptible to corrosion
Swellex	Mild steel, coated, stainless steel, and yielding	Immediate support and reinforcing. Initial small diameter can facilitate installation in very poor ground and long bolts from small excavations	Hole diameter critical to effectiveness. Relies on use of pump. Susceptible to corrosion
Hollow Groutable Bolts	Ingersoll Rand, Stelpipe	Immediate active support followed by longer term full column support.	Installation through mesh with jumbos can be awkward. Grouting can be difficult (especially with Stelpipes).
Combination Bolts	CT-Bolts	Immediate active support followed by longer term full column support.	Installation can be difficult through mesh with jumbos. Grouting can also be difficult.
Rebar	Mild steel, high tensile steel, continuous threading, resin or cement grout	High tensile capacity	Only effective once grout sets
Smooth Bar	Resin or cement grout	Capable of limited yield with cement grout	Only effective once grout sets

Type	Examples and Variations	Advantages	Disadvantages
De-stranded Hoist Rope	De-greased or not	Strong, yielding reinforcing	Labour intensive manufacture
Cone Bolts	Strata Control Systems, installed with cement grout and plate	Wax coated smooth bar, cone designed to yield through grout	Only performs with plate attached
Yielding Cable Bolts	Freyssinet	Mechanical, steel on steel mechanism	Relatively expensive
Cable Bolts	Plain strand, bulbed, de-bonded, multi-strand, mechanical, grouted, tensioned	Relatively cheap, strong reinforcing. Lengths cut to design requirements on site.	Requires specialist grouting equipment. Can corrode from centre of cable
Hoist Ropes	Locked coil, stranded, solid, hemp core	Very strong, yieldability dependent on extent of degreasing and rope core	Labour intensive installation, supply limited
Straps	Flat, W, mesh, cable	Used with or without mesh	Only useful for limited range of block size
Plates and Washers	Flat, domed, mesh, large/small	Domed plates provide limited initial yield capability and angled installation	
Timber Sets	Round poles, cut timber		Strength dependent on timber type, diameter and installation relative to load. Unsuitable for trackless working areas
Steel Sets	Leg bracing, Timber shielding, timber lagging, expanding foam filling	Visible performance. Very strong capacities available.	Time consuming installation. Relatively expensive.

Type	Examples and Variations	Advantages	Disadvantages
Steel Arches	Yielding	Visible performance, high load bearing capacities available	Limited range for excavation size variation
Concrete Lining	Pre-formed (e.g. ARMCO), cast in-situ	Pre-determined strengths	Bulky to move and relatively difficult to install
Shotcrete	With/without mesh	Can be pre-mixed and delivered where required. Reference 'Design of Shotcrete Support' section in 'Underground Excavations in Hard Rock'.	Susceptible to failure in high deformations. Quality governed by operator skills
Fibrecrete	Variation in fibre performance	Remains relatively intact following relatively high deformation	Relatively expensive
Polymers	Everbond, Mineguard	Thin, rapid setting	Relatively unproven, some are toxic.
Mesh	Diamond, Welded, Expanded	Varying strengths, yieldable, plain, galvanised, stainless steel	Can be time consuming to install
Lacing	Various patterns, installed in combination with mesh and bolts (with eyes/shepherd crooks)	Strong, suitable for large scale deformation and seismic areas	Labour intensive.
Cable Trusses	Various designs, fully grouted, mechanical shells, yielding	Strong surface support when used in combination with mesh	Requires extensive blocking /lagging when rock surface is concave
Cable Slings	Cables installed with split sets or Swellex	Can be installed without specialised equipment	Limited application
Timber Props	Poles, Cut timber	No specialised equipment required	Labour intensive

Type	Examples and Variations	Advantages	Disadvantages
Mechanical Props	Acrow Props, Camlok Props Screw, jacking, yielding, with/without headboards or load spreaders	Pre-set loads, yieldable, re-usable	Unsuitable for mechanised mining
Hydraulic Props	100kN, 200kN, 400kN, 1600kN, varying yield capacities (closure and velocities)	Pre-set loads, re-usable	Only suitable for narrow excavations. Capital intensive
Timber packs	Matpacks, combined elongate/slab packs	Suitable for large scale deformations	Labour intensive. Shrink with time. Fire risk unless treated. Only for narrow excavations
Grout packs	Cement/backfill grout	Can be tailored and designed to suit load/deformation requirements	Labour intensive plus requires a grout supply system
Composite Packs	Brick and timber	Can be designed to suit requirements	Labour Intensive
Cement Grout	Low Heat, pumped (pre-or post), cement capsules	Far cheaper than resin	Quality affected by water:cement ratio, quality of water and installation
Resin grout	Slow, medium, fast	Effective. Can be installed fast at end of hole, slow to collar, allowing full column grout with tensioning.	Expensive, diameter of capsule, bolt and hole must be matched. Requires special storage facilities.

7.4.1 Support Design

A few techniques are available for the determination of support requirements and support system design, including a few rules of thumb, empirical analyses using rock mass classification data and designs incorporating all available information.

The decisions to be made regarding the support system involve the type of support elements, the length of support element, the spacing between elements, and, if required, the thickness of shotcrete.

7.4.1.1 Rules of thumb

Some of the most basic methods of design, used for many years and still good for an initial design estimates, are the rules of thumb indicating the length and spacing of support element relative to excavation size. These rules were developed with experience gained in mines prior to the implementation of engineering design methods. The designs are suitable in the majority of conditions but should not be used for design purposes in this day and age without confirming the suitability of the design with other methods.

One rule of thumb is that the length of the support element should not be less than one third to one half the width of the excavation and the spacing between the bolts should not be less than half the bolt length. For example; a 6m x 6m drive would have 3m bolts on a 1.5m spacing. This rule has been applied to development and service excavations in South African mines for many years.

An additional provision to this rule is that the bolt spacing should be less than or equal to 4 times the average size of potentially unstable blocks.

Another rule indicates; 1.5m (5 feet) bolts for excavations less than 2.4m (8 feet); 1.8m bolts for excavations between 2.4m and 3m (8 to 10 feet); 2.4m bolts for excavations between 3m and 4m (10 to 12 feet) and 3m bolts for excavations greater than 4m (12 feet), with all bolts spaced at 1.2m (4 feet). This rule is applicable in mines where all development is undertaken using hand held airleg machines.

7.4.2.1 Rock mass classification methods

The rock mass classification methods described in an earlier section can be utilised for excavation support estimation. It should be noted that these empirical methods are very generalised and the conditions at a particular site could be outside the applicability of the methods. Some were also designed for Civil Engineering applications. These methods use a limited number of rock/rock mass properties – whereas to fully describe a rock mass would require details of at least than 40 parameters.

7.4.2.1.1 RSR

Wickham's Rock Structure Rating classification method contains a system for the design of support for tunnels. This is not widely used, having been eclipsed by the more detailed Q, RMR and MRMR systems.

7.4.2.1.2 Q

Rock mass classification using Barton's 'Q' system can be utilised to design the length and spacing of bolts and thickness of shotcrete for underground excavations. The additional factors which have to be taken into account include the size of excavation and the planned function of the excavation. To relate the Rock Tunnelling Quality index, Q, to support requirements requires the calculation of the equivalent dimension, D_e, and the excavation support ratio (ESR). The value of the excavation support ratio is related to the required lifespan and function of the excavation. Support length, support system design and maximum unsupported spans can all be estimated using the ESR.

	Excavation Category	**ESR**
A	Temporary mine openings	3-5
B	Permanent mine openings, water tunnels for hydro power excluding high pressure penstocks, pilot tunnels, drifts and headings for large excavations	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portal excavations	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories	0.8

After Barton (1974)

and $$D_e = \frac{\text{Excavation Span, Diameter or Height (m)}}{\text{Excavation Support Ratio (ESR)}}$$

The length of support calculation follows - please note that this can drastically underestimate support requirements relative to current safety attitudes and requirements for the prevention of fall of ground accidents. The maximum unsupported span can similarly overestimate the rock mass strength (where MUS = 2 x ESR x $Q^{0.4}$).

$$L = \frac{2 + 0.15B}{ESR}$$

Where L = length of support item and B = width of excavation

The Equivalent dimension can be used in combination with Q to estimate support requirements, as per attached Figure 1. This graphical method enables the support requirements to be estimated with regards to bolt length and spacing, shotcrete thickness etc. It should be noted that category 4 could also include the option of alternative areal coverage using mesh and/or straps - the updated design chart was specifically drawn up for shotcrete. A recent, 1996, paper by Barton covering the NMT is referenced.

7.4.2.1.3 RMR

Bieniawski's RMR method or rock mass classification can be used to estimate support requirements in relation to the calculated RMR value. The table below has been copied from the reference document 'Support of Underground Excavations in Hard Rock'.

Note that the designs are for 10m span rock tunnels, with a possible tendency to under-estimate support in the better ground categories.

Guidelines for excavation and support of 10m span rock tunnels in accordance with the RMR system

Rock mass class	**Excavation**	**Rock bolts (20mm diameter, fully grouted)**	**Shotcrete**	**Steel sets**
I - Very good rock *RMR*: 81-100	Full face, 3m advance	Generally no support required, except spot bolting		
II – Good rock *RMR*: 61-80	Full face. 1-1.5m advance. Complete Support 20m from face	Locally, bolts in crown 3m long, spaced 2.5m with occasional wire mesh	50mm in crown where required	None
III - Fair rock *RMR*: 41-60	Top heading and bench. 1.5-3m advance in top heading. Commence support after each blast. Complete support 10m from face.	Systematic bolts 4m long, spaced 1.5-2m in crown and walls with wire mesh in crown	50-100mm in crown and 30mm in sides	None
IV – Poor rock *RMR*: 21-40	Top heading and bench. 1.0-1.5m advance in top heading. Install support concurrently with excavation, 10m from face.	Systematic bolts 4-5m long, spaced 1-1.5m in crown and walls, with wire mesh	100-150mm in crown and 100mm in sides	Light to medium ribs spaced 1.5m where required
V – Very poor rock *RMR*: ≤ 20	Multiple drifts. 0.5-1.5m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6m long, spaced 1-1.5m in crown and walls with wire mesh. Bolt invert.	150-200mm in crown, 150mm in sides and 50mm on face	Medium to heavy ribs spaced 0.75m with steel lagging and forepoling if required. Close invert.

(After Bieniawski, 1989)

7.4.2.1.4 MRMR

Laubscher's MRMR method can also be used to determine support requirements based on support systems used in other mines with similar ground conditions. Copies of the relevant tables and graphs from Laubscher's 1990 paper are attached. Tables IX and X (Figures 2 and 3) describe relationships between Rock Mass Ratings, Mining Rock Mass Ratings and various support techniques. Relationships between the design rock mass strength (DRMS) and the maximum stress and mining environment stress are also indicated in Figures 4 and 5, for various support techniques.

7.4.3.1 Numerical methods

Numerical design of rock support required to prevent slippage along planes and falling of wedges is described in Hoek and Brown, 'Underground Excavations in Rock'. There are software packages available to calculate wedge sizes, support requirements and factors of safety.

7.4.3.1.1 UNWEDGE

UNWEDGE is an example of a commercial program for support design. This software can utilise input from the structural joint analysis program *DIPS*, or joints can be input manually. The geometry of an excavation is entered into the program either manually, or via a *dxf* file, and various support options can be assessed. The program accepts various bolt types, e.g. frictional, end anchored, with or without plates and with or without shotcrete. The type of bolt and the load bearing capacities, spacing, length and installation angle of the bolts can then be altered to investigate the sensitivity of factor of safety to different systems. The best way to learn the use of *UNWEDGE* is to assess various alternative support scenarios, commencing with the basic standard support for a particular mine.

7.4.3.1.2 *FLAC, PHASE2* etc

Geotechnical simulation packages such as *FLAC* and *PHASE2* can be used to assess the applicability and response of support to changing stress environments. These analyses will generally require a relatively long time to conduct and the accuracy of the results will be controlled by the rock mass properties. Rock mass properties at the periphery of excavations are not easily determined and hence the response of the rock mass alone to stress changes is difficult, even without consideration of support.

The programs can indicate the relative difference in deformation expected for unsupported and/or unreinforced excavations as opposed to reinforced and/or supported excavations.

More realistic results would be obtainable using 3-Dimensional programmes such as *FLAC-3D* and 3-D finite element programs, but the time required to set up and run simulations only make this worthwhile for very high capital intensive programmes such as underground power stations, public stadiums, nuclear repositories etc.

7.4.4.1 Holistic, site specific

Sole use of one of the previously discussed methods could easily produce a support system which does not take into account all major factors. Ideally the support system should be designed taking account **all** of the following factors;

Rock Mass Properties
Excavation Size and Shape
Planned Life and Function of Excavation
Excavation Orientation
Previous Support Performance
Legislated Guidelines
Corporate Requirements
Contractual Limitations
Stress
Rock Mass Deformation
Seismicity
Groundwater (flow rates, pressures and corrosiveness)
Excavation Method
Availability and Cost
Air Quality

These issues are briefly discussed below.

7.4.4.1.1 Rock mass properties

Rock mass properties should always be taken into account when designing support systems for underground excavations. As detailed previously the Q, RMR and MRMR systems of rock mass classification are all useful empirical methods of grouping the rock mass into categories with similar strength properties. Additional properties required for specific sites could include Fracture Toughness and other tests for the determination of strain burst proneness, slake durability to determine susceptibility to weathering and mineralogical evaluation to determine the presence of swelling minerals etc etc. The requirement for some of these tests only becomes apparent following drill core deterioration and problems during excavation. Local knowledge of rock behaviour is very important.

7.4.4.1.2 Excavation size and shape

Large excavations expose more structural weaknesses and, in high stress environments, the depth of fracturing may extend further into the rock mass. This may require longer bolts for stabilisation.

Narrow excavations may also limit the choice of support elements due to equipment and space limitations. Narrow excavations could also enable other elements to become useable, such as hand installed units, props etc. The shape of excavations in relation to foliation could require support elements in the back to be installed at an angle other than perpendicular to the wall. Bolts installed along foliation are not as effective as across foliation and designs should always bear this in mind.

7.4.4.1.3 Planned life and function of excavation

The required or expected life and function of an excavation, together with the environmental conditions and future access, must be considered when determining the most suitable support elements. Re-support and rehabilitation of excavations is expensive - the correct support installed to high standards the first time around will be more cost effective.

An excavation with a limited life span might be supportable with split sets as the main support item. The workable life of this type of unit could range from only a few months to a few years and is dependant upon individual site conditions. Excavations with required lifespans of twenty or more years should be supported with fully grouted, galvanised or stainless steel systems, dependant on the environment.

Areas which are required to last many years, and to which access will be lost, may require upgraded and corrosion proof systems. Shotcrete and concrete are long life systems suitable for drives and service excavations. Long life requirements should be specified in the material specifications - some additives can reduce the life of concrete and shotcrete. In some situations the use of concrete and shotcrete is impractical, e.g. raisebored ventilation shafts and timber lined hoisting shafts. A more suitable system could incorporate heavy galvanised fully grouted bars and plates (varying grades of galvanising are available - the more you pay the more you get), stainless steel expanded mesh, with additional corrosion protecting paint. Thicker steel, than that required for support purposes, for bars and mesh is one method of extending life. These types of support systems may seem like over-support on a grand scale but a collapse in an inaccessible ventilation shaft or ventilation drive could cost millions of dollars to rectify.

The use of an excavation for LHDs and/or trucks could also influence the support design. Elements which lose all functionality when a truck collides with the wall and removes plates or connections may require alternative support elements which function without plates or could require an additional protective coating of shotcrete. Drawpoints have similar requirements – support must be LHD and blast proof.

7.4.4.1.4 Excavation orientation

Excavations can be stable in one orientation yet unstable in another. The support requirements for alternative orientations could therefore vary considerably. The presence of major structures within the rock, the virgin and mining induced stress fields are major factors determining stability and support requirements relative to orientation. Also important is local knowledge regarding the stability of excavations in similar conditions.

In many situations, development (and other excavations, including stopes) across foliation and parallel to the major principal stress is the most stable orientation, requiring basic support. Excavations developed parallel to the foliation and perpendicular to the major principal stress could require additional bolts along the sidewalls to prevent slabbing failure, additional bolts and mesh across the backs and corners to prevent small wedges, and overall mesh coverage to prevent falls occurring if the area becomes de-stressed due to mining (normal stress across foliation reduced or even removed).

7.4.4.1.5 Previous support performance

Knowledge of the performance of a variety of support elements at the mine will assist in determining the support system. This relates to the ability to retain and hold smaller blocks, bolt failure mechanisms, optimum installation orientation, corrosion susceptibility and ease of installation.

7.4.4.1.6 Legislated guidelines

Legal guidelines for minimum support standards are already in place and could become more detailed in the coming year with the MOSHAB Code of Practice. This should not have the effect of removing engineering input into design. One of the main objectives of the Code of Practice is to ensure sufficient geotechnical or other satisfactory engineering input into support system design. Where there is no engineering input the guidelines could require the blanket installation of mesh (for example). With sufficient geotechnical input the DME could accept minimal support, if this is proven to be enough to prevent falls of ground.

7.4.4.1.7 Corporate requirements

Corporate standards may require far more support than is actually determined from geotechnical evaluation. A requirement for bolt and mesh in all development areas is a corporate requirement at some WMC mines for example. There are areas where this is oversupport by an order of magnitude. The time and hours required to alter corporate requirements should be weighed up against possible cost savings, perceived risk of fall of ground accidents (however minor), and reluctance of mine management to accept additional risks (re: public opinion, share price etc).

7.4.4.1.8 Contractual limitations

Existing contractual agreements exist on mines and whilst these should not affect support design they can in practice have an impact. A contract could, for example, only include contractual agreements for split sets and resin grouted rebars, whereas a design analysis indicates HGBs are more suitable. Contractual re-negotiation is always possible even though it could take up (sometimes reluctant) management time.

7.4.4.1.9 Stress

The design of support systems should take into account the virgin stress regime, mainly to determine the expected magnitude of rock mass deformation and susceptibility to seismicity. Maximum Principal Stress levels approaching UCS/2 should indicate that rock mass deformations could be a critical aspect when designing support. Virgin stress fields can be very variable due to the influence of geological structures and history. This should always be noted when interpreting stress fields into new areas.

Stress magnitudes and directions can change considerably with the commencement of stoping operations. Mining induced stress changes can be estimated using numerical modelling methods. This is particularly necessary close to large stoping excavations in high stress areas. Relating these stress changes to rock mass deformations and movement should indicate the most suitable reinforcing and/or support systems. Calibration of numerical modelling results to underground rock mass deformation monitoring enables more accurate prediction of deformation.

The total expected stress regime during the planned life of a stope access excavation could typically include periods of increasing stress, changes in stress direction, peak stresses and finally stress relaxation. This series of changes could, in turn, cause clamping of wedges, slabbing around pillars, increased fracturing in one or more corners and finally increased falls of ground following the relaxation of stresses. If this is known in advance, the support system would not rely on point anchored elements and would include full column frictional or grouted elements plus mesh.

7.4.4.1.10 Rock mass deformation

Matching rock mass deformation to support system capabilities is critical to the effectiveness and life of support and reinforcing elements. In low stress areas this is generally not applicable except where the stress regime becomes tensile and the rock mass opens up on foliation or other major planes. As stress levels increase with depth, or with the extent of mining, rock mass deformations can increase significantly. The possible presence of rocks which exhibit creep or rapid deformation rates should also be investigated. This type of rock behaviour would require yielding support to prevent premature failure.

All support elements have a limited ability to cope with rock mass deformations, ranging from tens of mm for fully grouted elements to a few metres for debonded, yielding cables.

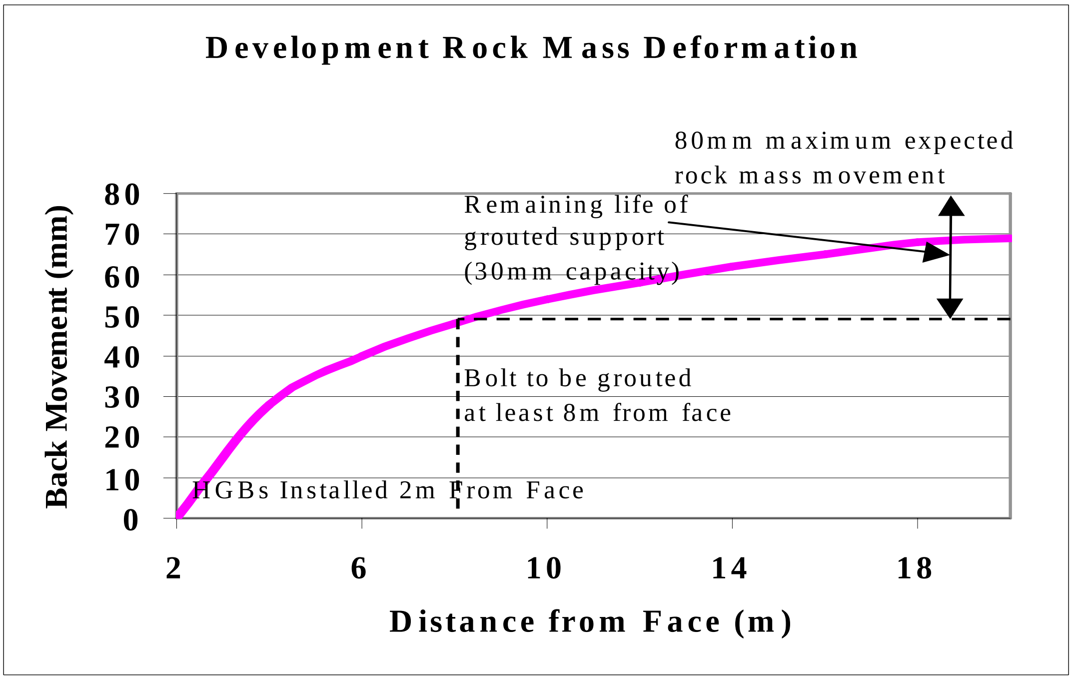
Prediction of rock mass deformations using numerical methods is possible but can greatly under-estimate deformations. If models have been calibrated with actual deformations the predictions will be more reliable.

Monitoring the response of the rock mass to excavation (including both the development and stoping phases) is the practical method required to quantify movement, although, obviously, this cannot be undertaken prior to mining. In currently operating mines monitoring methods such as closure pegs between the walls and backs of excavations, tell tale units installed with support elements, extensometers installed into the rock mass, and general survey techniques can all be used. The results from all these types of monitoring can be used for numerical model calibration.

Generally, the greater the expected rock mass movement the more critical this component becomes in the choice of support due to the limited number of yielding elements available.

Fully grouted elements such as mild steel rebar will cope with 30mm of movement per crack or moving feature. Split sets and other frictional elements will yield for a few hundred mm, at loads varying from 30 to 45 kN per meter of installed bar. The yield capability of mechanical rockbolts and de-bonded rebar depends on the steel properties and could range from 3 to 20% of the installed length (assuming the shell outperforms the bar).

An example of the application of such information is to determine at what distance from the face are hollow groutable bolts (HGBs) grouted. An example graph of measured back deformation relative to the distance from the face is attached. From this graph the initial deformation rate is steep, with 40mm of movement occurring within 6m of measurements starting (pegs installed at face, first measurement 2m from face). This deformation rate could cause premature failure of fully grouted elements within 10m of the face. Working back from an estimated maximum back deformation of 80mm it can be seen that HGBs should not be grouted closer than 8m from the face.



7.4.4.1.11 Seismicity

If an excavation is expected to be exposed to seismic activity, additional importance should be given to the ability of the support system to cope with rapid deformation. Sidewall deformation can exceed tens of cm, at velocities greater than 3m/s, due to seismic events. This can be analysed using the seismic section of the support resistance methodology, as discussed in the section 7.5, Support Effectiveness.

Bolts in seismic prone areas require rapid yield capabilities in excess of 30cm and the surface retaining elements, such as mesh or fibrecrete also need to cope with similar deformation (bulking). The complete support system should be strong, capable of yielding and containing ejected material and also capable of containing the bulked rock mass.

Cone bolts, yielding cable bolts, long de-bonded tendons (cable, smooth bar or rebar), diamond mesh and lacing are all suitable for rockburst protection of excavations. The minimum combination would be cone bolts plus mesh. Mesh and steel fibre reinforced shotcrete are also suitable but non-reinforced shotcrete is unsuitable.

7.4.4.1.12 Groundwater (flow rates, pressures and corrosiveness)

The presence of groundwater, especially in areas of high flow rates, and high corrosiveness due to low pH and/or high Cl, tend to force the design of support systems away from a reliance on thin walled steel support items. Opportunities do exist to extend the life of thin walled units, such as the use of galvanised or stainless steel, cement grouting (with Split Sets for example), and corrosion resistant coatings (Swellex for example). The workable underground life of such thin walled elements in corrosive environments is difficult to predict in laboratory conditions. Representative historical test and performance data should be built up prior to total reliance on such systems. A typical life of a thin walled unit in high salinity, and low pH conditions could be 12 months.

