SPAN DESIGN FOR ENTRY-TYPE EXCAVATIONS

by

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ABSTRACT

Underground entry-type excavations require higher factors of safety than do non-entry excavations yet not as high as those required for permanent underground structures. A review is made of underground excavation span design techniques and the conditions under which they can be applied. Shortcomings of these existing methods, as they are applied to cut and fill stopes and other entry-type excavations, are highlighted.

A design procedure specific to conditions found in entry-type mining is proposed. At the centre of the procedure is an empirical span design chart, called the "Stability Graph for Entry-Type Excavations", which provides a practical tool for mining engineers to design stable entry-type excavations. The development of this chart and its use as a design tool is a result of the statistical analysis of 172 stoping case histories collected at a large underground gold mine in northern Ontario.

The influence of artificial support in maintaining stability and increasing span is investigated. A report is given of a trial support program carried out at the same operation using a concentrated pattern of cable bolts to replace a post pillar in order to increase span.

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ACKNOWLEDGMENTS

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1. INTRODUCTION

In 1989, Placer Dome Inc.'s Detour Lake Mine undertook a major research focus in developing "Design Guidelines for Cut and Fill Stopes" in conjunction with CANMET. These guidelines make specific reference to optimum stope dimensioning, ground support, mine sequencing, and pillar extraction. This thesis will focus on the span design portion of this research project and the role that support can have in increasing the allowable span of entry-type excavations in general.

In designing spans for entry-type excavations, there are two limiting constraints which influence the design. First, the nature of entry-type mining is such that workers are exposed to freshly blasted ground, unlike non-entry stopes. Therefore, higher safety factors are required for the design of entry-type stope spans. Secondly, profitable mining often demands the maximum extraction of the ore, which is achieved by maximizing the spans between pillars. In addition, stope excavations are required for only a short duration and therefore the high safety factors which would be used for permanent underground civil engineering structures would be difficult to justify. This thesis will attempt to reconcile these conflicting design objectives by providing for the mining engineer a practical design tool developed specifically for spans in entry-type excavations.

1.1 BACKGROUND

In recent years, entry-type mining methods such as cut and fill, room and pillar, and shrinkage stoping have been replaced in many mining operations by lower cost, non-entry, bulk mining methods. In many mines, however, the nature of the orebody is such that more selective, entry-type mining methods are still desirable. In 1989, cut and fill stoping and other entry-type mining methods still accounted for 37.1% of the total tonnes of ore extracted from underground metal mines in Canada (CMJ, 1990). Over the last 50 years, Canadian mines have pioneered many innovations in cut and fill mining technology, including rock fill, cemented fill, undercut and fill, and post pillar cut and fill mining (Singh et al., 1980). This need for innovation will certainly continue as existing orebodies become depleted and mining reaches greater depths.

Improved design procedures developed particularly for entry-type mining methods can result in three major benefits for mining operations:

- improved worker safety;
- increased ore recovery; and
- reduced dilution.

In its presentation to the Provincial Inquiry into Ground Control and Emergency Preparedness in Ontario Mines in 1985, the Ontario Ministry of Labour provided statistics on mining related injuries over the twenty-two year period ending in 1984 (Stevenson, 1986). The statistics indicate that falls of ground are the single highest cause of death in the mining industry in Ontario. Two-thirds of these fatalities occur while scaling, drilling, or from falling pieces of loose. These categories are predominantly associated with work tasks at a freshly blasted face, as are encountered with-entry type mining methods. The trend towards bulk mining techniques, as well as mechanized scaling, bolting, and drilling in entry-type stopes is likely to reduce the accident frequency in coming years.

Improved design procedures and the use of alternative support measures can increase ore recovery in entry-type stopes. In cut and fill stopes for example, post pillars are commonly left in the ore as a means of support. This report will show how the use of cable bolts was successful in maintaining support in a cut and fill stope after the post pillar was mined out. In the future, the mining industry may face increasing social pressures to maximize extraction of the public resource they are licensed to exploit. Alternate support measures such as this may become more widespread if this is the case.

Improved excavation design, mining techniques, and support methods can contribute to reduced dilution in entry-type stopes. While dilution in entry-type stopes is usually lower compared to open stopes, in the event of large failures, considerable waste may have to mined before the stope can be rehabilitated.

There are no suitable methods for designing large open spans for entry-type stopes in jointed rock. Beam and plate theories, Voussoir block analysis, and numerical models which are described in Chapter 3, have been employed in the past. In general, however, they have been adopted from the field of civil engineering and are restricted by homogeneous, isotropic, and linear elastic assumptions about the rock mass. More recently, an empirical design method has been developed for the design of spans in non-entry stopes and has gained widespread acceptance in the mining industry in Canada. This method would not be suitable for entry-type mining methods, since the definition of stable in a cut and fill stope is much more conservative then what is considered stable in a longhole stope. Other empirical methods have been proposed as general purpose span design techniques for a range of excavations from temporary mine openings to nuclear power stations. In general, however, they have been derived from a database consisting primarily of civil engineering case histories which require long term stability and higher safety factors than those required for entry-type stoping.

1.2 RESEARCH METHODOLOGY

The first phase of this research involved a questionnaire sent to underground cut and fill operators in Canada to determine what type of span and pillar design methods were being practiced in Canadian mines. From the returned questionnaires, it was evident that there is not a commonly accepted method used by mining engineers to design stable excavation spans. Undocumented rule-of-thumb approaches and past-practice plays a large part in the design procedure at most mines. The problem with these procedures is that the experience of mining one orebody is not readily transferable to other orebodies. The results of the questionnaire did suggest that an empirical design method which would quantify these rule-of-thumb approaches, would be the best design method for predicting conditions of stability under varying conditions of rock quality and stope geometry. Empirically based design methods are gaining increasingly widespread acceptance in the mining industry. Procedures have been developed for such areas as:

- open stope dimensioning;
- cable bolt support design;
- prediction of dilution in open stopes;
- prediction of stand-up time; and
- support requirements.

The second phase of the project involved collection of span, rock quality, and stability data from a large number of cut and fill stopes to establish the empirical database. Placer Dome Inc.'s Detour Lake Mine, as a co-sponsor of this research project, provided access to its operation for the purpose of gathering these measurements over the period from December, 1989 to March, 1992. In addition, the mine made available a large database of stope span, rock quality, and stability data gathered at the mine before the project began. This information has been compiled on a *Stability Graph for Entry-Type Excavations* which plots the design span versus rock mass rating. The data was analyzed statistically to define regions on the graph as stable, potentially unstable, or unstable. This graph provides for the mining engineer a practical means of designing stable spans for entry-type stopes. The design procedure recognizes the need for a comparatively low safety factor that is required for short-term underground excavations.

The role of support systems such as post pillars and cable bolts is assessed and their affect on span is studied. Post pillars have been used successfully at Detour Lake Mine and elsewhere to increase the overall span which can be mined before instability occurs. To achieve greater ore recovery and mining efficiency, the Detour Lake Mine sought to replace the support provided by the post pillar with cable bolt support. From a research perspective this work would provide a means of estimating the increase in span which can be made possible with artificial support. A trial support project undertaken by Detour Lake Mine and described in Chapter 7 demonstrated the effect of replacing a post pillar with a concentrated cable bolt pattern.

It is intended that the empirical span design procedure proposed in this thesis be used as part of an integrated design philosophy which also combines analytical procedures, numerical modeling, and engineering judgment. This design approach as it is now practiced at Detour Lake Mine will be described in further detail in Chapter 3.

2. THE DETOUR LAKE MINE

2.1 INTRODUCTION

Placer Dome Inc.'s Detour Lake Mine (DLM) began production in 1984 as an 1800 tonne per day open pit gold mine (Figure 2.1). The pit reached an ultimate depth of 130 metres in 1987, at which time production commenced from underground operations. Mining is carried out using mechanized cut and fill, longhole, and captive cut and fill techniques. Approximately 80% of production at the time of this study came from mechanized cut and fill stopes, 10% from longhole, and 10% from captive cut and fill stopes. Productivity improvements and increased milling capacity have boosted the production rate to 2200 tonnes per day. Current ore reserves stand at approximately 6.0 million tonnes, sufficient for another 6.5 years of production. The orebody has been proven to a depth of 660 metres below surface and is open along strike at depth. The mine is serviced by an all-weather road from Cochrane, Ontario, and a gravel air strip at the site. Most employees commute by bus to the mine and work a schedule of seven days in and seven days out.

2.2 GEOLOGY

2.2.1 Regional Geology

The Detour Lake Mine is located on the northwest rim of the Abitibi greenstone belt, which hosts a series of Archean felsic, mafic, and ultramafic tuffs, flows, and intrusions, as well as volcaniclastic and chemical sediments (Miller, 1988). The deposit lies on the north limb of an east-west striking anticline. The lithologies strike on azimuth 070° to 080° and dip 60° to 80° north. Most of the ore discovered to date is located at or adjacent to the contact between mafic and ultramafic rocks.

The hangingwall rocks located to the north of a so-called chert horizon are iron rich mafic volcanics with increasing potassic alteration closer to the chert. The rocks to the south of the chert are magnesium rich mafic and ultramafics which have been identified as chloritic greenstone and talc-chlorite schist. Figure 2.2 is a plan view of the lithologic units associated with the Detour Lake Mine orebody.

2.2.2 Orogeny

The chert may have been formed as a mylonite zone, a chemical sediment, or a deformed felsic intrusive. Irrespective of its origin, the chert marker horizon occurs along a break in the stratigraphy that establishes the boundary between plastically deformed rocks to the south and brittly deformed rocks to the north. Overall, the mineralization appears to have developed in a wide fault zone that displays sinistral movement. Vertical movement is reverse, with the north block moving up relative to the south. This fault zone subsequently served as a conduit for mineralizing fluids.

The faulting was probably a response to the regional stress field in the vicinity of the Detour Lake Mine. The faulting strikes approximately 232° and dips 45°-60° north. This coincides with the major principal stress direction, which at DLM has a measured azimuth of 257° and an inclination of 32° north. In the brittle hangingwall rocks, the faulting is manifested as small-scale folds and flexures in the chert and quartz veins. In the plastically deformed footwall rocks, it results in small-scale prolate boudins. Figure 2.3 is a simplified geological model of the rock in the vicinity of Detour Lake Mine illustrating the direction of shearing relative to the major principal stress direction.

2.2.3 Mine Geology

There are three interrelated gold bearing zones, namely the Main Zone, Quartz Zone, and the Talc Chlorite Zone. Figure 2.4 illustrates the relationship of these three zones on the 360 metre Level.

2.2.3.1 Main Zone

The Main Zone contains 72% of proven reserves. It strikes east-west and has an average dip of 60° north. The Main Zone is lens shaped, with widths of up to 45 metres in the centre and pinching down to under 5 metres at each end. Above the 560 metre Level horizon, the orebody plunges at 45° west and has an average strike length of 200 metres. Below this elevation, the plunge gradually flattens to horizontal and the strike length increases (Figure 2.5). Present drilling indicates that the ore bottoms out at approximately 760 metres below surface. The orebody is open along strike below this depth.

The most persistent ore bearing feature is the chert horizon which dips at 60° north. Vertically dipping quartz-sulphide veins splay off this chert into the hangingwall. Immediately adjacent to the chert, the quartz-sulphide veining is quite dense. The veining pinches out, and in some cases the grade decreases, with increasing distance from the chert. Dropping these veins from the mining limit as they become too far from the chert accounts for the stepped hangingwall of stopes as illustrated in Figure 2.6. The footwall contact is more regular on strike than the hangingwall contact but it undulates locally, with dips varying from 30° to vertical.

2.2.3.2 Quartz Zones

The ore in the Quartz Zones, which comprises 5% of proven reserves, exists as gold bearing veins which diverge north from the chert but continue to carry grade for long distances. The ore generally consists of three to five 10 cm thick veins with a combined average width of 3 metres. The Quartz Zones have an east-west strike, a dip of sub-vertical to vertical, and a plunge of 45° west. Mining of the Quartz Zone is carried up as part of the Main Zone using mechanized cut and fill for up to 50 metres along strike, depending on scheduling constraints. Whatever ore remains further along strike is extracted by longhole methods.

2.2.3.3 Talc Zones

The Talc Zones are located to the south of the Main Zone and comprise 23% of proven reserves. They are discontinuous in plan and section with dips undulating locally from 20° to 70° north. Mineralization occurs in a relatively weak talc-chlorite schist which requires more ground support than the Main Zone. Strong fault structures containing several feet of gouge material control the distribution of mineralization. Mafic intrusive dykes ranging in width from 0.3 to 5 metres commonly cut through the mineralization. These features combine to make geological control and mining of the Talc Zones difficult, however, this can be offset by the generally higher grade.

The Talc Zones are mined simultaneously with the Main Zone using mechanized cut and fill techniques if they are close enough and if it is economical to do so. If the Talc Zones are too far from the Main Zone or if they are too narrow, they may be mined using captive cut and fill methods.

2.3 MINING METHODS

2.3.1 Primary Development

The mine is accessed from the hangingwall side of the ore body by a three compartment shaft sunk to a depth of 615 metres. Main levels have been driven from the shaft at 100 metre intervals. Five main mining levels have been established, namely:

- 260 Level;
- 360 Level;
- 460 Level; and
- 560 Level.

In addition, a connecting ramp system extends from surface to the 660 Level. Between each main level, two sublevels are driven in the hangingwall at 30 metre elevation intervals to access the orebody (Figure 2.7). The Main Zone is accessed by means of two crosscuts, referred to as attack ramps, driven from the hangingwall drift. Broken muck is hauled up the attack ramps to a central orepass system which passes ore to the 430 Level, where it feeds a crusher located on the 460 Level. Below the 460 Level, a fine ore bin feeds a loading pocket from which ore is skipped to surface.

Mechanized cut and fill stoping accounts for approximately 85 percent of the mine's output. Longhole stoping of the Quartz Zones contributes 10 percent to the mine's output and captive cut and fill mining of some of the Talc Zones provides the remaining 5 percent.

2.3.2 MCF Stope Development

Mechanized cut and fill stopes were started on the 260, 360, 460, and 560 Levels. Attack ramps were driven from these levels at +3% to facilitate drainage. Typically, a stope is mined in two halves, each accessed by a separate attack ramp. As one half is mined, the other half is filled. In this way, a constant mining rate in each stope can be assured. When a lift has been completed, the attack is backslashed to provide access to the next lift. This continues until the attack finally reaches an inclination of 20%, at which point another attack is driven at minus 20% from the next level, 30 metres above (Figure 2.8).

Sill pillars are left beneath each of the stopes on the 260, 360, 460, and 560 Levels, and a crown pillar is maintained between the 260 stope and the pit. The ultimate pillar thickness is variable, depending on stope span, rock quality, and stress conditions.

2.3.3 Drilling and Blasting

Two boom hydraulic jumbos drill horizontal breasts 5 metres high and 3.6 metres deep. The face can be up to 35 m wide, depending upon ground conditions and ore limits. 45 mm diameter holes are drilled on a 1 metre square pattern. The holes are loaded with pneumatically placed ANFO initiated by Nonel detonators.

A number of ground problems experienced at Detour Lake Mine have been attributed to poor drilling and blasting practices. For this reason, particular emphasis is placed on drilling flat, parallel back holes for ground control purposes. Back holes are spaced 0.6 metres apart and loaded with 25 mm Trimrite cartridges to produce a decoupled charge. Figure 2.9 shows the location, loading, and sequencing of holes in a typical breast face at DLM. Technical details of cut and fill blasting at DLM are provided in Table 2.1.

2.3.4 Mucking

Five-yard LHD's and 26-tonne trucks are used to muck out stopes. The ore is trammed to ore passes which intersect the hangingwall ramp and pass the ore to the 430 Level, where it is transferred to the coarse ore bin for crushing. Each lift is mucked out to within 0.4 to 0.6 metres of the sand fill in order to maintain a good mucking floor for the equipment to work on. Before a stope is filled, the remaining muck is removed down to the fill level.

2.3.5 Ground Support

In accordance with the Ontario Ministry of Labour's Policy on Ground Support, the freshly blasted area is scaled and ground support is installed immediately after the area is mucked out. Methods of artificial ground support in use at DLM include:

- mechanically anchored rockbolts;
- Swellex bolts;
- cable bolts;
- wire mesh; and
- steel straps.

2.3.5.1 Mechanically Anchored Rockbolts

Mechanically anchored rockbolts are used almost exclusively in development headings, Quartz Zones, and the Main Zone. Rock in these areas has a compressive strength of approximately 165 MPa making bail and wedge anchors effective. Spalling of rock around the collar of the hole is not a problem in these zones. For these reasons, the mechanically anchored rockbolts, when used with a steel plate, provide excellent active support. The purposes of the mechanically anchored rock bolt are:

- To provide immediate support to potentially unstable blocks which cannot be removed by scaling; and,

- To support key-blocks at the immediate surface of the excavation which in turn provide geometric support to the overlying rock.

The technical specifications of the mechanically anchored bolts employed at DLM are provided in Table 2.2.

Rockbolting is carried out either with stopers operated from scissor lift vehicles or from a rockbolting jumbo. A standard 1.2 metre rock bolt spacing is used which was found through experience to be suitable for most of the DLM rock mass. In areas where joint spacing is as little as 0.3 metres, a 1.0 m square pattern has been applied.

Quality control on the installation of these bolts is maintained through routine torque testing by bolting crews and supervisors and pull testing carried out by the engineering department.

2.3.5.2 Swellex Rockbolts

Swellex rockbolts are often preferred in the Talc Zones because the low rock strength does not permit proper anchorage with a mechanically anchored bolt. Spalling of the drill hole collar often occurs, which also renders mechanical rockbolts ineffective. The Swellex bolt is a friction stabilizer, providing anchorage along the entire length of the bolt. This holds the rock together, reduces joint separation, and ultimately helps the rock mass to support itself geometrically. The standard 1.2 metres pattern is also used for Swellex bolts. Quality control on the installation of Swellex rockbolts is maintained by pull-out tests conducted by the engineering department.

Super swellex bolts, similar to the standard swellex, are installed in areas where a potential wedge has been identified and which cannot be supported by standard 1.8 metre bolts. Super swellex are manufactured from a thicker steel and have a larger diameter than standard swellex. The installation pattern is designed for the specific failure geometry they are being used to stabilize.

Technical specifications for standard and super Swellex bolts are provided in Table 2.3.

2.3.5.3 Cable Bolts

Cable bolting is an effective means of stabilizing and supporting rock masses too large for conventional bolts and for pre-supporting cut and fill stopes. Cable bolts can be cut to any length and include end-holding devices that allow for ease of installation in up-holes. A typical cable bolt is made from 16 mm diameter, 7 strand, stress relieved degreased steel cable, having an ultimate strength of 25 tonnes. Cable bolts are grouted with a 0.4 water:cement mixture by weight.

Detour Lake Mine has carried out a number of support trials involving regular steel cables, steel birdcaged cables, and fibreglass birdcage cables. The mine has also demonstrated the use of concentrated cable bolt support as a means of replacing post pillars for support of wide spans in cut and fill stopes. This work will be discussed in Section 7 of this report.

2.3.5.4 Wire Mesh

Wire mesh, commonly 5 cm x 5 cm galvanized chain link mesh or 10 cm x 10 cm weld mesh, is used in high traffic areas, shops, refuge stations, and areas of stopes with excessive small pieces of loose. The main purpose of the chain link mesh is to prevent injury to personnel or damage to equipment by containing small pieces of loose. In high traffic and work station areas where personnel and equipment are often present, the mesh is installed as a long term safety measure to contain loose which may fall over an extended time period. The mesh is generally used in conjunction with mechanical rockbolts. A wooden plate is inserted between the mesh and the steel rock bolt plate to prevent cutting of the screen.

2.3.5.5 Steel Straps

Steel straps are used to prevent joints or cracks from opening and to reinforce pillar corners. Due to the higher costs of this type of support, they are used on a limited basis and only where local conditions make them necessary. Locations of installation are generally a front-line supervisor's decision. Steel straps used at DLM are made of 6 mm thick steel, 100 mm wide and vary in length from 1.2 m to 2.4 m. The steel straps are pinned with mechanically anchored or resin/rebar bolts.

2.3.6 Backfilling

When a lift has been mined out to the ore limits, the remaining 0.4 to 0.6 metre layer of muck is scraped off the floor down to the backfill. Any waste muck mined on or near the level is then placed in the stope prior to placing the hydraulic fill. The hydraulically placed backfill is contained by building up a bulkhead with muck to a level slightly higher than the planned fill level and covering it with fabrene. The fill is normally placed to within one metre of the back.

The sand fill used by Detour Lake Mine is obtained from a nearby esker sand borrow pit on surface. Percolation tests and sieve analysis are performed on the sand in the pit to ensure an adequate percolation rate before it is excavated. Sufficient sand is stockpiled during summer near the backfill plant for use throughout the year. The backfill is mixed to 65% solids and delivered at a rate of 100 tonnes per hour. It is transported to the stopes from surface through a combination of drill holes and 10 cm diameter Sclairpipe. The backfill mixture drains quickly, having an average percolation rate of 80 cm/hr. No other mechanisms to assist drainage are required. The drain water is collected in small sumps where it is collected and drained through drill holes to a large sump on the 460 Level. It is pumped in a single stage into the open pit on surface. Table 2.4 summarizes the characteristics of the backfill sand used at DLM and Figure 2.10 shows a typical sieve analysis.

The first lift of each mechanized cut and fill stope was filled with a 10:1 sand to cement mixture to facilitate sill pillar recovery. Rebar and screen were placed in the fill for added strength.

Table 2.1 Drilling and Loading Specifications for Cut and Fill Breasting

Boreholes	
Diameter	45 mm
Length (Drilled)	3.6 m
Length (Loaded)	3.4 m
Burden	1.0 m
Spacing	1.0 m

Explosives
ANFO (Nilite) for production holes
25 mm emulsified ANFO (Trimrite) in perimeter holes
Maximum Number of Holes per Delay = 20
Maximum Weight of Explosives per Delay = 105 kg

Table 2.2 Technical Specifications of Mechanically Anchored Rockbolts

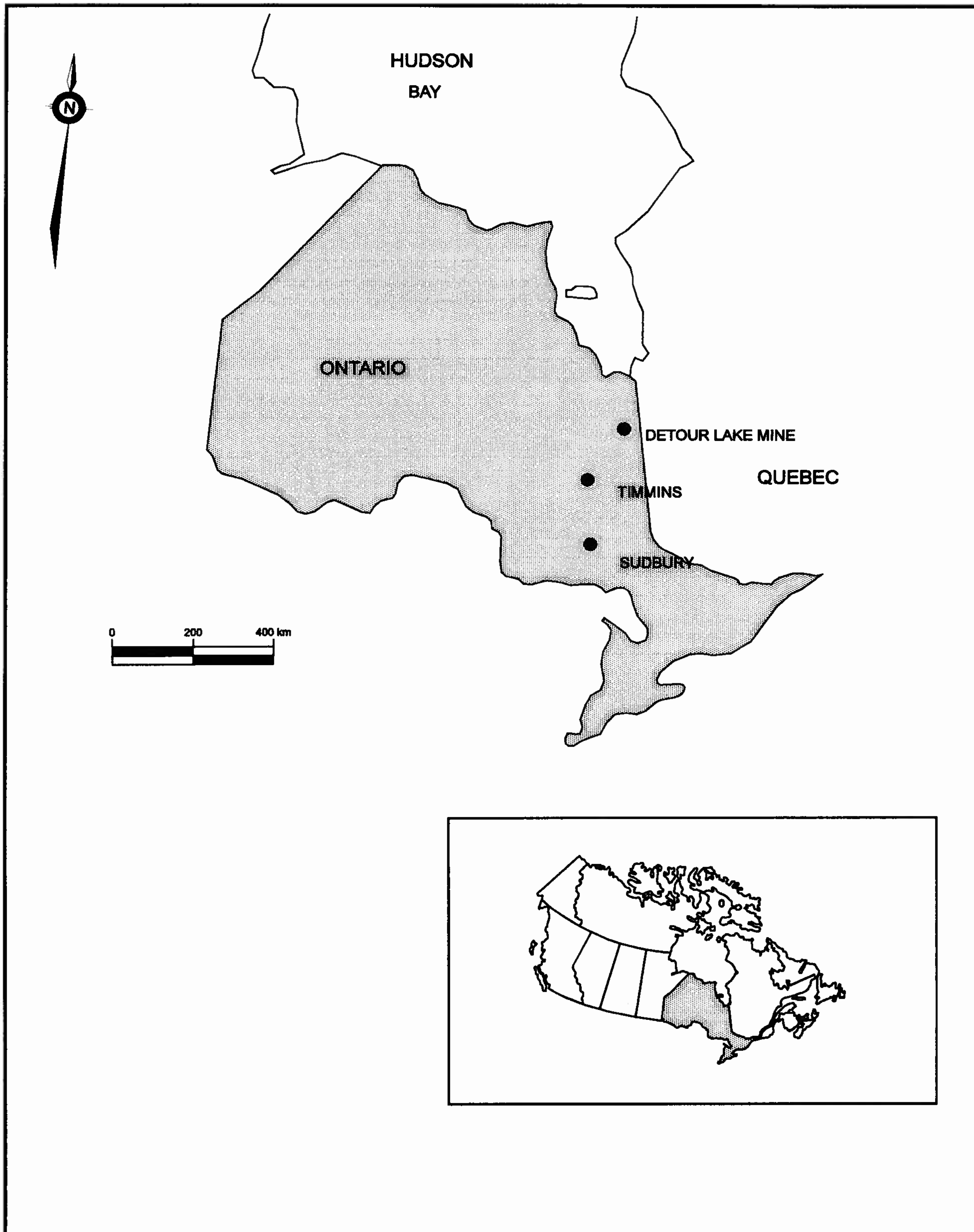
Type	LH Thread, Bail Type shell, forged head or threaded both ends, ASTM-F43Z-83 Standard
Steel Diameter	16 mm
Yield Load	12.5 tonnes
Ultimate Loads	16.3 tonnes
Bolt Lengths	1.8 m, 2.4 m, 3.0 m

Table 2.3 Technical Specifications of Swellex Bolts

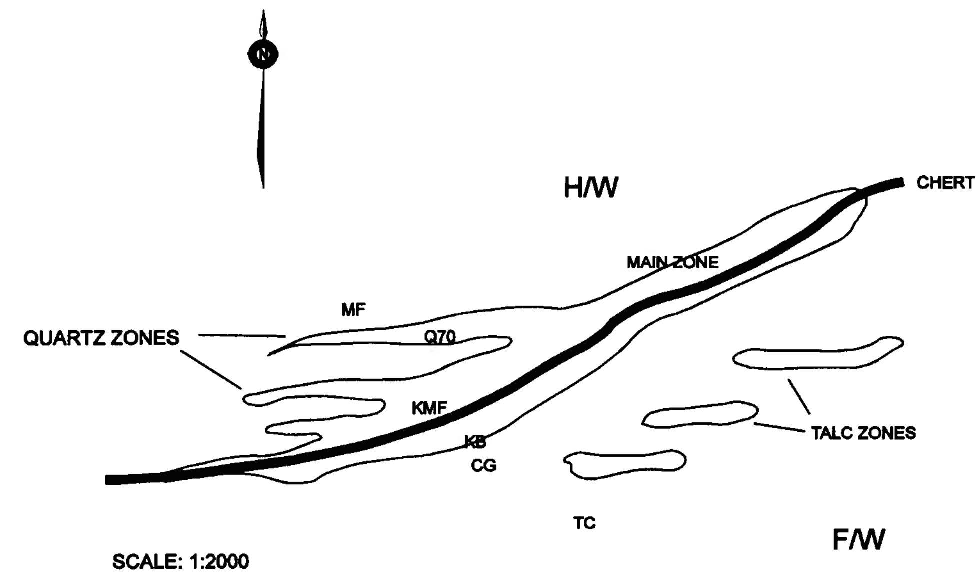
	Standard	Super
Tube Diameter	26 mm	52 mm
Yield Load (tube)	12 tonnes	24 tonnes
Ultimate Load (tube)	12 tonnes	24 tonnes
Bolt Lengths	1.8 - 6.0 m	3.6 - 6.0 m

Table 2.4 Detour Lake Mine Backfill Properties

Saturated Unit Weight	2,660 kg/m^3
Pulp Density	65%
Porosity of Settled Fill	0.38
Percolation Rate	0.8 metre/hour



LOCATION MAP - DETOUR LAKE MINE	FIGURE 2.1

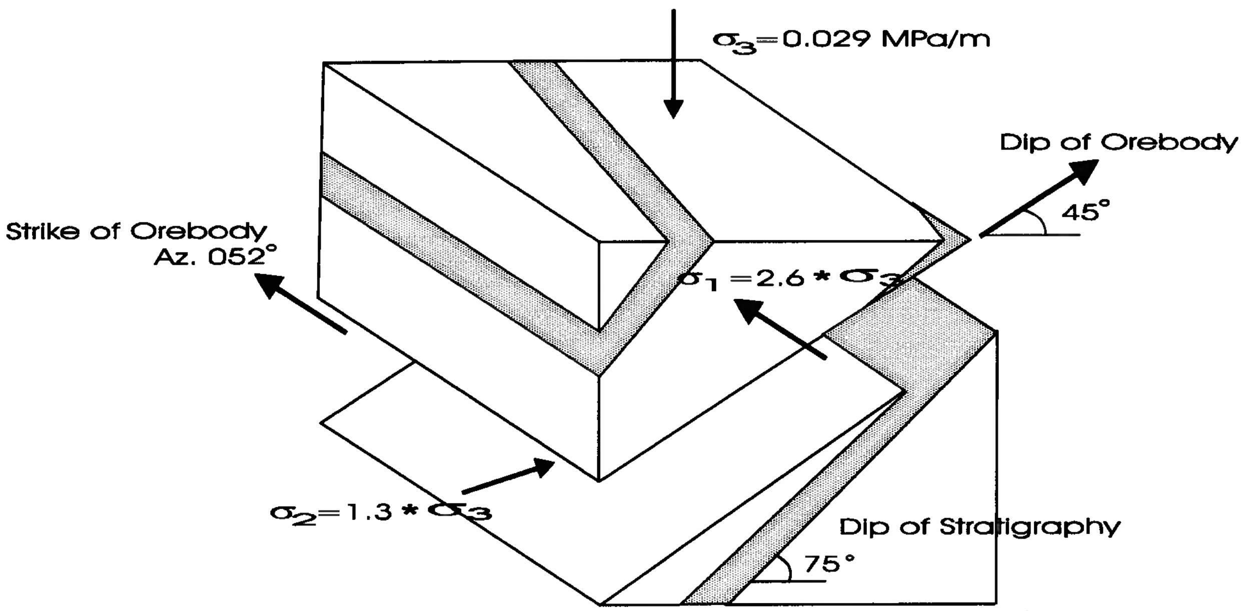


LEGEND

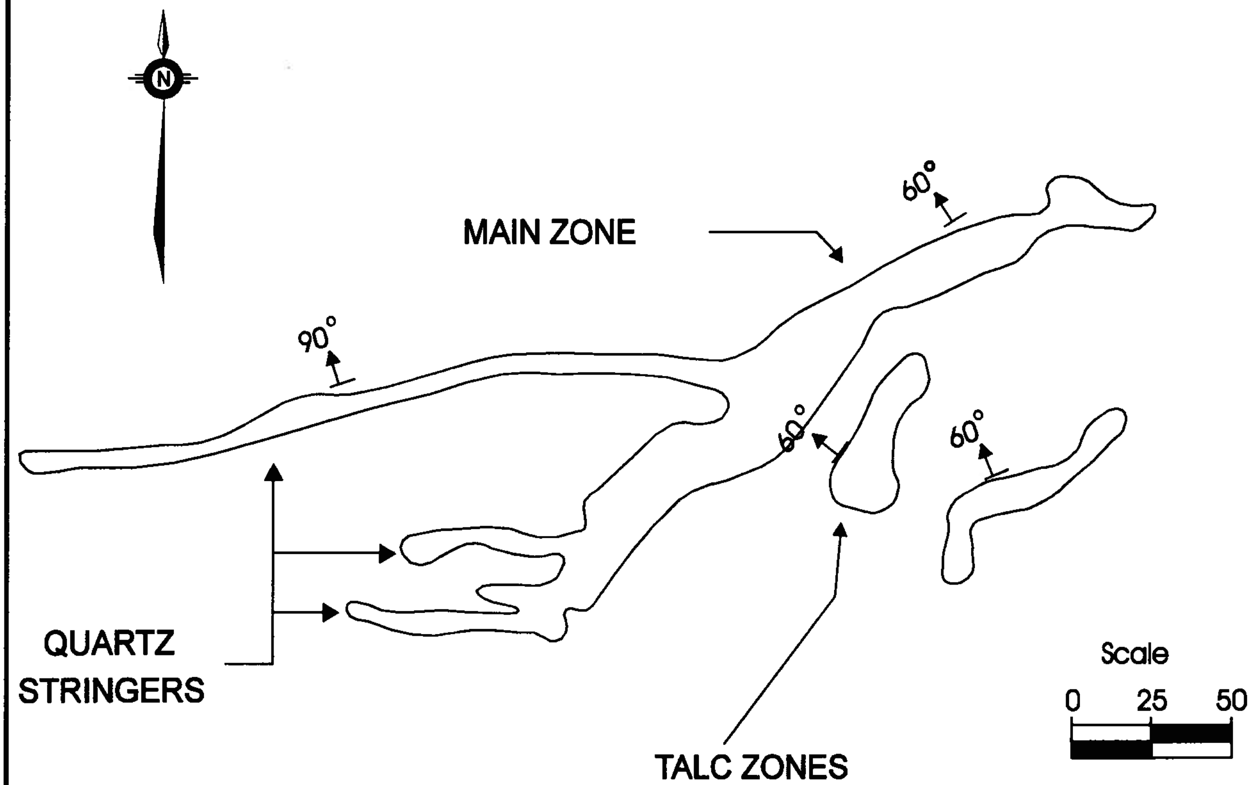
HW	MF = IRON RICH MAFIC VOLCANIC FLOWS	}	THOELIITIC BASALTS
	KMF = POTASSIUM ALTERED MAFIC VOLCANIC FLOWS	}	
	CHERT		
	KB = KOMATITIC BASALT	}	MAGNESIUM RICH MAFIC AND ULTRAMAFIC VOLCANICS
	CG = CHLORITIC GREENSTONE	}	
FW	TC = TALC CHLORITE	}	