The quality of groundwater can also affect the strength of cementitious grouts. Test programmes should be instituted to determine representative strengths of grout using local water and cement supplies. If the tests indicate unsatisfactory strengths, alternative water supplies or grouts should be assessed.

Fully grouted steel elements are generally considered to be more resistant to corrosion. Corrosive groundwater will attack steel units where rock movements have cracked grout, exposing steel - this could lead to premature failure due to general corrosion in mild steel units or embrittlement in high tensile steel units.

Cable bolts are also susceptible to this type of corrosion but in addition can be corroded from the inside out. Grouting of cable bolt units does not generally effect filling of the internal, inter-wire voids, especially with the use of thicker grout mixes and grout-first installation methods. Water exiting from the exposed end of the cable, or evidence of previous water flow, is an indication that there is a possibility of internal corrosion.

Underground observations of previously installed support are in many cases unable to determine the extent of corrosion. If in doubt, the elements should be pull tested (which only really assesses whether there is intact steel between the collar and a 50cm length of grout) and/or replaced.

7.4.4.1.13 Excavation method

Poor blasting techniques can cause extensive damage to the rock mass, leading to increased support requirements. The ultimate objective of good blasting should be to maintain the rock mass surrounding the excavation in an undisturbed condition, facilitating the self supporting properties of the rock mass. At the other extreme the use of Tunnel Boring Machines, raiseborers and roadheaders limit damage to excavations but also have a tendency to hide possibly critical discontinuities.

Good blasting techniques, including reduced peripheral and penultimate hole charge densities, well designed drilling patterns, explosive charges and hole timing can reduce support requirements. Such techniques reduce the incidence of small blocks, increasing the effectiveness of standard support patterns and reducing the need for systematic mesh installation (for example). Management procedures for the monitoring and control of overbreak are important to mine personnel involved in support design – overbreak could be an indicator of poor blasting practice leading to additional support, or could even indicate less competent ground conditions.

With many excavations requiring mesh to the face (for one reason or another), the impact of blast damage on support elements is becoming more critical. If mesh or hollow groutable bolts, for example, are being regularly damaged by blasting either the blasting should be investigated or the support should be upgraded to cope, or a combination of both. Upgrading mesh could resolve the blast damage problem but cause installation difficulties, as could changing from HGBs to rebars - engineers and management investigating changes to support items and support systems should consider all aspects.

7.4.4.1.14 Availability and cost

In some regions certain support items will not be available or will be available at excessively high costs. In these cases alternative support items should be utilised if suitable. An example of this is the use of pumped cement or cement capsules to replace resin capsules in hot, remote areas.

Equipment availability should also be considered – mining companies and contractors do not always have finances for specialised equipment. Absence of a Rockbolting jumbo could rule out resin capsules as a first line support. Specialised shotcreting or cable bolt grouting and tensioning equipment are also items of equipment not held as stock by most mining companies. Contractors are, however, usually available in Australia to supply and operate all types of equipment, albeit at sometimes elevated cost. Sometimes increased support costs can mean increased output, safety and reduced dilution –all aspects should be considered before writing off certain support systems.

7.4.4.1.15 Air quality

The installation of thin walled support elements in return/exhaust airway environments will generally lead to a reduced workable life due to increased humidity. Fully grouted elements would be more suitable for longer life in harsher return air environments. Differing grades of stainless steel and galvanised bolts are available, or could be made up, for long term installations, e.g. in return airways in blocky ground. Other options could include greasing or other protection of exposed surfaces and shotcrete or polymer membranes.

7.5.1 Support Effectiveness

Design of support systems is not the end of the story. Engineers designing the support, operators installing the support and managers controlling underground mines are all legally and morally bound to ensure such designs and recommendations are followed through effectively.

7.5.1.1 Installation procedures

The crews installing support, whether they are contractors or mining company employees, should have written procedures for the installation of all support elements. These procedures should have been developed by the contractor, mining company or support supplier and should have been checked by **all** parties. The procedures should also be checked by safety, occupational health or loss control departments.

Regular audits to determine whether written procedures are being adhered to are also recommended, especially following changes in contractors, management, other staff or ground conditions. Variances to agreed procedures and required changes should be communicated as soon as possible - delays could indicate your acceptance of lower or changed procedures. Difficulties in following set procedures could also require re-design of support systems to ensure areas are supported effectively and efficiently.

7.5.2.1 Testing programmes

Regular test programmes are required to determine the effectiveness of material specifications and installation procedures. This should include regular pull testing programmes (support suppliers should be willing to assist and co-operate in this regard), regular testing of grout cubes, shotcrete panels and cubes, and material testing of steel elements at suitably certificated laboratories.

If support materials or installation standards are consistently lower than those specified, changes could be required to support suppliers, elements or systems. Lower shotcrete strengths could require increased thickness to compensate.

7.5.3.1 Support resistance

The support resistance of a support system can be used to compare the relative strength of systems. The support resistance capacities of support systems required to retain specific blocks and thicknesses of rock can also be calculated from individual bolt strengths. Required installation patterns can be determined from this information.

The attached figure on the function of local support indicates the basics of the support resistance calculation. Both the rock load to be supported and the support capacity are rationalised in terms of force per unit area, kN/m^2.

Example:

A mine has historical data on fall of ground incidents accidents that indicate that 98% of falls are caused by slabs/beams of rock with a thickness of 0.75m or less. Assuming the density of rock is 3.0 the support will have to carry a load exceeding the deadweight of such beams;

$$= \rho g h = 3.0 \times 1000 \times 9.81 \times 0.75 = 22 \text{ kN/m}^2$$

where ρ = density of rock (kg/m3)

g = acceleration due to gravity

and h = thickness of beam

assuming a corporate factor of safety = 2

$\therefore$ load to be supported by system = 2 x 22 = 44 kN/m^2

(i.e. required support resistance)

Assuming the beam is relatively continuous between support elements and that 10 tonne (100kN) point anchored bolts are the support element of choice;

capacity of bolt = 100 kN

$\therefore$ required bolts per m^2 = $\frac{22 \times 2}{100}$ = 0.44 bolts/m^2

$\therefore$ the area per bolt = 1/0.44 = 2.27m^2/bolt

and $\therefore$ required bolting pattern = $\sqrt{2.27}$ = 1.5m x 1.5m

This calculation could have been calculated in reverse order. For example an area is currently bolted with 190 kN fully grouted rebars on a 2m pattern – is the support system sufficient?

support resistance of current system = $\frac{190}{2 \times 2}$ = 47.5 kN/m^2

This is in excess of the required 44kN/m^2 so the system is also sufficient.

Support resistance requirements can also be increased to take into account of additional seismic related forces. The formula for calculating the required support resistance becomes;

$$\text{support resistance} = \left(1 + \frac{\boldsymbol{v}^2}{2\boldsymbol{gd}}\right)\boldsymbol{\rho g h}$$

where $\boldsymbol{v}$ = velocity of rock
and $\boldsymbol{d}$ = distance rock can travel (= yield capacity of support)

Example:

The same mine as in the previous example, with a seismic velocity of 3m/s and rock movements of 0.05m, 0.1m and 0.3m (fully grouted bar, mechanical rockbolt and cone bolt, with load capacities of 170, 120 and 140kN respectively).

The required support resistance $= \left(1 + \frac{3^2}{2 \boldsymbol{x} 9.81 \boldsymbol{x} \boldsymbol{d}}\right) 3 \boldsymbol{x} 1000 \boldsymbol{x} 9.81 \boldsymbol{x} 0.75$

$= \left(1 + \frac{0.46}{\boldsymbol{d}}\right) 22$

= 224kN/m^2 for $\boldsymbol{d}$ = 0.05
= 123kN/m^2 for $\boldsymbol{d}$ = 0.1
and = 56kN/m^2 for $\boldsymbol{d}$ = 0.3

Applying a factor of safety of 2;
the required support resistances = 448kN/m^2 for $\boldsymbol{d}$ = 0.05
= 246kN/m^2 for $\boldsymbol{d}$ = 0.1
and = 112kN/m^2 for $\boldsymbol{d}$ = 0.3

Assuming mesh is installed for all systems, with a support resistance of 25kN/m^2, the support resistances required from the bolts are;

= 423kN/m^2 for $\boldsymbol{d}$ = 0.05
= 221kN/m^2 for $\boldsymbol{d}$ = 0.1
and = 87kN/m^2 for $\boldsymbol{d}$ = 0.3

The bolt spacing therefore;

$= \sqrt{\frac{170}{423}}$ = 0.63m x 0.63m for the grouted rebar

$= \sqrt{\frac{120}{221}}$ = 0.73m x 0.73m for the mechanical bolt

$= \sqrt{\frac{140}{87}}$ = 1.26m x 1.26m for the cone bolt

This simplified example illustrates the advantage of yielding support systems – they are capable of far greater work loads than non-yielding bolts and systems. Good references for this type of analysis are the South African 'An Industry Guide to Methods of Ameliorating the Hazards of Rockfalls and Rockbursts', 1988, and the 'Canadian Rockburst Research Program' Summary document, 1990-1995. Cone bolts and Yielding Super Swellex are examples of strong, yielding elements which are suitable for rockburst conditions.

Various examples for support resistance/pressure are attached at the end of this section, including typical calculated values for common rockbolts, steel sets, mesh, shotcrete and concrete.

7.5.4.1 Factor of safety and risk assessments

Factor of safety calculations, probability of failure and acceptable risk categories are all additional methods of determining whether the support system will be suitable and acceptable.

Factors of safety and probability of failure are discussed in Chapter 2 of 'Support of Underground Excavations in Hard Rock'. A factor of safety is equal to the strength of a system divided by the load of the system;

For example,

$$\text{Factor of safety} = \frac{\text{Strength of bolt}}{\text{Load required to just prevent wedge from falling}}$$

and

$$\text{Factor of safety} = \frac{\text{Strength of Pillar}}{\text{Stress on Pillar}}$$

The probability of failure is based on the distribution of strengths and loads e.g. pull test results and historical data on the height/weight of all observed falls of ground. These two distributions are then used to build a distribution for the factor of safety (reference page 17 of 'Support of Underground Excavations in Hard Rock'). From this type of distribution the percentage of samples with a factor of safety of less than 1 can be calculated, hence the probability of failure.

Risk assessment is a subject on its own, involving the building of matrices relating the probability of an event occurring to the effect and cost if the event occurs.

8.1 STOPING

In addition to the support and reinforcement elements, previously mentioned for use in development, there are support systems suitable for use only in certain stoping operations. These systems include pillars and backfill. Cable bolts and hydraulic props are also discussed in greater detail.

Stoping excavations are large compared to most development excavations but design of support systems follows the same format, rules and procedures.

8.1.1 Why

In addition to the requirement to enable safe access to orebodies, there is also a requirement to limit dilution and allow flexibility with regards to mining method.

Stoping excavations are generally larger, in at least one dimension, than development excavations. Short support elements and retaining methods such as mesh and shotcrete, are only applicable in relatively small sections of stopes. The importance of longer elements, such as long cable bolt reinforcing, regional support in the form of pillars and backfill, increases with the size of stopes and total area/volume mined.

The design stage of stoping, including the location of access development, the choice of mining method and mining equipment and optimisation of the extraction sequence all become increasingly important due to the difficulty in supporting these large excavations. The option of using remote access equipment has increased the safety aspect of some mining methods, but the support and reinforcement design must still be weighed against the cost of equipment damage, the cost of dilution and the cost of lost production due to collapse.

8.2.1 Support Types

In addition to the standard support and reinforcement items used in development, stoping operations also utilise pillars and backfill. These, together with additional comments on hydraulic props and cables are discussed below.

8.2.1.1 Tendon, steel, timber and concrete support

The list of support included in the previous chapter includes all support and reinforcement items which could be used in stoping operations.

8.2.2.1 Pillars

Pillars can be considered as a passive support system, designed to control or limit movement in the hangingwall, footwall or orebody (crown or sill pillars) of stoping areas.

Pillars can function to stabilise hangingwalls, footwalls, crowns, weak structures such as faults, and to protect underground infrastructure such as shafts, development traversing orebodies, and surface infrastructure. Regional pillars can also function to separate the effects of one mining region on another. The pillar design formulae described below are applicable for all pillars, but this section is aimed at internal stope stability pillars for hangingwall and footwall stabilisation. One of the functions of pillars in stoping environments is to reduce the hydraulic radii of hangingwalls and crowns. Reference should be made to the attached section of 'Cablebolting in Underground Mines' to the application of the hydraulic radius method.

A few standard empirical formulae are used to estimate pillar strengths. The pillar strength is then related to the expected pillar stresses and factor of safety determined. The majority of pillar design work has been conducted for coal mines, where room and pillar mining is used on a large scale. These formulae consider the internal pillar strength, not the possibility of punching into the footwall or hangingwall. When the orebody is stronger than the footwall or hangingwall, and small, high stress pillars are planned, the possibility of pillar punching should be considered.

An example of a rule of thumb for pillar stability is a critical stress level of 2.5 times the UCS for regional pillars in deep South African gold mines, with a level of 3.5 capable of causing foundation failure. Another rule of thumb is that pillar with width to height ratios of less than 2 will yield, of over 5 will be stable and from 2 to 5 require strength calculations.

Pillar stress levels can be calculated from tributary area calculations for large areas, or from numerical simulations using 2-Dimensional, e.g. *FLAC*, *UDEC*, *EXAMINE*, *PHASE2*, or, preferably, 3-Dimensional software e.g. *MAP3D*, *EXAMINE3D*, *FLAC3D*, *3DEC*, *BESOL*, *MINSOL*. Note that this list of programs only includes examples, and does not include all available software. The methods of determining applicable stress levels from the results of numerical analyses depends on the program, element size (if applicable) and whether 'softer' material properties have been used on the pillar peripheries. Elastic models for example will generally predict higher stress levels on the pillar edge than experienced in real life due to the code not taking into consideration varying levels of confining stresses.

Examples of pillar formulae are as follows. This is not a complete list of available formulae but is the author's preferred list. For width to height ratios of over 5 the squat pillar formula should be used and where the width to height ratio exceeds 10 the pillars can be assumed indestructible. These assumptions and formula are generalisations and all aspects of local conditions should also be considered when designing pillars. Stiff pillars incorrectly designed can fail catastrophically.

Salamon

Salamon and Munro’s study of coal mining pillar behaviour in the 1960s concluded with an empirical formula for pillar strength.

$$Q = 7.2\frac{W^{0.46}}{H^{0.66}}$$

Where Q = Pillar Strength (MPa)
W = Pillar width (m)
and H = Pillar Height (m)

This formula was derived for the square pillars typically used in coal mines. The recommended design factor of safety is 1.6 for standard conditions, but can range from 1.5 for better than average conditions to 1.7 for unfavourable conditions, e.g. internal joint sets. These factors of safety were derived from statistical analyses of pillar performance covering a large number of world-wide coal pillars. Comments from the study regarding superimposition of pillars concluded that superimposition was not necessary where the inter-orebody spacing was greater than twice the bord (room) width and/or if the inter-orebody spacing was greater than 0.75 times the centre to centre pillar spacing. These generalised guidelines can be inapplicable for very high and/or deviatoric stress fields.

This formula and the recommended factors of safety are probably as good a starting point as any for the design of pillars.

Hedley and Grant

Hedley and Grant determined their pillar strength formula from studies of pillar behaviour at the Elliot Lake uranium mines in Canada. These pillars were essentially rib pillars and the formula is detailed below.

$$Q = 133\frac{W^{0.5}}{H^{0.75}}$$

Note that the factor 133 is site specific and dependent on rock mass strength, with the mean orebody strength (UCS) from the Elliot Lake study 75MPa.

The factors of safety calculated during an Elliot Lake study were;

1.1 for unstable rib pillars (trackless mining)
1.3 for stable rib pillars (airleg mining)
1.7 for stable crown pillars
and 2.1 for stable sill pillars

Stacey and Page

Stacey and Page refer to the Salamon and Hedley pillar formulae and also a squat pillar formula for pillars with width to height ratios of 4.5 or greater. At these ratios the pillar cores tend to be less affected by stress increases and as the width to height ratio increases above 10 the pillars become basically indestructible.

A simplified version of the squat pillar formula is as follows;

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

Where P_s = Pillar strength

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

$$V = W^2_{eff}.H$$

$$k = \text{Design Rock Mass Strength (MPa)}$$

and $$W_{eff} = \frac{4 \text{ x pillar area}}{\text{Pillar perimeter}}$$

This simplified formula is applicable for W:H ratios greater than 5.

Considerable work has been undertaken throughout the world on pillar design, including a study by Potvin, Hudyma and Millar (1988). A stope pillar stability graph from the referenced study is attached (Figure 6). More recent formulae are being introduced following continued research, including a study by Lunder, Pakalnis and Vongpaisal, and these require various stress indications from simulation.

8.2.3.1 Backfill

Backfill can also be considered as a passive support system, used to provide a more regional resistance to movement in the hangingwall, footwall and orebody rock masses of stoping areas.

The regeneration of stress levels in backfill is generally below those required to take meaningful loads away from adjacent blocks of intact rock on a local scale, e.g. pillars in an open stope layout. The load due to the height of fill is also minimal compared to in-situ and mining induced stress levels, e.g. 0.8 MPa maximum for a 40m high stope.

Backfill can prevent, or limit, large scale caving type failures and, with cement, can be used to control inelastic rock movement adjacent to stopes. This is important in open stoping environments where backfill will act to limit the open spans in the hangingwall and/or crown.

8.2.4.1 Hydraulic props

Hydraulic props are only used in narrow, generally shallow dipping deposits where large or rapid closure rates are or can be experienced. South African gold and platinum mines and longwall coal mines are examples.

In coal mines, the rock is weak enough to mine with mechanised cutting and shearing equipment, with straight faces. This facilitates automated, remote movement of props as the face advances. In hard rock mines the face of narrow deposits is advanced using drill and blast mining and the subsequent irregular face shapes and hangingwall and footwall surfaces lead to a requirement for manual movement of props. This is labour intensive and would only be applicable, in Australia, where highly stressed, high grade remnants were being mined. Using props during mining of old pillars is an example.

Hydraulic props used in South Africa are typically of 400kN and 200kN capacity, with 1600kN units for specialist applications. Headboards, footboards and/or load spreaders are used to provide greater areal coverage between props. 400kN props would be pumped to 200kN for example with 150mm minimum yield remaining, with slow or rapid release, up to 3m/s.

In summary these units are high load bearing capacity, re-useable, blast and rockburst proof, but expensive, with high labour requirements and only suitable for narrow deposits.

8.2.5.1 Cable bolts

Cable bolts are flexible tendons composed of multi-wire strand, capable of being installed as reinforcing elements relatively deep into the rock surrounding existing or proposed excavations. This reinforcement is typically required in larger excavations, such as stopes, where other, rigid, tendons are of insufficient length. Cable bolts are discussed in ‘Support of Excavations in Hard Rock’ and in detail in ‘Cablebolting in Underground Mines’.

Cablebolt reinforcing design can be undertaken using rules of thumb, analytical methods (eg wedges analyses such as *UNWEDGE*), numerical analyses (eg *PHASE^2*) and empirical methods. The rules of thumb are as for development support design, with the length of reinforcing elements half the stope span, and the spacing between elements half the length. This is very simplified and spacing between the bolts is overestimated.

Analytical methods (including *UNWEDGE*) are similar to development support design. Numerical methods such as *PHASE^2*, *FLAC*, *CSTRESS* and *CABLEBND* are also similar to development support design. Empirical methods, as add-ons to the Q system, are also available, and are discussed in the support design section.

Two major additional factors to consider during the design of cable bolt reinforcing are the drilling and grouting equipment capabilities and the cost of reinforcing versus the cost of probable dilution without reinforcing. These factors will probably place limits on the practical length and density of cable bolt reinforcing.

8.3.1 Support Design

8.3.1.1 Rules of thumb

As explained in the development section some of the most basic methods of design, used for many years and still good for an initial design estimate, are the rules of thumb indicating the length and spacing of support element relative to excavation size.

An adjustment to the rule of thumb used for development is that the length of the reinforcing element should not be less than half the width of the excavation and the spacing between the bolts should not be less than half the bolt length. For example; a 10m x 10m stope should have 5m bolts on a 2.5m spacing. This is very basic, but can form the basis of a design, for fine tuning with *UNWEDGE* and/or *PHASE2* etc.

8.3.2.1 Rock mass classification methods

Barton's 'Q', Bieniawski's RMR and Laubscher's MRMR rock mass classification systems can be expanded to design the length and spacing of bolts for stoping. This is, however, probably over-extending the initial intended applicability of the methods and should be used only as an initial guideline.

The Q system can be extended to cover the design of stope stability and cable reinforcement design. The Mathews and Potvin stability methods are discussed in the relevant section from 'Cablebolting in Underground Mines' by Hutchinson and Diederichs, 1996, which is attached.

This empirical methodology is discussed briefly below. The first step is to access the parameters for calculation of Q,

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

and then to calculate Q',

where
$$Q' = \frac{RQD \times J_r}{J_n \times J_a}$$

The J_W and SRF factors are omitted, with the stress effects being used at a later stage.

The next stage is to determine N',

where
$$N' = Q' xAxBxC$$

and **A** = Rock Stress Factor

B = Joint Orientation Factor

C = Gravity Adjustment Factor

The method of calculating these factors is more fully described in Hutchinson and Diederichs, 1996. The next step is to calculate the hydraulic radius of the stope surface under investigation, ie the hangingwall or crown,

where,
$$HR = FaceArea / Perimeter$$

The results of the N' and HR calculations are plotted against each other to assess the probable stability of the surface, as indicated on Figure 7. The calculations for Q' and N' are typically undertaken for the worst case, best case and expected values. HR calculations are determined for realistic stope dimensions (strike length, orebody width, and stope height). This plot indicates the expected stability of the stope surface related to worldwide case studies. The same values can then be plotted on empirical stope cable bolt design graphs for single and twin cable bolt spacings and cable bolt lengths (see attached section). Another example, from Hoek et al, 1995, is also attached, as Figure 8, for cablebolt designs related to the ratio RQD/J_n:HR.

8.3.3.1 Numerical methods

The methods described in the development section are also applicable to stoping, particularly *UNWEDGE* and *PHASE2.* Stoping excavations are generally large, however, and the programs should be capable of modelling longer rock reinforcing elements such as cable bolts. The programs *CSTRESS* and *CABLEBND* can also be used for more detailed performance analyses (reference Cablebolt reinforcement section of 'Support of Underground Excavations in Hard Rock').

Rules of thumb and initial guidelines from Rock Mass Classification methods can be a good starting point for assessing alternative support and reinforcement designs.

8.3.4.1 Holistic, site specific

As discussed in the development support section, sole use of one of the previously discussed methods could produce a support and/or reinforcing system which does not take into account all major factors. Ideally the support system should be designed taking account **all** of the following factors;

Rock Mass Properties
Excavation Size and Shape
Planned Life and Function of Excavation
Excavation Orientation
Previous Support Performance
Legislated Guidelines
Corporate Requirements
Contractual Limitations
Stress
Rock Mass Deformation
Seismicity
Groundwater (flow rates, pressures and corrosiveness)
Excavation Method
Availability and Cost
Air Quality

These issues were discussed in the development section. In addition, the grade of the orebody can determine the finances available for support and reinforcement, especially when dilution is considered. The cost of loading and transporting waste or low grade dilution and then treating it can work out very expensive. Prevention of such dilution using efficient support and reinforcement could easily pay for itself. The funds available for such dilution control depend on many site specific factors, such as blasting costs, loading costs, haulage costs, hoisting costs, treatment plant recoveries and the extent to which waste material fed into the treatment plant has to be treated, including possible tailings disposal.

One of the major differences between development and stoping is that many stoping areas have a working life less than that of development. In many cases time dependant rock failure is not critical after the ore has been mined, and if it is critical the stope can be filled.

8.4.1 Support Effectiveness

Monitoring of stope support and reinforcing effectiveness can be conducted as per development support plus there is the additional scope to measure the overbreak using survey instrumentation.

8.4.1.1 Stope surveys

Surveying of stopes using remote, laser based equipment such as the Optech Cavity Monitoring System (CMS) enables the effectiveness of stope reinforcing and support to be assessed. Stope boundaries and profiles can be related to planned mining outlines, geological boundaries, crown support and cable bolt reinforcing. As indicated in the example in Figure 9, the effectiveness of cable bolt reinforcing could be assessed by this method - there is an unreinforced central area of the crown which collapsed, there are shallow dipping cables which did not perform and there are hangingwall cables which performed satisfactorily. There is also an area in the footwall which collapsed but which might not be reinforceable with available access.