PLAN VIEW OF LITHOLOGIC UNITS
DETOUR LAKE MINE

FIGURE 2.2

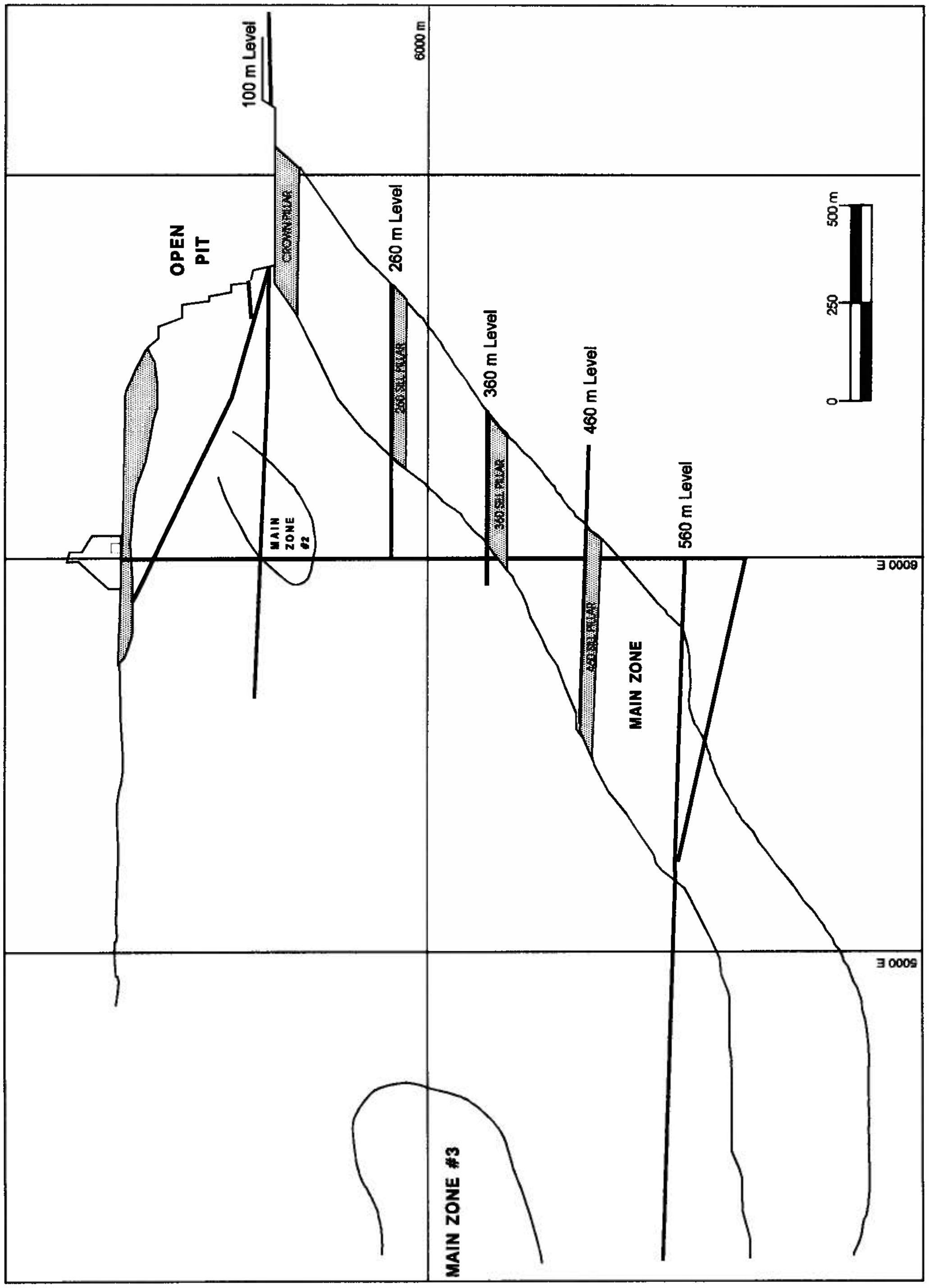


STRUCTURAL GEOLOGY MODEL AND ORIENTATION OF PRINCIPAL STRESSES AT DETOUR LAKE MINE	FIGURE 2.3

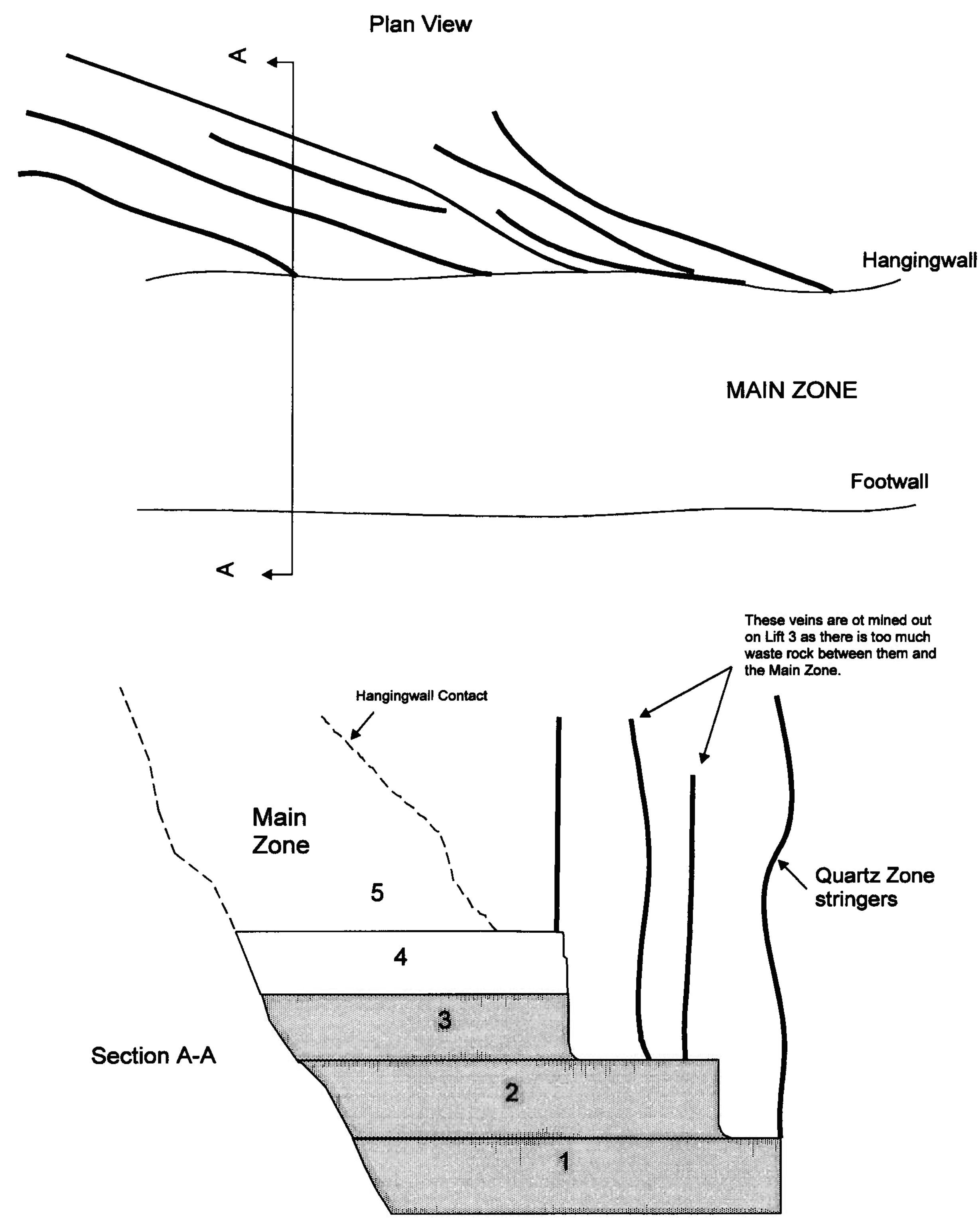


PLAN VIEW OF ORE ZONES
DETOUR LAKE MINE

FIGURE 2.4

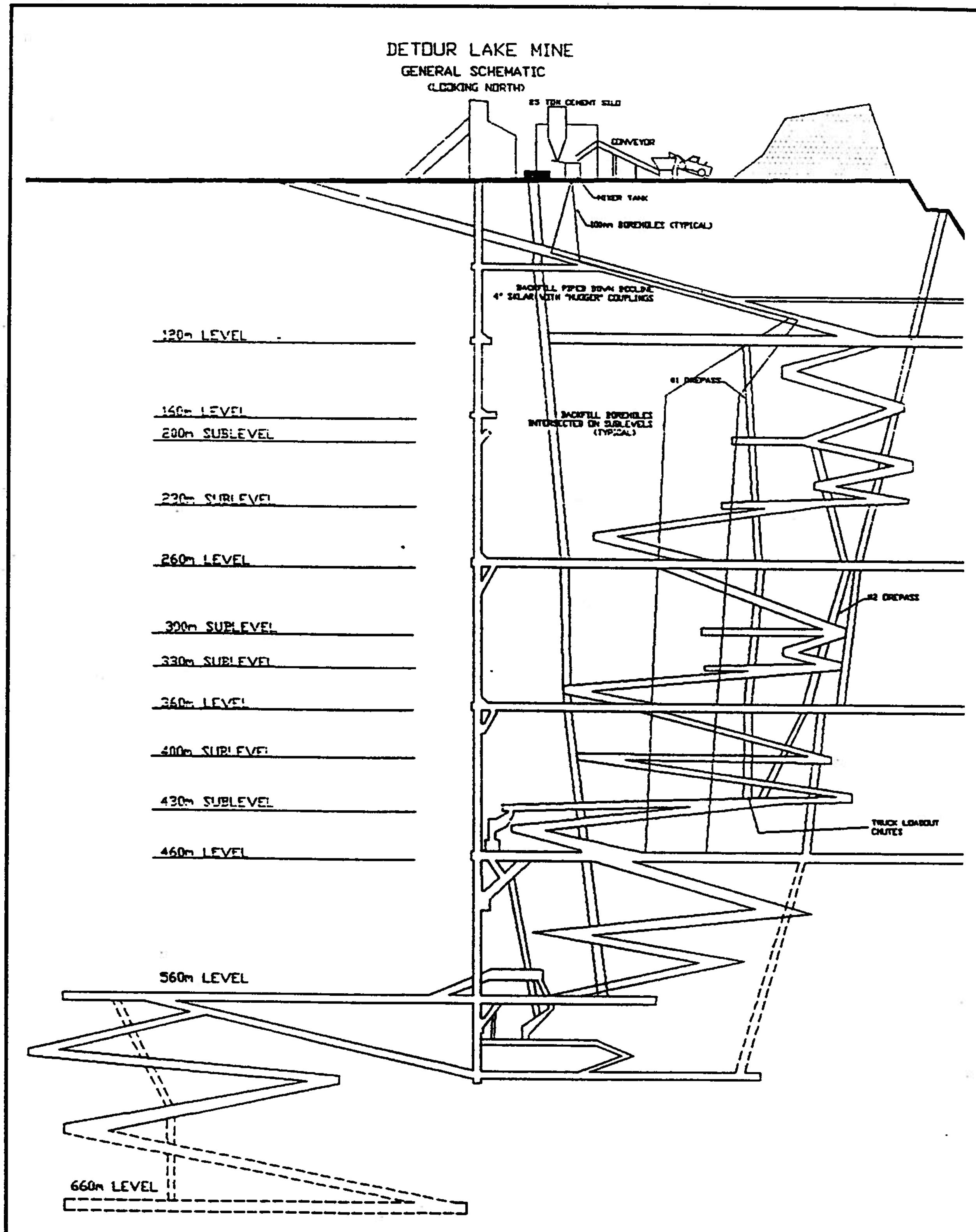


LONGITUDINAL SECTION OF MAIN ZONE DETOUR LAKE MINE LOOKING NORTH	FIGURE 2.5

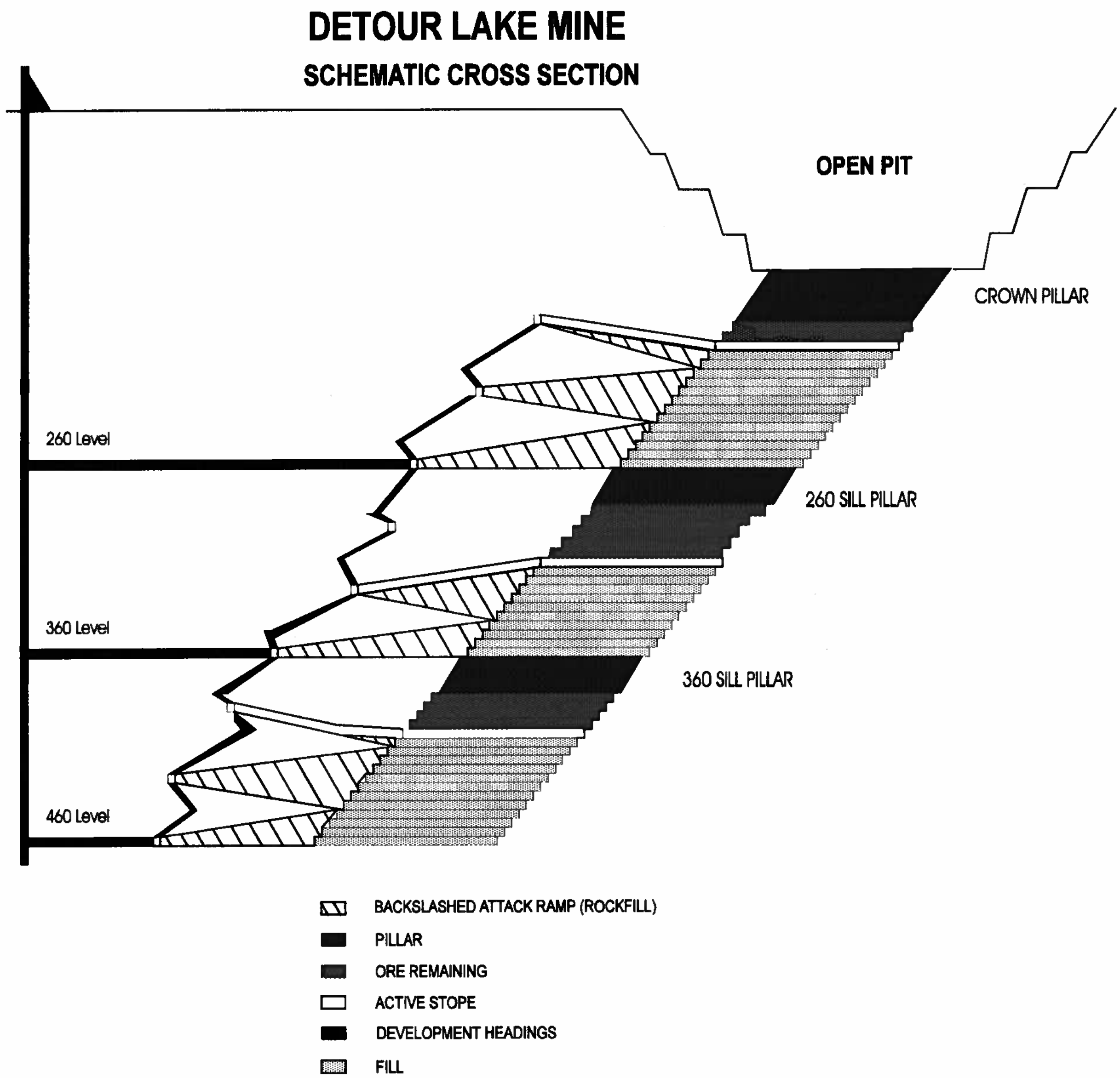


STEPPED HANGINGWALL CONTACT OF MAIN ZONE
DETOUR LAKE MINE

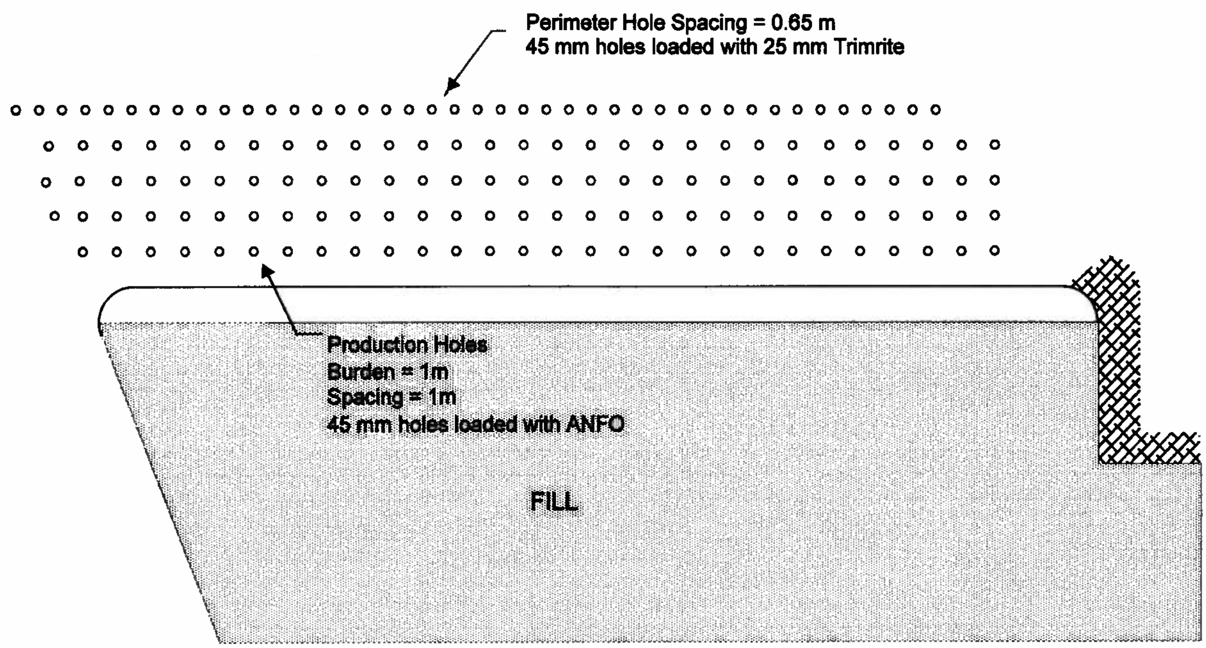
FIGURE 2.6



PRIMARY DEVELOPMENT DETOUR LAKE MINE	FIGURE 2.7

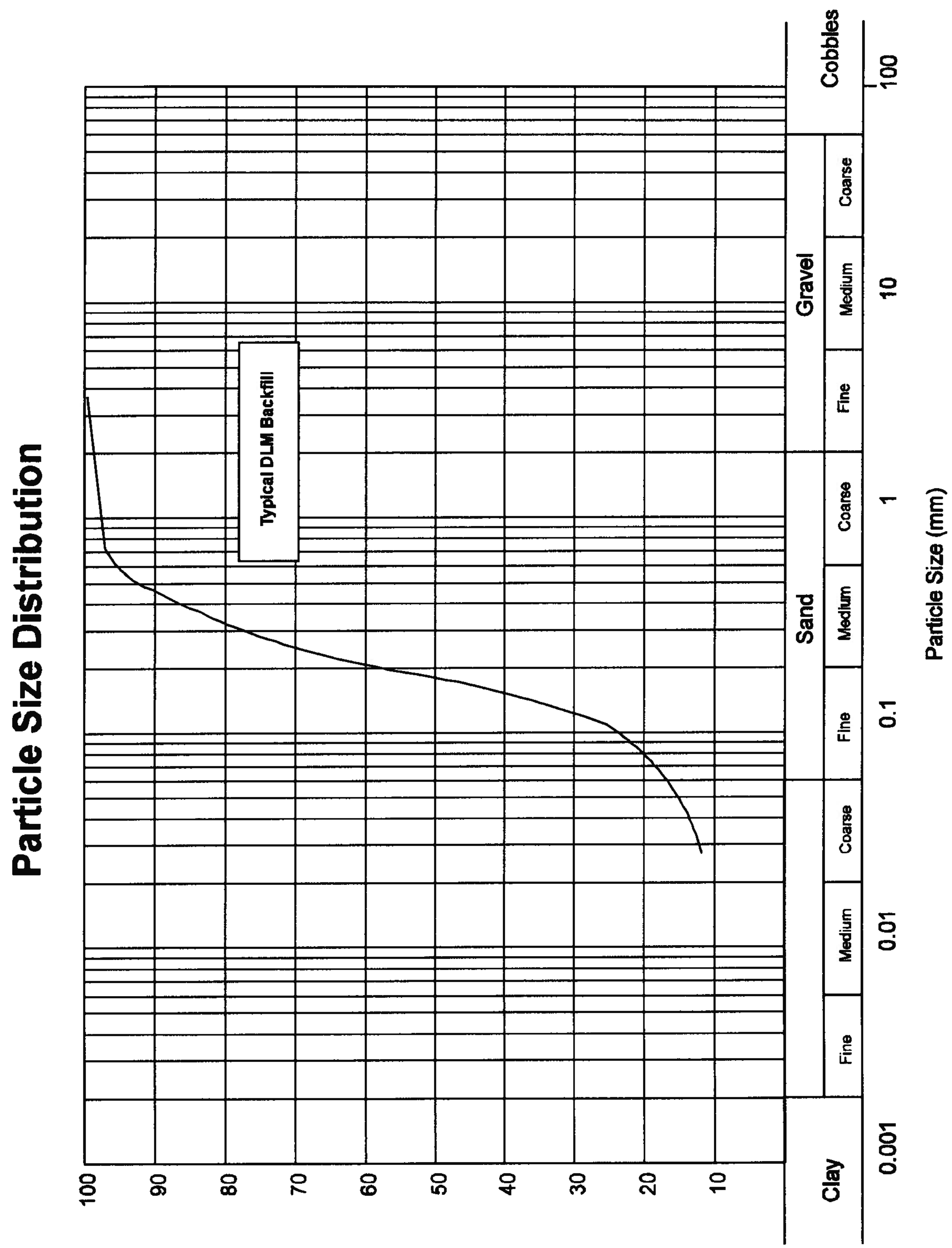


MECHANIZED CUT AND FILL STOPE DEVELOPMENT SCHEMATIC VIEW DETOUR LAKE MINE	FIGURE 2.8



CUT AND FILL BREAST DRILLING AND LOADING
DETOUR LAKE MINE

FIGURE 2.9



BACKFILL GRAIN SIZE DISTRIBUTION
DETOUR LAKE MINE

FIGURE 2.10

3. REVIEW OF DESIGN METHODOLOGIES

3.1 LITERATURE REVIEW

The design of underground structures is a relatively recent practice compared with the time man has been mining underground (Obert, 1973). The first types of design were simple rule-of-thumb approaches which are practiced even to this day. Requirements for greater mining efficiency and higher safety standards have made necessary a more reliable and effective approach to the design of underground structures.

Many design methods have been developed over the years and they can be classified into three categories:

- Analytical Approximations;
- Numerical Simulations; and
- Empirical Methods.

Analytical approximations include closed form solutions, limit equilibrium techniques, photoelastic modeling, and physical modeling. Analytical methods usually involve gross simplifications of the excavation geometry and rock properties. These simplifications can place severe restrictions on their application to real mining problems.

Numerical simulations, based on finite difference, finite element, boundary element, or distinct element methods, are gaining increased usage thanks to the availability of software and the low cost for personal computers. Numerical simulations are becoming increasingly sophisticated with 3-D modeling packages now commercially available at reasonable cost.

Empirical design methods, which involve the application of knowledge based on documented experience with similar mining conditions, are gaining increased acceptance in the mining industry. This requires a database of observations relating the stability of the underground structures to mine geometry, the rock mass characteristics, and other factors which influence stability. Empirical methods have been made possible in part by widespread acceptance of rock mass classification systems.

These design methods will be discussed in detail as they are applied to the six failure modes which account for instability of underground openings namely:

- beam or plate failure;
- Voussoir block failure;
- wedge failure;
- chimney failure;

- rock mass failure; and
- stress induced failure.

These failure mechanisms are illustrated in Figure 3.1.

3.1.1 Beam and Plate Failure

Beam and plate failure analyses assume the rock mass behaves as an elastic beam or plate. The analysis methods have been adapted from civil engineering solutions for bending of homogeneous, isotropic, and linear elastic materials such as concrete. Obert *et al.* (1967) provide a good treatment of beam and plate failure analysis. In applying this type of analysis to the stability of underground structures, the following simplifying assumptions must be made:

- In the case of beam failure, the strike length of the opening must be twice the width (beam span); and in the case of plate failure analysis, the strike length of the opening must be between 0.5 and 2 times the width;

- The rock must be hard, massive, and free of jointing to a degree that it is reasonable to consider it as homogeneous, isotropic and linear elastic;

- The beam must be continuous with the stope walls so the beam ends are considered to be fixed;

- In the case of beam bending, no loads are applied along the strike (plane strain condition); and,

- The beam is considered to have a uniform thickness.

3.1.1.1 Beam Failure

The two potential failure modes for beams are tensile (flexural) failure and shear failure, as indicated in Figure 3.2.

(a) Tensile Failure

For a horizontally layered roof, subjected to gravity loading, the maximum tensile stress is given by:

$$\sigma_{\max} = \frac{\gamma_a S^2}{2t} + \frac{pS^2}{2t^2} \tag{3.1}$$

where,

σ_{max}=maximum tensile stress in beam
S = span of roof layer
p = any uniformly distributed load (i.e. a filled stope above)
γ_a = adjusted unit weight of the lowest strata

t = thickness of roof layer

The adjusted unit weight (γ_a), of the lowest strata used in the formula above to account for the weight of overlying strata, is calculated as follows:

$$\gamma_a = \frac{E_1 t_1^2 (\gamma_1 t_1 + \gamma_2 t_2 + \gamma_3 t_3 + \ldots + \gamma_n t_n)}{E_1 t_1^3 + E_2 t_2^3 + E_3 t_3^3 + \ldots + E_n t_n^3} \qquad (3.2)$$

where,

E_n = Young's Modulus of nth layer
γ_n = unit weight of nth layer
t_n = thickness of nth layer

Setting $\sigma_{max} = R_o$, where R_o is the Modulus of Rupture (outer fibre tensile strength), and rearranging, the following span design equation for shear failure of horizontal strata can be derived:

$$S = \sqrt{\frac{2t(R_o - \sigma_o)}{F_s(\gamma_a + p/t)}} \qquad (3.3)$$

where,

R_o = modulus of rupture
F_s = factor of safety

(b) Shear Failure

When the ratio of strata thickness to span exceeds approximately 0.2, shear failure begins to dominate over flexural failure (Obert *et al.*, 1967). For a horizontally layered roof, subjected to gravity loading, the maximum shear stress is given by:

$$\tau_{max} = \frac{3\gamma S}{4} \qquad (3.4)$$

The shear strength τ of the beam is given by:

$$\tau = c' + \sigma_n(\tan\phi') \qquad (3.5)$$

where,

σ_n = horizontal compressive stress
c' = cohesion on the plane of shear acting over the compressed zone
ϕ' = friction angle on the plane of shear

By rearranging these equations, and defining the factor of safety F_S as the ratio of shear strength to shear stress, the maximum allowable span to resist shear failure of the back in horizontally bedded strata is given by:

$$S = \frac{4(c' + \sigma_n \tan\phi')}{3\gamma_a F_s} \quad (3.6)$$

3.1.1.2 Plate Failure

In cases where the orebody has a strike length to width ratio of between 0.5:1 to 2:1, the back should be treated as a slab spanning two directions with fixed support on all four sides (Figure 3.3). The maximum tensile stress for such a plate occurs at the centre of the long edge. The maximum tensile stress for bending of such a plate is given by:

$$\sigma_{max} = \frac{\beta q S b}{t^2} \quad (3.7)$$

where,

β = coefficient which varies with the span ratio (See Figure 3.3b)
S = short span of plate
b = long span of plate
t = thickness of plate
q = loading on plate per unit area

Setting σ_{max}=R_o, the maximum stable span is given by:

$$S = \frac{\sigma_{max} t^2}{\beta q b F_s} \quad (3.8)$$

Shear failure of the plate is analogous to chimney failure, which will be discussed later.

3.1.2 Voussoir Block Failure

3.1.2.1 Voussoir Beams

(Evans, 1941) was the first to consider analyzing stope backs as discrete blocks as in a masonry or Voussoir arch. It has been recognized since Roman times that arching can greatly increase the load bearing capacity of a beam. The Voussoir beam model was modified by Beer and Meek (1982) and is illustrated in Figure 3.4. The concept conveyed in this figure is that the line of lateral thrust within such an arch, when traced on the beam span, approximates a parabolic arch.

Voussoir beam theory makes the following assumptions about the rock mass being analyzed:

- The rock mass is assumed to be cut by linear discontinuities trending along strike, such that the back can be assumed to be composed of discrete blocks;

- It is assumed that there is no horizontal compressive stress in the back transferred from the surrounding rock; and,

- No tensile strength develops between individual blocks (c=0).

3.1.2.1 (a) Analysis Procedure

Because the solution to the problem is indeterminate, two assumptions are required for the analysis. First, the line of thrust is assumed to be parabolic, as mentioned; and secondly, the load distribution at the centre of the beam and the abutment contact is assumed to be triangular (Figure 3.4(b)). The triangular end load operates over a length *nt* where,

$$n = 1.5\left(1 - \frac{z}{t}\right) \tag{3.9}$$

where,

n = lateral load to depth ratio
z = arch height
t = beam thickness

Applying moment equilibrium around the centroid of the half beam yields:

$$\frac{\gamma}{8} tS^2 = \frac{f_c ntz}{2} \quad \text{or,} \quad f_c = \frac{\gamma S^2}{4nz} \tag{3.10}$$

where,

f_c = the horizontal compressive stress at the centre of the beam
γ = unit weight of beam
S = horizontal span of beam

Assuming that the shape of the thrust arch acting in the beam is parabolic, the arc length L, can be expressed by:

$$L = S + \frac{16z^2}{3s} \tag{3.11}$$

where,

L= arc length of parabolic thrust profile
z = height of arch
S = horizontal span

The resultant force acts through the centre of each force distribution, so the initial moment arm for f_c is given by

$$z_o = t - \frac{2nt}{3} \tag{3.12}$$

where,

z_o = initial moment arm of f_c

t = beam thickness

As the beam deflects, the arch goes into compression and shortens by a length ΔL. If the arch height z_o is shortened in compression due to ΔL, the new moment arm (z) can be computed by:

$$z = \sqrt{\left(\frac{3s}{16}\right)\left(\frac{16z_o^2}{3S} - \Delta L\right)} \tag{3.13}$$

The incremental elastic shortening of the arch, ΔL is given by:

$$\Delta L = \frac{f_{av} L}{E} \tag{3.14}$$

where,

f_{av} = the longitudinal stress in the beam
E = Young's Modulus of the beam

The average longitudinal stress in the beam is now estimated by considering the stresses in only a quarter of the beam, as shown in Figure 3.4(c). At a distance *S/4* from the abutment, the stress distribution is uniform over the arch depth. The average longitudinal stress f_{av} for this quarter of the beam, and hence for the entire beam, is given by:

$$f_{av} = \frac{1}{2} f_c \left(\frac{2}{3} + \frac{n}{2}\right) \tag{3.15}$$

An explicit solution for the loading in the beam and beam deformation is not possible. An iterative procedure is required, which begins with assuming a value for the initial load to depth ratio, *n*. An initial value of $n=0.5$ will normally produce a stable solution. The procedure involves calculating sequentially f_c, f_{av}, L, ΔL, z, and n. The process is repeated with the load to depth ratio n, used to calculate f_c. Iterations continue until stable load to depth ratios are obtained.

(b) Failure Modes

Beer and Meek (1982), identified three possible failure modes for Voussoir arches (Figure 3.5):

- Crushing at the hinges formed in the upper portion of the centre of the beam and at the lower abutment contacts;
- Shear at the abutment when the limiting shear resistance T (tan ϕ) is less than the required abutment vertical reaction force V, (W/2); and,

- Buckling of the roof beam with increasing eccentricity of lateral thrust giving rise to a *snap through* mechanism.

Crushing or compressive failure is analyzed by comparing the maximum longitudinal compressive stress f_c to the uniaxial compressive strength of the beam. The factor of safety against compressive failure of the beam is then:

$$F = \frac{UCS}{f_c} \tag{3.16}$$

The factor of safety against shear failure is defined by the frictional resistance to shearing divided by the shear stress caused by the weight of the beam. The resistance to shearing is given by:

$$F = T\tan\phi = \frac{f_c nt(\tan\phi)}{2} \tag{3.17}$$

The abutment shear force (V) is:

$$V = \frac{W}{2} = \frac{\gamma St}{2} \tag{3.18}$$

The factor of safety (F_S) against shear failure at the abutments is given by:

$$F_S = \frac{f_c n}{\gamma S}\tan\phi \tag{3.19}$$

Buckling failure will occur when the moment arm *z* becomes negative; that is, when the centroid of the centre force distribution is lower than the abutment lateral force distribution. A check must should be made in the iteration procedure described above to determine if *z* is negative and, therefore, if buckling failure occurs.

3.1.2.2 Voussoir Plates

In cases where the span to length ratio is greater than about 0.5, plane strain conditions do not apply. It is necessary to consider the roof of the stope as a plate supported on four edges. Tensile cracks will develop on the lower surface of the plate along the lines of maximum tensile stress. Beer and Meek, (1982) suggest this pattern of cracking results in two triangular segments on each end of a rectangular plate, and two trapezoidal segments on the sides (Figure 3.6). For a square plate, four equally sized triangular segments would be formed. Referring to Figure 3.6, the shape of the segments is given by:

$$y = \frac{S}{2}\left[\sqrt{k^2 + 3} - k\right] \tag{3.20}$$

where,

S = short side of plate
b = long side of plate
y = height of triangular segment
k = width to height ratio

Clearly, since the weight and moment arms of the trapezoidal segments are greater, their behaviour will control overall roof stability.

(a) Analysis Procedure

The weight of the trapezoidal segment in Figure 3.6 is given by:

$$W = \frac{\gamma St}{2}(b - y) \tag{3.21}$$

The centroid is located at a distance *x* from the plate edge where,

$$x = \left(\frac{Sb}{b - y}\right)\left(\frac{1}{4} - \frac{yk}{3S}\right) \tag{3.22}$$

Applying moment equilibrium about the centroid yields:

$$\gamma S^2 bt\left(\frac{1}{4} - \frac{yk}{3S}\right) = \frac{f_c ntbz}{2} \qquad \text{or,} \qquad f_c = \frac{\gamma S^2\left(\frac{1}{4} - \frac{yk}{3S}\right)}{nz} \tag{3.23}$$

The average longitudinal stress acting in the *x* direction as indicated in Figure 3.6 is given by Equation 3.14. In the *y* direction, f_{av} is approximated by:

$$f_{av}^{y} = \frac{7kf_c}{12} \tag{3.24}$$

Using equations 3.14 and 3.23, the elastic shortening of the arch caused by deflection of the plate is given by:

$$\Delta L = \frac{f_{av}L(1-k\nu)}{E} \tag{3.25}$$

where,

ν = Poisson's Ratio

In order to solve for the state of stress in the plate, an iterative procedure is followed similar to the procedure for Voussoir beams, except that Equations 3.9 and 3.13 are replaced by Equations 3.22 and 3.24 respectively. The rest of the procedure is identical.

(b) Failure Modes

The failure modes for plates are similar to those for a beam. Compressive or crushing failure will occur at the top of the plate at the centre of the span or at the lower side of the abutment contacts. The factor of safety is given by Equation 3.18.

The factor of safety against shear failure is defined by the frictional shear resistance due to the maximum longitudinal compressive stress, f_c divided by the shear stress due to the weight of the plate. It is calculated by:

$$F_s = \frac{f_c nb \tan\phi}{\gamma S(b-y)} \tag{3.26}$$

As with Voussoir beams, buckling will occur if the moment arm z becomes negative. A check should be made during the iterative process to determine if buckling occurs.

3.1.3 Structurally Controlled Failure

Structurally controlled failure (wedge failure) is a relatively common occurrence in underground metal mines. Wedges are delineated by intersecting discontinuity planes and the back or wall of an excavation (Figure 3.7). Failure can occur by sliding along one of the planes in the case of wedge on a wall or by fall-out from the back. The frequency, condition, and orientation of the jointing combined with the size of the excavation determine the size of potential wedges. The stress level around the excavation can also influence the stability of wedges; however, most design procedures assume the immediate back to be in a relaxed state.

3.1.3.1 Stereonet Analysis Techniques

Potential failure mechanisms can be analyzed quickly using a stereonet. A good introduction to the use of stereonets for this purpose is provided by Hoek and Brown, (1980). The first step is to determine the orientation of the dominant joints sets which are prevalent in the area of concern. In Figure 3.8(a) three joint sets are plotted on a lower hemisphere, equal angle stereonet. These joints form three release planes, and with the roof of the excavation, form a tetrahedral wedge. It can be seen that the vertical line through the centre of the stereonet lies inside the triangle created by the three great circles defining the joint sets. This condition indicates that a vertical free fall of the wedge is kinematically possible.

If the three great circles representing the joint sets intersect to form a wedge and the vertical line at the centre of the stereonet lies outside the triangle formed by the great circles, instability can only be possible by sliding along one of the discontinuities. In order for sliding to occur, the plane on which sliding takes place must be steeper than the angle of friction which is represented by a continuous circle, as shown in 3.8(b). If the entire triangle falls outside of this circle, the wedge will be stable. In the example shown, the friction circle intersects Joint Set A, so sliding will occur along this plane.

Sidewall failure can also be analyzed using stereonet projection. Figure 3.9(a) shows two joint sets and a 70° dipping wall. Since this line represents a wall on each side of the excavation, the failure modes on each side of the line must be assessed. Where two planes intersect in the wall of an excavation, sliding failure is possible if the plunge of the intersection is less than the dip of the wall and greater than the angle of friction. This condition is illustrated in Figure 3.9(a). On the northeast wall of the excavation, sliding will occur in the direction of the plunge of the intersection of Joints A and B.

Another useful procedure for determining if sliding will occur on a plane or on the line of intersection of the planes is discussed by Hocking, (1976). This procedure is illustrated in Figure 3.9(b). If the plunge of the line of intersection of two planes falls between the dip of the wall and the internal friction angle circle, and if the dip direction of either of the planes falls between the dip direction of the wall and the trend of the line of intersection, sliding will occur on that plane.

3.1.3.2 Computational Techniques

A vector analysis technique for determining the stability of underground wedges, published in Hoek and Brown, (1980) has been adapted into an underground wedge stability computer program, *UNWEDGE*, produced by the University of Toronto. This program enables the user to graphically input the excavation geometry, joint patterns, and joint strength properties for use in the analysis. The program can analyze wedges created by three intersecting joints in the roof or side walls of an excavation. The size and shape of the wedges can be displayed around the perimeter of the excavation. For a defined wedge, the program determines if the wedge is stable, falls out under gravity, or slides along one or two of the planes and computes factors of safety. An example of an *UNWEDGE* analysis is given in Figure 3.10. Such programs are useful for determining whether specific wedges, which have been identified in a stope, will be stable or unstable. They can also assist the ground control engineer in the design of artificial ground support systems.

Where the potential for wedge type failures is recognized, the span should be limited to control the maximum size of the wedge to one that can be supported with artificial ground support.

3.1.4 Chimney Failure

Chimney failure occurs when the entire sill pillar or crown pillar above a stope slides as a block into the stope (Figure 3.11). This type of failure is not common but has been responsible for some large scale failures in the past. Chimney failures occur in very schistose rock masses or in orebodies where the footwall and hangingwall are defined by weak discontinuities.

Hoek, (1989) has developed an equation for determining the factor of safety against shear on the sides of the failure block. The factor of safety against downward sliding is given by:

$$F = \left(\frac{2}{\gamma_r}\right)\left(\frac{\tau_{xz}}{y} + \frac{\tau_{yz}}{x}\right) \tag{3.27}$$

where,

γ_r = unit weight of rock
x = length of end wall
y = length of sidewall
z = depth of block
τ_{xz} = shear strength along end wall
τ_{yz} = shear strength along side wall

and,

$$\tau = c + (\sigma - u)\tan\phi \tag{3.28}$$

where,

c = cohesion along shear plane
σ = horizontal stress
u = groundwater pressure

Using this approach, chimney failure is defined to occur when the weight of the pillar exceeds the total shear strength developed on the sides of the block. The shear strength used in this equation is the shear strength along the sliding plane, which is a function of the cohesion, friction angle, horizontal stress, and water pressure along the shear plane.

3.1.5 Rock Mass Failure

General rock mass failure or caving is characterized by a gradual failure of loose rock into the stope. Given sufficient time, the failure could continue to cave until the void is filled or it could stop when a stable shape has been created by the caving. Figure 3.12 illustrates these two conditions. Clearly, the susceptibility of a stope to a rock mass failure is dependent upon many factors, the most important of which are:

- joint spacing;
- joint orientation;
- joint condition;
- groundwater conditions;
- stress conditions;
- excavation geometry; and
- rock hardness.

Empirical design techniques are the only methods available for analyzing the susceptibility of a rock mass to caving. All empirical methods rely on rock mass classification systems which attempt to quantify the rock mass parameters which contribute to weakness. Classification systems have been used to a limited extent in the past to predict stable spans, stand-up times, and support requirements in underground openings. Unfortunately, however, much of the data has been compiled from civil engineering case histories which require higher safety factors than mining operations. Other empirical design methods have been developed largely from open stoping (Potvin, 1988) or block caving databases (Laubscher, 1981). The two most common systems which have been used to develop span design graphs and which have gained broad acceptance in the mining industry are the Norwegian Geotechnical Institute (NGI)- Q Rating System and the South African Council for Scientific and Industrial Research (CSIR) Geomechanics Rock Mass Rating system.

3.1.5.1 NGI-Q Rock Mass Classification System

The NGI Tunneling Quality Index (Q) proposed by Barton, Lunde, and Lien (1974) is based on 200 tunneling case histories in Scandinavia. The tunneling index value Q is defined by:

$$Q = \left(\frac{RQD}{J_n}\right) \times \left(\frac{J_r}{J_a}\right) \times \left(\frac{J_w}{SRF}\right) \tag{3.29}$$

where,

RQD = Deere's Rock Quality Designation
J_n = Joint Set Number
J_r = Joint Roughness Number
J_a = Joint Alteration Number
J_w = Joint Water Reduction Factor

SRF = Stress Reduction Factor

The first quotient, RQD/J_n is a rough measure of the relative block size. The second quotient J_r/J_a represents the interblock shear strength. Rough, tight, unaltered joints will have higher shear strengths than smooth, open, and altered joints. The third quotient, J_w/SRF is a measure of the active stress in the rock mass. SRF can be a measure of the loosening load in the case of shear zones, a measure of the induced stress around the opening in the case of competent rock, or a measure of the squeezing or swelling load in plastic, incompetent rock. The factor J_w is a measure of the water pressure which reduces the effective shear strength of the joints. The ratings applied to individual parameters for the NGI Q system are provided in Table 3.1 of this report.

Barton *et al.* have also defined two other factors for relating the Tunneling Index (Q) to the span which can be supported.

$$D_e = \frac{\text{Excavation Span or Height (m)}}{\text{ESR}} \tag{3.30}$$

where,

D_e = Equivalent Dimension
ESR= Excavation Support Ratio

The excavation support ratio (ESR) is analogous to a safety factor and is dependent on the purpose of the excavation. The ESR ranges from 0.8 for underground nuclear power stations to 3-5 for temporary mine openings. A complete explanation of excavation support ratios is provided in Table 3.2. From the table, it can be seen that the ESR for temporary mine openings is based on only 2 observations. Therefore, great care must be exercised if this procedure is to be used for design of mine openings. Figure 3.13 shows the relationship between the Tunneling Index, Q and the equivalent dimension, D_e. This figure shows a sharp line dividing the zone requiring support from the zone requiring no support. The equation of this line is given by:

$$\text{Maximum Unsupported Span (m)} = 2(\text{ESR})Q^{0.4} \tag{3.31}$$

In practice, there is a zone of potential instability which is not easily defined. Figure 3.12 has been modified in Figure 3.14 to show span versus Q for the excavation support ratios 3 and 5 defined to be the upper and lower bounds for temporary mine openings.

3.1.5.2 CSIR Rock Mass Rating

The geomechanics classification system developed by Bieniawski (1976) is a general purpose rock mass classification system which has been used to predict stable spans, stand-up time, and support requirements. The rock mass rating (RMR) is defined as the sum of six parameters which can be obtained in the field or estimated from borehole data.

$$\mathrm{RMR} = \mathrm{A} + \mathrm{B} + \mathrm{C} + \mathrm{D} + \mathrm{E} + \mathrm{F} \tag{3.32}$$

where,

A = unconfined compressive strength of intact rock
B = Deere's Rock Quality Designation
C = spacing of discontinuities
D= condition of discontinuities
E = groundwater conditions
F = orientation of discontinuities

The range of values for each parameter is given in Table 3.3. Since the original publication of the classification system, Bieniawski has made several updates to the ratings; however, the 1976 paper by Bieniawski is considered the basic reference for this work and is the basis for other empirical studies (Hoek *et al.*, 1994). The ratings for each parameter are summed up to obtain a value between 0 and 100. Based on this rating, the rock is categorized into five classes ranging from very poor rock to very good rock (Table 3.3).

Bieniawski has related the span to stand-up time and rock mass rating for tunneling and mining case histories in Figure 3.15. This graph illustrates the wide band defining the unstable and potentially unstable zone.

Bieniawski (1976) has proposed the following relationship between the NGI Q rating and the RMR based on 117 case histories analyzed:

$$\mathrm{RMR} = 9 \ln \mathrm{Q} + 44 \tag{3.33}$$

Based on this relationship, Bieniawski has compared the maximum unsupported span as predicted by the NGI and CSIR rock mass classification systems. In Figure 3.16 it can be observed that the RMR is more conservative than the NGI system, which is probably a reflection of the different tunneling practice in Scandinavia and the considerable experience they have in the particular rock conditions found there.

The geomechanics classification provides guidelines for selecting the support for an opening. These support requirements are also dependent upon the size and shape of the excavation, the construction method, and the stress around the opening. Support classifications for the geomechanics classification are provided in Table 3.4.

Unal (1983) has proposed an equation for determining the support load using the rock mass rating.

$$P = \left(\frac{100 - RMR}{100} \right) \gamma S = \gamma h_t \tag{3.34}$$

where,

P = support load
h_t = rock load height (m)
S = tunnel width (m)
γ = unit weight of rock (N/m^3)

The variation of rock load per unit length of tunnel with span and rock mass rating is presented in Figure 3.17. This is analogous to determining the height of the relaxed zone of rock in the back of a stope which must be supported. The rock load height is plotted against tunnel width in Figure 3.18.

3.1.5.3 Modified Q -Rating

Potvin (1988) has developed an empirical design method for predicting stable spans in open stopes based on a modified NGI - Q rating. The work is based upon an earlier study by Mathews *et al.* (1980) which looked at 26 ope stope case histories taken from three mines and 29 case histories taken from the literature. The current Modified Stability Graph design technique is supported by 175 case histories collected in more than forty Canadian underground mines. The chart known as the Modified Stability Graph is constructed by plotting the modified stability number, N' versus the hydraulic radius of the design surface (Figure 3.19). The modified stability number N' is given by the following equation:

$$N' = Q' \times A \times B \times C \tag{3.35}$$

where,

Q' = modified NGI tunneling index
A = rock stress factor
B= rock defect orientation factor
C = orientation of design surface factor

The modified NGI rating is taken to be the first two quotients of the NGI rating given in Equation 3.29; that is, the stress reduction factor (SRF) and water pressure (J_w) terms have been ignored. The stress condition is accounted for in Factor A. The values for factors A, B, and C can be obtained from the graphs in Figure 3.19. Each surface plotted on the graph was classified as stable, unstable, and caved; and from this data, three zones were defined: stable zone, supportable zone, and caving zone.

By calculating the parameters Q', A, B, and C for a given stope surface, the stability number can be plotted on the Modified Stability Graph to determine the potential for instability. It is important to recognize that the database was developed from open stoping case histories and that the design method would be unconservative for entry-type mining methods such as cut and fill.

3.1.5.4 Mining Rock Mass Rating

Another rock mass classification system which has been developed is the Mining Rock Mass Rating (Laubscher, 1990). The MRMR takes account of the changes that a rock undergoes in a mining environment by taking the original CSIR rock mass rating and then adjusting it for weathering, mining induced stress, joint orientation, and blasting effects. Laubscher has proposed a slightly different way of calculating the RMR, as follows:

$$\mathrm{RMR} = \mathrm{IRS} + \frac{\mathrm{FF}}{\mathrm{m}} + 40[\mathrm{D} \times \mathrm{E} \times \mathrm{F} \times \mathrm{G}] \tag{3.36}$$

where,

IRS = unconfined compressive strength rating
FF/m = fracture frequency per metre
D = large scale joint expression (i.e. wavy, planar, stepped)
E = small scale joint expression (i.e. rough, smooth)
F = joint wall alteration
G = joint infilling

The Mining Rock Mass Rating (MRMR) is defined as:

$$\mathrm{MRMR} = \mathrm{RMR} \times [\mathrm{W} \times \mathrm{J} \times \mathrm{B} \times \mathrm{T}] \tag{3.37}$$

where,

W = rating adjustment for weathering
J = rating adjustment for joint wall orientation
B = rating adjustment for blasting practice
T = rating adjustment for induced mining stress

The ratings for each RMR parameter and the adjustments are given in Table 3.5. The total stress parameter is the most difficult parameter to establish and may require an adjustment of 60% to 120%. Laubscher recommends that the mining induced stress be determined from published stress distribution diagrams in the case of simple excavations, or from numerical modeling studies in the case of complex geometries.

Laubscher has also attempted to define the strength of the rock mass (RMS) in terms of the IRS and the RMR. Noting that large scale rock specimens give IRS values which are 80% of the small scale IRS values, the rock mass is assumed to have a strength of 0.8 IRS if it had no joints at all. The RMS is calculated by subtracting the IRS rating, A, from the full RMR rating, to give a rating out of 80 such that:

$$\text{RMS} = \text{IRS}\left(\frac{\text{RMR} - \text{A}}{80}\right)\left(\frac{80}{100}\right) = \frac{\text{IRS}(\text{RMR} - \text{A})}{100} \tag{3.38}$$

The Design Rock Mass Strength (DRMS) is defined as the strength of an unconfined rock mass in a specific mining environment. It could, for example, be compared to the mining induced stress in a pillar to compute a safety factor. In general, the immediate surface of an opening can be considered as unconfined. The depth of this zone depends on the size and shape of the opening. The same adjustments used to obtain the MRMR are used to obtain the DRMS.

$$\text{DRMS} = \text{RMS} \times [\text{W} \times \text{J} \times \text{B} \times \text{T}] \tag{3.39}$$

This MRMR classification system has been successfully applied to assessing the suitability of a rock mass for block caving. The objective in block caving is to open up a span which will remain unstable and continue to cave while the caved rock is gradually drawn out of the stope. The lower limit of what is considered a caveable rock mass could be considered to be the upper limit of the stable span of a stope. Laubscher (1990), has constructed a span design chart plotting MRMR versus the hydraulic radius (Figure 3.20). Case histories were categorized as being stable (requiring only key block support), caving, or in a transition zone between the two where more intensive support was required to maintain stability. This curve is not directly comparable to the RMR and Q span design curves presented earlier, since the hydraulic radius and not the span is presented.

3.1.5.5 Golder Crown Pillar Study Database

Carter *et al.* (1990) have undertaken an empirical evaluation of crown pillar stability involving 237 individual pillar case histories. In most cases, sufficient data was present in the records for the authors to assign a CSIR Rock Mass Rating and an NGI Q value. A method was developed for relating the geometric factors and rock mass parameters controlling pillar stability to the observed stability of the pillar. On the basis of their study, the authors considered the most important geometric factors controlling crown pillar stability to be:

- span (S);
- thickness (T);
- strike length (L);
- foliation/ore dip (θ); and
- unit weight of rock (γ).

These factors were appropriately combined to obtain a Crown Geometry Number (C_g), defined as:

$$C_g = f\left(\frac{F_w \times F_{st} \times F_{sr}}{F_\theta}\right) \tag{3.40}$$

where,

F_{st} = span to thickness ratio = S/T
F_θ = stope inclination factor = (1 - 0.4cosθ)
F_{sr} = span ratio factor = S/(1+S/L)
F_W = specific gravity (γ)

The Crown Geometry Number is inversely proportional to stability. As the weight factor (F_W) increases, the weight of the pillar increases and stability decreases. The span to thickness ratio (F_{st}) is a historical rule-of-thumb approach for assessing pillar stability. As this factor increases, the pillar stability decreases. The span ratio factor (F_{sr}), which is equal to twice the hydraulic radius, recognizes that when the length of the stope is greater than four times the span, failure is controlled by the short span. For shorter strike lengths, stability is controlled by two-way spanning. The stope inclination factor is equivalent to Factor C in the Modified Stability Method described above. It recognizes that a vertical or steeply dipping stope is more unstable than a shallow dipping one.

The square root of C_g was taken to obtain a final empirical expression, C_S, termed the Scaled Crown Span.

$$C_s = S\sqrt{\left(\frac{\gamma}{T(1+S_r)(1-0.4\cos\theta)}\right)} \tag{3.41}$$

The scaled span has been plotted against the rock mass rating for each case history in Figure 3.22. An empirical fit line proposed by Barton (1974) provides a good dividing line between the stable and unstable cases when superimposed on the crown pillar data. Carter *et al.* have added a hyperbolic sine term to account for the non-linear trend to increasing stability in very good rock masses. The following expression was developed to describe this line:

$$Critical\,C_s = 3.3Q^{0.43}\left[\sinh^{0.0016}(Q)\right] \tag{3.42}$$

Therefore, knowing the RMR or Q value for an area, the Critical Span can be obtained from Figure 3.21. Knowing the pillar thickness, stope length, stope dip, and unit weight of the rock, the minimum pillar thickness can be calculated by solving for T in Equation 3.41. The maximum allowable span can be calculated by rearranging Equation 3.41 and solving the binomial equation for *S*.

$$S^2 - A^2 + \frac{A^2 S}{L} = 0$$

where,

$$A = C_s \sqrt{\left(\frac{T}{\gamma}\right)(1 - 0.4\cos\theta)} \tag{3.43}$$

As part of this research project, the raw data used Carter's crown pillar study was reanalysed solely on the basis of span and is shown in Table 3.6 and Figure 3.22. The individual points on the graph are defined as stable or unstable. A value is found adjacent to an observation on the graph if the crown thickness above the stope is less than 4 metres. These points have a very low thickness to span ratio where failure could result from beam failure rather than a rock mass failure.

3.1.6 Stress Induced Failure

Stress induced failure is the result of mining induced stresses which exceed the strength of the rock mass. In competent, massive, elastic rock, this type of failure can take the form of rockbursting. In a jointed rock mass, a gradual yielding failure may take place. In cut and fill stopes, failure caused by high stress is most likely to occur in the back of the stope as the sill pillar width becomes smaller with each lift (Figure 3.23). It may also occur in very stiff post pillars. An analysis of the potential for this type of failure must take the form of analyzing the induced mining stresses at the boundaries of an excavation and comparing it to the rock mass strength.

Except in the early stages of development, most mines have complex excavation geometries in which stresses at the excavation boundaries cannot be estimated using closed-form solutions such as those which have been compiled by Poulos and Davis (1974). In recent years a wide variety of computer programs have become available which are capable of modeling excavations in two or three dimensions and carrying out a stress analysis. The three main types of numerical analyses in use today are:

- The Boundary Element Method;
- The Finite Element Method; and
- The Distinct Element Method.

These types of analyses are also useful for determining the stresses around excavations and in turn serve as input for other design methods as described previously. Numerical modeling has many other practical applications for rock mechanics engineers including:

- pillar design;
- open stope span design/dilution studies;
- shaft and service tunnel layouts;
- analyses of complex excavation geometries;
- stope sequencing studies; and
- parametric design studies.

3.1.6.1 Limitations of Numerical Modeling

With any numerical modeling procedure for determining stresses, the accuracy of the analysis depends on the accuracy of three main input parameters:

- The stress-strain relation(s) of the material(s);
- The pre-mining stress conditions; and
- The model geometry.

Many numerical models (BEAP3D, EXAMINE) assume the materials to be linear elastic, and isotropic. This assumption is considered accurate for intact drill core material but would not normally represent the stress-strain relationship of the rock mass. Inhomogeneity of the rock mass can usually be modeled by using different material parameters for different groups of elements in the model.

The pre-mining stress conditions must be determined as input for the analysis. Both the magnitude and direction of the principal stresses are required. These values are normally determined from in-situ stress measurements.

3.1.6.2 The Boundary Element Method

An overview of two-dimensional boundary element stress analysis is provided by Hoek and Brown (1980). In general terms, the problem is to determine the stresses around an excavation given a two dimensional stress field as shown in Figure 3.24(a). This procedure describes the method of solving a two-dimensional problem; however, the 3-D problem is solved in a similar fashion. Prior to excavation, the rock provides support for the area outside the excavation. This can be represented by normal and tangential tractions, as shown in Figure 3.24(b). The magnitudes of the tractions will vary along the surface, depending upon the shape. When the opening is excavated, the stress at the boundary is reduced to zero. This is equivalent to introducing negative tractions at the boundary, as shown in Figure 3.24(c). The final stress at the boundary can be considered to be the superposition of the original stress state and the stresses induced by the negative surface tractions.

The section shown in Figure 3.24(c) can be considered the true or actual situation. In order to model the problem, the boundary must be discretized into segments or elements. Each element is subjected to a fictitious force acting in the plane of the section and with components F_n and F_t as shown in Figure 3.24(d). These forces are assumed to act uniformly over the length of the element. An iterative procedure is used to adjust each of the fictitious forces in such a way that the normal and shear stresses at the centre of each element are equal to the normal and shear tractions. The stress at any point away from the boundary can be computed from standard expressions which sum the effects of the fictitious forces. These stresses are then added to the stresses from the original stress field to obtain the final stress. The elastic displacements are computed from standard solutions for displacements in an infinite medium due to point or distributed loads.

(a) 2-D Boundary Element Modeling

Two-dimensional boundary element programs can be used to model a cross-section of an opening where the dimension of the opening normal to the section is very long relative to the section dimensions (plane strain conditions). Most commercially available programs assume the medium to be linearly elastic and isotropic. The program *EXAMINE-2D* (Curran et al., 1989) was used in the course of this study for modeling of cut and fill stopes and sill pillars. The program utilizes a graphical interface for input of the excavation geometry and viewing of stresses and displacements. The program can use Mohr-Coulomb or Hoek-Brown failure criteria to compute factors of safety. Figure 3.25 is an example of the output created by *EXAMINE-2D* showing contours for the major principal stress around a vertical cross section of the Detour Lake Mine.

(b) 3-D Boundary Element Modeling

The shape of many underground excavations and the influence of neighbouring excavations make 2-D plane strain analysis inappropriate in many circumstances. In such cases a three-dimensional analysis may be required. The program *BEAP-3D* (CANMET, 1993) was used in the course of this research project to analyze the complex excavation geometry at Detour Lake Mine.

BEAP-3D, or Boundary Element Applications Package, is a powerful numerical modeling package designed specifically for modeling three-dimensional underground openings. The version used was capable of modeling up to 1000 elements. The program is designed to run from a Sun Sparc Workstation operating under *Open Windows 3.0* or from a PC operating under Windows 3.1. The program utilizes a graphical preprocessor called *MINE DESIGNER* for creating a model geometry file as well as a graphical post processor called *VIEWBEAP* for viewing stresses.

BEAP-3D has excellent graphics for viewing excavations and stress distributions; however input of the excavation geometry is still very time consuming, particularly when a high degree of detail is required. For this reason, three-dimensional modeling is still not widely used at mine sites. Further improvements to these types of programs including linkages to existing mine design software will no doubt increase the use of this software in the near future. Figure 3.26 is an example of the output from BEAP-3D showing the principal stress contours around the stopes and the pit of the Detour Lake Mine. The results of the modeling of DLM will be discussed further in Chapter 5 of this report.

(c) 2D versus 3D Modeling

Pakalnis (1991) has made a study of the relative differences in results between 2D and 3D boundary element models. In general it was found that in the hangingwall of tabular stopes, the zone of relaxation predicted by 2D analysis is much larger than 3D for various stope geometries. The magnitudes of the tensile stresses in 2D are greater than in 3D. In the backs of stopes, the compressive stresses predicted by 2D modeling are greater than those evaluated by 3D modeling for various geometries. Since 2-D results have been shown to be conservative under many conditions, it can remain a useful tool as a first check. If stresses are found to be acceptably low using the 2-D model, 3-D modeling will not be required.

3.1.6.3 The Finite Element Method

An introduction to the finite element method applied to the field of rock mechanics can be found in Brady *et al.* (1985). Briefly, the finite element method involves discretizing the domain to be studied into elements. The domain is subjected to initial stresses p_{xx}, p_{yy}, and p_{zz} shown in Figure 3.27(a). Appropriate boundary conditions are applied at the boundary of the domain to render the problem statically determinate. For each element, appropriate functions are chosen which define the displacement of any point within the element in terms of the nodal displacements. The solution procedure results in a stiffness matrix, [k], for each element, based on the shape of the element, and an element load vector based on nodal tractions and boundary pressures such that,

$$[k]\ [\delta] = [f] \tag{3.44}$$

The element stiffness matrices are assembled into a global stiffness matrix in such a way that compatibility of displacements is maintained. For the global system,

$$[\Delta] = [K]^{-1}[F] \tag{3.45}$$

The system is solved for the global nodal displacements; and, because the strain is the derivative of the displacements, the nodal strains can be computed. Stresses are then computed using a two dimensional elasticity matrix involving Young's Modulus and Poisson's Ratio.

The finite element method does offer more flexibility in terms of defining the stress-strain relations than does the boundary element method. In theory, each element could have a different stress strain relation but in practice, groups of elements are the same. Although most codes which have been developed for rock mechanics applications rely on a simple, linear-elastic, stress-strain relationship, more complicated codes can be developed to model post peak strength behaviour. The disadvantage of the finite element is that it is very time consuming to set up the model for all but the simplest problems because the entire domain must be discretized rather than just the boundary as is the case in boundary element models. The number of elements required also makes it very costly in terms of computer time and data storage requirements.

3.1.6.4 The Distinct Element Method

The distinct element method treats the domain being modeled as an assemblage of blocks (Figure 3.28). This may certainly be appropriate for many excavations where stability is controlled by structural discontinuities. Where the stiffness along these discontinuities is much less than the stiffness of the block, the block can be considered rigid and displacement only occurs along the discontinuities. Therefore, the block shape does not change during the analysis; rather, the system adjusts to the prescribed boundary conditions by movement parallel and normal to the joint surface. Difficulties with the distinct element method arise in defining the stress-strain relationships parallel and perpendicular to the discontinuity. The problem is compounded if each discontinuity has a different strength. Furthermore, setting up the model geometry for actual mine problems is cumbersome and time consuming. These factors may account for why the procedure is not widely used in the mining industry except as a research tool.

3.2 SURVEY OF OTHER CUT AND FILL OPERATORS

A survey questionnaire was distributed in 1991 to cut and fill operations in Canada. The survey's objectives were to:

- identify how operations were designing cut and fill stope spans;
- obtain an operation's typical stable stope span and rock mass quality;
- obtain span and rock quality data from stopes which had experienced a fall of ground;
- identify how operations were designing post pillars and sill pillars;
- determine types of instrumentation mines were using to predict instability; and
- determine how cable bolts are being utilized in cut and fill stopes.

The following five companies representing 9 operations responded to the questionnaire:

- Westmin Resources - Myra Falls Operations, Campbell River, B.C.;
- Placer Dome Inc., - Dome Mine, Timmins, Ontario;
- Hudson Bay Mining & Smelting -Trout Lake Mine, Trout Lake Manitoba;
- Inco Ltd. - Sudbury Operations, Sudbury, Ontario; and,
- Falconbridge Inc. - Sudbury Operations, Sudbury, Ontario.

The returned questionnaires are provided in Appendix A. Table 3.7 summarizes the span design data obtained from this questionnaire. It is noteworthy that empirical design is used at least in part in three of the four operations which require span design. Figure 3.29 is a plot of the span versus RMR for these five operations. The points on the graph plot as critical or design. A critical point indicates that based upon past practice it was demonstrated that exceeding the span for the given rock quality results in instability. A design point indicates the design mining span for an operation. This would have been determined through numerical modeling, past experience, or orebody constraints. It does not necessarily imply that larger spans would be unstable.

A number of pillar design methods are used by these mines and are identified in Table 3.8 and Table 3.9. Empirical design based on past experience at individual operations is the most common method, however two of the mines reported using the Hedley pillar formula for design (Hedley, 1972). Surprisingly, only one of the operations reported using numerical modeling in the pillar design process.

Cable bolts were found to be used by all of the operations in specific situations where bad structure or a low quality rock mass has been identified. No firm rules were identified for where cable bolts would be installed. Table 3.10 gives some detail of the bolting practice at these operations. One mine reported an experimental support project involving replacing post pillar support by cable bolt support.

3.3 DESIGN PHILOSOPHY

In the author's experience, the best approach to span design is an integrated one which combines elements of analytical solutions, empirical design, and numerical modeling. This approach has been successfully applied at Detour Lake Mine for designing safe yet practical spans in wide cut and fill stopes. The different analysis techniques are considered necessary given the different types of failure mechanisms described earlier. Solving a solution from two or more approaches also provides confidence in the design if the two solutions can be made to closely agree. Of course, any final design must also comply with existing regulatory statutes such as those covering the minimum size of barrier pillars.

Before any design can be implemented, a detailed fabric analysis of the rock mass must be carried out and the intact rock strength parameters of the rock must be determined. This is required in order to obtain the fundamental parameters which are required regardless of the design procedure or failure mechanism. The intact rock strength parameters which should be determined prior to carrying out a design are:

- Uniaxial Compressive Strength (UCS);
- Young's Modulus (E);
- Poisson's Ratio (μ); and
- Unit Weight (N/m^3).

The fabric analysis must provide sufficient information on the structural characteristics of the rock mass for it to be classified using either the NGI or CSIR classification systems. At Detour Lake Mine, this information is obtained from diamond drilling and geotechnical mapping of each lift. Structural data is compiled on stereonets to determine the dominant joint sets. Faults or continuous joints are mapped on a daily basis by the geology staff.

As the second step of the design process, the rock mechanics engineer must decide which is the controlling failure mechanism. At Detour Lake Mine for example, the presence of faults or continuous joints in the back which dip at less than 30°, can lead to wedge failure irrespective of span or the overall rock quality. Structural failure, therefore is the first failure mechanism which must be accounted for in the design. Prediction of structural failure requires a good database of geotechnical observations for the area under design as well as ongoing visual inspection. A stereonet analysis of the structures defining the wedge is carried out to determine whether failure is kinematically possible. The computer program UNWEDGE is used to determine the size of the potential groundfall and to assess the support requirements. If structural failure is predicted, the span could be reduced to prevent formation of the wedge. Alternatively, more intensive ground support could be specified to support the wedge. At this stage, numerical modeling would be useful for determining the extent of the relaxed zone in the immediate back.

The next step is to decide which other failure mechanisms and design methods are relevant. At Detour Lake Mine and most underground metal mines, beam and plate failure can be ruled out as a failure mechanism given that discontinuities are generally present and these theories assume horizontally stratified intact rock. Voussoir block theory does not apply at DLM since the jointing pattern does not meet the strict criteria set out above. Chimney failure is unlikely at DLM because there is not any unfavourable structure on the hangingwall or footwall or strong foliation parallel to the orebody. At the ultimate stope height where the pillar width is low, a quick assessment for chimney failure should be made using Equation 3.27. Numerical modeling may also be required at this stage in order to determine the horizontal stresses on the pillar such that the shear stress in Equation 3.27 may be calculated.

After structural failure, rock mass failure is the next most likely mode of failure. A rock mass failure assessment is made using an empirical approach utilizing a database of observations from stoping case histories at Detour Lake Mine. These observations have been compiled and plotted on a span versus RMR graph to enable future prediction of stable spans given the RMR of the stope. This approach has proven successful at predicting stable spans at DLM. The DLM database and empirical design method will be described in greater detail in Chapter 6.

Finally, the potential for stress induced failure is assessed using 2-D and 3-D boundary element modeling. Stope design at DLM is an ongoing process. New structures and changes in the overall rock mass due to stress redistribution can develop from lift to lift, which may warrant design changes. The DLM design procedure for cut and fill stopes is outlined in the diagram shown in Figure 3.30. The development of an empirically based stability graph specifically designed for entry-type stopes is the focus of this study.