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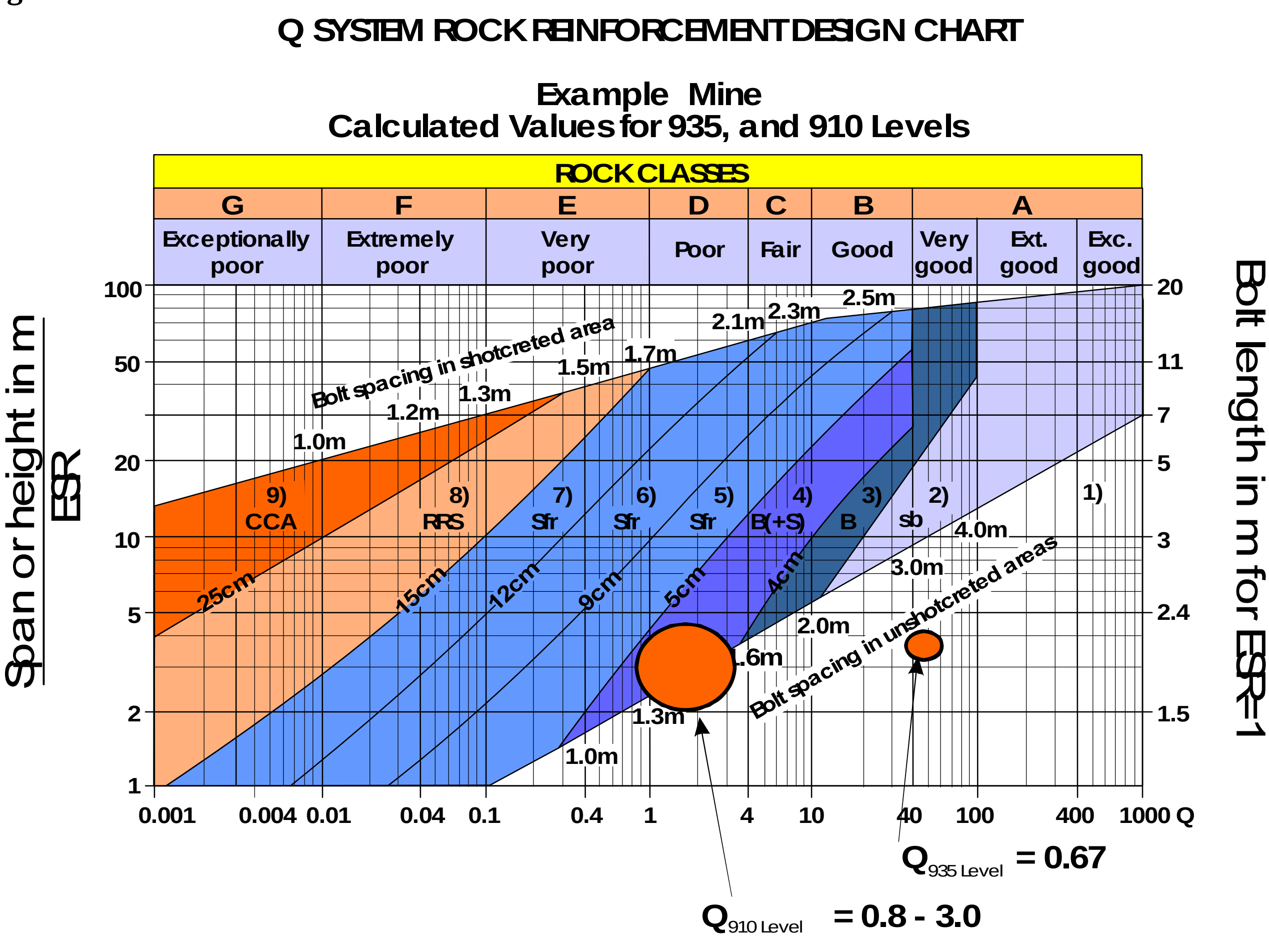
Figure 1



$$\text{Rock mass quality } Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$$

REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot Bolting, sb
3) Systematic Bolting, B
4) Systematic bolting (and unreinforced shotcrete, 4-5cm), B (+S)
5) Fibre reinforced shotcrete and bolting, 5-9cm, Sfr +B
6) Fibre reinforced shotcrete and bolting, 9-12cm, Sfr + B
7) Fibre reinforced shotcrete and bolting, 12-15cm, Sfr +B
8) Fibre reinforced shotcrete >15cm, reinforced ribs of shotcrete and bolting, Sfr, RRS+B
9) Cast Concrete lining, CCA

(after Grimstad et al., 1993)

Figure 2

MRMR Design Table

TABLE IX

SUPPORT* PRESSURE FOR DECREASING MRMR

	RMR									
MRMR	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
	— Rock reinforcement—plastic deformation —									
1A										
1B										
2A										
2B	a	a								
3A	b	b	a	a						
3B	b	b	b	b	b	c				
4A	r	r	c	c	c	d	d			
4B				d	e	f	f	c+l		
5A						f/p	h+f/p	h+f/l	h+f/l	
5B							h+f/p	f/p	t	t

* The codes for the various support techniques are given in Table X.

After Laubscher, 1990

Figure 3

MRMR Support Reference Table

TABLE X
SUPPORT TECHNIQUES

Rock reinforcement

- a Local bolting at joint intersections
- b Bolts at 1 m spacing
- c b and straps and mesh if rock is finely jointed
- d b and mesh/steel-fibre reinforced shotcrete bolts as lateral restraint
- e d and straps in contact with or shotcreted in
- f e and cable bolts as reinforcing and lateral restraint
- g f and pinning
- h Spilling
- i Grouting

Rigid lining

- j Timber
- k Rigid steel sets
- l Massive concrete
- m k and concrete

(j–m: Low deformation)

- n Structurally reinforced concrete

Yielding lining, repair technique, high deformation

- o Yielding steel arches
- p Yielding steel arches set in concrete or shotcrete

Fill

- q Fill

Spalling control

- r Bolts and rope-laced mesh

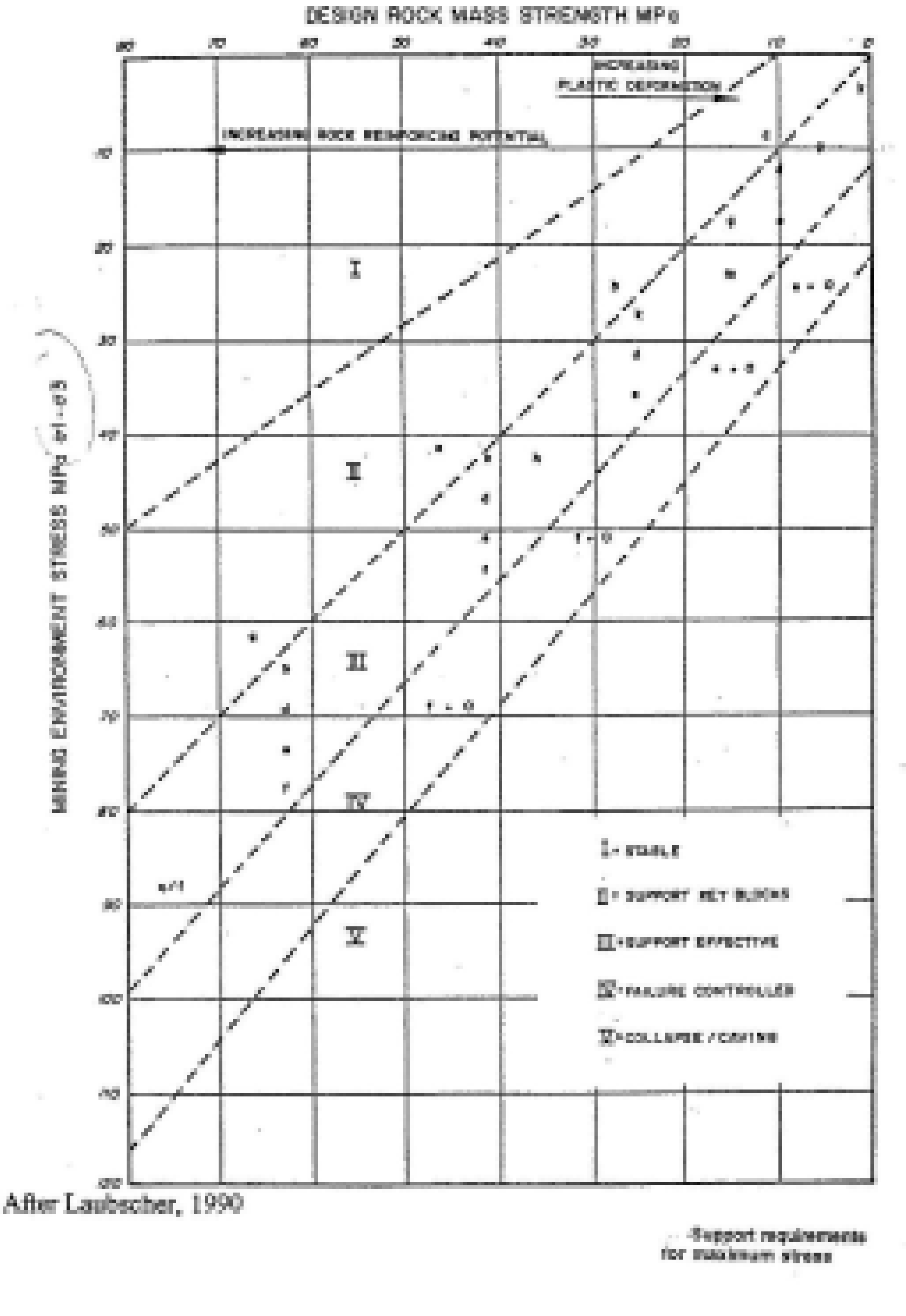
Rock replacement

- s Rock replaced by stronger material
- t Development avoided if possible

After Laubscher, 1990

Figure 4

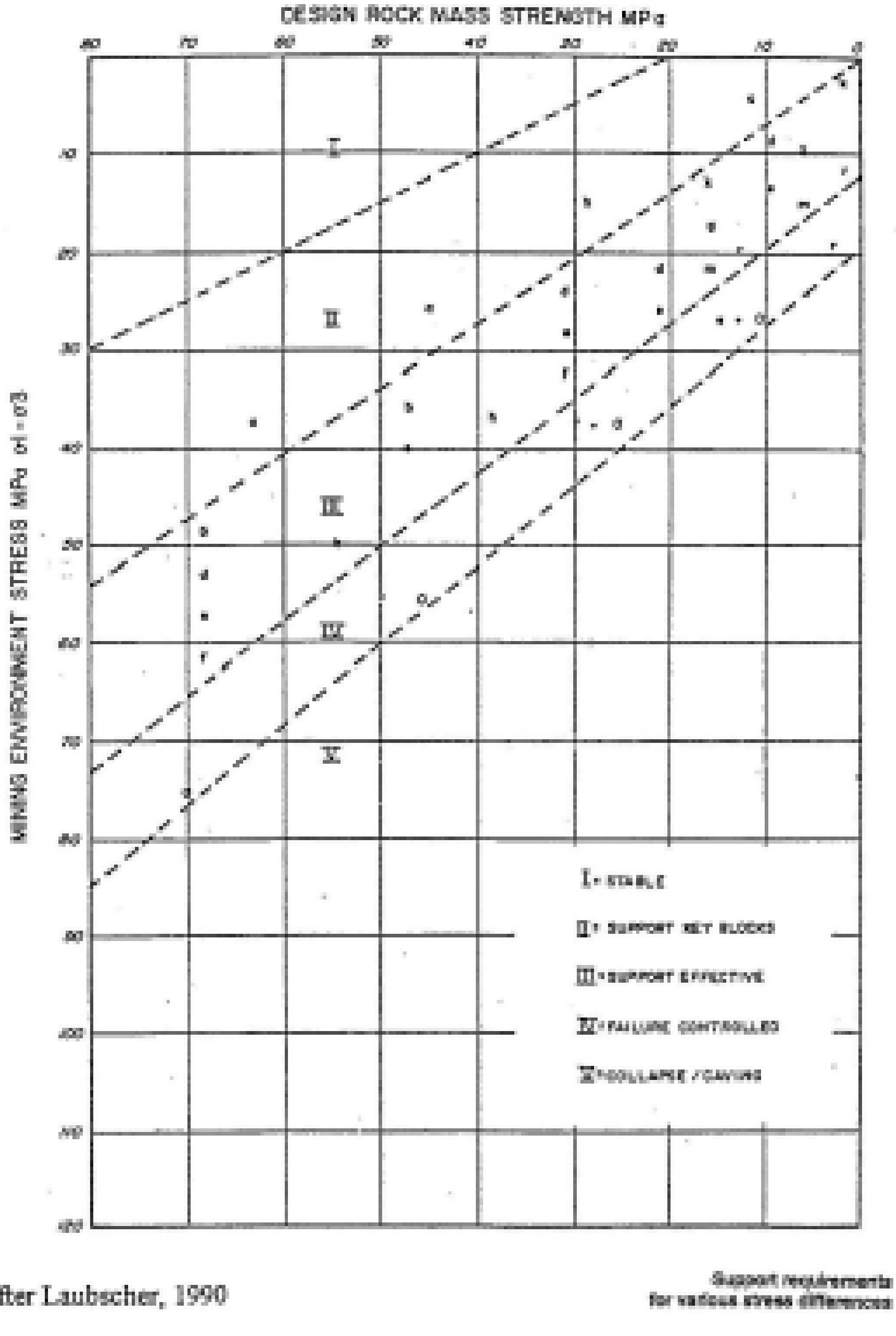
DRMS Maximum Stress Support Graph



After Laubscher, 1990

Figure 5

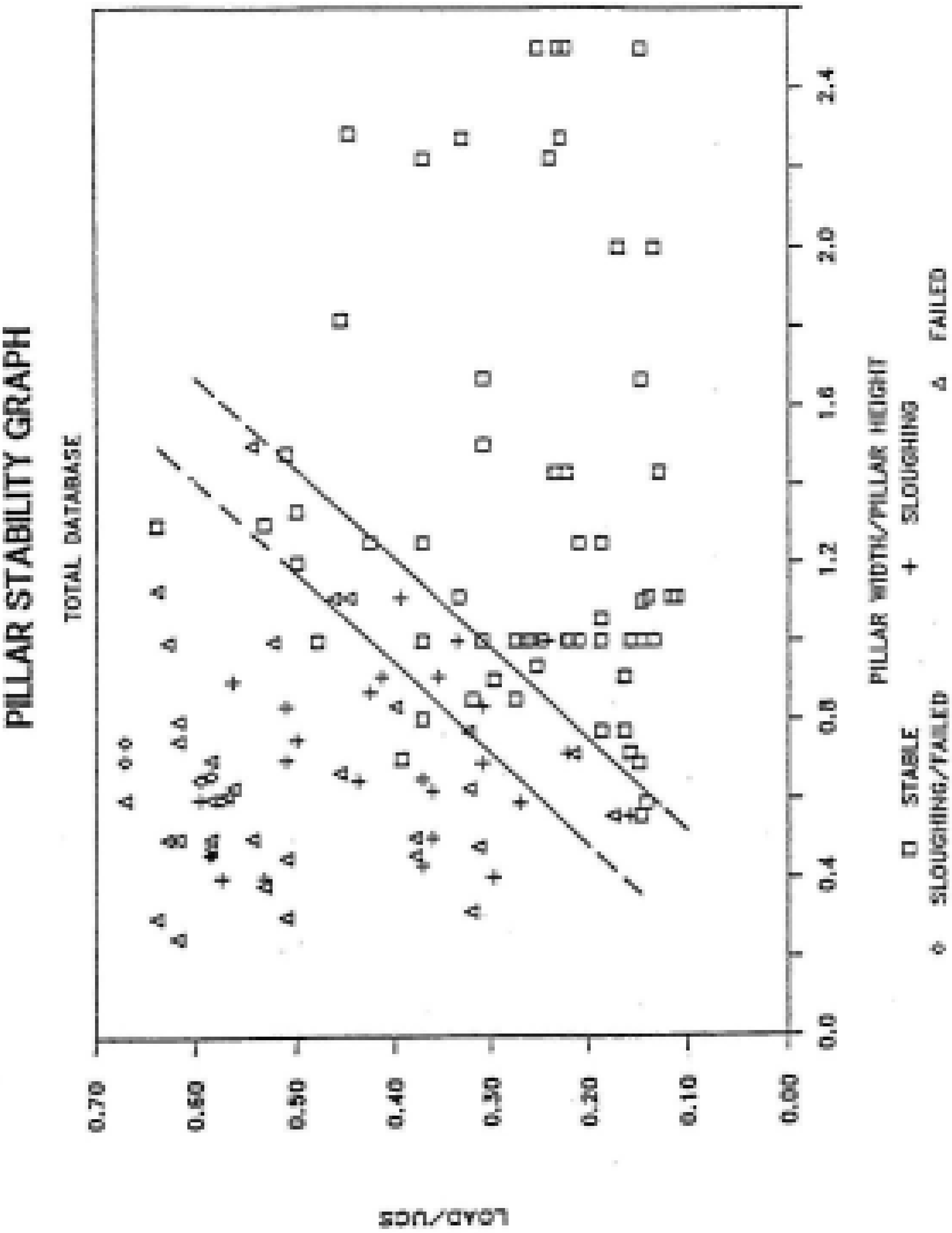
DRMS Mining Environment Stress Support Graph



After Laubscher, 1990

Figure 6

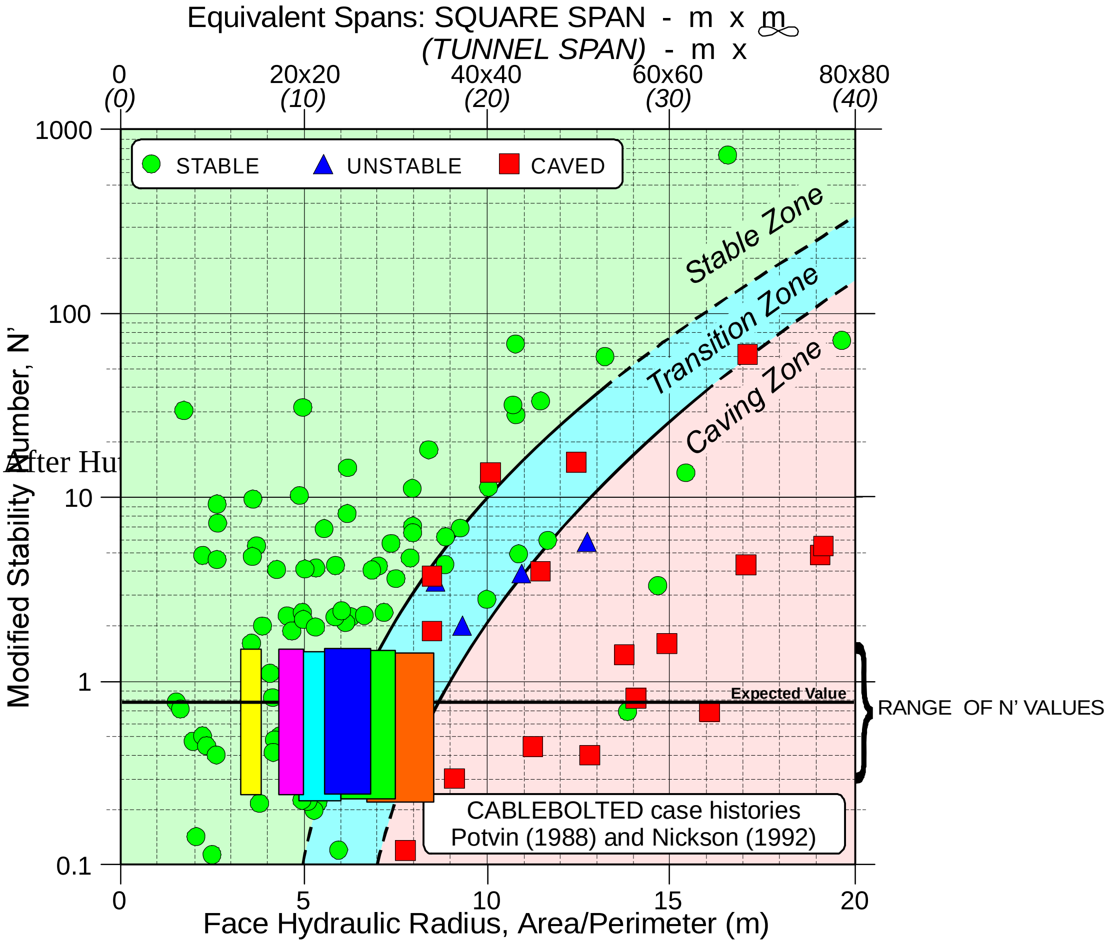
Pillar Stability Graph



After Potvin et al, 1988

Figure 7

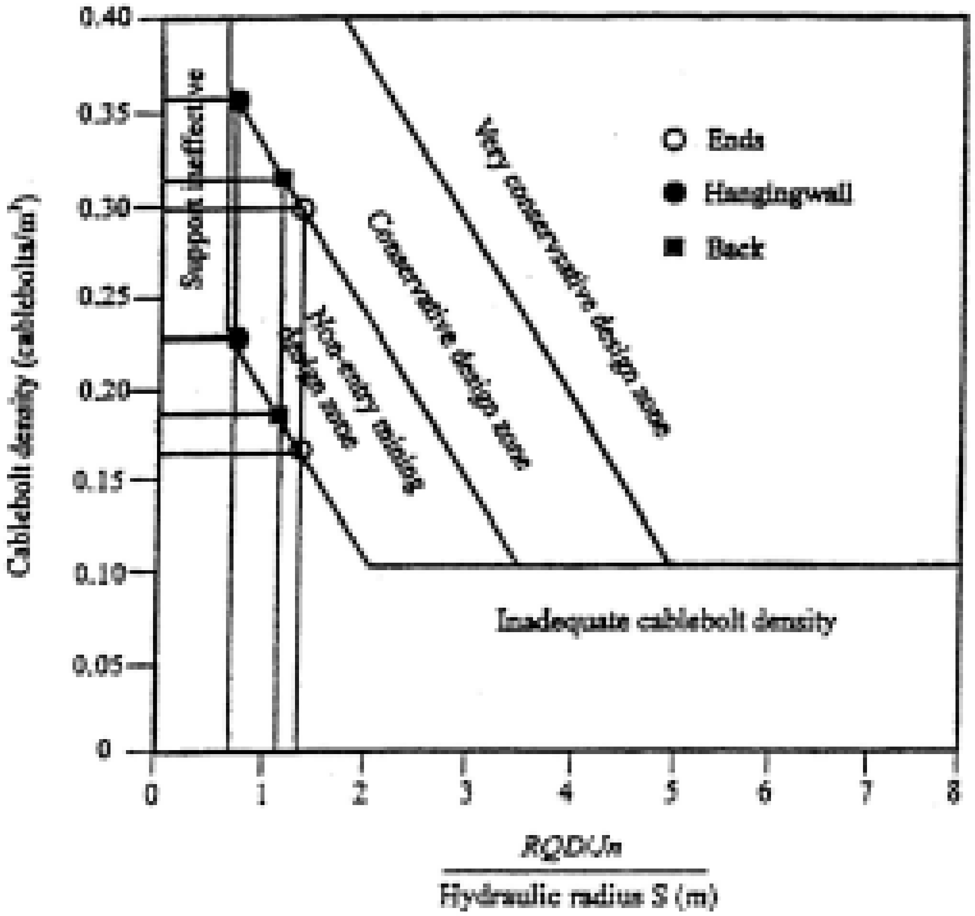
Example Stope Stability Graph



Calculated Stability of Cablebolted Crowns of
(10, 15, 20,25, 30 and 40m Strike Lengths, 20-30m Widths)

Figure 8

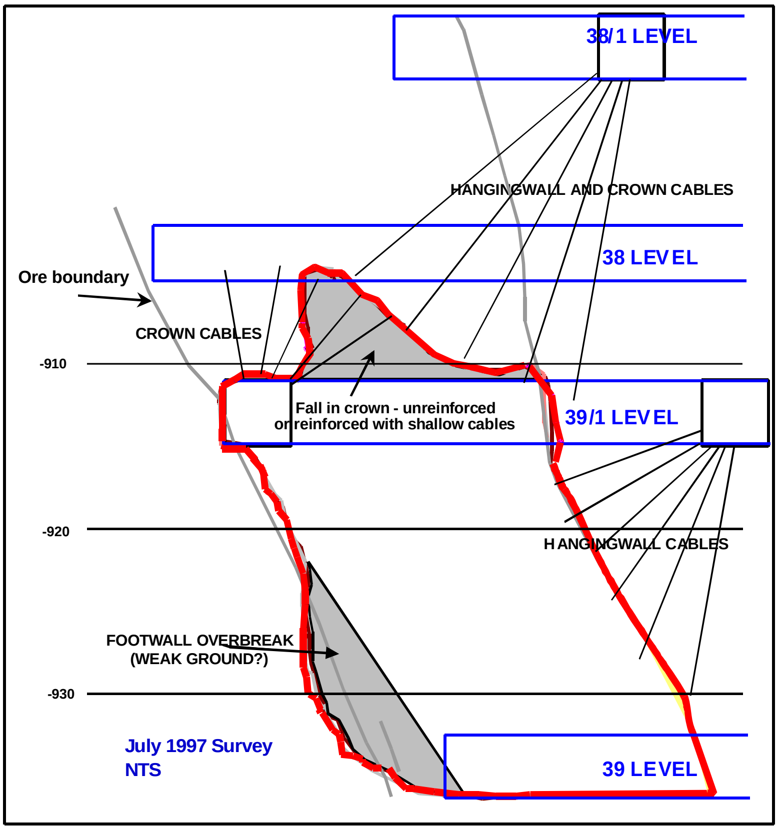
Hangingwall Cablebolt Design Graph



After Hoek et al, 1995

Figure 9

Results of interpretation into Optech Cavity Monitoring System (CMS) survey.