Table 3.1 NGI - Q Classification System Rating for Individual Parameters (Adapted from Barton *et al.*, 1974)

Parameter	Item and Description	Value
RQD	**Rock Quality Designation**	
	The total length of core pieces over four inches in length in an interval divided by the length of the interval and expressed as a percent.	0-100
J_n	**Number of Sets of Discontinuities**	
	Massive	0.5
	One Set	2.0
	One Set Plus Random	3.0
	Two Sets	4.0
	Two Sets Plus Random	6.0
	Three Sets	9.0
	Three Sets Plus Random	12.0
	Four or More Sets	15.0
	Crushed Rock	20.0
J_r	**Roughness of Discontinuities**	
	Non-continuous joints	4.0
	Rough and wavy	3.0
	Smooth and wavy	2.0
	Rough and planar	1.5
	Smooth and planar	1.0
	Slickensided and planar	0.5
	Filled discontinuities	1.0
J_a	**Filling and Wall Rock Alteration, Essentially Unfilled**	
	Healed Joints	0.75
	Staining only, no alteration	1.0
	Slightly altered joint walls	2.0
	Silty or sandy coatings	3.0
	Clay coatings	4.0
	Filling and Wall Rock Alteration, Filled Joint	
	Sand or crushed rock filling	4.0
	Stiff clay filling less than 5 mm thick	6.0
	Soft clay filling less than 5 mm thick	8.0
	Swelling clay filling less than 5 mm thick	12.0
	Stiff clay filling more than 5 mm thick	10.0
	Soft clay filling more than 5 mm thick	15.0
	Swelling clay filling more than 5 mm thick	20.0
J_w	**Water Conditions**	
	Dry, or inflow < 5 litres/minute locally	1.0
	Medium water inflow	0.66
	Large inflow, unfilled joints	0.5
	Large inflow, filled joints with washout	0.33
	Large inflow, filled joints, high transient flow	0.2 to 0.1
	Large inflow, filled joints, high continuous inflow	0.1 to 0.05
SRF	**Stress Condition Class**	
	Loose rock with clay filled discontinuities	10.0
	Loose rock with open discontinuities	5.0
	Shallow depth (50 m or less) rock with clay filled discontinuities	2.5
	Rock with tight unfilled discontinuities, medium stress	1.0

Table 3.2 Excavation Support Ratios

	Excavation Category	ESR	No. of Cases
A.	Temporary Mine Openings	3-5	2
B.	Vertical Shafts: Circular Section Rectangular or Square Section	 2.5 2.0	 - -
C.	Permanent Mine Openings, water tunnels for hydropower (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations.	1.6	83
D.	Storage rooms, water treatment plants, minor highway or railway tunnels, surge chambers, access tunnels.	1.3	25
E.	Power stations, major highway or railway tunnels, civil defense chambers, portals, intersections.	1.0	73
F.	Underground nuclear power stations, railroad stations and factories.	0.8	2

Table 3.3 Geomechanics Classification of Rock Masses (after Bieniawski, 1976)

A. Classification Parameters and their Ratings

<table>
<tr><td colspan="3">PARAMETER</td><td colspan="7">RANGE OF VALUES</td></tr>
<tr><td rowspan="3">1</td><td rowspan="2">Strength of Intact Rock Material</td><td>Point Load Strength Index</td><td>>8 MPa</td><td>4-8 MPa</td><td>2-4 MPa</td><td>1-2 MPa</td><td colspan="3">For this low range uniaxial tests are preferred</td></tr>
<tr><td>Uniaxial Compressive Strength</td><td>>200 MPa</td><td>100-200 MPa</td><td>50-100 MPa</td><td>25-50 MPa</td><td>5-25 MPa</td><td>1-5 MPa</td><td><1 MPa</td></tr>
<tr><td colspan="2">Rating</td><td>15</td><td>12</td><td>7</td><td>4</td><td>2</td><td>1</td><td>0</td></tr>
<tr><td rowspan="2">2</td><td colspan="2">Drill Core Quality, RQD</td><td>90%-100%</td><td>75%-90%</td><td>50%-75%</td><td>25%-50%</td><td colspan="3"><25%</td></tr>
<tr><td colspan="2">Rating</td><td>20</td><td>17</td><td>13</td><td>8</td><td colspan="3">3</td></tr>
<tr><td rowspan="2">3</td><td colspan="2">Spacing of Discontinuities</td><td>>3 m</td><td>1 .0 - 3.0 m</td><td>0.3-1.0 m</td><td>50 - 300 mm</td><td colspan="3"><50 mm</td></tr>
<tr><td colspan="2">Rating</td><td>30</td><td>25</td><td>20</td><td>10</td><td colspan="3">5</td></tr>
<tr><td rowspan="2">4</td><td colspan="2">Condition of Discontinuities</td><td>Very rough surfaces Not continuous No separation Unweathered wall rock</td><td>Slightly rough surfaces] Separation < 1mm,slightly weathered walls</td><td>Slightly rough surfaces Separation < 1mm Highly weathered walls</td><td>Slickensided surfaces OR Gouge <5 mm thick OR Separation 1-5 mm continuous</td><td colspan="3">Soft gouge > 5mm thick OR Separation less than 5mm Continuous</td></tr>
<tr><td colspan="2">Rating</td><td>25</td><td>20</td><td>12</td><td>6</td><td colspan="3">0</td></tr>
<tr><td rowspan="4">5</td><td rowspan="3">Groundwater</td><td>Inflow per 10 m tunnel length</td><td colspan="2">None</td><td><25 litres/min</td><td>25-125 litres/min</td><td colspan="3">>125 litres/min</td></tr>
<tr><td>Ratio $\frac{\text{Joint Water Pressure}}{\text{Major Principal Stress}}$</td><td colspan="2">0</td><td>0.0-0.2</td><td>0.2-0.5</td><td colspan="3">>0.5</td></tr>
<tr><td>General Conditions</td><td colspan="2">Completely Dry</td><td>Moist only (Interstitial Water)</td><td>Water under moderate pressure</td><td colspan="3">Severe water problems</td></tr>
<tr><td colspan="2">Rating</td><td colspan="2">10</td><td>7</td><td>4</td><td colspan="3">0</td></tr>
</table>

B. Rating Adjustment for Discontinuity Orientation

Strike and Dip Orientations of Joints		Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable
	Tunnels	0	-2	-5	-10	-12
Ratings	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. Rock Mass Classes Determined From Total Ratings

Rating	81-100	61-80	41-60	21-40	<20
Class No.	I	II	III	IV	V
Description	Very Good Rock	Good Rock	Fair Rock	Poor Rock	Very Poor Rock

D. Meaning of Rock Mass Classes

Class Number	I	II	III	IV	V
Average Stand-up Time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the Rock Mass	>400 kPa	300-400 kPa	200-300 kPa	100-200 kPa	<100 kPa
Friction Angle of the Rock Mass	<45°	35°-45°	25°-35°	15°-25°	<15°

Table 3.4 Geomechanics Classification Guide for Excavation and Support in Rock Tunnels (after Bieniawski, 1984)

Rock Mass Class	Excavation	Support		
		Rockbolts (20mm diam., fully bonded)	Shotcrete	Steel Sets
Very Good Rock RMR: 81-100	Full Face: 3m advance	Generally no support required except for occasional spot bolting		
Good Rock RMR: 61-80	Full Face: 1.0-1.5 m advance; Complete support 20 m from face	Locally bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None
Fair Rock RMR: 41-60	Top Heading and Bench: 1.5-3.0 m advance in top heading; Commence support after each blast Complete support 10m from face	Systematic bolts 4m long,spaced 1.5-2.0 m in crown and walls with wire mesh in crown	50-100mm in crown and 30 mm in sides	None
Poor Rock RMR:21-40	Top Heading and Bench:1.0-1.5 m advance in top heading; Install support concurrently with excavation - 10m from face	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh	100-150 mm in crown and 100 mm in sides	Light ribs spaced 1.5 m where required
Very Poor Rock RMR: <20	Multiple Drifts: 0.5-1.5 m advance in top heading; Install support concurrently with excavation; shotcrete as soon as possible after blasting	Systematic bolts 5-6 m long Spaced 1-1.5 m in crown and walls with wire mesh Bolt invert	150-200 mm in crown 150 mm in sides and 50mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert

Table 3.5 Mining Rock Mass Rating System Parameter Ratings and Adjustments (after Laubscher, 1990)

ASSESSMENT OF JOINT CONDITION						
Parameters	Description					
			Dry	Moist	Mod. Pressure	High Pressure
D						
LARGE SCALE JOINT EXPRESSION	wavy	multi-directional	100	100	95	90
	wavy	unidirectional	95	90	85	80
	curved		85	80	75	70
	slight	undulation	80	80	75	65
	straight		75	70	65	60
E						
SMALL SCALE JOINT EXPRESSION	rough	step/irreg	95	90	85	80
	smooth	stepped	90	85	80	75
	slickensides	stepped	85	80	75	70
	rough	undulating	80	75	70	65
	smooth	undulating	75	70	65	60
	slickensided	undulating	70	65	60	55
	rough	planar	65	60	55	50
	smooth	planar	60	55	50	45
	polished	planar	55	50	45	40
. JOINT WALL ALTERATION	weaker than wall rock		75	70	65	60
G						
JOINT FILLING	non	coarse	90	85	80	75
	softening	medium	85	80	75	70
	sheared	fine	80	75	70	65
	soft	coarse	70	65	60	55
	sheared	medium	60	55	50	45
		fine	50	45	40	35
	gouge thickness	<amplitude of irreg	45	40	35	30
	gouge thickness	>amplitude of irre	30	20	15	10

FRACTURE FREQUENCY PER METRE			
Ave/m	Ratings		
	1 set	2 sets	3 sets
0.1	40	40	40
0.15	40	40	40
0.2	40	40	38
0.25	40	38	36
0.3	38	36	34
0.5	36	34	31
0.8	34	31	28
1.0	31	28	26
1.5	28	26	24
2	26	24	21
3	24	21	18
5	21	18	15
7	18	15	12
10	15	12	10
15	12	10	7
20	10	7	5
30	7	5	2
40	5	2	0

INTACT ROCK STRENGTH	
MPa	Rating
>185	20
165-185	18
145-164	16
125-144	14
105-124	12
85-104	10
65-84	8
45-64	6
35-44	5
25-34	4
12-24	3
5-11	2
1-4	1

ADJUSTMENTS FOR WEATHERING

Degree of Weathering	Potential Weathering and Adjustments, (%)				
	1/2 year	1 year	2 years	3 years	+4 years
Fresh	100	100	100	100	100
Slight	88	90	92	94	96
Moderate	82	84	86	88	90
High	70	72	74	76	78
Complete	54	56	58	60	62
Residual Soil	30	32	34	36	38

PERCENTAGE ADJUSTMENTS FOR JOINT ORIENTATION

No. of Joints defining the block	Number of faces inclined away from the vertical				
	70%	75%	80%	85%	90%
3	3		2		
4	4	3		2	
5	5	4	3	2	1
6	6	5	4	3	2.1

PERCENTAGE ADJUSTMENTS FOR BLASTING EFFECTS

Technique	Adjustment, %
Boring	100
Smooth Wall Blasting	97
Good Conventional Blasting	94
Poor Blasting	80

Table 3.6 Crown Pillar Study Data (Carter, 1990)

Case No.	RMR (%)	Q	Span (m)	Condition
12A	80	54.6	40	U
12B	80	54.6	80	U
14	45	1.1	23	U
21	35	0.4	22	U
22A	50	1.9	15	U
22B	50	1.9	21	U
25A	25	0.1	8	U
25B	25	0.1	11	U
26	50	1.9	13	U
27	55	3.4	23	U
28	15	0.0	19	U
29	75	31.3	30	U2
35	25	0.1	3	U
36	58	4.7	20	U
38	50	1.9	6	U1
39	50	1.9	65	U
41	55	3.4	60	U
46	20	0.1	3	U
50	20	0.1	15	U
52	48	1.6	12	U
53	54	3.0	12	U
54	85	95.2	43	U3
57	55	3.4	6	U3
59	70	18.0	10	U4
62	80	54.6	10	U
64A	45	1.1	3	U1
64B	45	1.1	5	U1
1	75	31.3	27	S
2	70	18.0	30	S
3	0	0.0	43	S
4	75	31.3	40	S
5	65	10.3	14	S
6	45	1.1	11	S
7	60	5.9	5	S
8	60	5.9	15	S
9	60	5.9	73	S
10	85	95.2	18	S
11	35	0.4	6	S
12	80	54.6	55	S

Case No.	RMR (%)	Q	Span (m)	Condition
13	50	1.9	30	S
14	45	1.1	5	S
15	45	1.1	4	S
16	85	95.2	6	S
17	70	18.0	3	S
18	80	54.6	18	S
19	80	54.6	21	S
20	70	18.0	12	S
21	45	1.1	22	S
22	50	1.9	17	S
23	60	5.9	45	S
25	30	0.2	11	S
30	50	1.9	16	S
31	60	5.9	3	S
32	50	1.9	6	S
33	75	31.3	3	S
34	50	1.9	3	S
35	25	0.1	3	S
36	58	4.7	10	S
37	60	5.9	3	S
40	75	31.3	6	S
42	75	31.3	5	S
43	75	31.3	3	S
44	60	5.9	10	S
45	45	1.1	10	S
46	50	1.9	3	S
47	60	5.9	20	S
48	70	18.0	32	S
49	45	1.1	6	S
51	41	0.7	17	S
55	45	1.1	15	S
56	50	1.9	30	S
57	65	10.3	6	S
58	65	10.3	10	S
60	60	5.9	6	S
61	45	1.1	6	S
62	80	54.6	10	S
63	60	5.9	6	S
65	36	0.4	4	S
66	85	95.2	21	S
67	65	10.3	60	S
68	56	3.8	25	S
69	56	3.8	24	S
70	70	18.0	32	S

U = Unstable
S = Stable
A number following an unstable condition (U2) refers to an unstable case where the thickness of the pillar was very small. The number refers to the pillar thickness in metres. For these cases, beam failure rather than rock mass failure may have been the failure mechanism.

Table 3.7 Span Design Methodology at Canadian Cut and Fill Mines

Design Method	Falconbridge Ltd. Sudbury Mines	Placer Dome Inc. Dome Mine	HBM&S Trout Lake Mine	Inco Limited Sudbury Mines	Westmin Resources HW Mine
Mathews Stability Graph Method	•				
Numerical Methods	•				
Past Experience		•		•	•
Not Designed (Full Extraction)			•		

Table 3.8 Post Pillar Design Methodology at Canadian Cut and Fill Mines

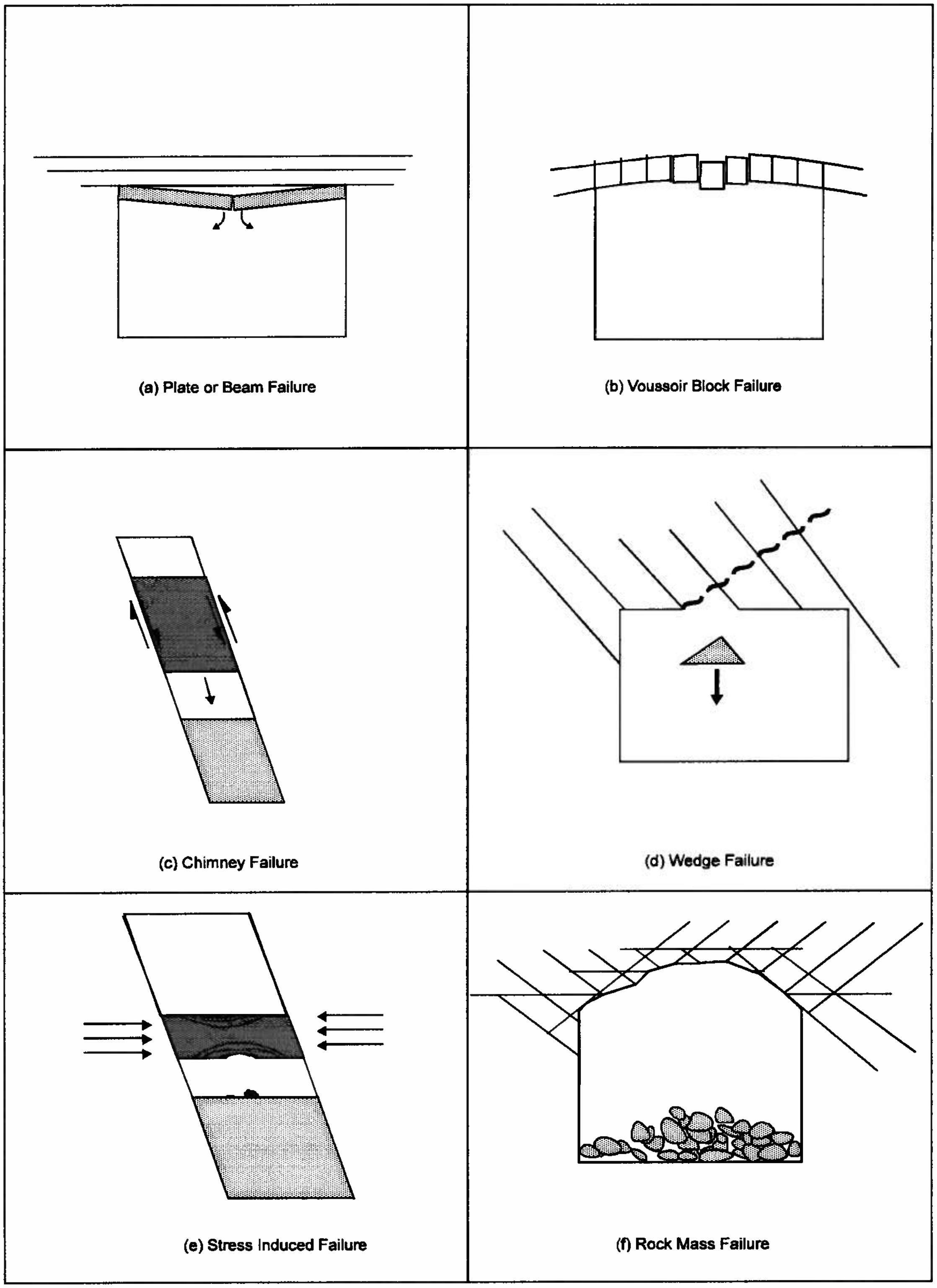
Design Method	Falconbridge Ltd Sudbury Mines	Placer Dome Inc. Dome Mine	HBM&S Trout Lake Mine	Inco Limited Sudbury Mines	Westmin Resources HW Mine
Methods Based on Hedley Formula	•				•
Past Experience		•		•	
Tributary Theory					•

Table 3.9 Sill Pillar Design Methodology at Canadian Cut and Fill Mines

Design Method	Falconbridge Ltd. Sudbury Mines	Placer Dome Inc. Dome Mine	HBM&S Trout Lake Mine	Inco Limited Sudbury Mines	Westmin Resources HW Mine
Based on Production Requirements	•			•	
Past Experience		•			
Designed to Support Load of Fill Above Pillar			•		

Table 3.10 Cable Bolting Support Design Methodology at Canadian Cut and Fill Mines

Design Method	Falconbridge Ltd. Sudbury Mines	Placer Dome Inc. Dome Mine	HBM&S Trout Lake Mine	Inco Limited Sudbury Mines	Westmin Resources HW Mine
Noranda/Potvin Method	•				
Numerical Methods	•				
Past Experience at Minesite		•			•
Analytical Methods		•		•	•
Experience at Other Mines			•		•