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Various extracted tables and diagrams for;

Support Function (Ref: Hoek, Kaiser and Bawden, 1997)
Types of Friction Stabilisers
Support Selection (Ref: Budavari, 1983)
Support Resistance Calculation (Ref: COMRO, 1988)
Support Resistance/Pressure Examples (Ref: Hoek, Kaiser and Bawden, 1997)
Support Pressure Examples (Ref: Hoek and Brown, 1980)
Support Characteristics Under Dynamic Loading (Ref: Hedley 1992)
Support Requirements for Rockburst Conditions (Ref: Hedley 1992)

GUIDE TO THE SELECTION OF SUPPORT FOR TUNNELS

Rock type		Hard rock - UCS greater than 175 MPa		Soft rock - UCS less than 100 MPa	
Field Stress Intensity		**(a) Massive (spacing between joints ≥ width of excavation)**	**(b) Jointed, laminated, blocky (spacing between joints $\leq {}^{1}/_{10}$ width of excavation)**	**(a) Massive (definition as for hard rock)**	**(b) Jointed, laminated, blocky (definition as for hard rock)**
(A) LOW STRESS (Vertical Component less than 30 MPa)	(i) Increasing	Failure of corners and sides possible if tunnel is badly shaped, therefore if increase is considerable: *Rockbolting - types 1,3 or 4. **Fabric - types (a) or (b) on sidewalls only.	Rockbolting - closely spaced small diameter bolting of types 1 or 4, throughout. Fabric - on sidewalls only, type (b) or (c) or (d) for very closely-jointed rock.	Anticipate failure of sidewalls even if tunnel is well shaped. Rockbolting of types 1, 3 or 4 in roof and 2, 3 or 4 with fabric (a) or (b) in sidewalls.	Mass concrete. Immediate application of shotcrete might be sufficient to provide temporary support during excavation.
	(ii) Static	No support.	Shotcrete alone should provide adequate support. Alternatively, rockbolting of small diameter type 1 or 4 closely spaced in roof only. 'Rams-horn' caps, closely cribbed to roof also acceptable. Traditional concrete lining methods acceptable.	No support required if machine cut or smooth-blasted to appropriate shape. Shotcrete to prevent weathering if necessary.	Rockbolting of roof with small diameter bolts of type 1 or reinforcement of type 3, if possible. Alternatively, 8 cm thick shotcrete by itself might be adequate.
	(iii) Decreasing	No support.	Shaped with respect to prominent bedding planes. Shotcrete alone should be quite adequate. Alternatives as in static case above. Traditional concrete lining methods acceptable.	As with static case above.	As in static case above.

GUIDE TO THE SELECTION OF SUPPORT FOR TUNNELS

Rock type		Hard rock - UCS greater than 175 MPa		Soft rock - UCS less than 100 MPa	
Field Stress Intensity		(a) Massive (spacing between joints ≥ width of excavation)	(b) Jointed, laminated, blocky (spacing between joints ≤ $^1/_{10}$ width of excavation)	(a) Massive (definition as for hard rock)	(b) Jointed, laminated, blocky (definition as for hard rock)
(B) HIGH STRESS (Vertical component greater than 60 MPa)	(i) Increasing	Shape and smooth-blast if possible. Moderate stress increase: rock reinforcement of type 4 with fabric (b) or (c). Considerable increase: intensive bolting of type 2 or possibly type 3 with fabric (d) or (e). Conventional bolts (1) should not be used. Mass concrete in any form is to be avoided.	Smooth-blast and shape with respect to bedding planes with curved sidewalls and arched roof in cross-cuts. Support as for massive rock with increased density of possible smaller diameter bolts, and fabric (c), (d) or (e) throughout.	Conventional arches, even when lagged and waste-packed are inadequate. Shotcrete applied immediately behind advancing face may delay sidewall fracture and permit effective, intensive bolting of type 2 or 3 and fabric (d) or (e). Bolts of type 1, 3 or 4 and fabric (b) or (c) for roof. Mass or cast concrete in any form is to be avoided.	Spiling, steel sets and concrete. Alternative suggestions: temporary support during development as for static case below, followed by close-spaced bolting of types 2, 3 or 4 and fabric (d). Alternatively, yielding arch or circular arch well-studded, lagged and waste-packed.
	(ii) Static	Shape and smooth-blast. Rockbolting - types 1, 3 or 4. Fabric - types (a), (b) or (c) on sidewalls only, alternatively shotcrete without mesh is adequate. Concrete-monolithic lining acceptable if required for other purposes.	Shape with respect to bedding planes and smooth blast. Bolting of types 1, 3 or 4 more closely spaced depending on joint spacing. Fabric (b) or preferably (c) on sidewalls only. Shotcrete by itself, 8 cm thick should be adequate.	As in case above except that type 2 bolts not necessary and 3 and 4 quite acceptable. Fabric of type (c) adequate for sides. Traditional forms of concrete lining acceptable.	Pre-cementation and circular steel arches with concrete. Reinforcing cage of grouted rods followed by pipe and rail sets. Alternative suggestions: immediate application of shotcrete as in low stress with rockbolting 1, 3 or 4 and fabric (d).
	(iii) Decreasing	Same as for static case except concrete lining not recommended whether reinforced or monolithic.	Same as in massive case except increased spacing of smaller diameter bolts may be necessary to suit jointing. Shotcrete applied over mesh may crack in roof but should be unaffected in sidewalls. Traditional forms of concrete liable to serious roof failure.	As in static case above.	As in static case above.

*Type of reinforcing
(1) conventional rockbolts (low yieldability)
(2) yielding rock-studs
(3) fully grouted rods or tubes
(4) fully grouted ropes or re-bar

**Fabric between reinforcing elements
(a) steel strapping
(b) rope lacing
(c) wire mesh
(d) wire mesh and shotcrete
(e) wire mesh plus lacing

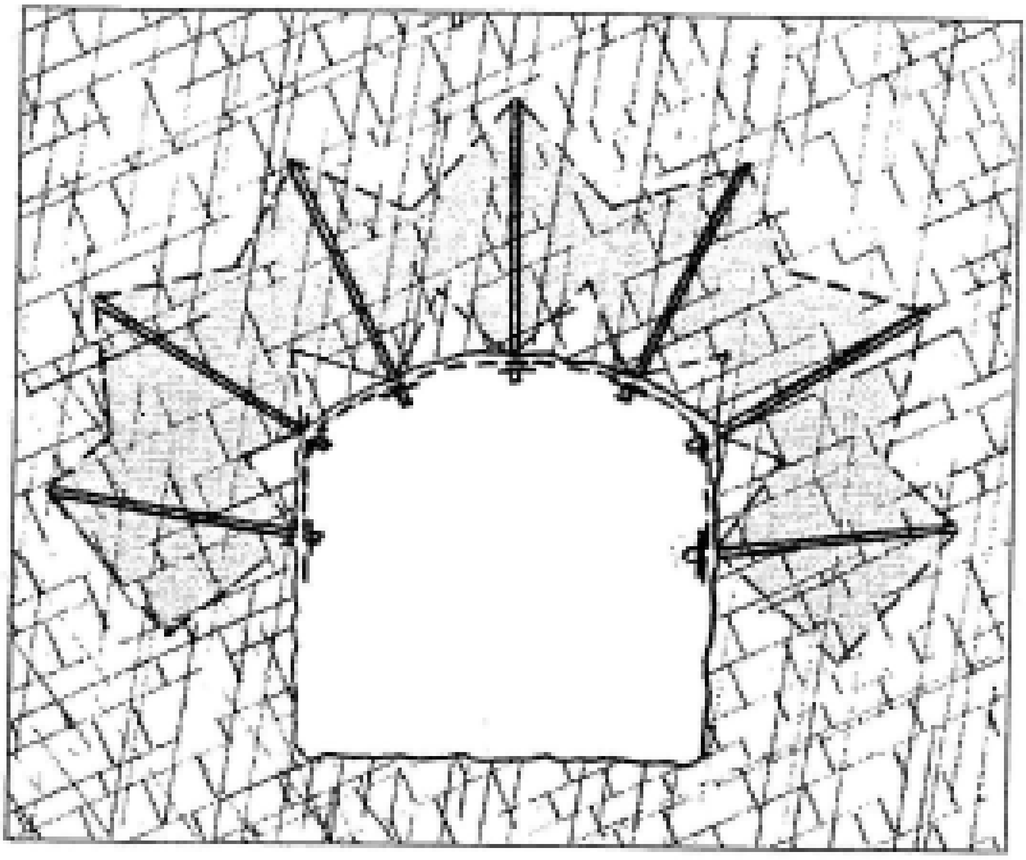


Figure 11.11: Pattern bolting for the support of heavily jointed rock which can fail by ravelling or squeezing. The shading represents a zone of compressive stress in which interlocking of individual rock pieces is retained and a self-supporting arch is created. Mesh or shotcrete should be applied to the excavation surface to retain small blocks and wedges in the stress-free zones between the rockbolts.

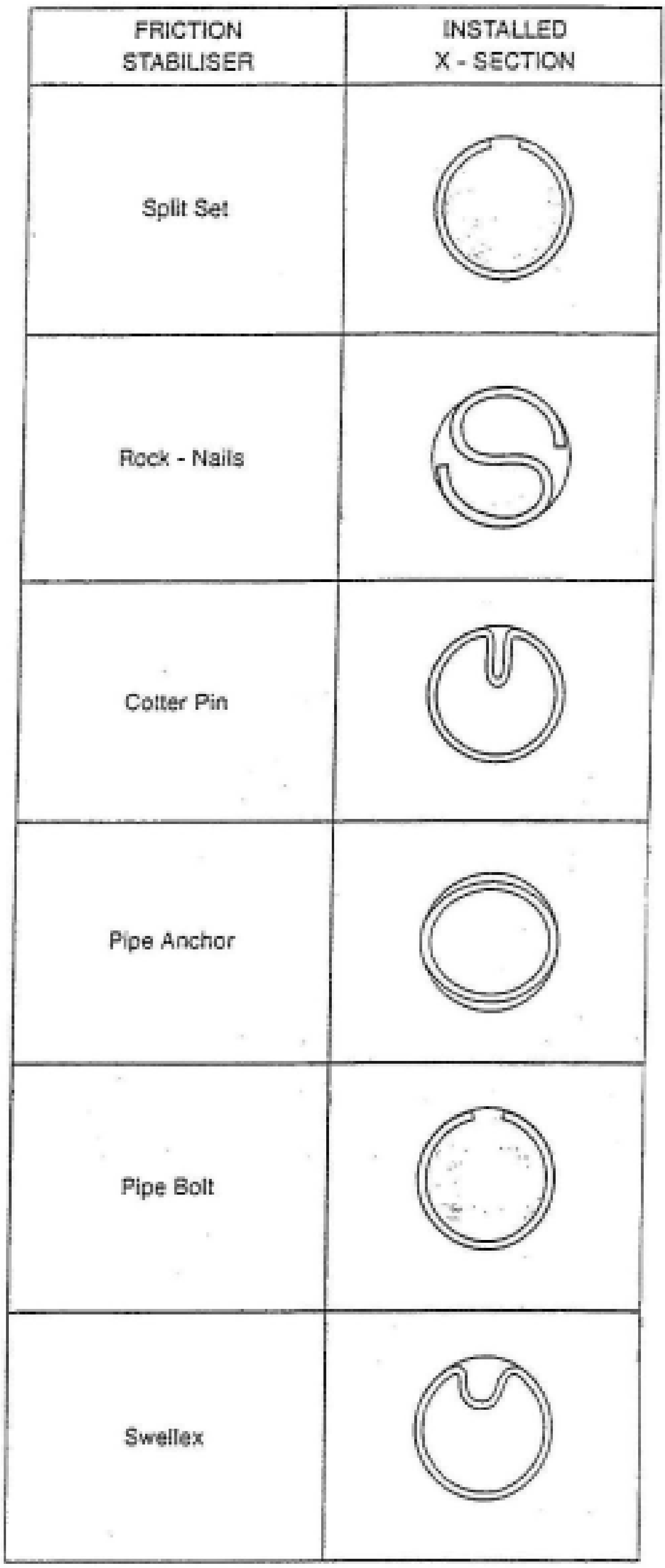
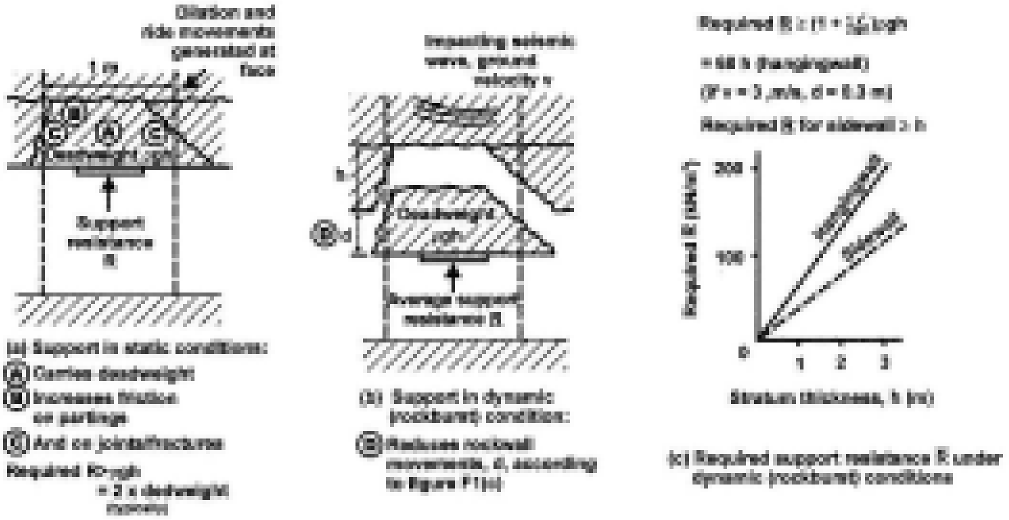


Figure 3.17 Cross sections and radial force distributions for selected friction stabilisers.



Presentation to Australian Centre for Geomechanics

FUNCTIONS OF LOCAL SUPPORT
(Ref:COMRO 1988)

AUSTRALIAN MINING CONSULTANTS

Scale: N.T.S	Drawn:	Date: Sept-98	SelectSupport.cdr

Table 9.1: Approximate support characteristics for different support systems installed in circular tunnels of various diameters.

Support type	Tunnel diameter - m	4	6	8	10	12
Very light rockbolts [1] 16mm dia. Pullout load =0.11 MN	Maximum pressure - MPa	0.25	0.11	0.06	0.04	0.03
	Max. elastic displacement - mm	10	12	13	14	15
Light rockbolts [1] 19mm diameter Pullout load =0.18 MN	Maximum pressure - MPa	0.40	0.18	0.10	0.06	0.04
	Max. elastic displacement - mm	12	14	15	17	18
Medium rockbolts [1] 25mm diameter Pullout load = 0.27 MN	Maximum pressure - MPa	0.60	0.27	0.15	0.10	0.07
	Max. elastic displacement - mm	15	16	17	19	20
Heavy rockbolts [1] 34mm diameter Pullout load =0.35 MN	Maximum pressure - MPa	0.77	0.34	0.19	0.12	0.09
	Max. elastic displacement - mm	19	21	22	23	24
One day old shotcrete 50mm [2] UCS = 14 MPa, Ec= 8500 MPa	Maximum pressure - MPa	0.35	0.23	0.17	0.14	0.12
	Max. elastic displacement - mm	3	5	6	8	10
28 day old shotcrete 50mm [2] UCS = 35 MPa, Ec= 21000 MPa	Maximum pressure - MPa	0.86	0.58	0.43	0.35	0.29
	Max. elastic displacement - mm	3	5	6	8	9
28 day old concrete 300mm UCS = 35 MPa, Ec= 21000 MPa	Maximum pressure - MPa	4.86	3.33	2.53	2.04	1.71
	Max. elastic displacement - mm	3	4	6	7	9
Light steel sets 6I12 [3] Spaced at 1.5m, well blocked	Maximum pressure - MPa	0.33	0.18	0.12	0.08	0.06
	Max. elastic displacement - mm	7	7	8	8	9
Medium steel sets 8I23 [4] Spaced at 1.5m, well blocked	Maximum pressure - MPa	6)	0.37	0.25	0.17	0.13
	Max. elastic displacement - mm		8	9	10	10
Heavy steel sets 12W65 [5] Spaced at 1.5m, well blocked	Maximum pressure - MPa	6)	6)	0.89	0.66	0.51
	Max. elastic displacement - mm			9	11	12

Notes: [1]Rockbolts are mechanically anchored and ungrouted. Bolt length is assumed to be equal to 1/3 of the tunnel diameter and bolt spacing is one half bolt length. [2]Values apply to a completely closed shotcrete ring. For a shotcrete lining applied to the roof and sidewalls only, the maximum support pressure is at least an order of magnitude lower. [3]6 inch deep I beam weighing 12 lb per foot. [4]8 inch deep I beam weighing 23 lb per foot. [5]12 inch deep wide flange I beam weighing 65 lb per foot. [6]The minimum radius to which I beams can be bent on site is approximately 11 times the section depth. In the case of wide flange beams the minimum radius is approximately 14 times the section depth.

TABLE 17 - MAXIMUM SUPPORT PRESSURES FOR VARIOUS SYSTEMS.

Support system | *Tunnel radius*	r_i 1m 39in	r_i 2.5m 98in	r_i 5m 197in	r_i 10m 394in
A - SHOTCRETE - 5cm (0.05m)/ 2 inches thick shotcrete. $\sigma_{c.conc.}$ = 14 MPa/2000 psi after 1 day.	p_{smax} 0.65 MPa 95 psi	p_{smax} 0.27 MPa 39 psi	p_{smax} 0.14 MP 20 psi	p_{smax} 0.07 MPa 10 psi
B - SHOTCRETE - 5cm(0.05m)/ 2 inches thick shotcrete. $\sigma_{c.conc.}$= 35 MPa/5000 psi after 28 days	1.63 MPa 236 psi	0.68 MPa 99 psi	0.34 MPa 50 psi	0.17 MPa 25 psi
C - CONCRETE - 30cm(0.30m)/ 12 inches thick concrete. $\sigma_{c.conc.}$ = 35 MPa/5000 psi after 28 days.	7.14 MPa 1036 psi	3.55 MPa 515 psi	1.93 MPa 279 psi	1.00 MPa 146 psi
D - CONCRETE - 50cm(0.50m)/ 19.5 inches thick concrete. $\sigma_{c.conc}$ = 35 MPa/5000 psi after 28 days.	9.72 MPa 1410 psi	5.35 MPa 775 psi	3.04 MPa 440 psi	1.63 MPa 236 psi
E - STEEL SETS - (6 I 12) space 2m/79 in.. Blocked 2θ=22½°, σ_{ys} = 248MPa/36 000 psi.	0.61 MPa 88 psi	0.18 MPa 27 psi	0.07 MPa 10 psi	0.02 MPa 3 psi
F - STEEL SETS - (8 I 23) space 1.5m/59 in.Blocked 2θ=22½°, σ_{ys} = 248MPa/36 000 psi.	1.59 MPa 230 psi	0.50 MPa 72 psi	0.18 MPa 27 psi	0.06 MPa 9 psi
G - STEEL SETS - (12 W 65) at 1m/39 in. Blocked 2θ=22½°, σ_{ys} = 248MPa/36 000 psi.	7.28 MPa 1055 psi	2.53 MPa 366 psi	1.04 MPa 150 psi	0.38 MPa 55 psi
H - VERY LIGHT ROCKBOLTS - 16mm/⅝in. Ø at 2.5m/98in. centres. Mechanical anchor. T_{bf} = 0.11MN/25 000 lb.	0.02 MPa 2.6 psi	0.02 MPa 2.6 psi	0.02 MPa 2.6 psi	0.02 MPa 2.6 psi
I - LIGHT ROCKBOLTS - 19mm/¾". Ø at 2.0m/79in. Mechanical anchor. T_{bf} = 0.18MN/40 000 lb.	0.045MPa 6.5 psi	0.045MPa 6.5 psi	0.045MPa 6.5 psi	0.045MPa 6.5 psi
J - MEDIUM ROCKBOLTS - 25mm/1" Ø at 1.5m/59in centres. Mechanical anchor. T_{bf} = 0.267MN/60 000 lb.	0.12 MPa 17 psi	0.12 MPa 17 psi	0.12 MPa 17 psi	0.12 MPa 17 psi
K - HEAVY ROCKBOLTS - 34mm/1⅜" at 1m/39in centres. Resin anchored. T_{bf} = 345 MN/ 150 000 lb.	0.34 MPa 49 psi	0.34 MPa 49 psi	0.34 MPa 49 psi	0.34 MPa 49 psi

Table 7.2 - Possible support characteristics under dynamic loading

Type of Support	Load Capacity kN	Stretch or Slip mm (1)	Support Resistance kN/m^2 (2)	Peak Particle Velocity m/s (3)	Energy Absorbed kJ/m^2 (2)
Mechanical bolts	120	20	83	0.46(4)	0.6(4)
Yielding bolts (5)	65	200	45	2.6	9.0
Rebar, regular	150	25	104	1.0	1.3
Rebar, smooth	130	75	70	1.6	3.4
Cable bolts	230	15	160	0.9	1.2
Split sets	50	100	35	1.6	3.5
Swellex, annealed (6)	100	50	70	1.6	3.5
Welded wire mesh (7)	30	200	20	1.2	2.1
Chain link mesh (7)	35	270	25	1.5	3.3
Steel cable, lacing	90	30	62/cable	0.8	0.9/cable

Notes: (1) 2 m long supports; (2) supports at 1.2 m (4 ft) centres;
(3) from Equations 7.15 or 7.16 using a slab thickness of 1 m;
(4) bolts installed with 40 kN tension and 10 mm stretch;
(5) Ortlepp, 1969; (6) Herron, 1988; (7) Pakalnis and Ames, 1983.

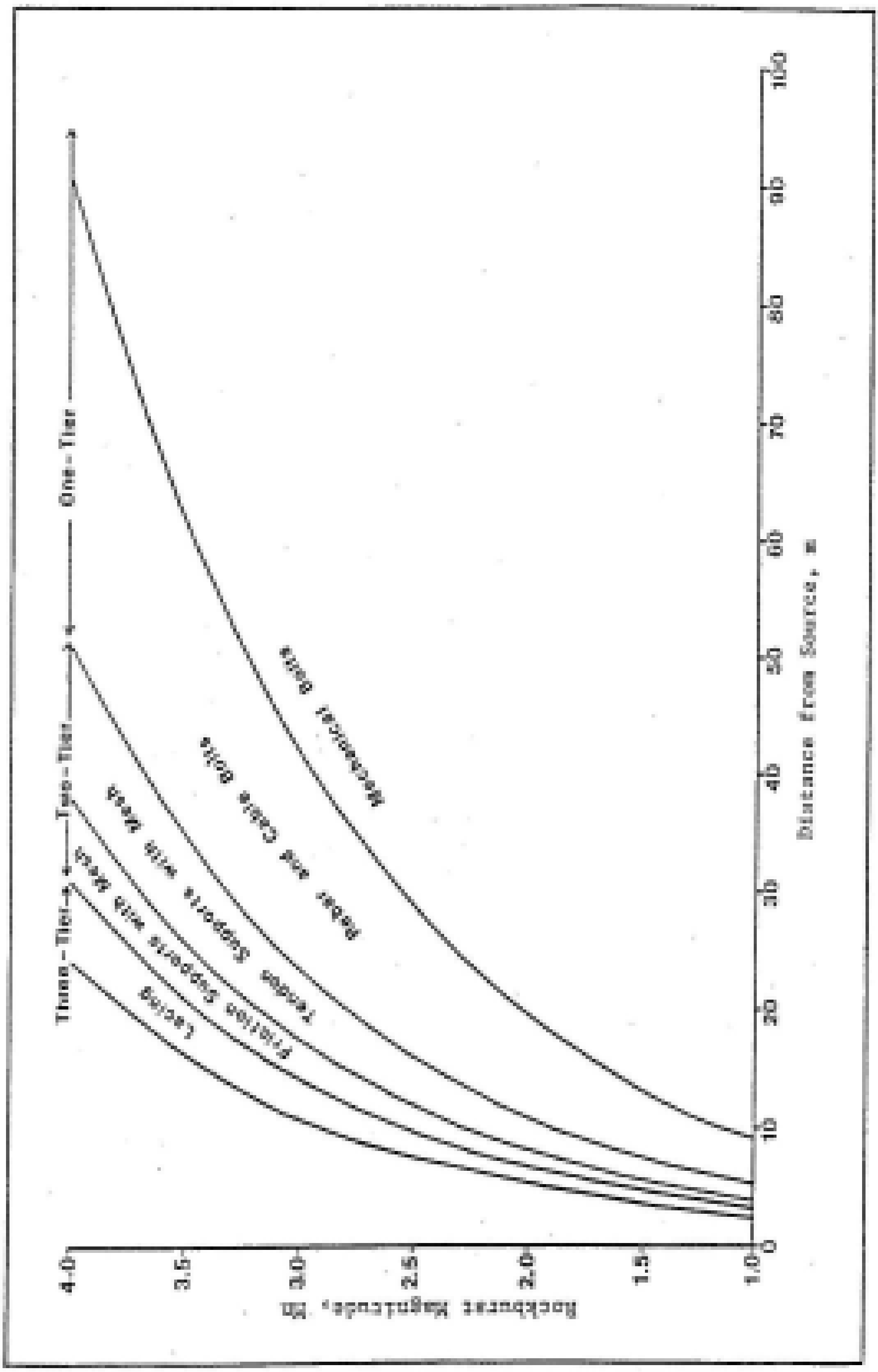


Fig. 7.13 - Conceptual support requirements for rockburst conditions.