(a) Plate or Beam Failure

(b) Voussoir Block Failure

(c) Chimney Failure

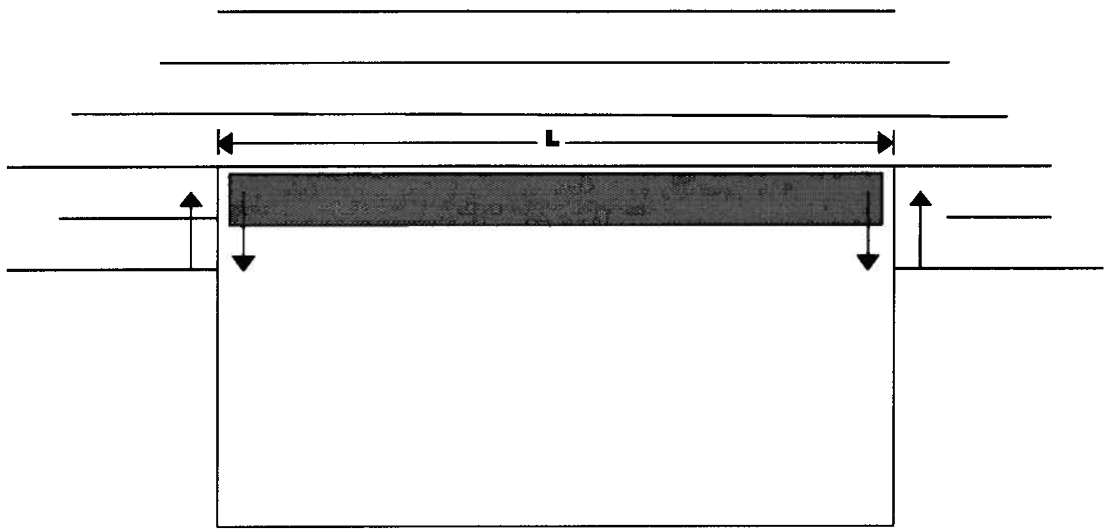
(d) Wedge Failure

(e) Stress Induced Failure

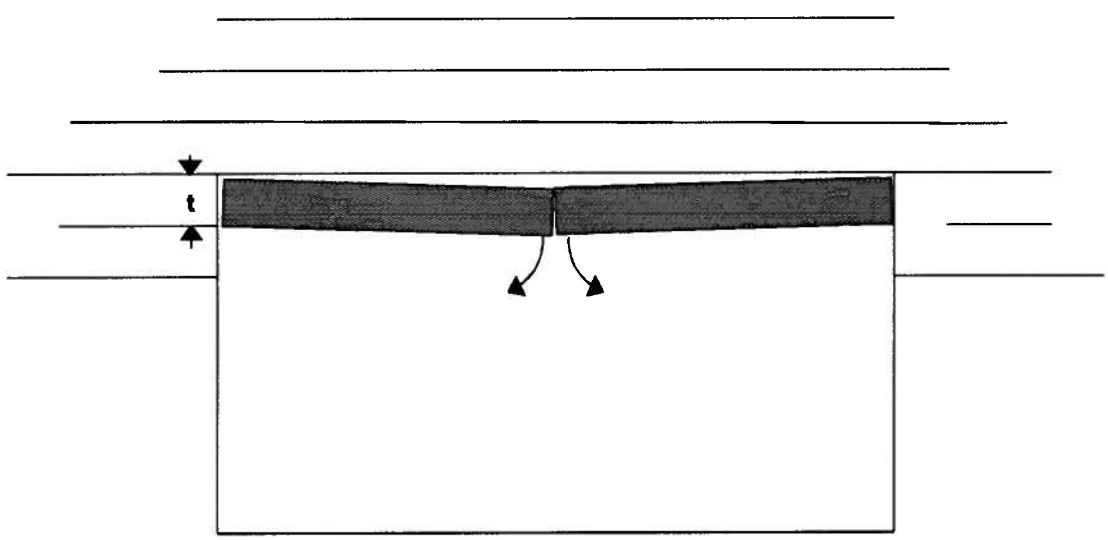
(f) Rock Mass Failure

UNDERGROUND ROOF FAILURE MECHANISMS

FIGURE 3.1

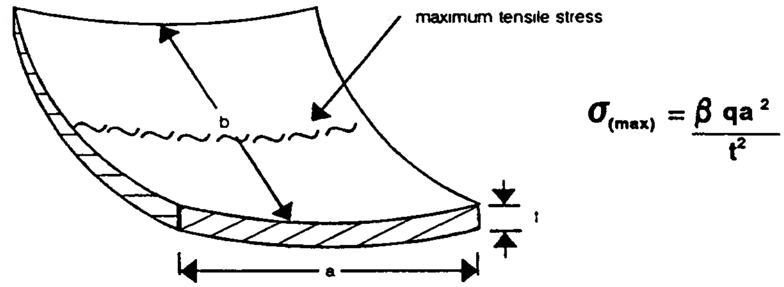


Beam Shear Failure

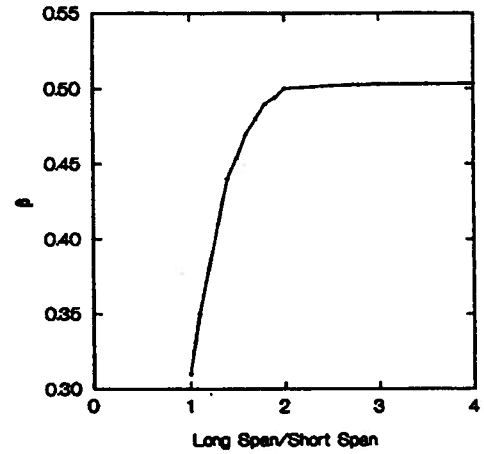


Beam Tensile Failure

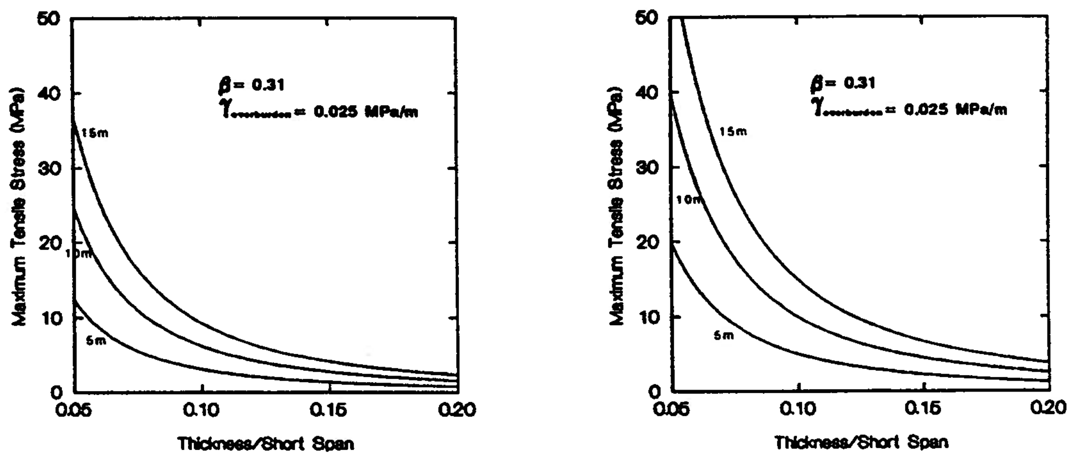
BEAM FAILURE MECHANISMS	FIGURE 3.2



(a) Plate Geometry

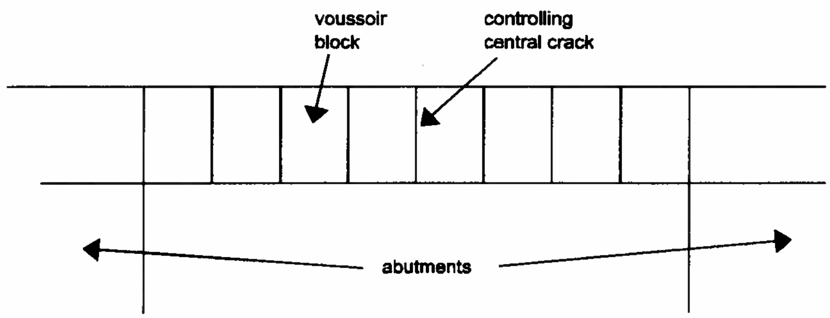


(b) Beta versus Span Ratio

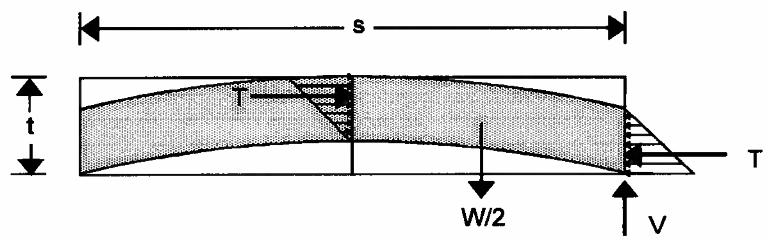


(c) Maximum Tensile Strength

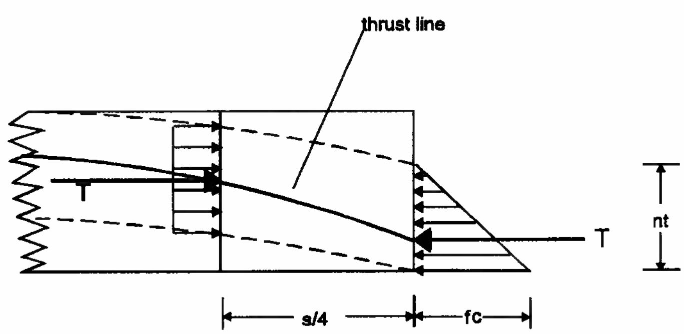
PLATE FAILURE	FIGURE 3.3



(a) Voussoir Block Excavation Roof



(b) Parabolic Arch Developed In Roof

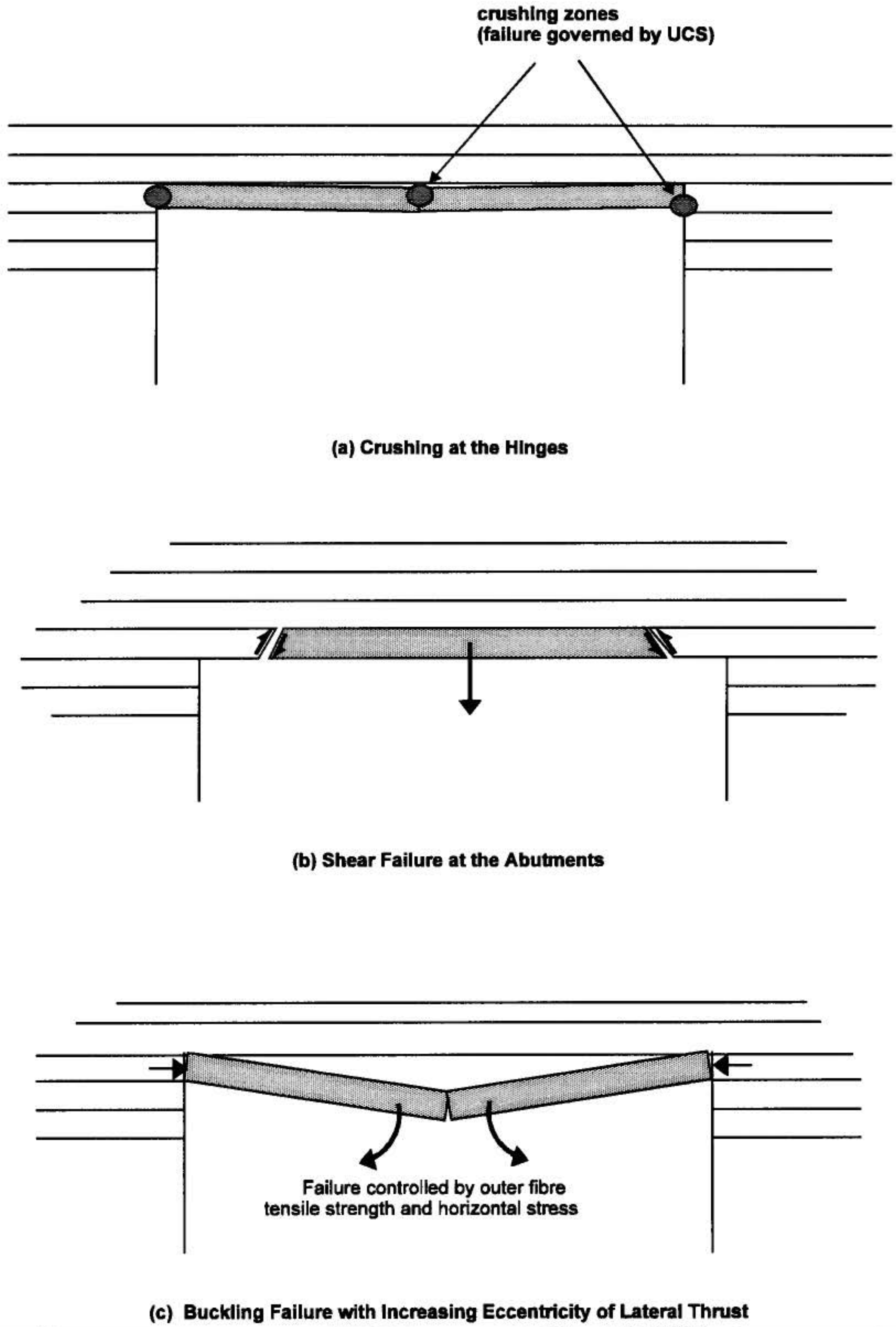


(c) Line of Thrust In Beam to Abutments

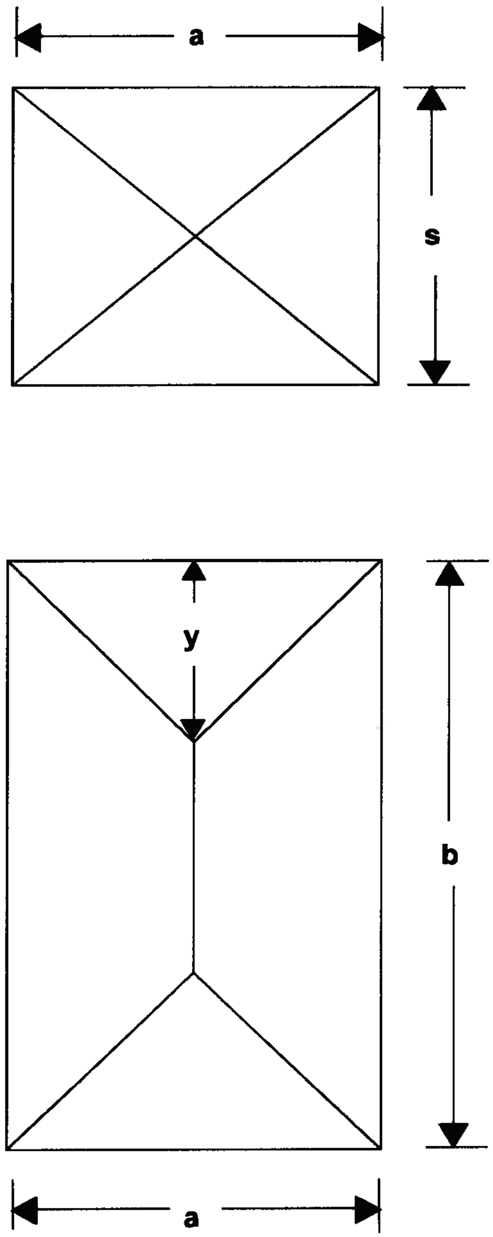
(after Brady and Brown, 1985)

VOUSSOIR BLOCK THEORY

FIGURE 3.4



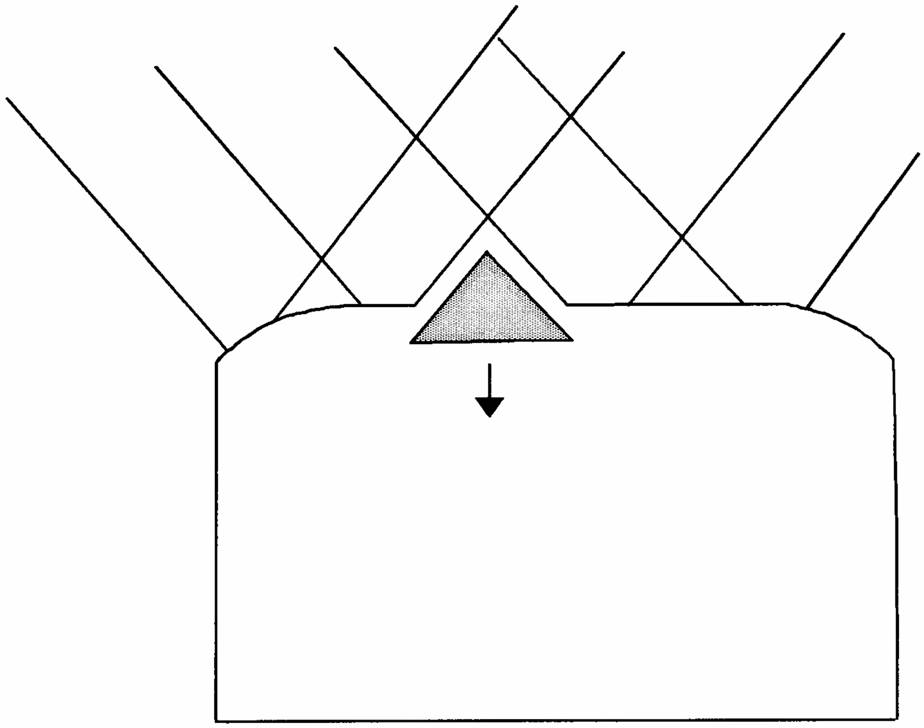
VOUSSOIR ARCH FAILURE MODES — FIGURE 3.5



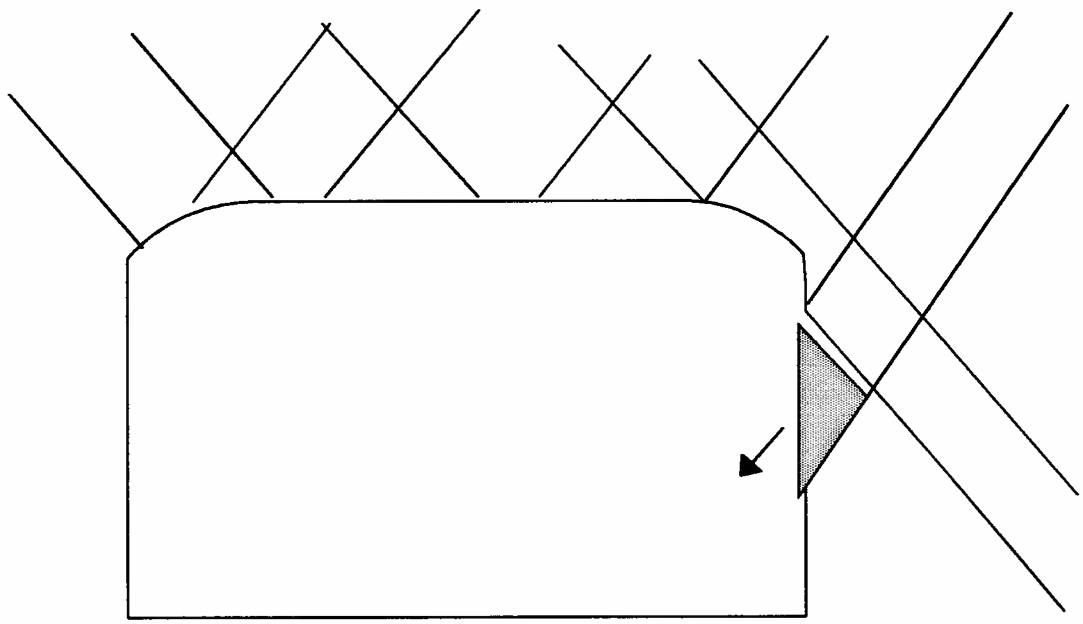
(after Beer & Meek, 1982)

VOUSSOIR PLATE FAILURE MODES

FIGURE 3.6

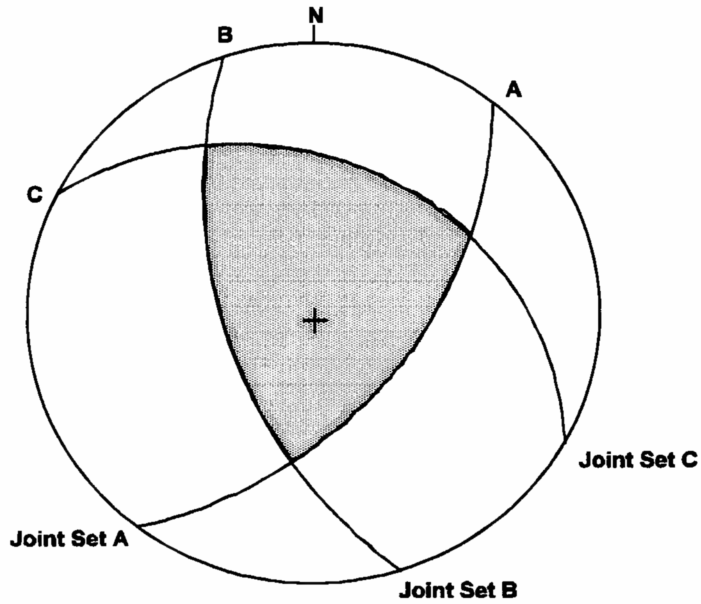


(a) Gravity Wedge Failure in Roof

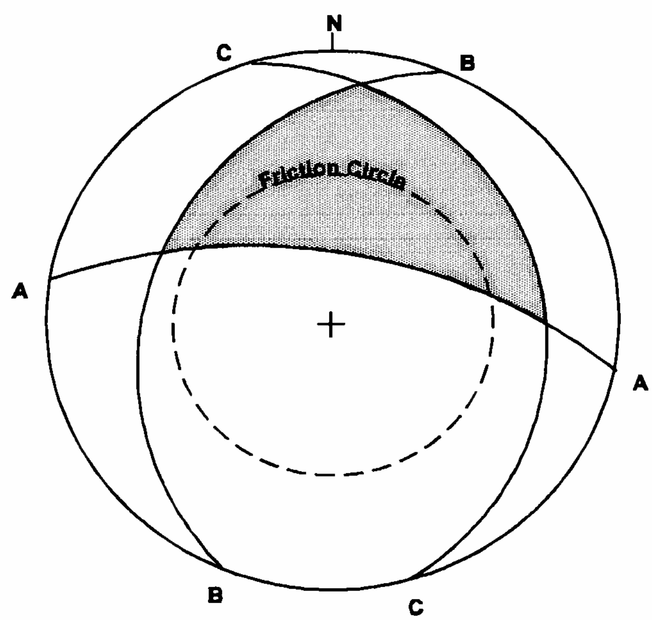


(b) Sliding Wedge Failure in Wall

WEDGE FAILURE	FIGURE 3.7

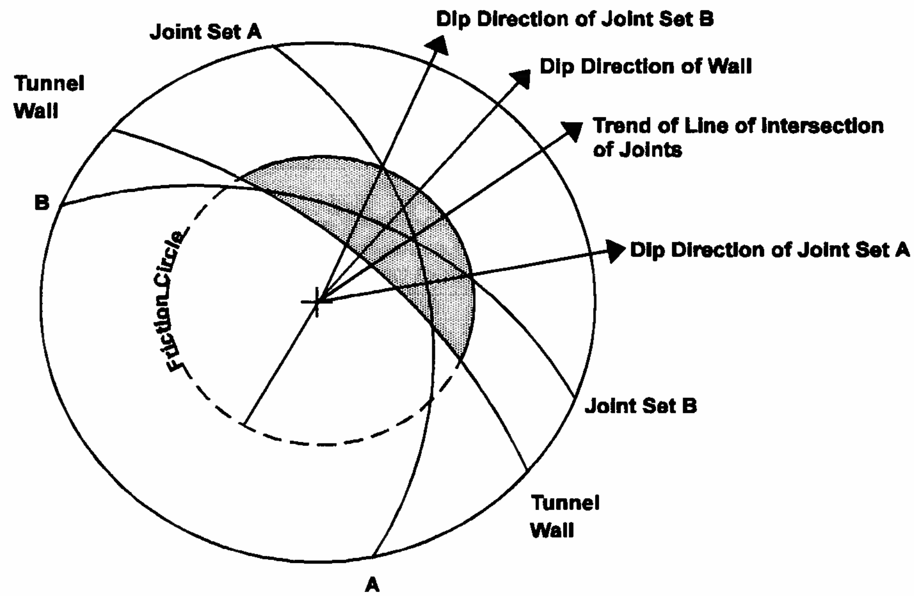


(a) Free Fall Wedge Failure

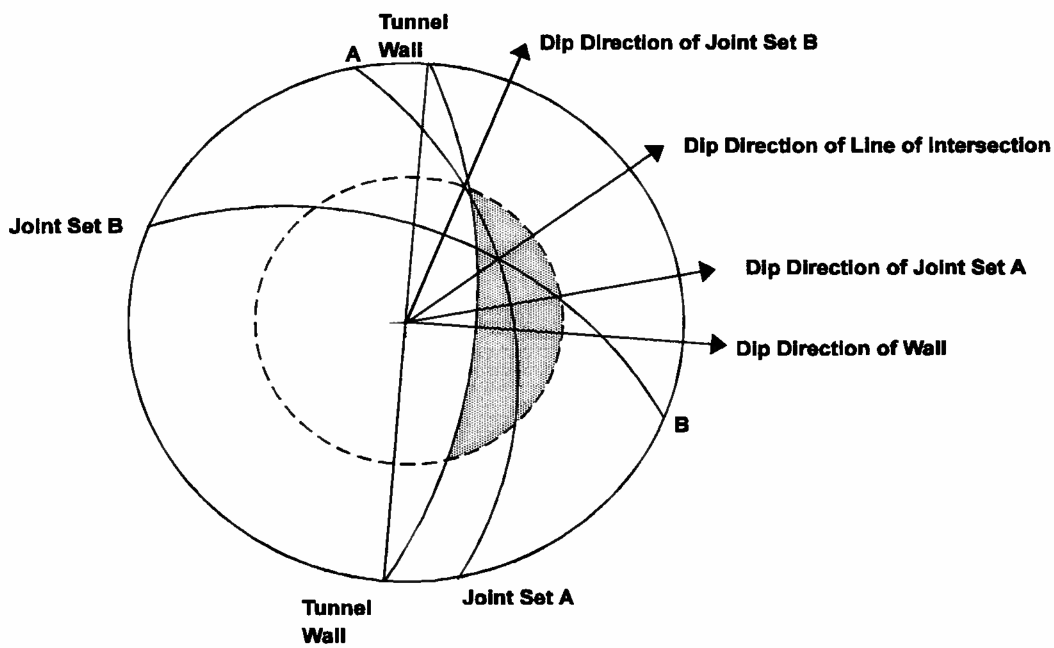


(b) Sliding Wedge Failure

STEREONET ANALYSIS OF WEDGE FAILURES IN BACK	FIGURE 3.8

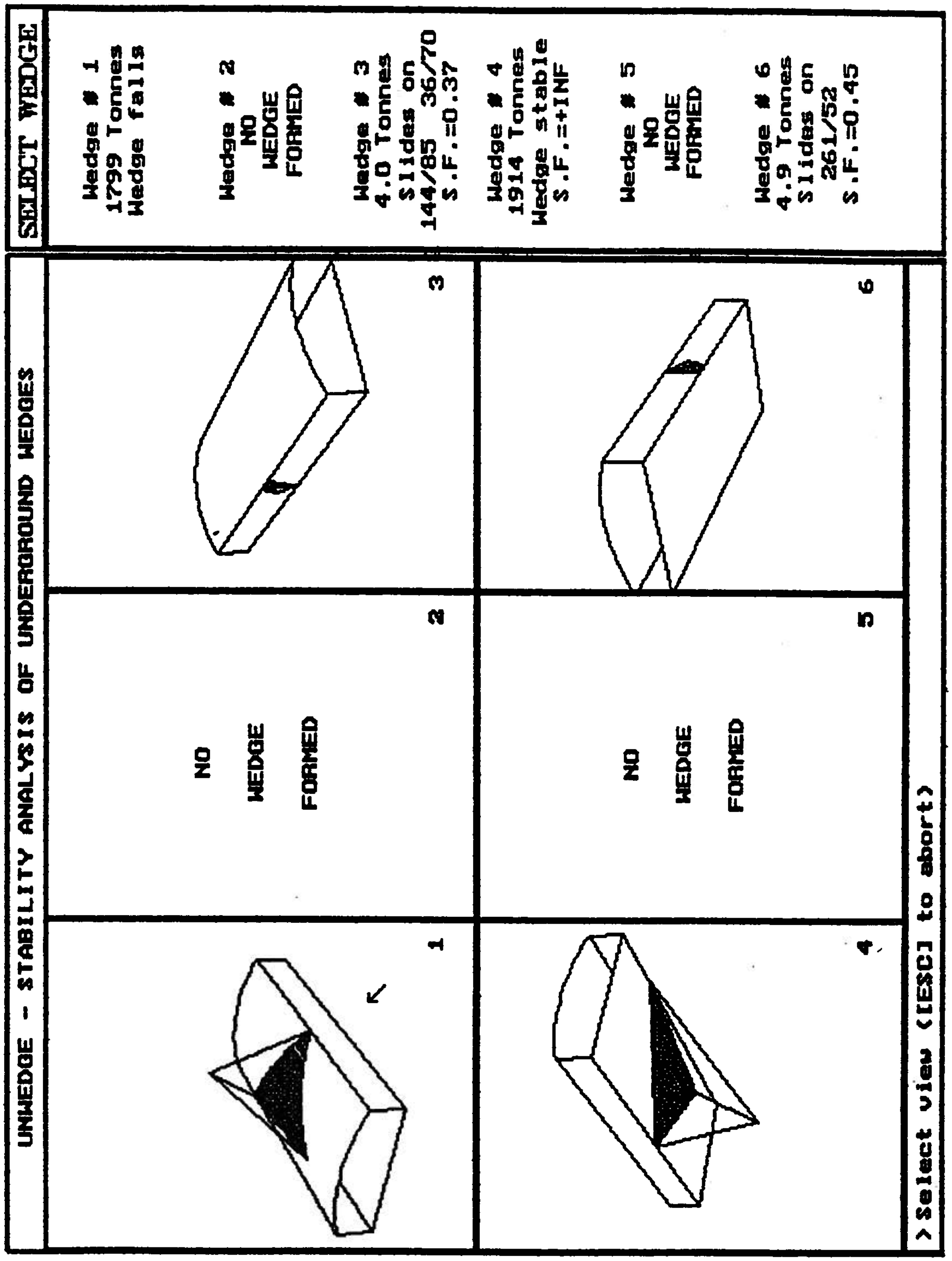


(a) Sliding on Line of Intersection of Joints

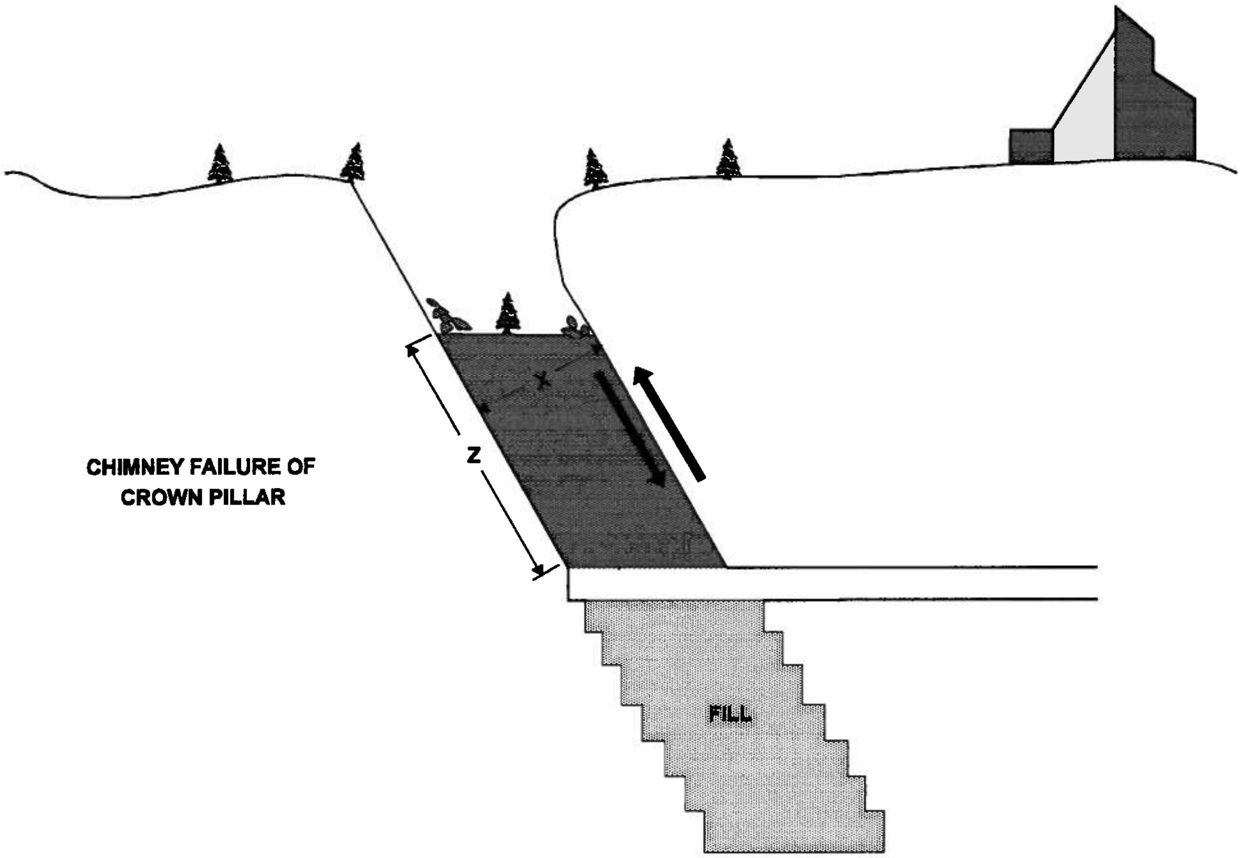


(b) Sliding on Joint A

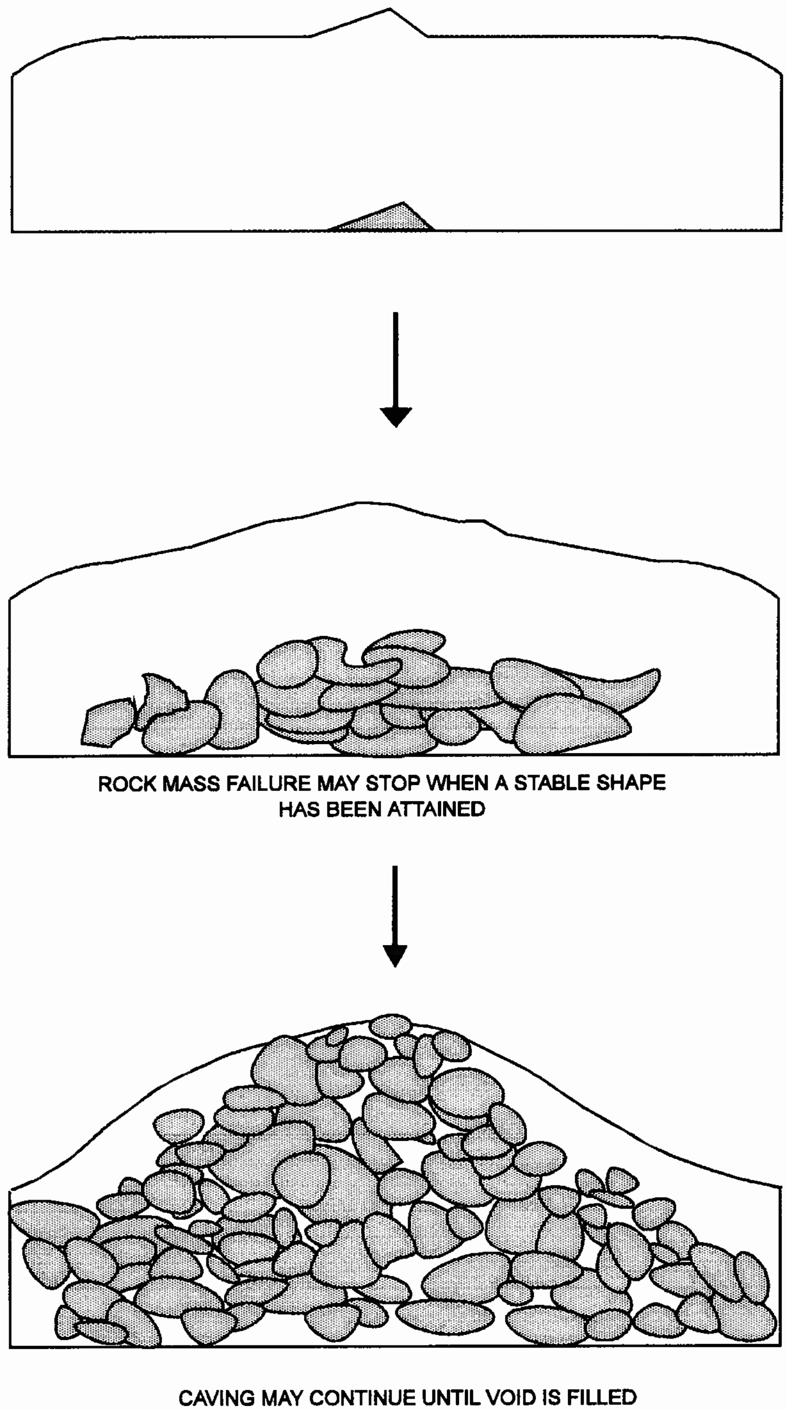
STEREONET ANALYSIS OF WEDGE FAILURES IN WALLS	FIGURE 3.9



UNWEDGE ANALYSIS OF WEDGE FAILURE	FIGURE 3.10

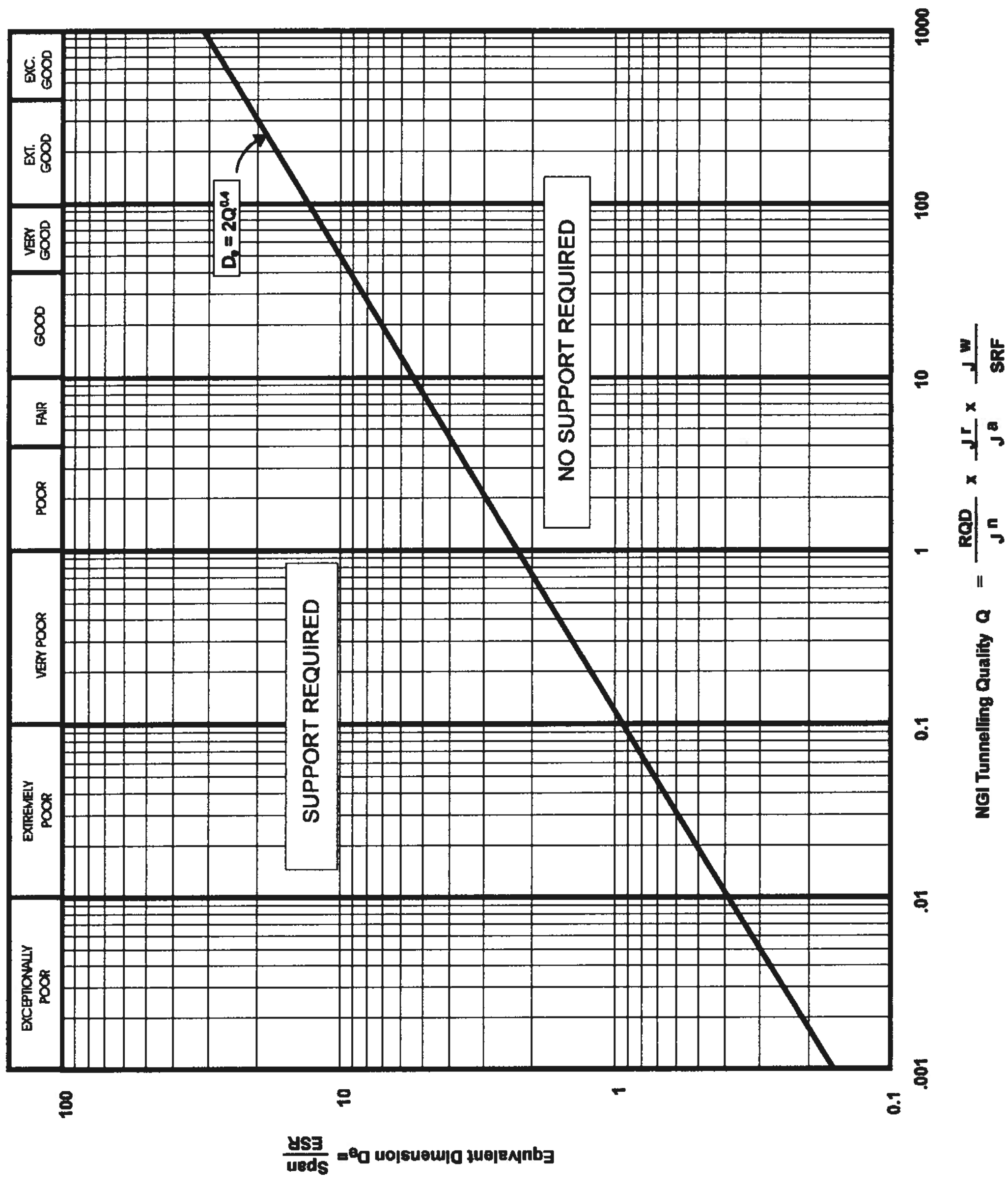


CHIMNEY FAILURE	FIGURE 3.11



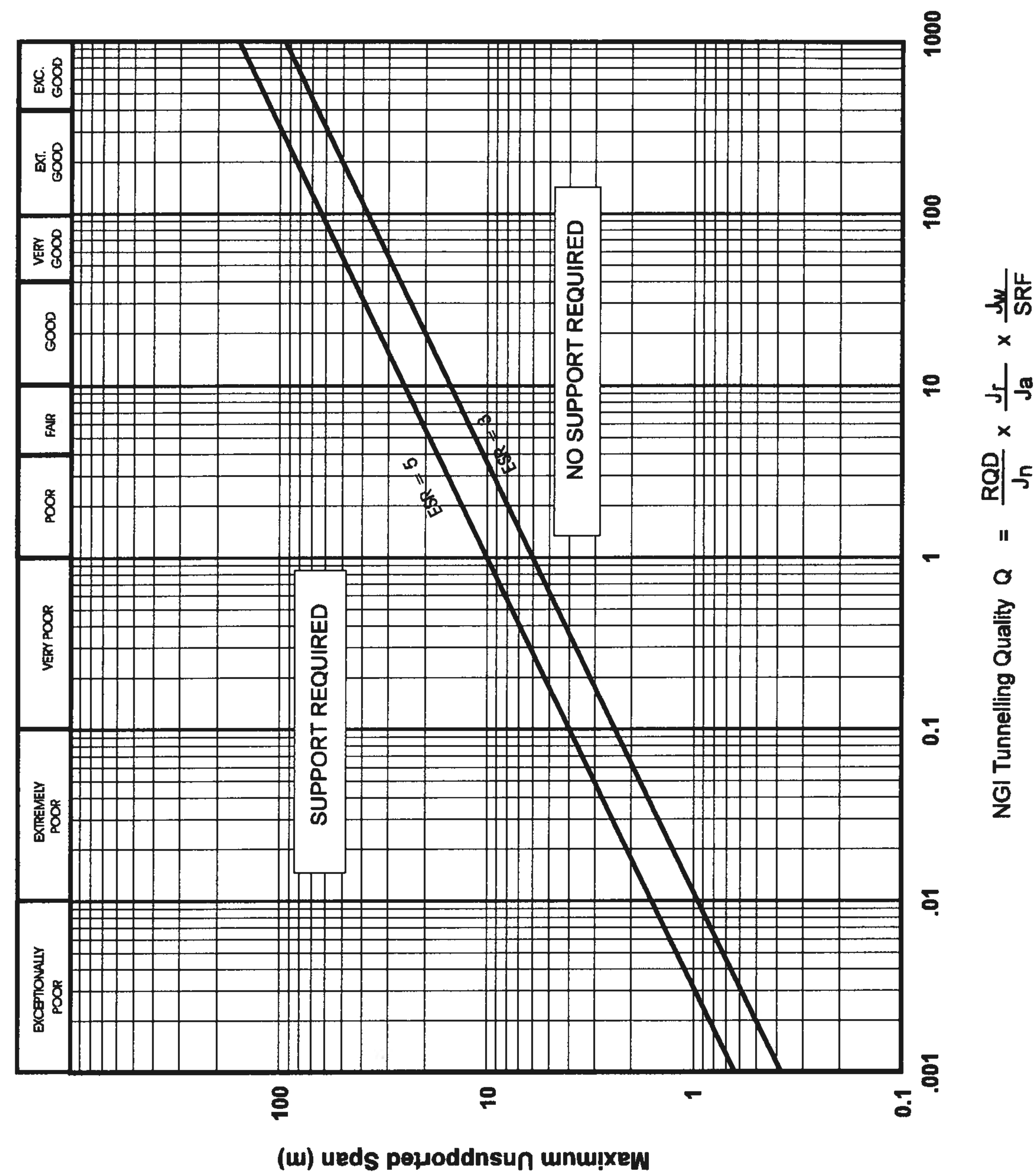
PROGRESSIVE ROCK MASS FAILURE

FIGURE 3.12

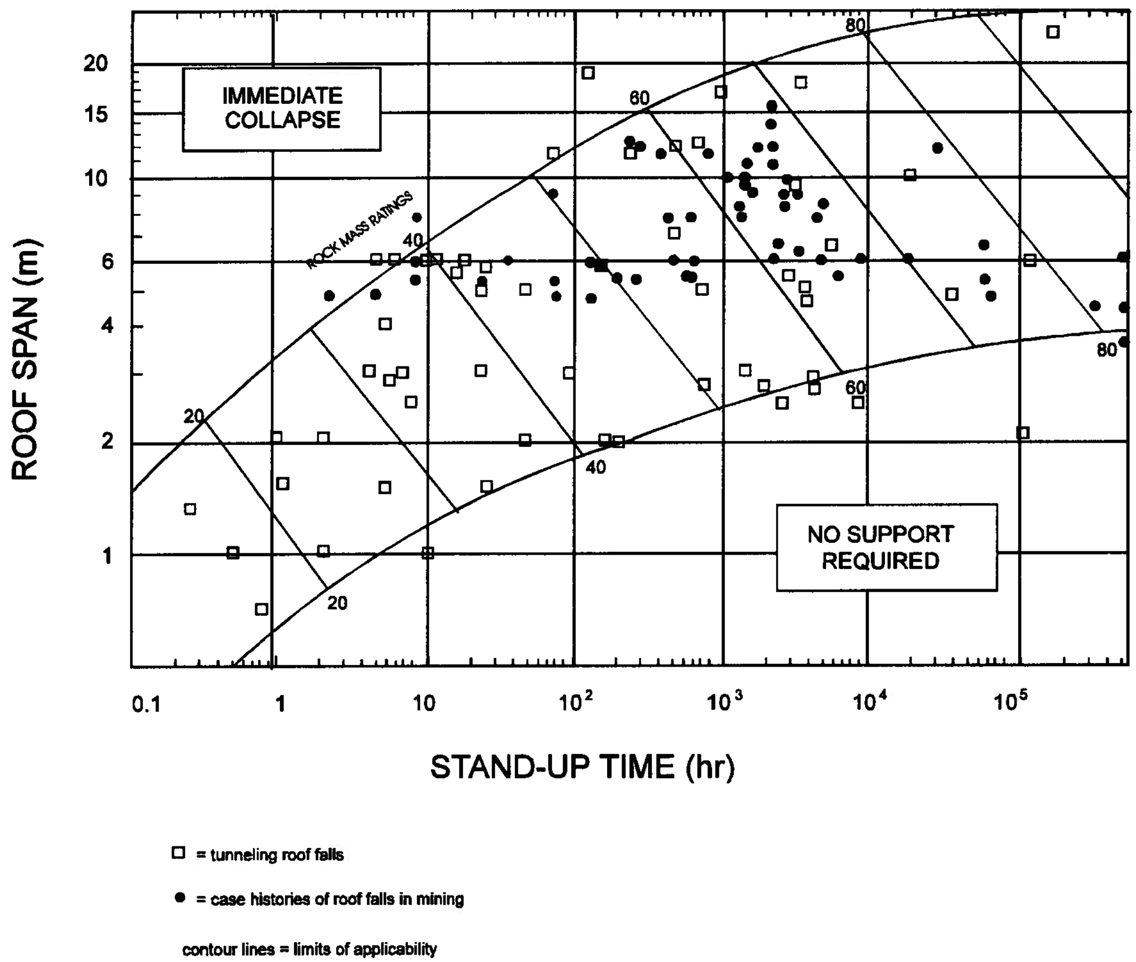


RELATIONSHIP BETWEEN NGI TUNNELING INDEX, Q AND EQUIVALENT DIMENSION, D_e (after Barton et al., 1974)

FIGURE 3.13

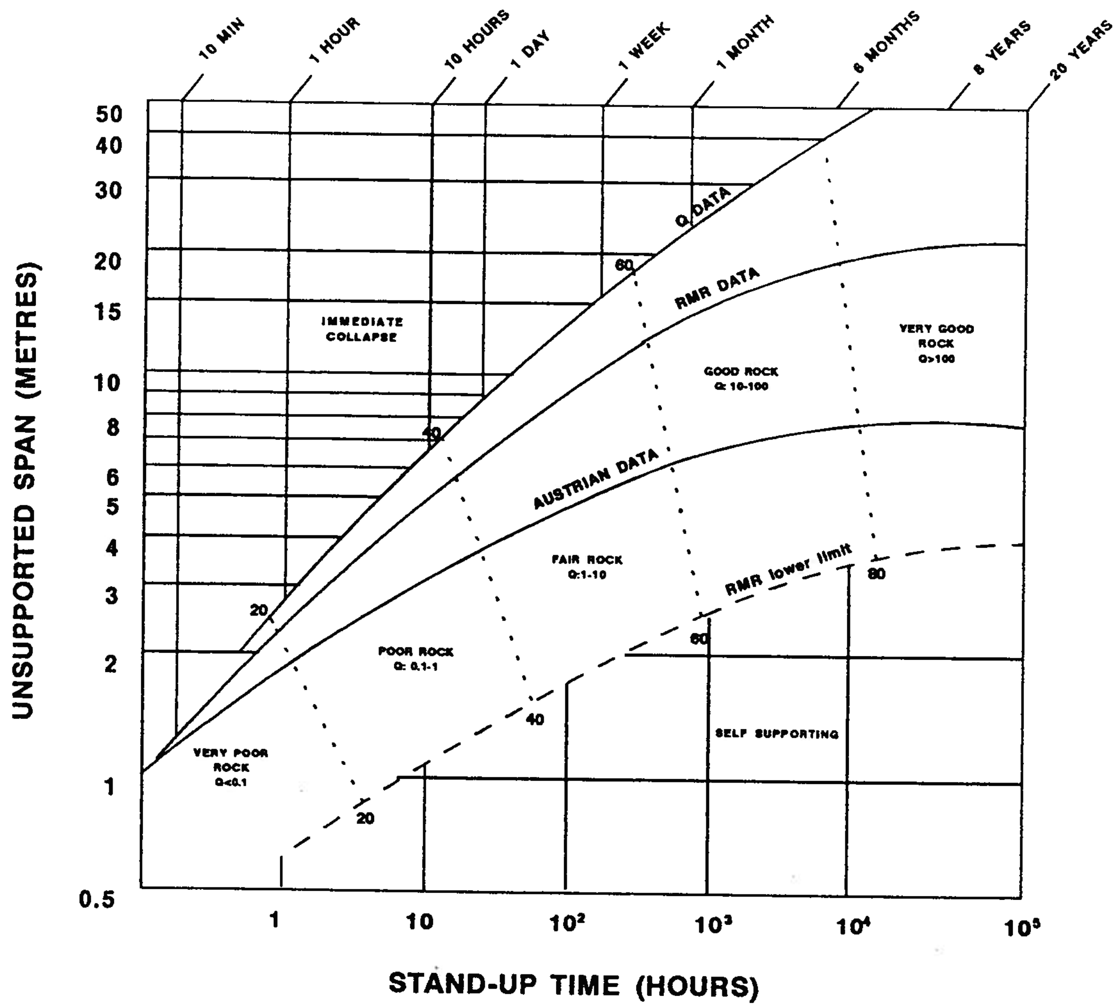


RELATIONSHIP BETWEEN THE NGI TUNNELING INDEX, Q AND SPAN FOR TEMPORARY MINE OPENINGS	FIGURE 3.14



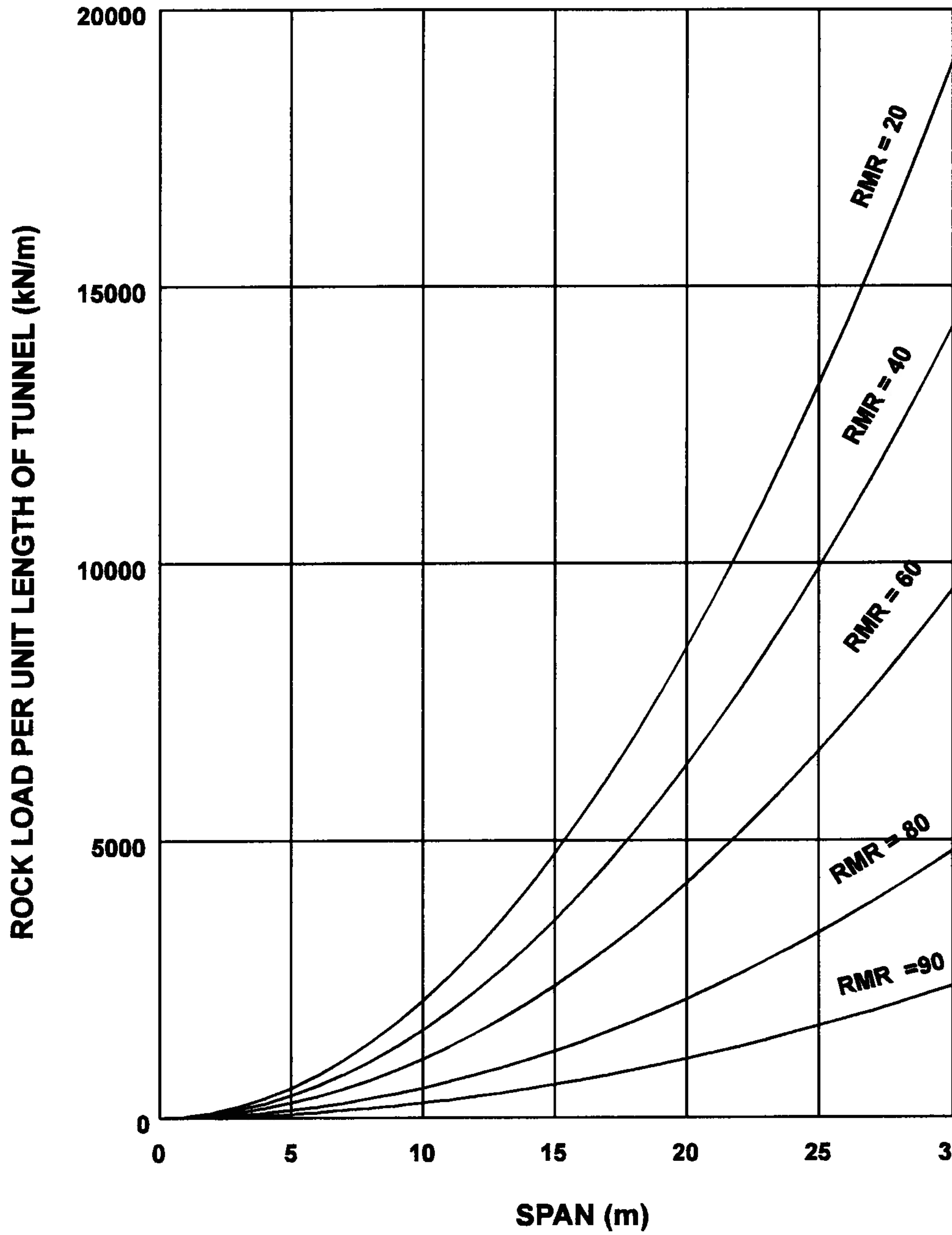
(after Bieniawski,1984)

RELATIONSHIP BETWEEN SPAN, RMR, AND STAND-UP TIME	FIGURE 3.15



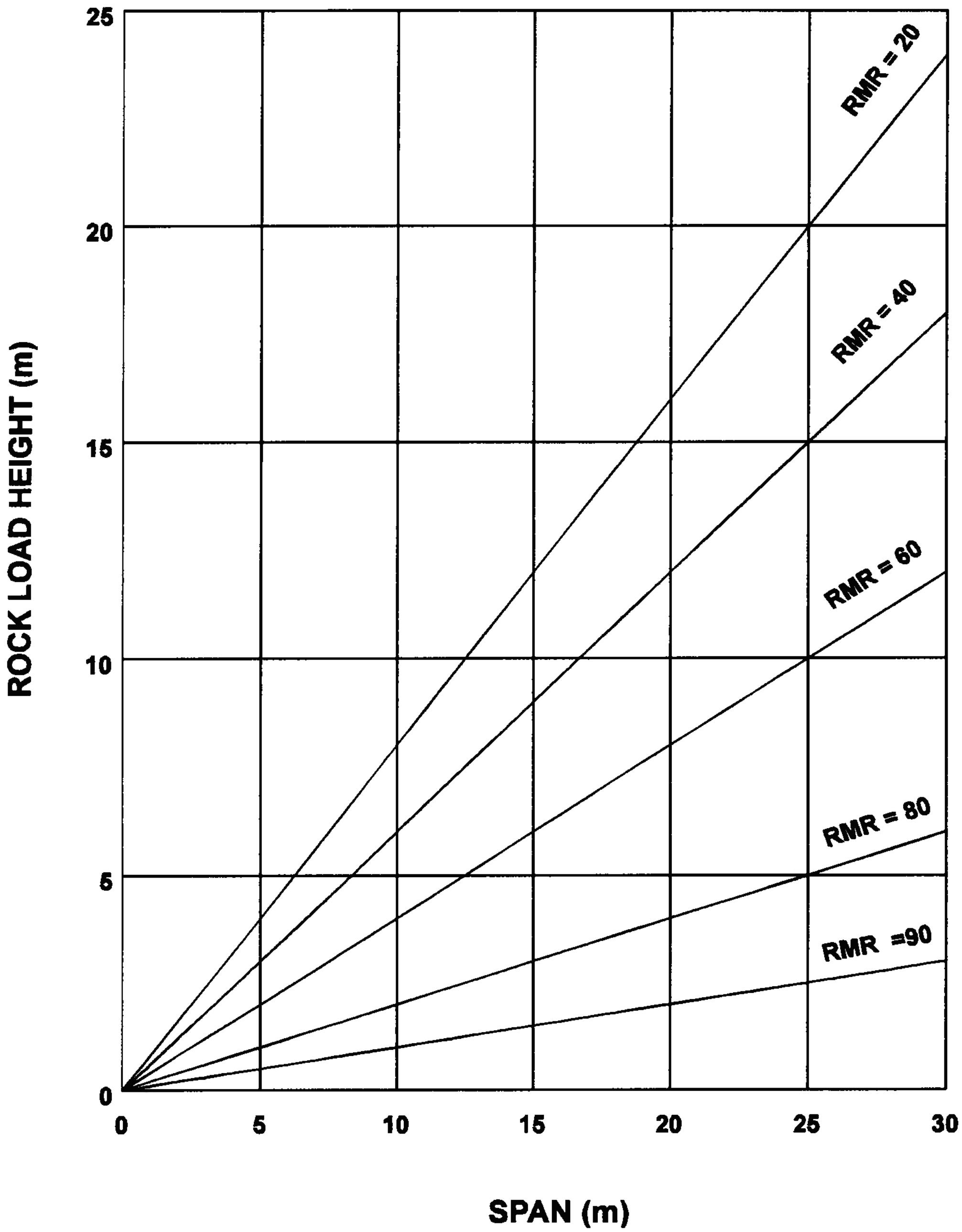
(after Bieniawski, 1974)