Extract from "Cablebolting in Underground Mines'
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Hutchinson DJ and Diederichs MS

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2.17 Empirical Design of Open Stopes and Support: Mathews/Potvin Stability Graph Method

Classical empirical tools such as *RMR* and *Q* were developed from a database composed primarily of civil engineering tunnels at low to moderate depth. These tools have proven invaluable to the tunnelling engineer. The recommendations derived from these systems for dimensioning and support, however, often result in conservative designs for large temporary or non-entry mining excavations.

While these systems are appropriate for high traffic mining roadways, lunchrooms and equipment rooms where stability must be paramount, they are difficult to apply to the problem of dimensioning and support design for large open stopes. These limited access areas can be designed as temporary structures and in the case of non-entry stopes, can tolerate limited local fallout of small rock blocks provided that dilution is minimized and overall stability is maintained. These criteria permit a more economical design suitable to mining.

RMR (Bieniawski, 1989; 1993) allows for design modification based on reduced stand-up times for mining while *Q* (Barton et al., 1974) attempts to include mining applications through the use of Equivalent Support Ratio. Laubscher and Taylor (1796) modified *RMR* and introduced a classification system for caving operations and for stability of mining excavations. Readers are referred to Hoek et al. (1995) for additional discussion of these methods.

Large scale open stoping methods such as Vertical Crater Retreat, AVOCA, Longhole and Blasthole Stoping rely on the selection of a limiting stope dimension. Ideally these stopes can be designed to be self supporting. When ground conditions or the need for larger stopes mandates the use of support, cablebolting is the most logical choice and has been successfully applied. Mathews et al. (1981) proposed an empirical method for the dimensioning of open stopes based on Q' and on three factors accounting for stress, structural orientation and for gravity effects. The method is used to dimension each face of the stope separately based on a combination of these three factors and on the hydraulic radius (calculated as *surface area / perimeter*) of the face. The hydraulic radius accounts for shape as well as size of the face.

Potvin (1988) modified this original method and calibrated it using 175 case histories. Nickson (1992) added case histories and further investigated Potvin's support design guidelines. These case histories include hangingwalls, footwalls, ends and backs from a wide variety of mining environments. Other case histories can be found throughout recent literature (Bawden, 1993; Bawden et al. 1989; Greer, 1989). The method has been expanded by the authors in this handbook to provide improved support guidelines.

2.17.1 Modified Stability Number, N'

The classification of the rockmass and of the excavation problem itself is accomplished in the Modified Stability Graph Method through the use of the Modified Stability Number, *N'*, as specified by Potvin (1988), Potvin and Milne (1992) and Bawden (1993). This parameter is similar to the value *N* proposed by Mathews et al. (1981) but has different factor weightings. Canadian mines use Potvin's *N'* while at present mines in Australia, for example, use Mathews' analysis and *N*. Only *N'* (Potvin) will be considered here. This method has been referred to as the *Potvin* method, the *Mathews/Potvin* method, the *Modified Stability Graph* method and the *Stability Graph* method. The latter label will be used for the rest of this discussion for clarity and brevity.

N' is based initially on *Q'*, where;

$$Q' = \frac{RQD}{Jn} x \frac{Jr}{Ja}$$

and where;

RQD/Jn is a measure of block size for a jointed rock mass

Jr/Ja is a measure of joint surface strength and stiffness

Modified Stability Number *N'*;

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

where;

A is a measure of the ratio of intact rock strength to induced stress. As the maximum compressive stress acting parallel to a free stope face approaches the uniaxial strength of the rock, factor *A* degrades to reflect the related instability due to rock yield.

B is a measure of the relative orientation of dominant jointing with respect to the excavation surface. Joints which form a shallow oblique angle (10-30°) with the free face are most likely to become unstable (i.e. to slip or separate). Joints which are perpendicular to the face are assumed to have the least influence on stability.

C is a measure of the influence of gravity on the stability of the face being considered. Overhanging stope faces (backs) or structural weaknesses which are oriented unfavourably with respect to gravity sliding have a maximum detrimental influence on stability.

Table 2.17.1: Range of values (*for hard rock mining):

Range	*RQD/Jn*	*Jr/Ja*	*A*	*B*	*C*	*N'*
Maximum	0.5 - 200	0.025 - 5	0.1 - 1	0.2 - 1	2 - 8	**0.0005 - 8000**
*Typical**	2.5 - 25	0.1 - 5	0.1 - 1	0.2 - 1	2 - 8	**0.1-1000**

2.17.2 Stability Graph Method - Input Parameters

Compute the value of hydraulic radius, *HR*:

- $HR = \dfrac{Area\ (m^2)}{Perimeter\ (m)} = \dfrac{w \times h}{2(w + h)}$ (units of m)

where *A* and *B* are the two dimensions defining the stope face to be analyzed.

Compute the modified stability number, *N'*:

- Measure or calculate the value of *RQD, Jn, Jr* and *Ja* as described in Section 2.14.5
- Compute $Q' = RQD/Jn \times Jr/Ja$.

From the charts that follow:
- Evaluate Rock Stress Factor *A*.
- Evaluate Joint Orientation Factor *B*
- Evaluate Gravity Adjustment Factor *C*
- Obtain $N' = Q' \times A \times B \times C$

Plot point (*HR,N'*) on stability graph and determine stability and design zone.

Rock Stress Factor A

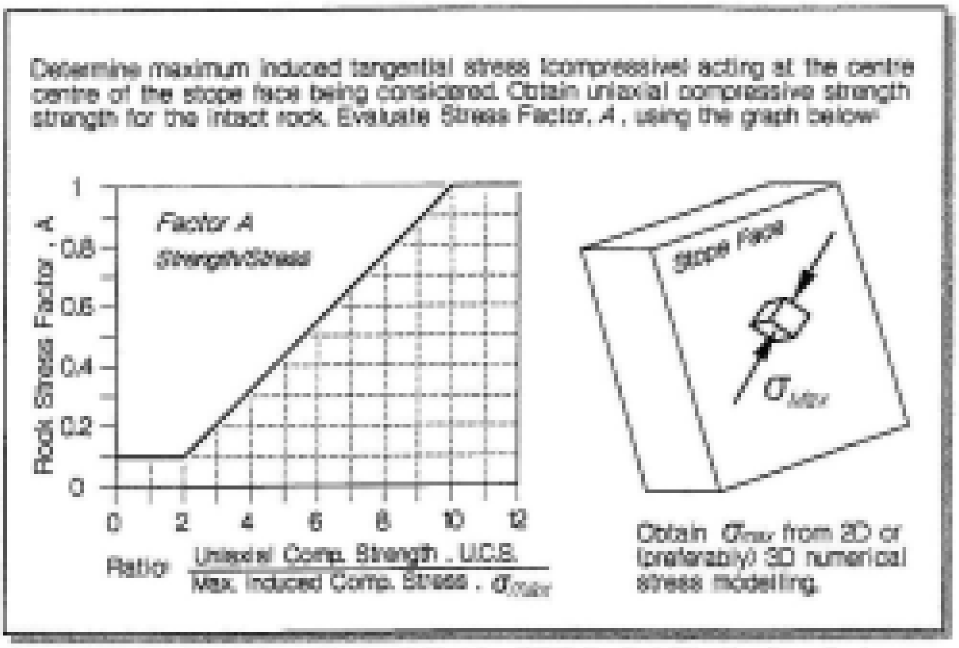


Figure 2.17.1: Rock Stress Factor *A* (Potvin, 1988) for Stability Graph analysis

Joint Orientation Factor, B

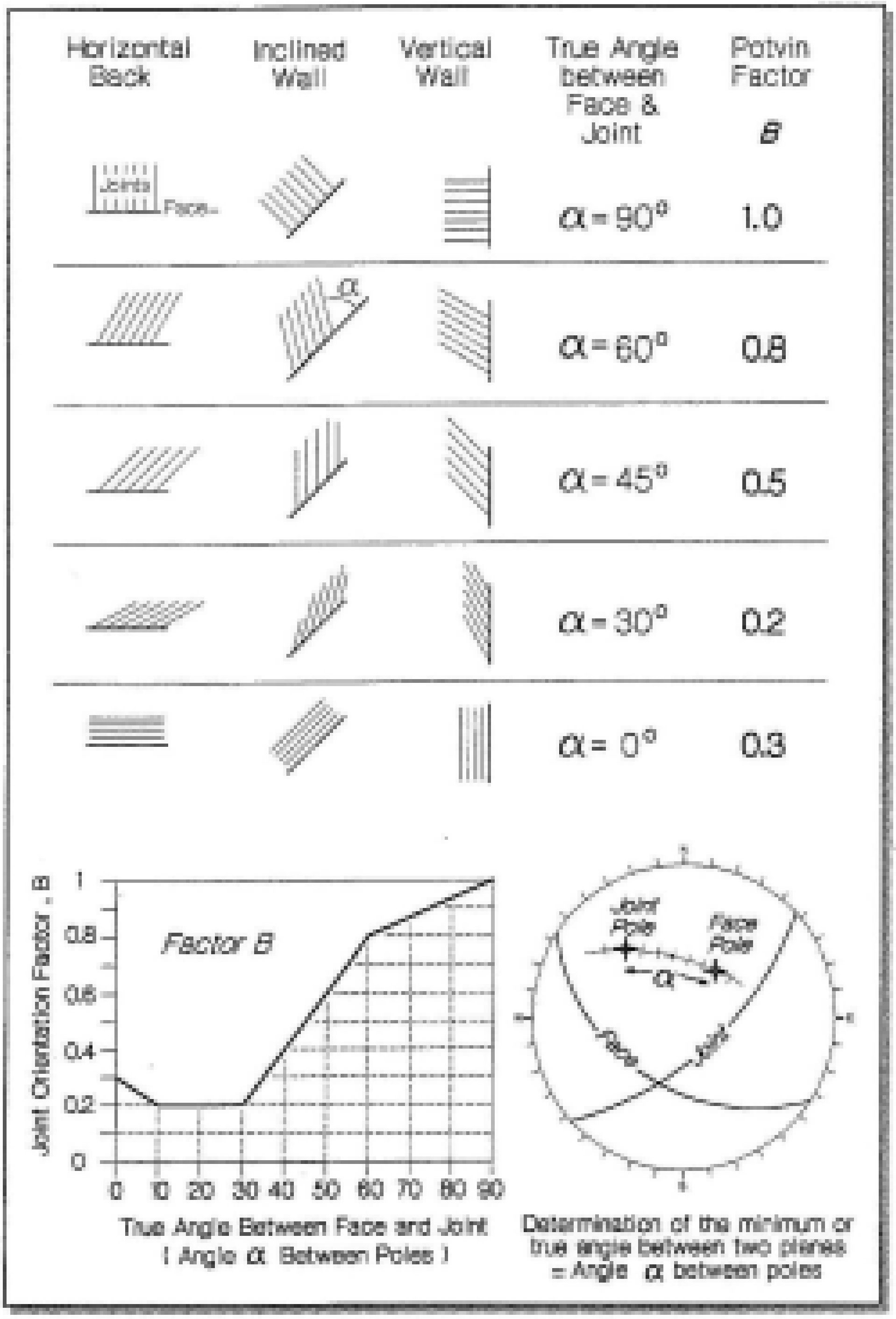


Figure 2.17.2: Determination of Joint Orientation Factor, B, for Stability Graph analysis

Joint Orientation Factor, B: Example Determination

The true angle between two planes is not immediately given by the relative dips and strikes of the planes. It must be calculated as shown on the following page or estimated from a stereonet as in this example.

Consider the hangingwall face and associated joint sets (Figure 2.17.3a). Determination of *B* involves only the pole to the face and the mean poles for each joint set 1, 2 and 3.

Using a series of small circles (cones) centred on the face pole, the angle (cone angle) from this pole to each of the joint set poles can be estimated as in Figure 2.17.3b). These small circles (cones) can be generated by hand (Goodman, 1980; Priest, 1985) or they may be automatically generated by a computer program such as DIPS (Hoek et al., 1995) as shown here. Cones drawn at 10, 30, 45, 60, and 90 degrees provide sufficient resolution to determine factor *B*.

The true angle between planes is given by the smallest angle between poles to the planes. Figure 2.17.3.b) illustrates how to determine that the angle from the face to set 1 = 20°,to set 2 = 53°, and to set 3 = 71°.

In Figure 2.17.3c), the angle contours have been replaced by corresponding Joint Orientation Factors (B). This shows clearly that joint set A is critical and that the factor, *B*, should be set to 0.2 for the Stability Graph analysis.

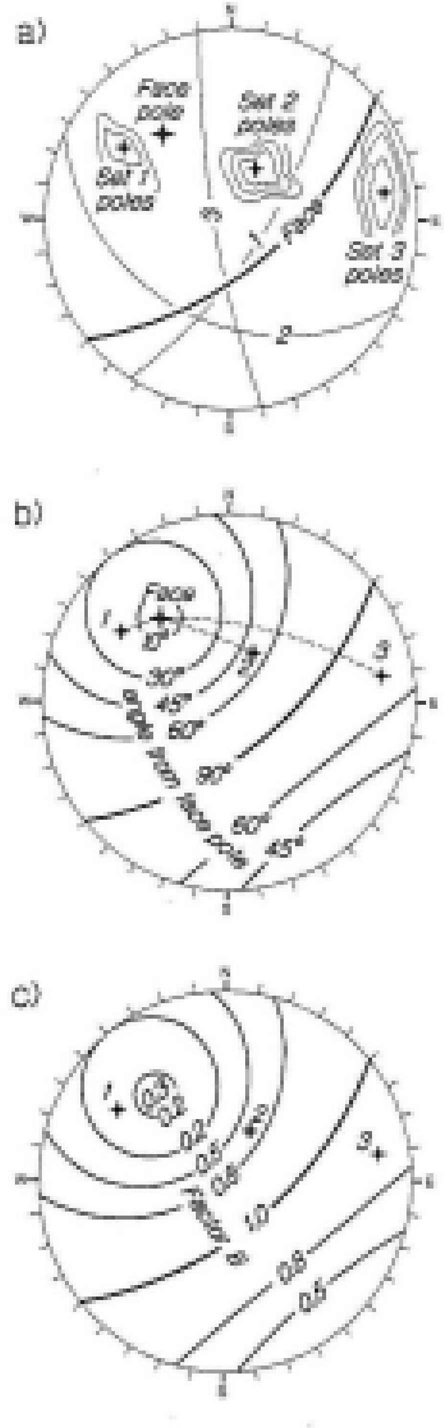


Figure 2.17.3: Estimation of true interplane angle and Joint Factor B

Joint Orientation Factor, B: Direct Calculation of Interplane Angle

It is possible to determine directly the true interplane angle between the stope face (wall plane) and the joint plane using the following simple procedure.

Given the *Dip* and the *Dip Direction* for a plane, the *Trend* and *Plunge* of the corresponding pole (normal vector) can be calculated:

$$T = Trend = Dip\ Direction + 180°$$
$$P = Plunge = 90° - Dip$$

For a stope wall plane, w, and a joint plane, j, the direction cosines with respect to the global coordinate grid (North, East, Down) are denoted by N, E and D respectively and are calculated as follows:

For the stope wall:

$$N_w = \cos(T_w)\cdot\cos(P_w)$$
$$E_w = \sin(T_w)\cdot\cos(P_w)$$
$$D_w = \sin(P_w)$$

For the joint plane:

$$N_j = \cos(T_j)\cdot\cos(P_j)$$
$$E_j = \sin(T_j)\cdot\cos(P_j)$$
$$D_j = \sin(P_j)$$

Next calculate the dot product, $\mathbf{w\cdot j}$, between the wall face and the joint plane:

$$\mathbf{w\cdot j} = N_w N_j + E_w E_j + D_w D_j$$

Finally, the true interplane angle, α, is given by:

$$\alpha = \cos^{-1}(\mathbf{w\cdot j}) = a\cos(\mathbf{w\cdot j})$$

This calculation can easily be solved using a calculator or can be implemented in a spreadsheet or computer program.

Once this true interplane angle is calculated, it is possible to assign a Joint Orientation Factor, B.

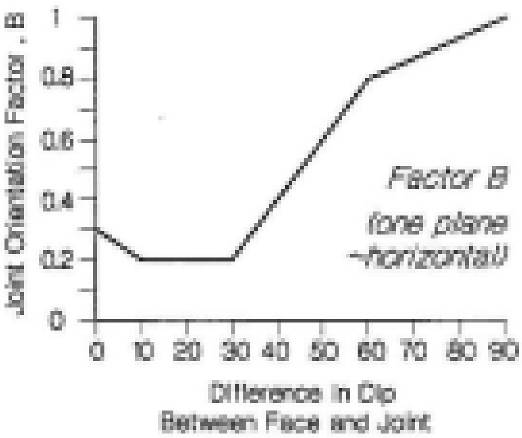
Joint Orientation Factor, B
Simplified approach (special cases)

It is important to remember that measurements such as Dip and Dip Direction or Strike are made relative to a global coordinate system. They cannot be used directly to calculate the true angle between two planes since the applicable coordinate system must be changed to be relative to one of the faces. Therefore the procedures discussed on the previous pages must be implemented.

The calculation of interplane angle is simplified, however, when one of the planes is approximately horizontal or near vertical (Dip ≈ 0 or Dip ≈ 90). In the case of true angle calculation for determination of Factor, *B*, this condition must apply to either the stope face or the joint plane (or both).

Horizontal Joint or Horizontal Stope Face (Back):

Consider only the difference in Dip between the stope face and the joint plane using the graph at right to determine *B*. When one plane is approximately horizontal, then the difference in Dip approximates the true interplane angle.



Near Vertical Joint or Near Vertical Stope Face:

The difference in Strike (or in Dip Direction) must also be considered in the case of vertical features. Note that this relationship as presented by Potvin (1988) should only be used when one of the planes is near vertical.

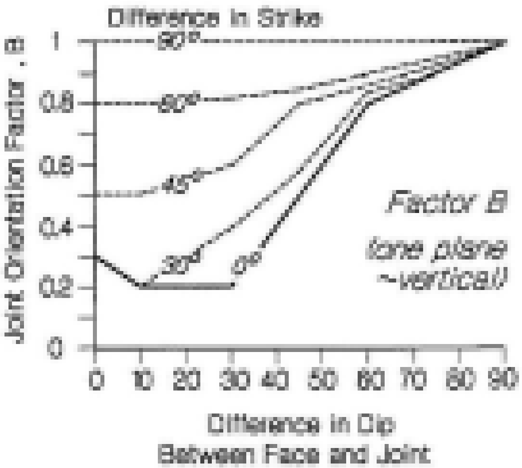


Figure 2.17.4: Simplified special cases for determining factor B

Gravity Adjustment Factor, C

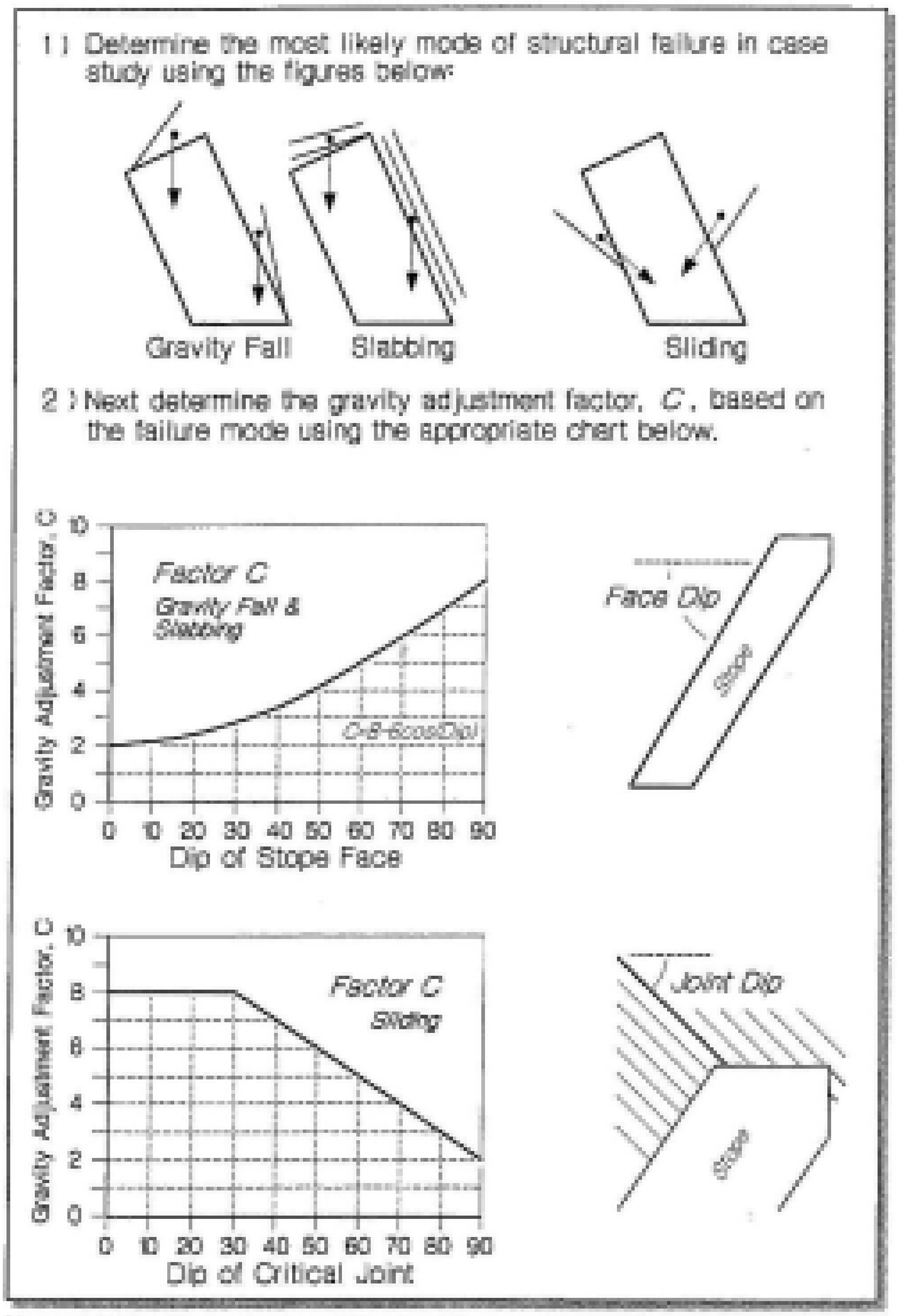


Figure 2.17.5: Determination of Gravity Adjustment Factor, C, for Stability Graph analysis

Hydraulic Radius

Before proceeding with the application of the Stability Graph, it is necessary to understand the nature of the hydraulic radius, *HR*. In short, *HR* is calculated by dividing the area of a stope face by the perimeter of that face as shown at right.

Most classification systems (e.g. *RMR* and *Q*) define stability and support zones with respect to a single value of span. This is because these methods are derived from tunnelling databases in which the long span can be assumed to be infinite and in which the short span is therefore the critical dimension. If this short span is kept constant and if the long span is reduced (to square dimensions, for example), the stability increases as a result of the increased confinement and rigidity provided by the extra two abutments. A face with a dimension ratio greater than 10:1 can be treated as a (tunnel) span equivalent to the shorter dimension.

Hydraulic radius more accurately accounts for the combined influence of size and shape on excavation stability. It is useful to become familiar with the range of "spans" for a given hydraulic radius. This will provide a means of comparison with other design methods which do not use hydraulic radius. Figure 2.17.6 illustrates these limits for a fixed hydraulic radius of 5 m. Note that although it is possible to apply this method to mining tunnels, the method has been calibrated for open stopes with finite dimensions and with lower priority for safety.

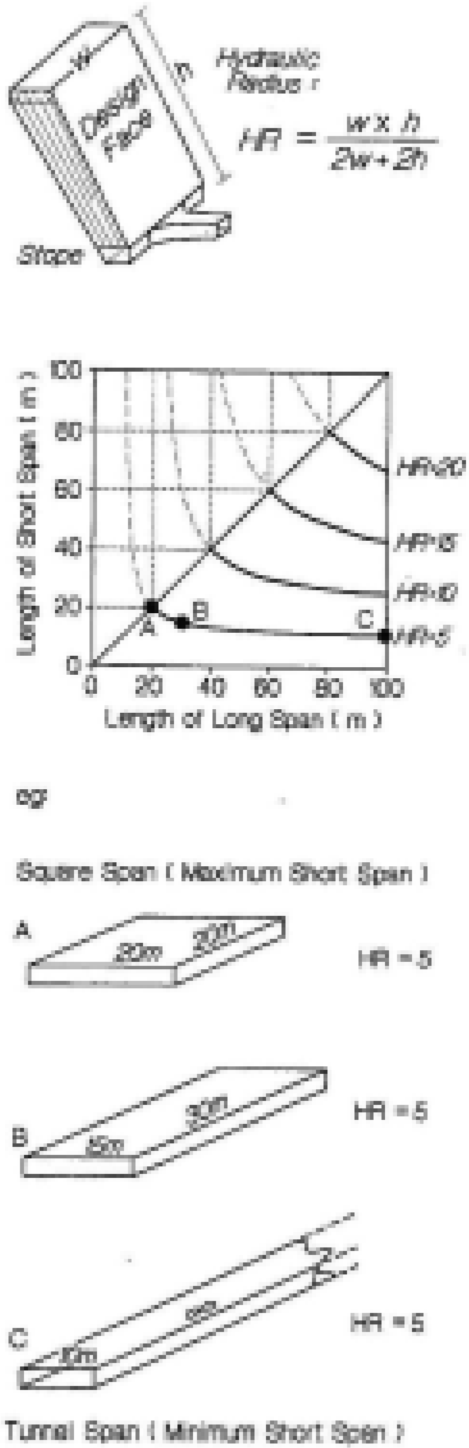


Figure 2.17.6: Hydraulic Radius, *HR*

2.17.3 Open Stope Case History Database

No-Support Limit

176 case histories by Potvin (1988) and 13 by Nickson (1992) of unsupported open stopes are plotted on the Stability Graph shown below. The modified stability number, *N'*, and the hydraulic radius, *HR*, were calculated for each case study as outlined in the previous sections. ***Stable*** stopes exhibited little or no deterioration during their service life. ***Unstable*** stopes exhibited limited wall failure and/or block fallout involving less than 30% of the face area. ***Caved*** stopes suffered unacceptable failure. Potvin plotted a *Transition Zone* defined by these cases to separate the *Stable* zone from the *Caving* zone. The upper boundary of this zone represents a recommended no-support limit for design. For a calculated value of *N'*, determine the maximum hydraulic radius for a stable stope face.

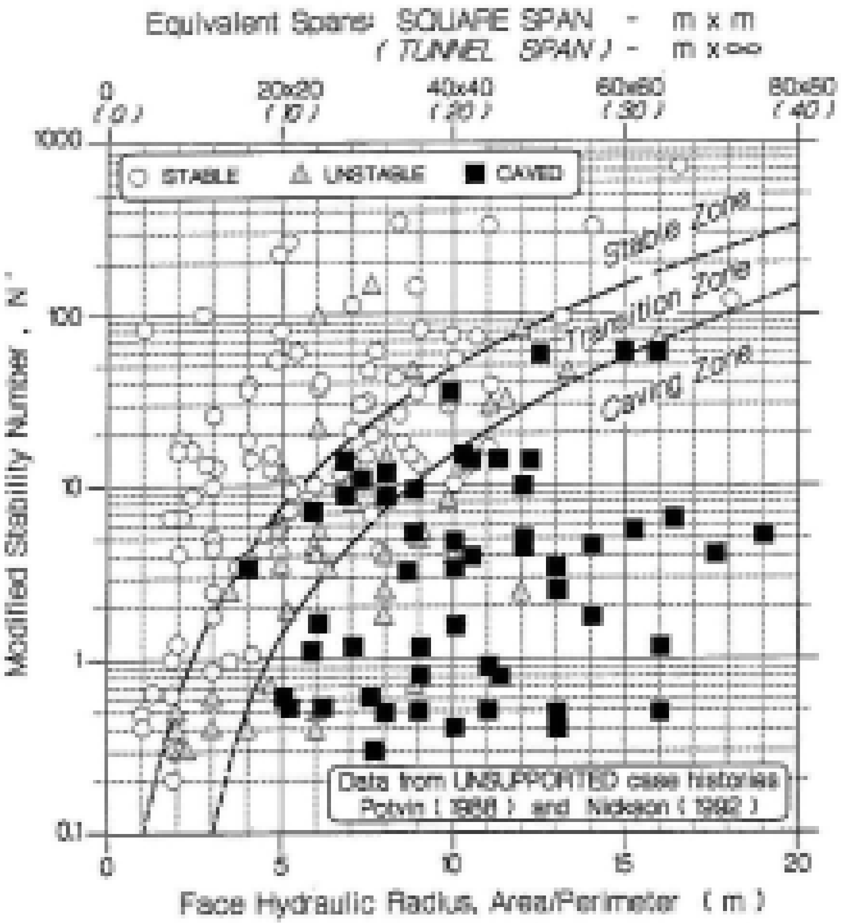


Figure 2.17.7: Database (Potvin, 1988; Nickson, 1992) of unsupported stopes

Limits of Cablebolt Effectiveness

Potvin (1988) and Potvin and Milne (1992) also collected 66 case histories of open stopes in which cablebolt support had been used. Nickson (1992) added an additional 46 case studies to this database which is illustrated below. Cablebolted stopes exhibit improved stability leading to larger stable spans (greater hydraulic radii). While this database does not take into account issues such as quality control, it does provide a reasonable demonstration of cablebolt effectiveness.

Potvin plotted a limit for cablebolt effectiveness which Nickson modified using statistical methods and additional data. The upper curve plotted below represents the limit of *reliable* cablebolt performance. Nickson proposed a zone as shown below to indicate the maximum stable hydraulic radius for cablebolted stopes (upper bounding curve) and the reduction in confidence until cables can no longer be assumed to be providing any degree of useful stope support (lower bound). Below this zone caving is inevitable.

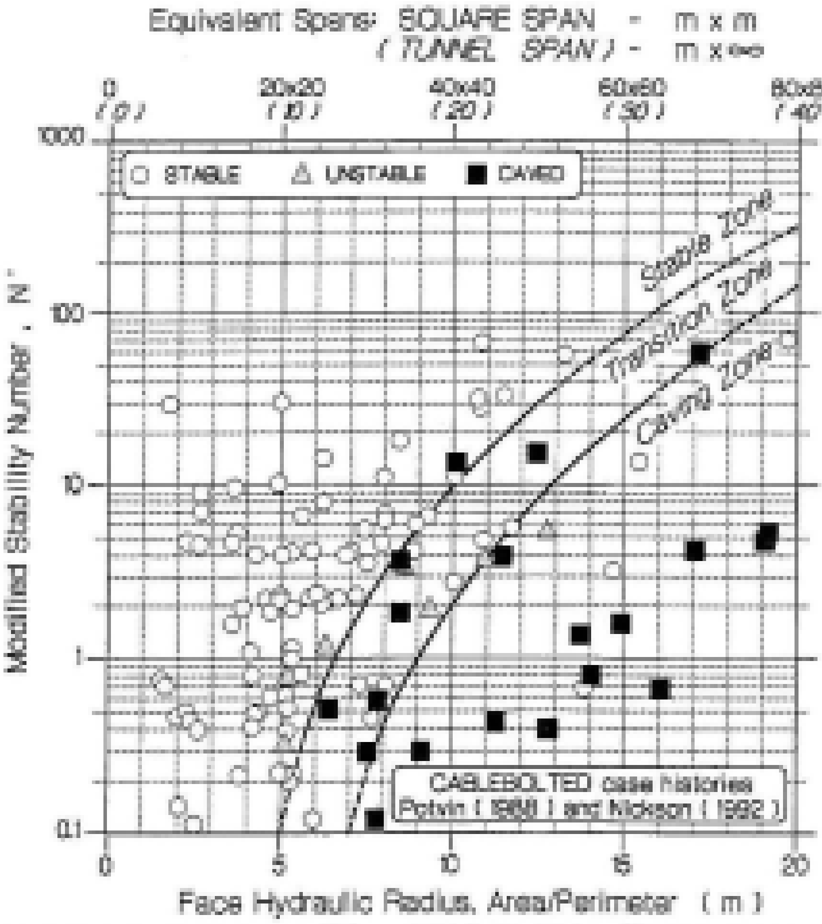


Figure 2.17.8: Database of cablebolt-supported stopes

Stability Graph - Stope and Support Design Zones

The recommended limits for unsupported and supported stopes are combined, along with the respective transition zones to obtain the design chart presented below. This graph allows the engineer to determine, from a calculated value or range of *N'*, the maximum recommended stope size and shape for an unsupported or supported case. A stope which plots well above or to the left of the uppermost design curve is capable of remaining stable without support for a reasonable service time. (Note that non-entry conditions are assumed here and that light patterned rockbolt support and mesh may be required for personnel safety in other areas). A stope which plots well into the lower-right quadrant is likely to suffer major instability with or without support. The cablebolt design zone gives the range in which cablebolts should be needed and effective. Clearly, the actual effectiveness is reduced further right and down within this design zone. As *HR* is increased or if *N'* deteriorates within this zone, the risk of failure is increased, and standup-times are reduced requiring tighter cablebolt patterns and longer bolts.

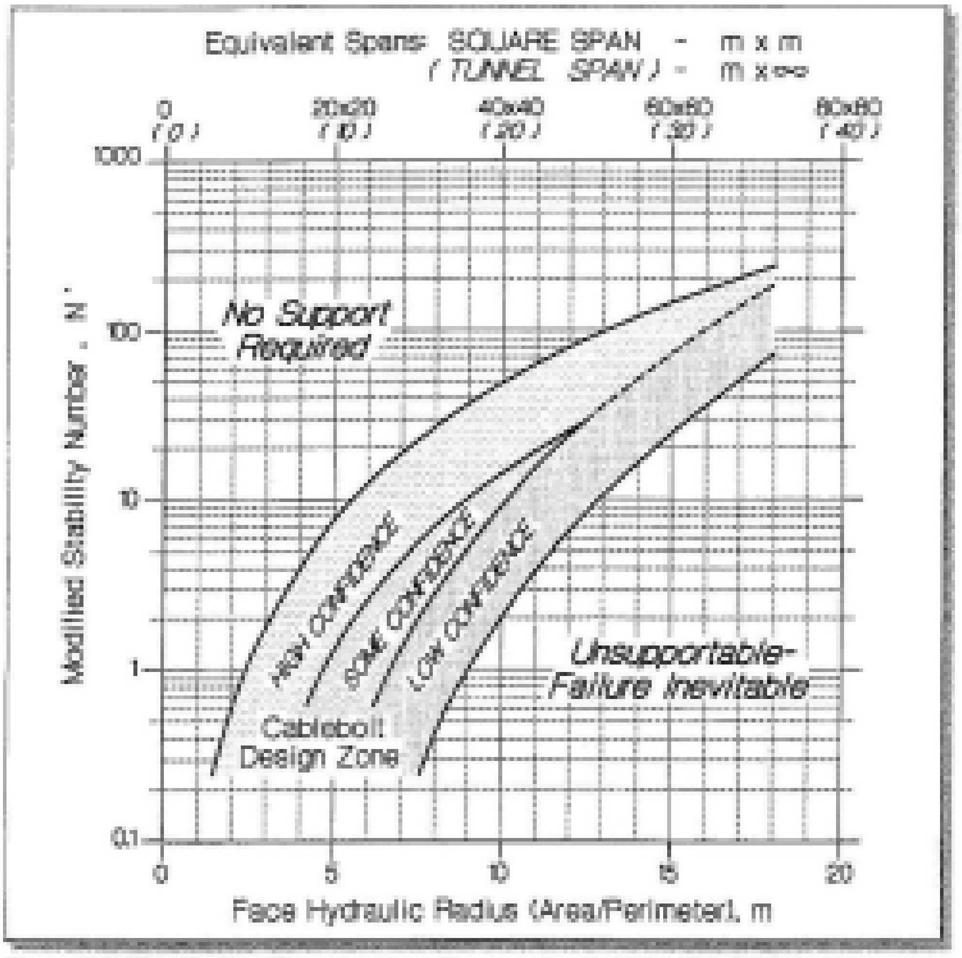


Figure 2.17.9: Design Zones for Open Stopes using Stability Graph Method

Cable Support Recommendations - Potvin (1988)

Based on his original database, Potvin (1988) determined crude guidelines for the design of patterned cablebolting. Specifically he proposed design charts for cablebolt length and cablebolt density. Cablebolt length is the length of the individual cablebolt (minimum length) and cablebolt density represents the number of cablebolts per unit area of stope face.

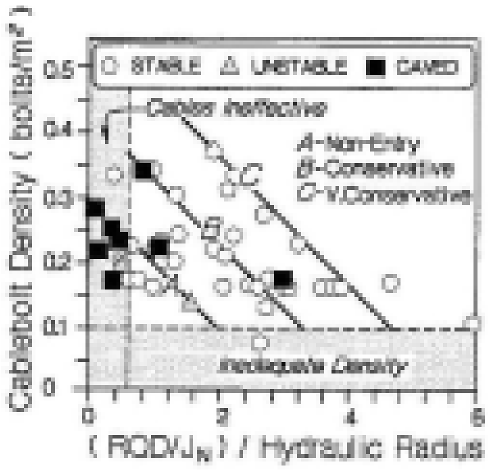
For the design of ***cablebolt density***, Potvin selected as the key empirical parameter, *(RQD/Jn)/HR*. This represents a measure of relative block size with respect to the excavation size. When this number was small it was expected that an increased cablebolt density would be necessary to ensure stability. The resultant design chart is shown at right. Note the different zones shown here.

Based on this data set, Potvin proposed that cablebolts were ineffective when *(RQD/Jn)/HR* was less than 0.6. In addition, note that the practical minimum cable density is 0.1 corresponding to a square pattern of approximately 3x3m. Three cablebolt density design lines are given which correspond to different degrees of conservatism. Non-entry stopes may require a lower cablebolting density than a main haulage drift for example.

The ***cablebolt length*** used in each case study was plotted against the hydraulic radius. This follows logic based on classical rules of thumb relating bolt length and span. A representative line based on current practice is shown and corresponds approximately to:

$$Length = 1.5 \times HR$$

up to a practical maximum of 15m at a hydraulic radius, *HR*=10m.



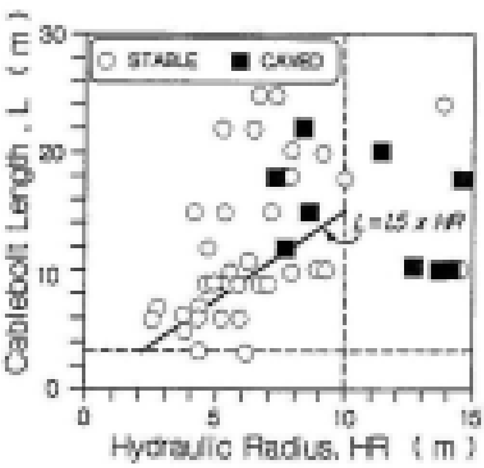


Figure 2.17.10: Guidelines for cablebolt density and length for regular patterns (after Potvin, 1988)

Cablebolt Density (bolts/m² of face) - Local Unravelling

Potvin (1988) plotted cablebolt densities used in case histories against *(RQD/Jn)/HR* based on the assumption that relative block size was in principle the governing empirical parameter for stope face stability and support effectiveness. Nickson (1992), however, applied statistical techniques in an investigation of many possible parametric combinations. For the combined cablebolted stope database of Potvin and Nickson, *(RQD/Jn)/HR* actually gave a very poor correlation to cablebolt density based on current practice. This is illustrated by the scatter in Figure 2.17.10. It is proposed here that the absolute block size represented by *RQD/Jn* should control local block fallout from the face and therefore should strongly influence ultimate stability of the stope. If cablebolts are spaced too far apart, unravelling will occur between bolts, progressively leading to more serious instability. The corresponding graph based on the Potvin/Nickson database is shown in Figure 2.17.11. The design zone plotted provides a crude recommended design range for cablebolt density in open stope applications. This design zone should not be applied to permanent openings or in high traffic areas, where safety is a critical issue, unless accompanied by primary support such as rockbolts and screen.

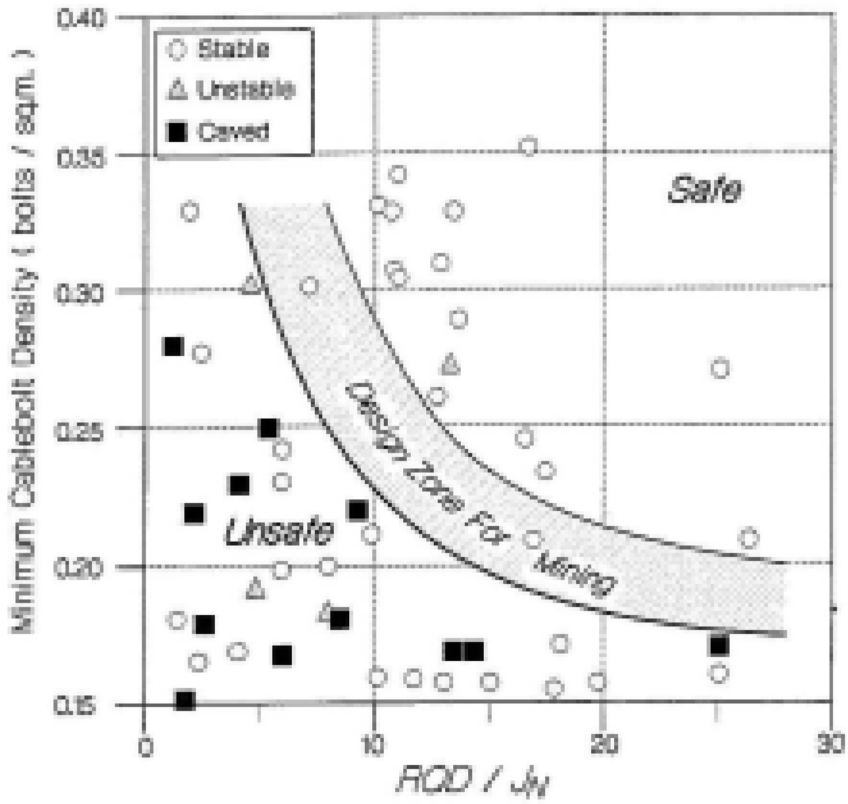


Figure 2.17.11: Guidelines for cablebolt density to control local unravelling

Cablebolt Density or Spacing - Stope Face Support

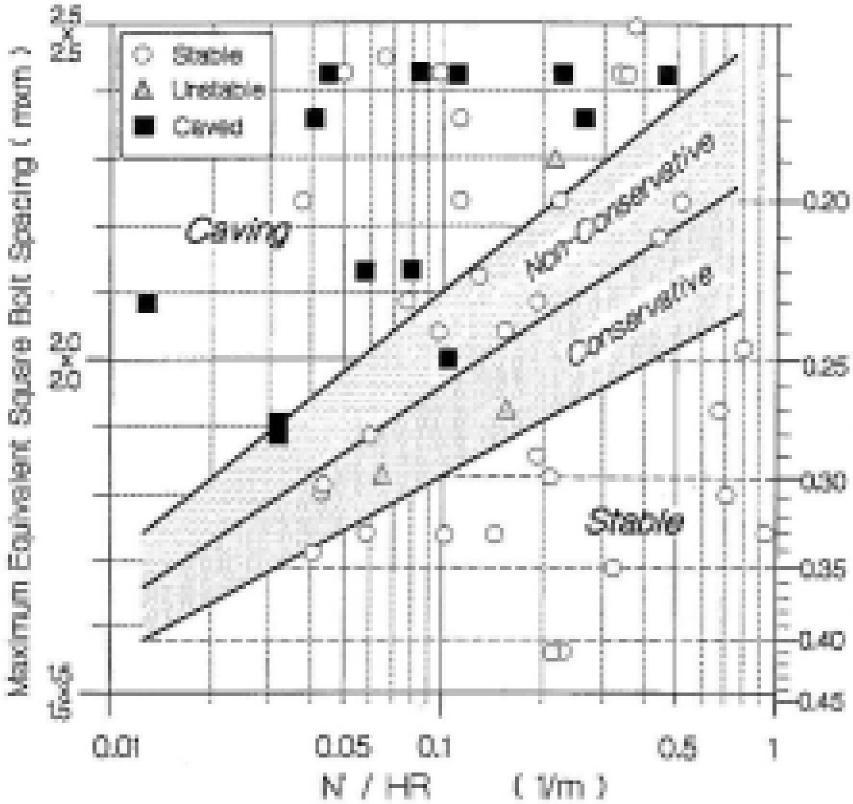


Figure 2.17.12: Guidelines for cable spacing and density - overall stope face stability

Nickson (1992) showed that the best empirical correlation with respect to cablebolt density was obtained by plotting density with respect to the parameter *N'/HR*. The logic here is similar to Potvin's usage of *(RQD/Jn)/HR*, except that *N'* contains additional information about stope inclination, stress related fracturing (parameter *A*) and favourable or unfavourable joint orientations. Nickson derived a relationship based on current practice without considering the degree of support effectiveness. The design zones proposed above in Figure 2.17.12 do relate to this degree of success. While the data scatter is great due to the trial-and-error nature of present design practice, there appears to be a reasonable limit to cablebolt effectiveness as delineated by the cluster of caved cases in the upper portion of this plot. The ***non-conservative*** zone can be used as a guide for non-entry conditions or where dilution is not critical. The ***conservative*** zone is applicable to stope backs above drilling horizons and other areas where entry is permitted. Note the two vertical scales used here. These illustrate the relationship between cablebolt density and the cable spacing of an equivalent square pattern. Use Figures 2.17.11 and 2.17.12 together to determine the critical (maximum recommended) spacing.

2.17.4 Semi-Empirical Cablebolt Design Approach

It is possible to combine the information gained from empirical methods with mechanistic assumptions and logic to develop a more sound semi-empirical design methodology. Figure 2.17.13 illustrates this approach. The ***No-Support Zone*** and the ***Unsupportable Zone*** are derived as previously discussed from examination of over 350 case histories. Within the cablebolt design zone (shaded area), however, it is possible to assume some basic support functions and modify the design accordingly. The ***Reinforcement*** zone implies that the rockmass is still partially stable, requiring cables to merely hold together the constituent blocks to form a self-supporting arch or beam. Cable spacings and lengths along the upper boundary of this zone are derived directly from the analysis in Section 2.18.12 using a back calculated rockmass stiffness. In the ***Support*** zone, however, the cables must bear the full load of the failed or loosened rockmass. Spacings and lengths along the lower boundary of this zone are therefore derived from conservative civil engineering experience (Section 2.16.5). The transition between these two extremes is continuous across the shaded zone. ***Retention*** recommendations based on ravelling failure (Figure 2.17.12) are superimposed on the above results. The maximum spacing and minimum length required to effectively carry out all of the support functions considered are then plotted in the following charts. The zone marked ***Retain*** in Figure 2.17.13 is the zone in which this function is critical with respect to spacing of cablebolt support. In the other zones, reinforcement and support dictate the maximum allowable spacing.

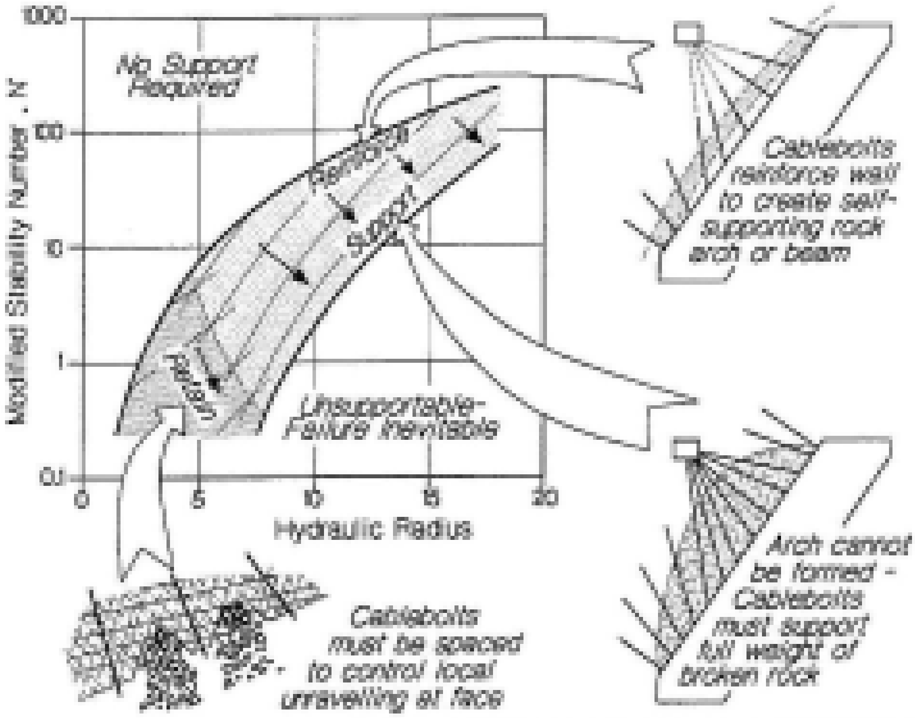


Figure 2.17.13: Five design zones for cable support of open stopes

Maximum Design Spacing for Single Strand Cablebolts

Based on the assumptions illustrated in Figure 2.17.13, recommended cablebolt spacings (for an equivalent square pattern) have been calculated for the range of *reinforcement-support* across the shaded cablebolt support zone. Where the maximum spacing so determined exceeds the recommended spacings obtained from Figure 2.17.12, *unravelling* between and around cables is assumed to dominate stability and Figure 2.17.12 therefore controls the design. The composite result is the cablebolt spacing design chart shown below in Figure 2.17.14.

For a given value of *N'* and *HR* plotting within the shaded cablebolt design zone, it is possible to determine the maximum (critical) spacing of single cables in a square pattern to ensure stability. Note that minimum cablebolt density, *Dc*, is related to maximum equivalent square spacing, *Sc*, as follows:

Cable Density, *Dc* (bolts/m^2): $Dc = Sc^{-2}$ Sc = Cable Spacing (m)

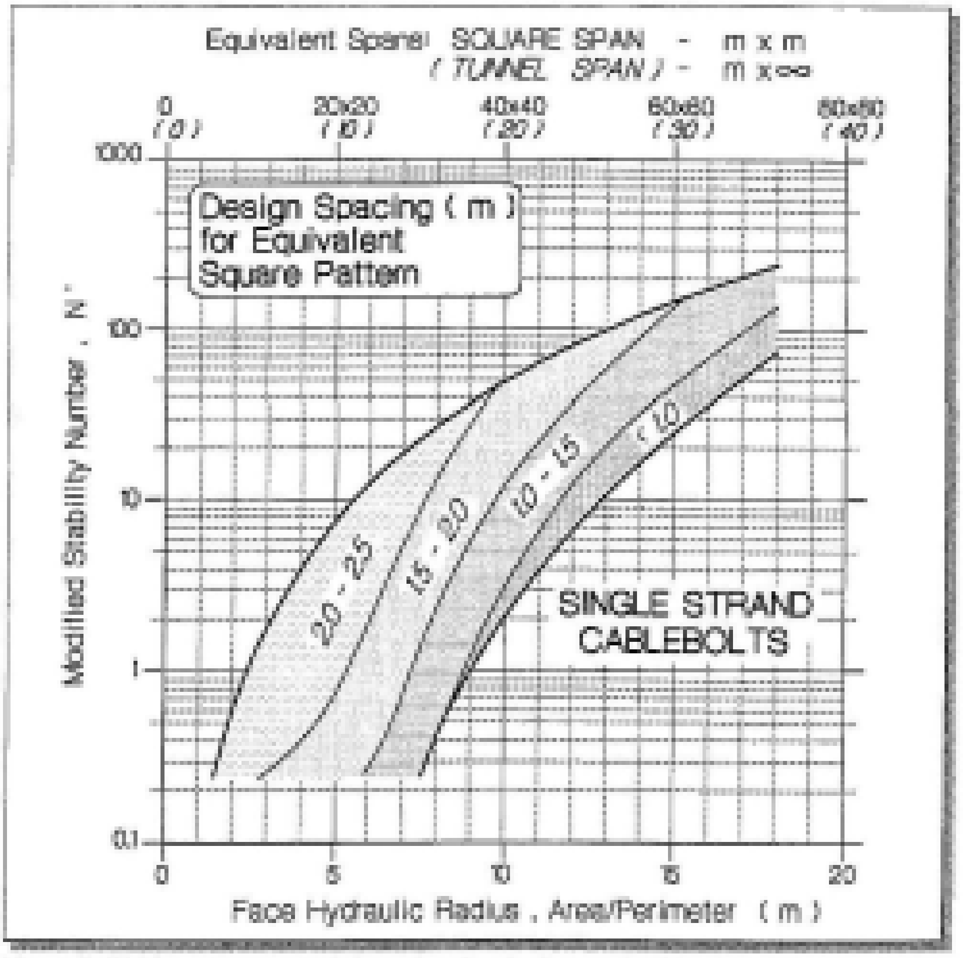


Figure 2.17.14: Recommended spacing for single strand cables (regular pattern)

Minimum Design Length for Cablebolts (Single/Double Strand)

Support design at the outer limits of the *Reinforcement* and the *Support* zones illustrated in Figure 2.17.13 are based on limiting conditions of arch/beam reinforcement and deadload estimation respectively. Based on parametric analysis using conservative parameters derived from N', these analyses yield the bounding values for spacing discussed in the previous sections and for length as shown below in Figure 2.17.16.

Recommended lengths for cement grouted cablebolts differ from resin grouted or mechanical bolt recommendations in the literature. This is due to the necessary addition of a reliable anchor length beyond the zone of supported rock. In the case of beam analysis and deadload estimation, this corresponds in the figure below to 2m beyond the stabilized beam or failed zone respectively. Note that increasing length does not always imply increased capacity (controlled by strand density). These lengths are based primarily on cable coverage of the supported zone.

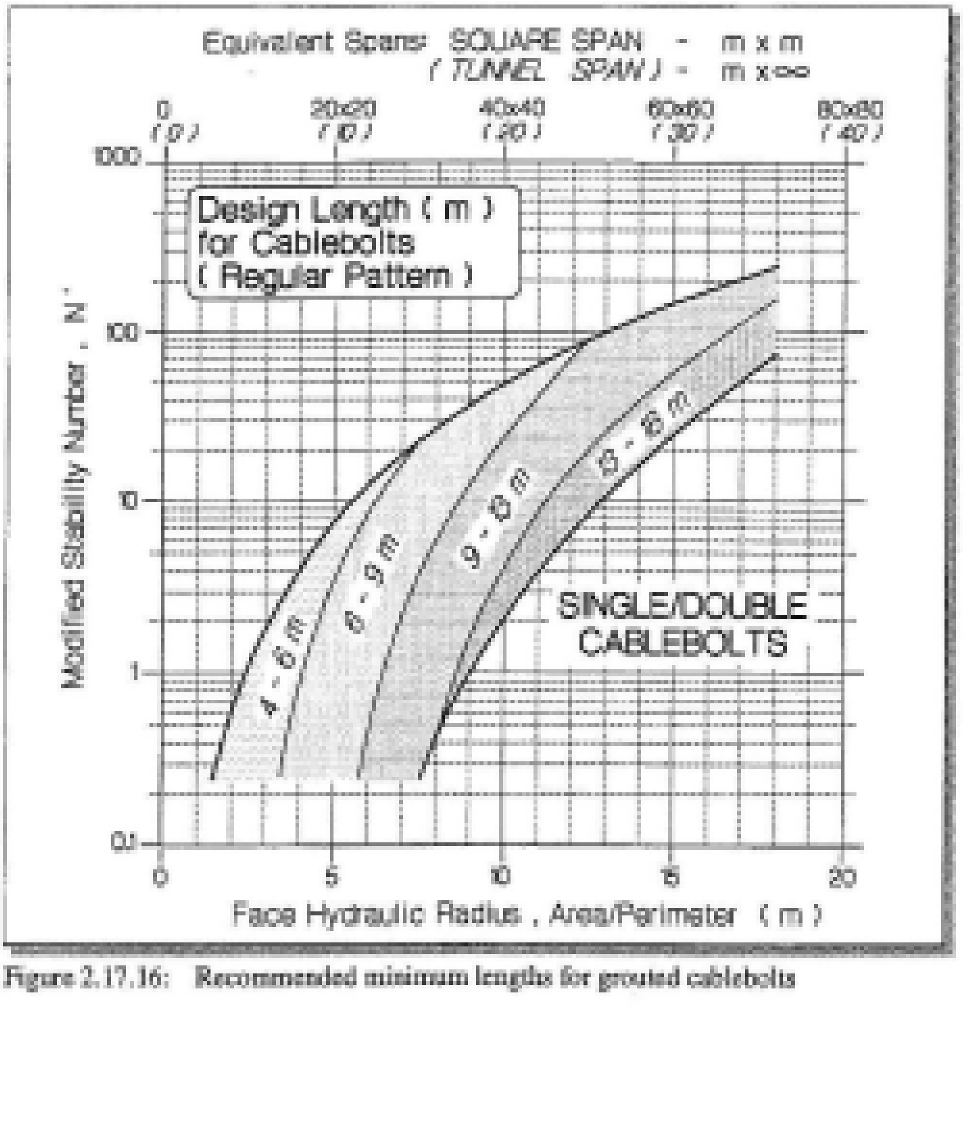


Figure 2.17.16: Recommended minimum lengths for grouted cablebolts

Cablebolt Spacing and Length of Regular, Uniform Arrays

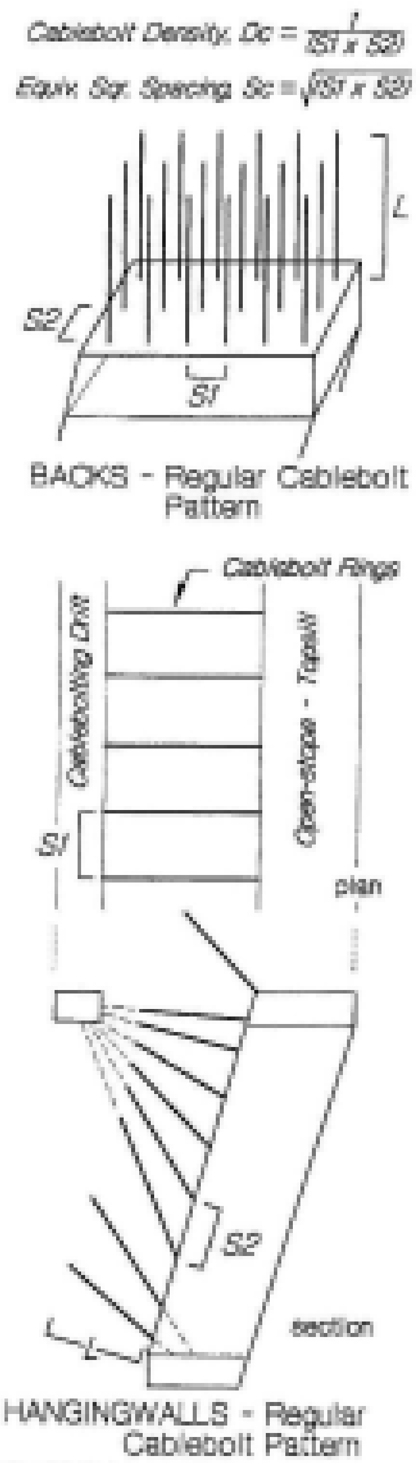


Figure 2.17.17: Regular Patterned Support

All of the preceding discussion concerning the modified stability graph and recommendations for cablebolt spacing and length apply to a regular or patterned array of cables; a constant distribution of bolts across the face area of the stope and an arrangement behind the face such that neighbouring cables are within 40 degrees of being mutually parallel. The example cablebolt patterns in Figure 2.17.17 illustrate the ideal application of these guidelines.

Cables should be spaced as close to square as possible if designed using the recommendations in this section. For example, patterns such as 1.5m x 2m (equivalent square = 1.7m x 1.7m) or 2m x 2.5m (= 2.2m x 2.2m) are acceptable, whereas a pattern of 1m x 3m may not perform as well as the equivalent square pattern (1.7m x 1.7m).

Cable spacing should also be uniform (i.e. spacing should not vary more than 20% over the stope face). The density of tight clusters of cables bounding larger areas of unsupported stope face cannot be averaged over the whole area and equated to an average density or equivalent spacing. The design of this type of system is handled differently as shown on the following page.

Cable lengths are specified for cablebolts which are within 30-40 degrees of perpendicular to the stope face. Normally the length refers to the perpendicular distance between the face and the end of the cables. Actual cable length will depend on the cable angle.

Line or Point Anchor Arrays - Sub-Span Design

In many cases in mining, access constraints do not allow the installation of a regular uniform pattern of cablebolts in a back or hangingwall. In addition, mining influences such as blasting, induced stress change or rock relaxation may limit the effectiveness of a distributed cable pattern (Section 2.6). This is particularly the case in foliated hangingwalls. Often it may be preferable, therefore, from both an operational and an engineering viewpoint to install line anchors as shown below at prescribed intervals. These anchors reinforce a local volume of rock, limiting internal displacements and preventing dilation. This artificial rockmass block or rib then acts as an effective abutment for adjacent spans (Fuller, 1983).

This support system should only be used in rockmasses dominated by a single lamination parallel to the stope face or joint sets perpendicular to this face (few oblique joints). Blast control is critical to avoid damage to the unsupported span and displacement rate monitoring may be a useful design verification tool here.

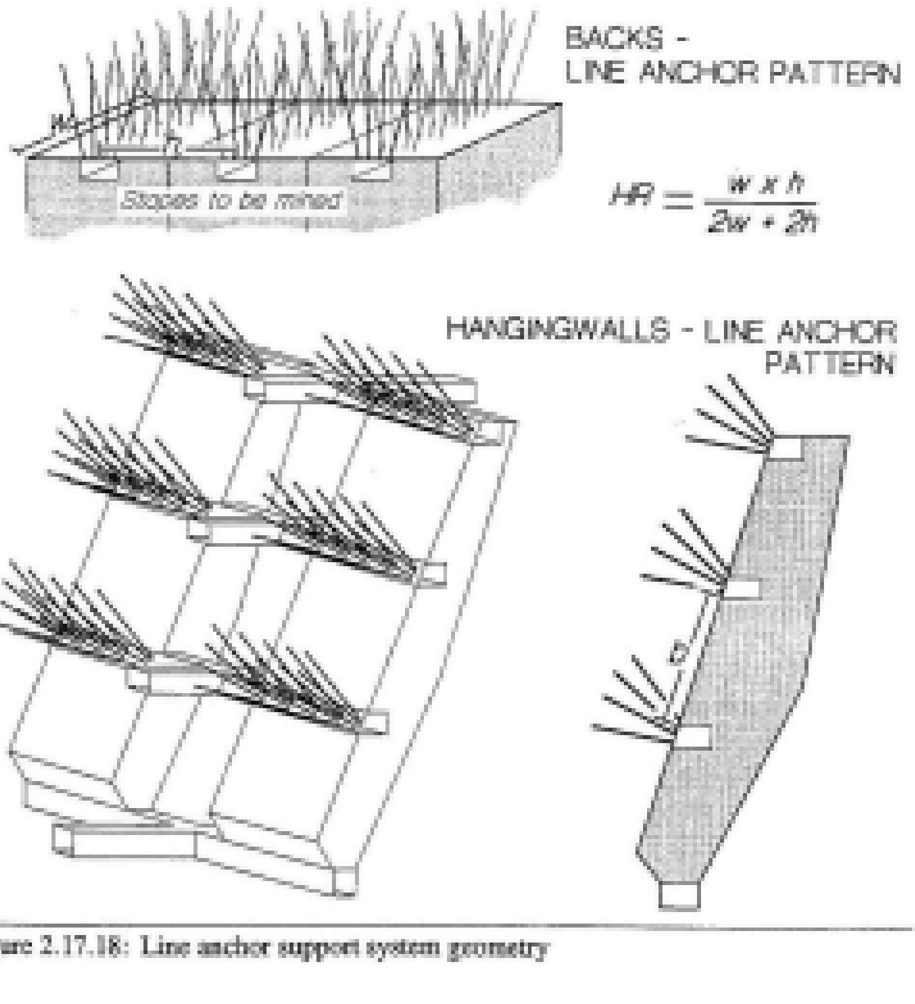


Figure 2.17.18: Line anchor support system geometry

As such these anchors must have a locally dense arrangement (<1.5 m spacing at collar) and 4-6 cablebolts in each ring. These cables should then be plated. This is to ensure limited internal movement within this reinforced "abutment". The Modified Stability Graph can then be used directly to dimension the unsupported sub-spans (a x b in Fig. 2.17.18). These sub-spans (unsupported spans) may be strung together providing a huge operational benefit by allowing a much larger stope to be opened without immediate backfilling. There is a limiting relationship, however, between the unsupported sub-span and the overall "supported" span (or hydraulic radius of total open stope face). Nickson (1992) compared 13 case histories of line anchored hangingwalls and proposed the crude relationship illustrated in Figure 2.17.19.

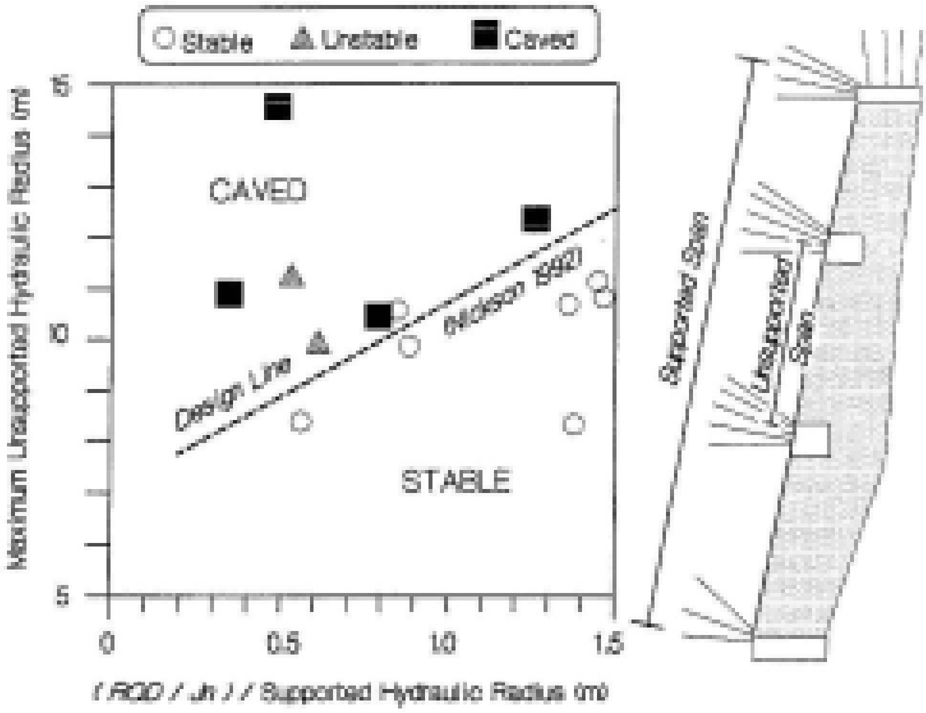


Figure 2.17.19: Crude relationship relating overall (supported) span to unsupported sub-span. Applicable to hangingwalls only (data from Nickson,1992).

Note that the database is extremely limited and so caution must be exercised when using this graph. Calibration to local conditions will be necessary.

The relationship above should not be applied to shallow dipping hangingwalls or backs. This method is designed for non-entry stopes should not be applied to stope faces in areas where regular human access is necessary without additional primary support such as rockbolts and screen to control small block fallout.

2.17.5 Stability Graph - Examples

Consider the following examples of open stope scenarios. These four cases have been deliberately chosen to result in the same hydraulic radii, *HR* and the same values of Modified Stability Number, *N'*.

Table 2.17.2: Four example applications of the Stability Graph

	CASE A Hangingwall	CASE B Back	CASE C Hangingwall	CASE D Back
Problem Description				
Depth	200 m	600 m	150 m	1000 m
Wall Stress	10 MPa	20 MPa	8 MPa	60 MPa
RQD	40	60	85	90
Joint Sets	2	2 + random	3 + random	2 + random
Joint Surface	Smooth Planar; Slightly Altered	Rough Undulating; Unaltered Stained	Rough Planar; Slightly Altered	Slickensided Undulating; Unaltered Stained
Rock Type	Foliated Schist	Bedded Limestone	Gneiss	Massive Sulphide
Rock Strength	80 MPa	115 MPa	160 MPa	180 MPa
Input Parameters				
Wall Dimensions	20 m X 40 m	18 m X 55 m	25 m X 30 m	22 m X 34 m
RQD/Jn	40 / 4 = 10	60 / 6 = 10	85 / 12 = 7.1	90 / 6 = 15
Jr/Ja	0.5	3.0	0.75	1.5
A	0.78	0.52	1.0	0.21
B	0.3	0.3	0.3	1.0
C	8.0	2.0	6.0	2.0
Stability Graph Coordinates				
HR	6.7	6.8	6.8	6.7
N'	9.4	9.4	9.6	9.6
Status	STABLE	CAVED	UNSTABLE	CAVED

These examples are illustrated in Figure 2.17.20 on the following page. Note the obvious differences in stope dimensions and in geometrical and geomechanical environment. Yet the plotted results for the four cases are indistinguishable. This illustrates both a strength and a weakness of the Stability Graph method. Like all empirical methods it is a general design method which allows us to formulate preliminary designs in the face of limitless variety and complexity. Other design techniques are normally very problem specific and cannot be universally applied. Once the preliminary design is established, however, the method does not provide for the fine tuning which must occur to adapt the design to the specific problems encountered in each case.

Case A, for example, is a thinly laminated rockmass with a second discontinuous joint set at 90 degrees from the main lamination. Even though the RQD is low due to the foliation, the stope wall is vertical and as such should be inherently stable unless disturbed by poor blasting or excessive span development. Cables are unlikely to improve stability within economic limits in this case. The stress is low compared to the rock strength so gravity is likely to be a dominant control. This case is suitable to Voussoir beam analysis (Section 2.18.12).

Case B is a competent blocky rockmass above a relatively wide sill span. The main lamination would suggest beam analysis. The cross jointing, however, is oblique to the back and is unlikely to allow complete arch development in the horizontal back. Patterned roof bolting will be necessary in this case

Case C represents a strong gneiss with moderate structural density. Even though the block size is small, the joint surfaces are very rough and tightly interlocked. The stresses are low but the steep dip of the wall will maintain compression and improve stability. Patterned cablebolting from a hangingwall drift should prove effective in this case.

Case D appears to be the highest quality rockmass as indicated by the large values for *RQD/Jn* and *Jr/Ja*. The stability graph analysis does not consider the sheared contact which forms the hangingwall. It is likely that this contact will shear due to stresses in over the back. These stresses are high and this slippage may be unstoppable. The vertical jointing will form release planes resulting in a large free full span wedge which must be supported. Cables must be designed to withstand large displacements or they will snap as the wedge slips.

These examples show that while the Stability Graph method is an invaluable tool for initial dimensioning and support design for open stopes, it is not the final word. If the stope plots well into the stable zone or well into the caving zone, then the respective result is fairly reliable. If the stope plots close to or within the cablebolt design (support required) zone, then further mechanistic analysis should be carried out to confirm the validity of critical assumptions and recommendations of the stability graph method.

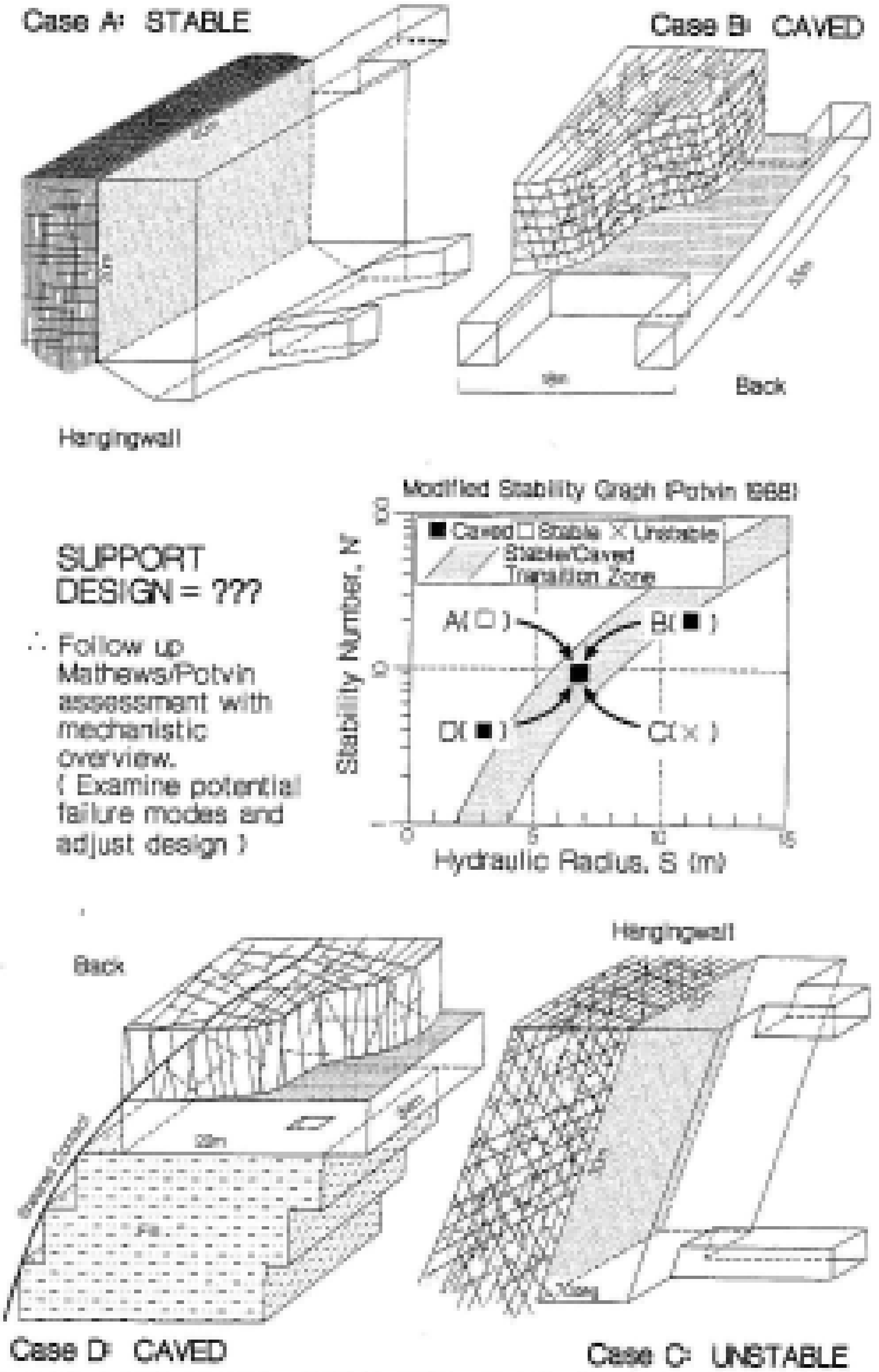


Figure 2.17.20: Four application examples for the Stability Graph Method. Note that very different design problems can result in the same position on the graph.

2.17.6 Stability Graph Method - Limitations

There are certain assumptions inherent in the application of the Stability Graph Method. Observe the following limitations when using the method for stope and support design.

Inadequate Fill

The estimation of stable hydraulic radii determined from the graph or used as input for stability evaluation assumes that the span being considered is fully bounded. This assumption is valid for unfilled stopes which are surrounded by fill (as in alternate block sequencing, for example). The surrounding fill must be tight to the walls and back of the stopes in order to be considered supporting elements. If this is not the case as shown in Fig. 2.17.21a), the true effective span for analysis may be much larger than the nominal stope panel. The same is true if the fill is highly compressible. In such a case, the Stability Graph Method is not applicable to the design of the unmined panel 7 in Fig 2.17.21b), for example.

Corners-Designed and Accidental

As shown in Fig 2.17.21c), corners or bulges can be created in stope walls though poor design ("chasing grade") or though the upward caving of mined stopes below. In either case, the stability graph cannot be used to evaluate the stability of either the span above the corner nor the overall span. The corner so created, will dominate the stability of the entire stope and will likely cause major stability problems. Such corners, either deliberate or accidental should be avoided.

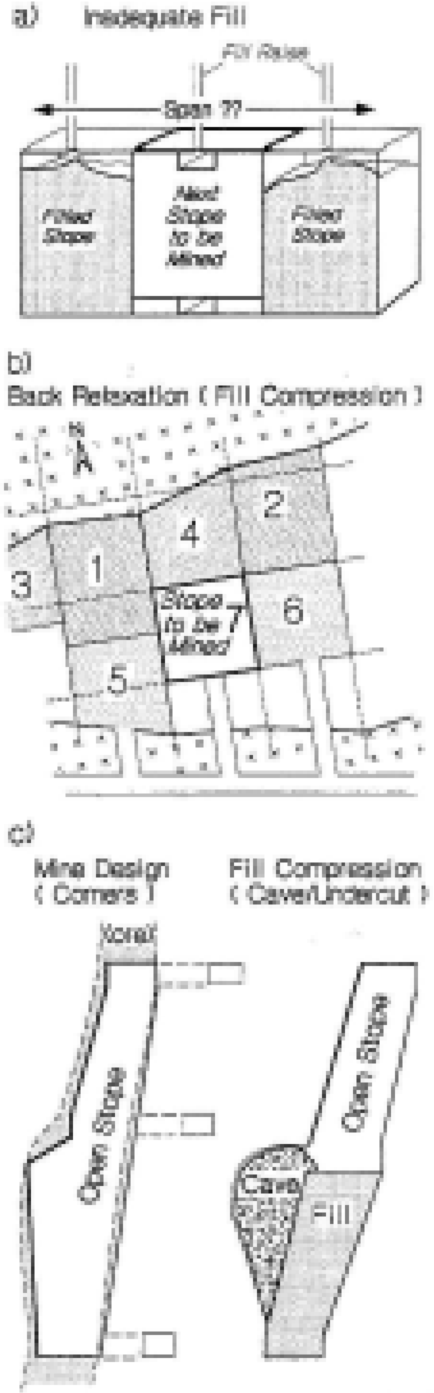


Figure 2.17.21: Limitations for use of Modified Stability Graph

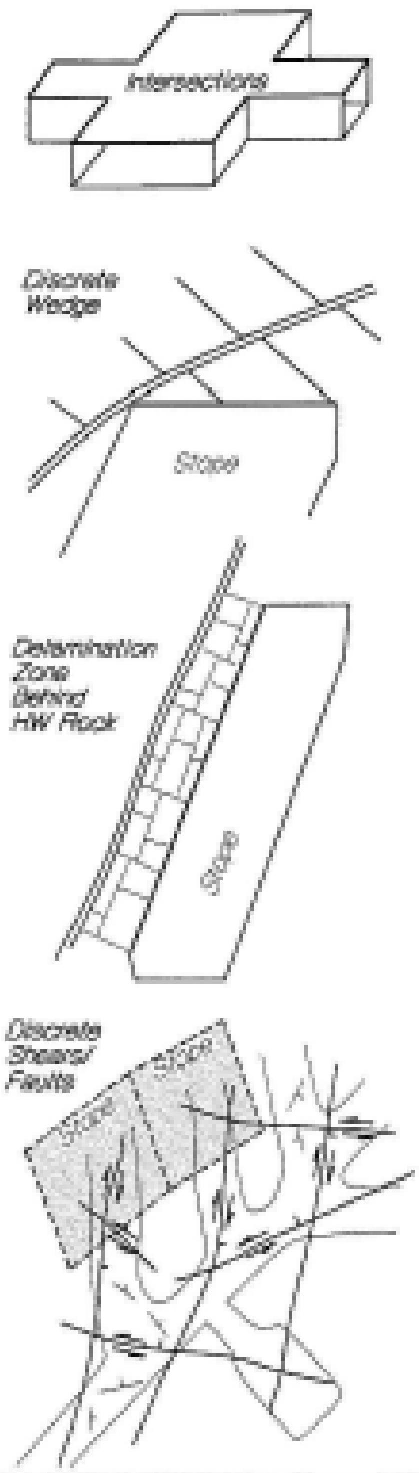


Figure 2.17.22: Limitations of the Stability Graph Method

Intersections

While the Stability Graph Method was calibrated for open stopes, it can be used for large mining tunnels and sills provided that a conservative approach is taken for safety reasons. The method should not be used, however, to design intersections. The assumption of a bounded span is not valid here. Intersections are normally less stable than the associated tunnels. In addition, it is not possible to calculate an equivalent *HR* for an intersection.

Discrete Wedges

The stability graph design approach is applicable to moderately structured rockmasses with distributed or ubiquitous structure. Discrete structural features such as large wedges which may form in sill backs must be considered separately.

Delamination Zones

Large stable spans may be predicted in cases with structurally sound wall rock. If this wall rock is bounded by a weak layer close (within 20% of the span) to and parallel to the wall as shown at left, the stability of the resultant beam will be reduced. Beam analysis methods may be more appropriate for design.

Discrete Shear Structures

Large scale structure (length > stope dimensions) will control stope stability under stress and gravity. Discontinuum analysis methods must be used for design. The Stability Graph results will not accurately predict stability.

2.17.7 Stability Graph-Calibration to Local Conditions

The initial database of approximately 350 case histories is impressive in size and scope. It is, however, incapable of accurately predicting stability in every possible situation. The database, for example, reflects Canadian practice. This immediately implies a bias towards Canadian conditions. In short, the method provides an excellent starting point for design but it must be calibrated on-site in every new mining environment. This involves maintaining an up-to-date database of stope dimensions, rockmass parameters, and stability status.

Bawden (1993) uses a data set from Greer (1989) to demonstrate this concept as illustrated in Figure 2.17.23 below (note the truncated axes for more detail). In this case, due to unique conditions at the mine site, significant caving and instability was observed in stopes which the method predicted to be stable.

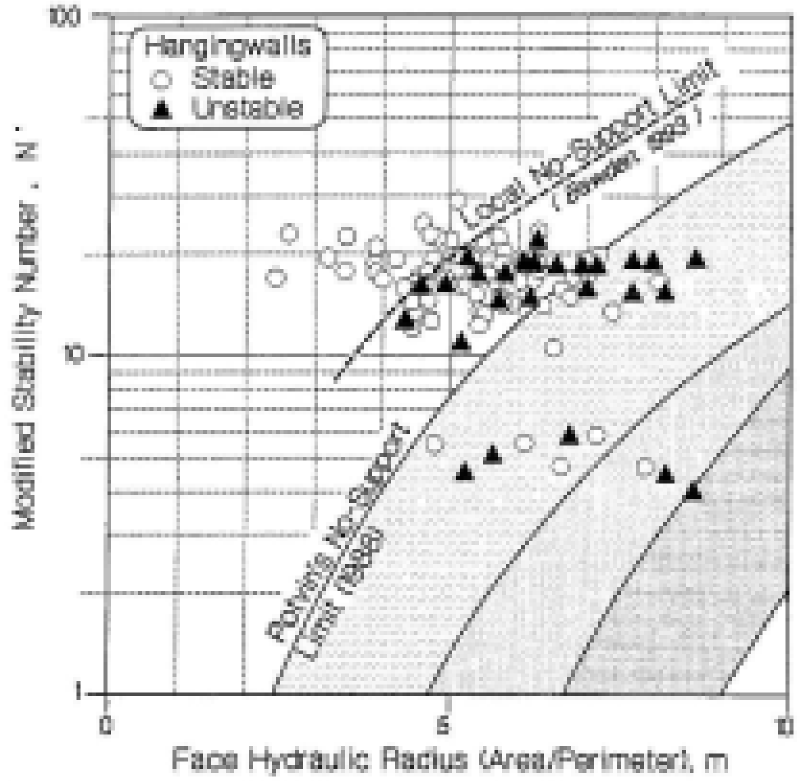


Figure 2.17.23: Example of local site calibration. New design line (dashed) can be used for future stope design (after Bawden, 1993).

If such a local database is maintained, then the Stability Graph can be calibrated for local conditions. The dashed design line proposed by Bawden for the above data bounds the caved and unstable stopes. This line should now be used as the no-support limit for future mine design at this site.

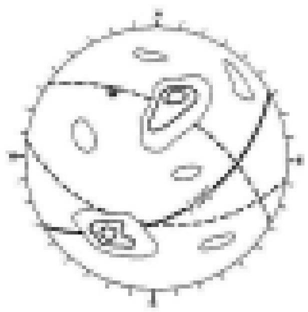
2.17.8 Parametric Analysis

The quality of a rockmass is never well defined. For overall mine design and budgeting purposes at a preliminary stage it may be adequate to design based on average rockmass conditions. Assuming worst case parameters may prove impractical from an economic perspective while designing based on the best possible conditions would clearly be imprudent.

In order to understand the consequences of this variability at a given site, it is useful to employ a bounded analysis for the Stability Graph method by tabulating reasonable ranges for the input parameters (limit ranges to one tabulated category or one standard deviation for each parameter and only use variability as required or impractically large solution ranges will result) and then calculating an expected range for N'. Consider the following example input for the hangingwall of a mine employing a modified AVOCA mining method:.

Table 2.17.3: Data sheet for *parametric* design example

DEPTH	500m
STOPE HEIGHT	20m
NOMINAL PANEL WIDTH	30m
WALL STRESS	20 - 30 MPa
HW DIP	65 degrees
HW ROCK	gneiss
UCS	120 - 180 MPa
JOINTS	2 + random; rough & planar; stained - slightly altered



PARAMETER	LOWER BOUND	EXPECTED	UPPER BOUND
RQD	70	75	80
Jn	6	6	6
Jr	1.5	1.5	1.5
Ja	2	2	1
A	0.3	0.4	0.5
B	0.2	0.3	0.4
C	5.5	5.5	5.5
N'	2.9	6.2	22

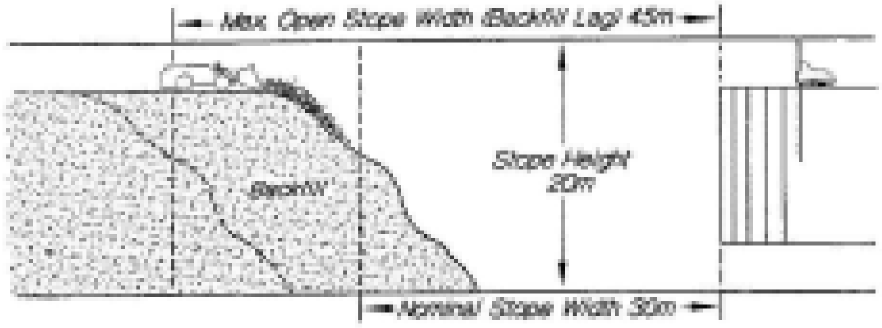


Figure 2.17.24: AVOCA Stope example - Fill Lag can be up to 1/2 stope width

Due to operational delays and scheduling problems, it is expected that the backfill front as illustrated in Figure 2.17.24 can lag behind the design position (relative to the blasting face) by as much as 1/2 of the nominal panel width. It must be considered then that the width can vary from 30m to 45m. The hydraulic radius, HR, must therefore be assumed to vary in the range:

From: $\dfrac{20 \times 30}{40 + 60} = 6$ To: $\dfrac{20 \times 45}{40 + 90} = 7$

The combined range of N' and of HR can be plotted on the Stability Graph as shown in Figure 2.17.25. Support is clearly required in this case. When worst case conditions occur, significant stability problems may result if support is inadequate. The decision to enhance support beyond average requirements must be based on risk to personnel and equipment and on the costs, losses and delays associated with unexpected dilution. This method can be expanded to involve probabilistic methods similar to those outlined in Hoek et al. (1995) and Harr (1987).

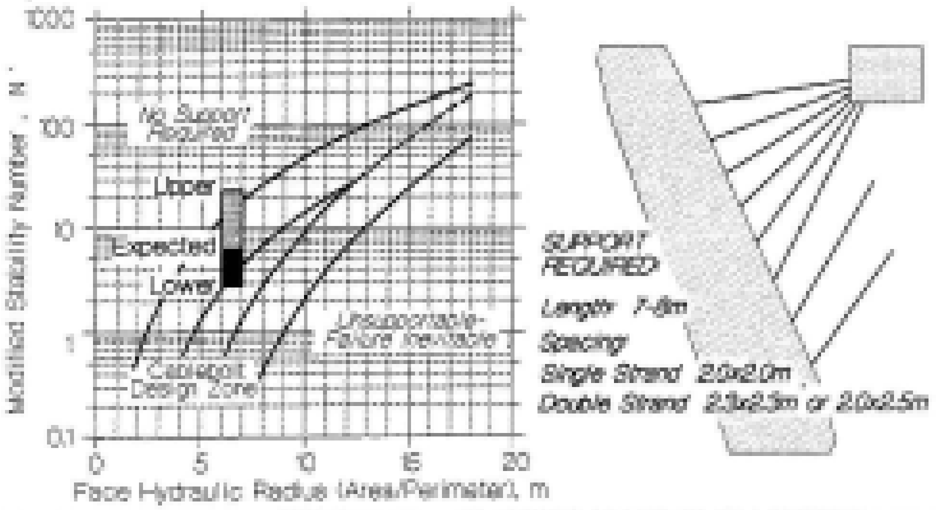


Figure 2.17.25: Example design range and recommendations

2.17.9 Probabilistic Analysis

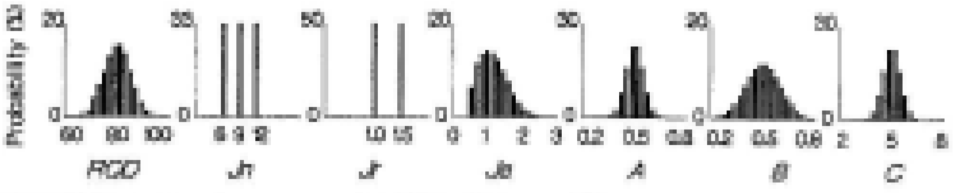


Figure 2.17.26: Example input distributions resulting from in situ variability

Another approach to incorporating input variability into the analysis is through the use of a probabilistic approach. Input parameters can be assigned distributions as shown in Figure 2.17.26. If a single recorded value for any parameter is sampled at random from the database, these histogram distributions show the relative likelihood of the sample equating to a particular value or range. Distributions can be obtained from real field data using statistical techniques (Harr, 1987; Pine, 1992; Rosenbleuth, 1981) or commercial simulation software (Hoek et al., 1995; Carter, 1992; Diederichs and Kaiser, 1996), and can be used in a Monte Carlo style analysis. In this analysis, a large number of calculations for N' are generated from different combinations of values for the above parameters. The frequencies with which each parameter falls within different ranges for use in the calculation are reflected in the distributions in Figure 2.17.26. Several hundred such calculations result in the distribution for N' shown in Figure 2.17.27 a). If this distribution is superimposed on the instability limits at a given *HR*, the *probability of instability* or of *caving* (Figure 2.17.27.b) is equal to relative area of the distribution which falls below the respective limit.

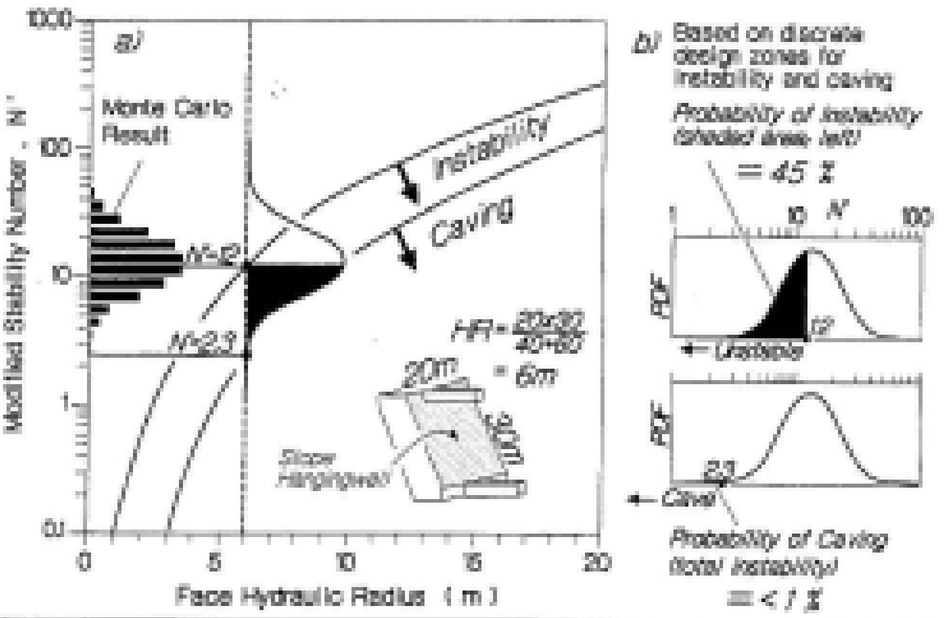


Figure 2.17.27: Probabilities of (b) instability and caving based on (a) Monte Carlo analysis (after Diederichs and Kaiser, 1996)

2.17.10 Dilution and the Stability Graph

The instability and caving limits in the Modified Stability Graph are based loosely on the apparent area of instability across the stope face. If the volume of failure is considered and divided by the volume of the ore in the stope, a value for dilution is obtained (Section 1.2). For a simple rectangular geometry, and if the stope thickness does not change, it is possible to plot contours of expected average dilution on the Stability Graph (Figure 2.17.28). Note that these contours are likely to be site-specific and depend on the stope thickness (5m in the example below). Based on local site experience, a *dilution vs HR* relationship for any rock quality *N'* can be obtained and used in economic analyses to optimize stope dimensions (Elbrond, 1994; Planeta et al., 1990; Diederichs and Kaiser, 1996).

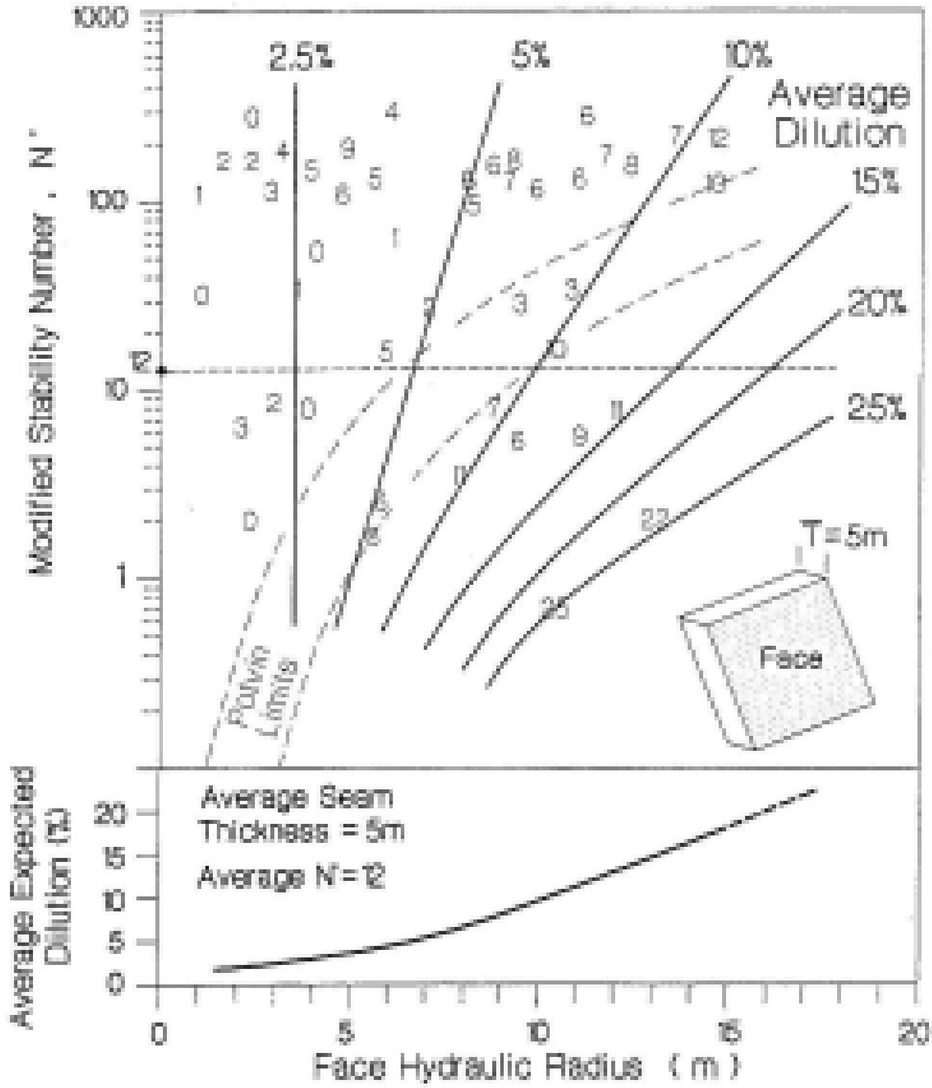


Figure 2.17.28: Site-specific average expected dilution (data from Pakalnis et al., 1995)