COMPARISON OF SPAN VERSUS STAND-UP TIME FOR NGI AND RMR DATA	FIGURE 3.16



(after Unal, 1983)

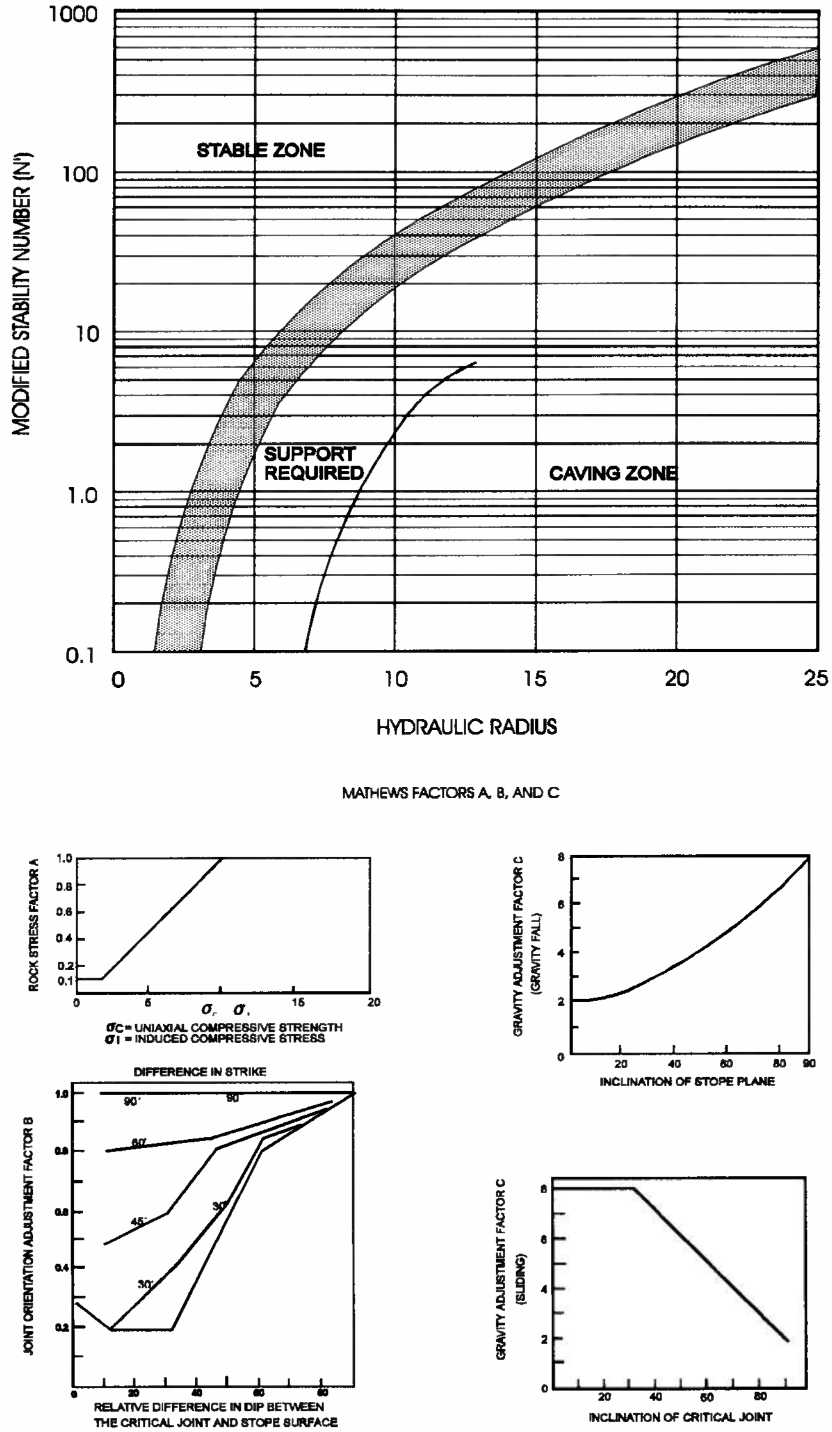
VARIATION OF ROCK LOAD AS A FUNCTION OF SPAN	FIGURE 3.17



(after Unal, 1983)

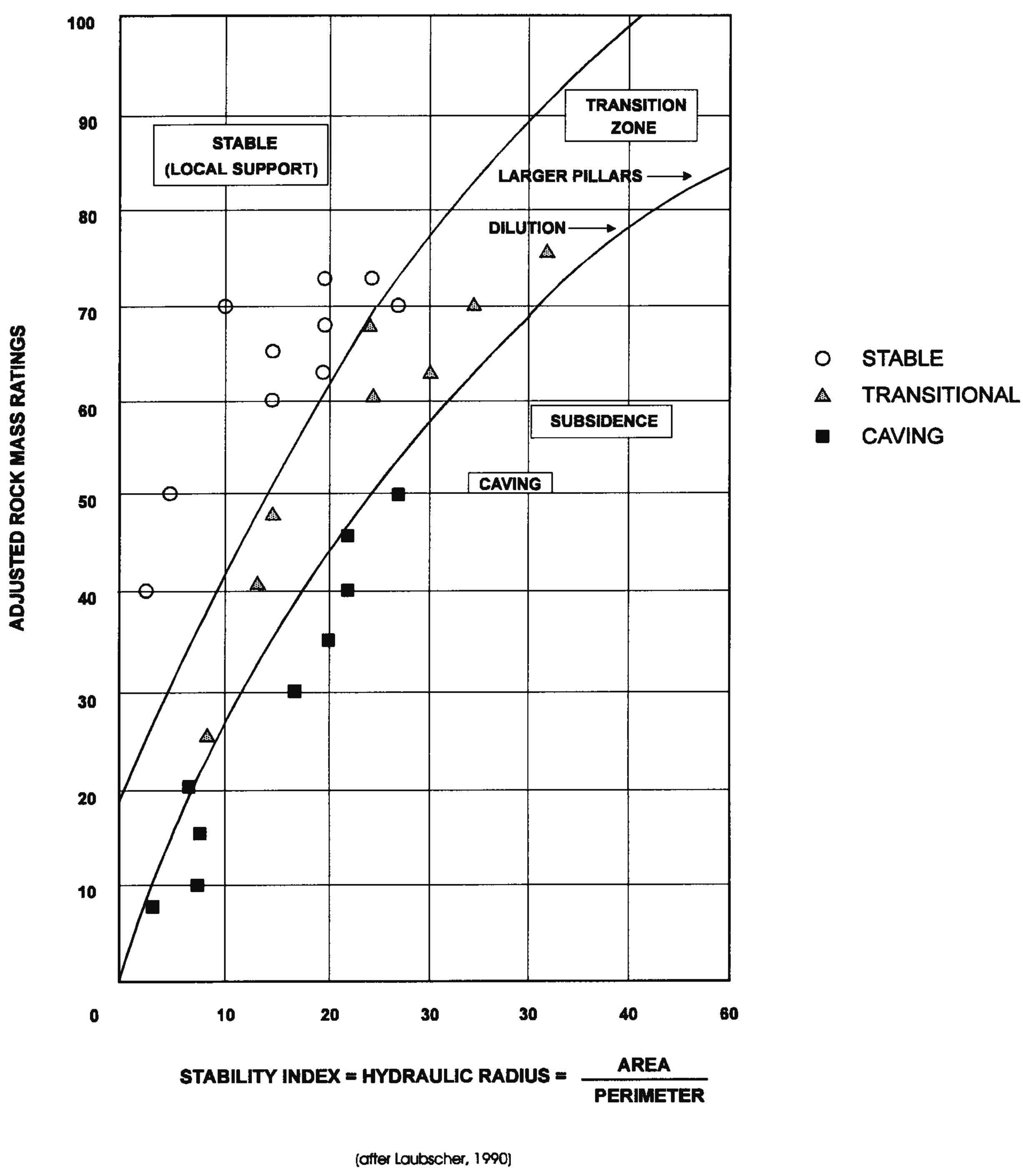
VARIATION OF ROCK LOAD HEIGHT AS A FUNCTION OF SPAN	FIGURE 3.18

MODIFIED STABILITY GRAPH



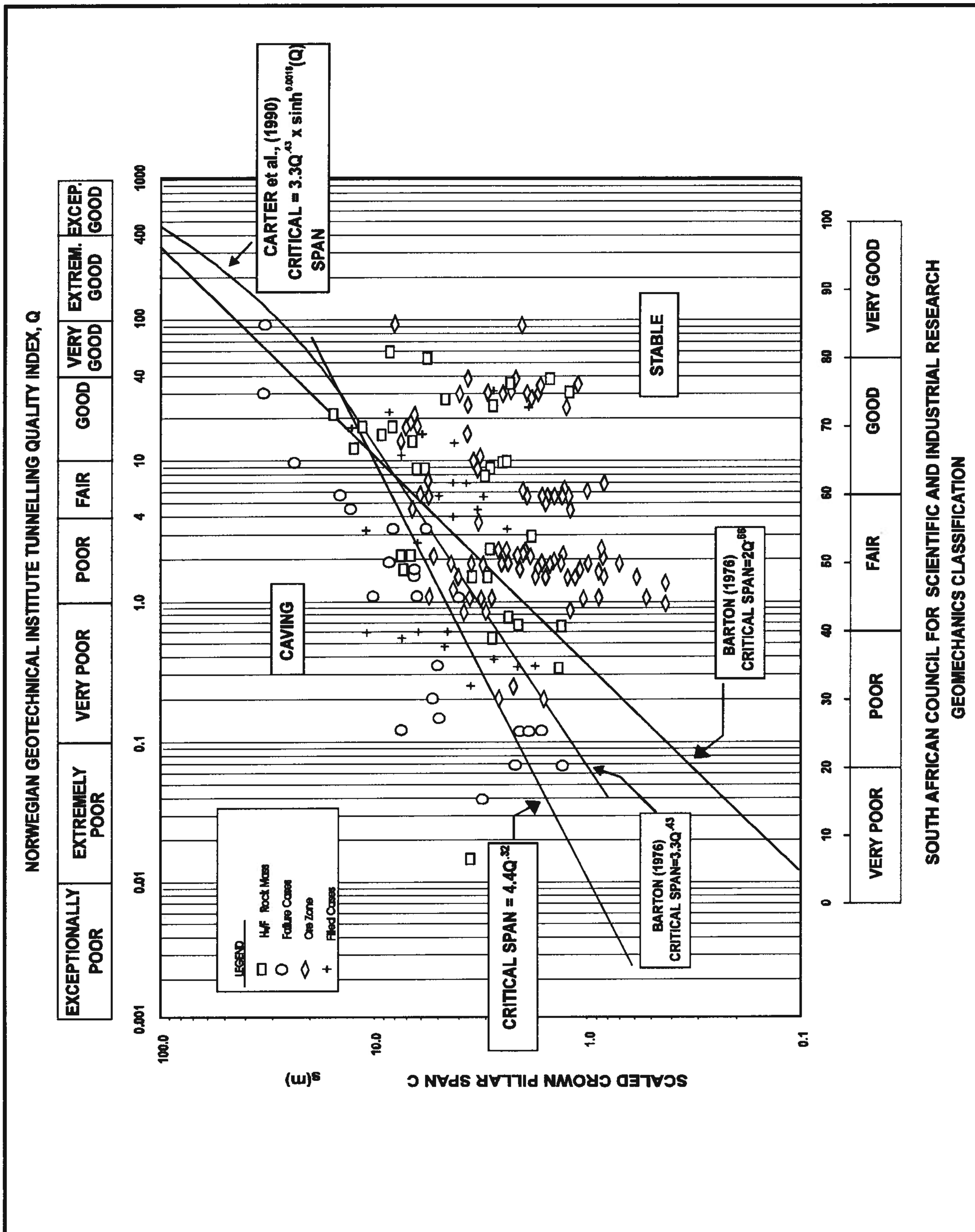
(after Potvin and Milne, 1992)

MODIFIED STABILITY GRAPH	FIGURE 3.19

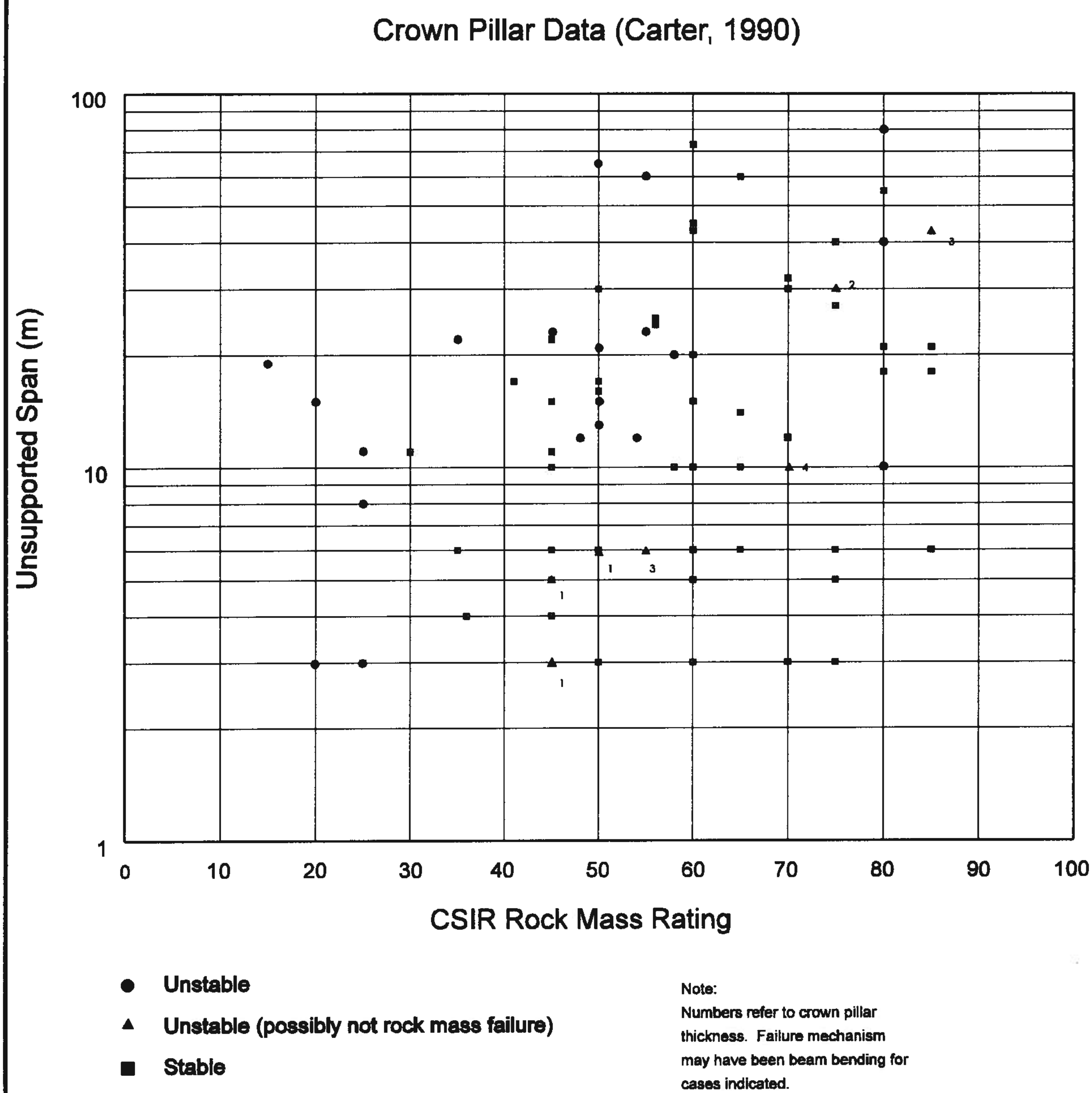


MINING ROCK MASS RATING VERSUS HYDRAULIC RADIUS

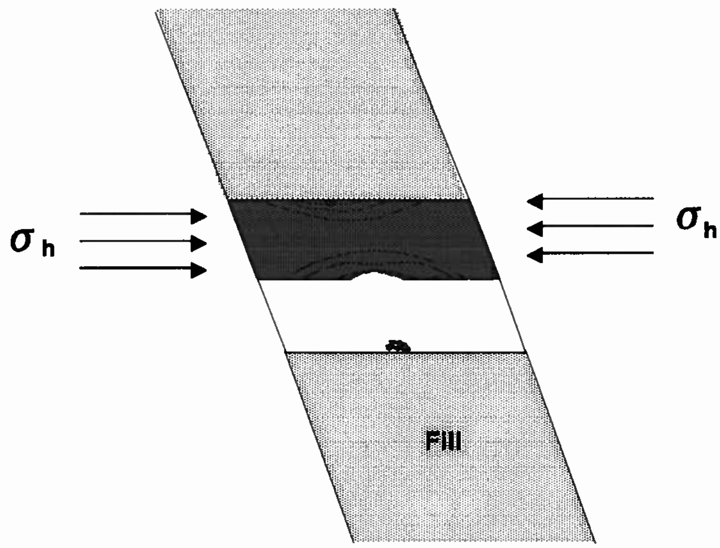
FIGURE 3.20



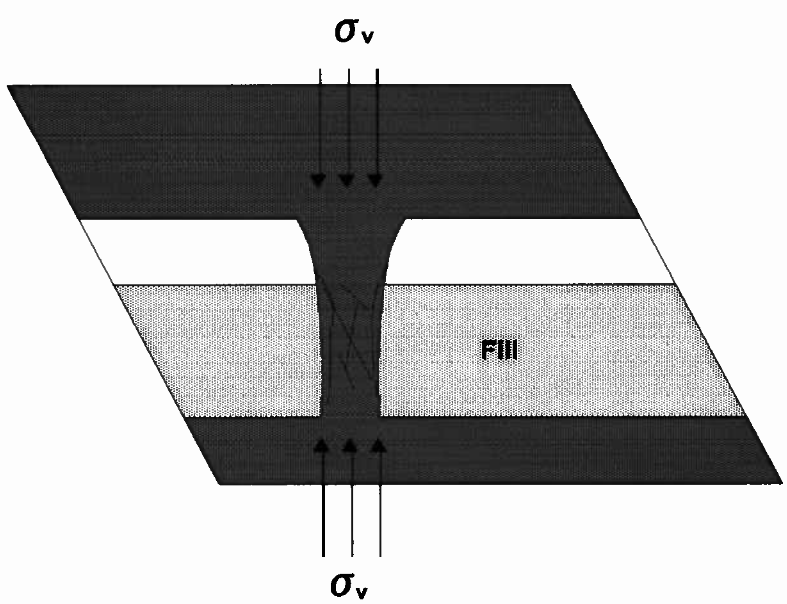
SCALED SPAN VERSUS ROCK MASS RATING (after Carter, 1990)	FIGURE 3.21



CROWN PILLAR STUDY DATA ANALYZED IN TERMS OF SPAN	FIGURE 3.22

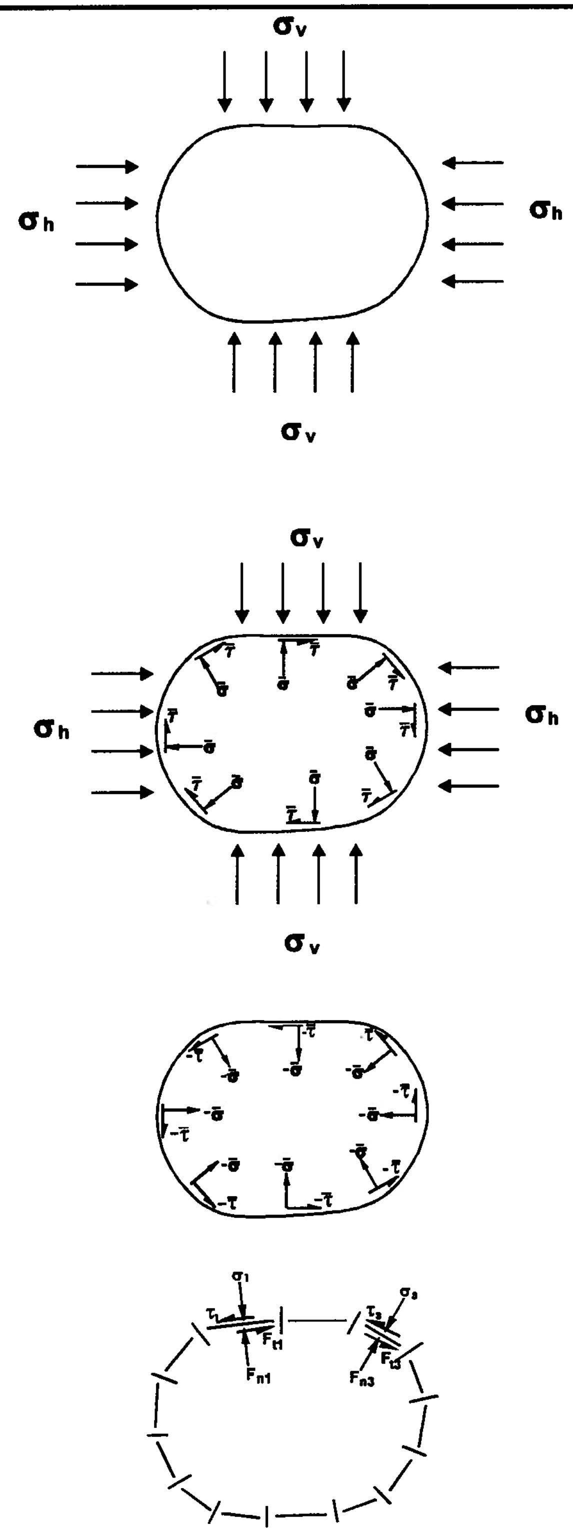


(a) High Horizontal Stress Developing in Sill Pillar



(b) High Vertical Stress Develops In Post Pillars

STRESS INDUCED FAILURE IN PILLARS	FIGURE 3.23



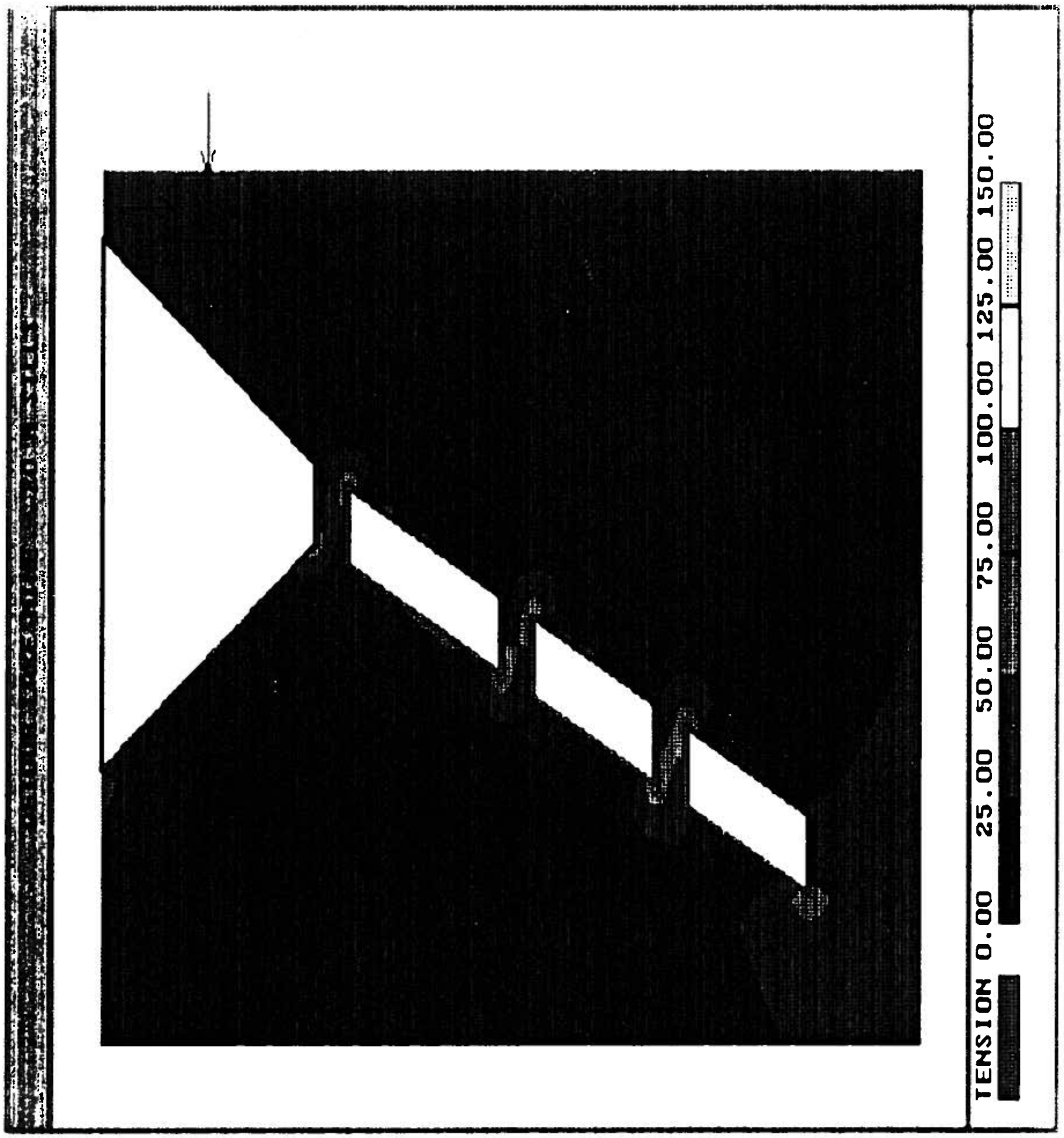
(a) The problem to be solved

(b) Tractions on potential boundary before excavation of hole in an infinite plane

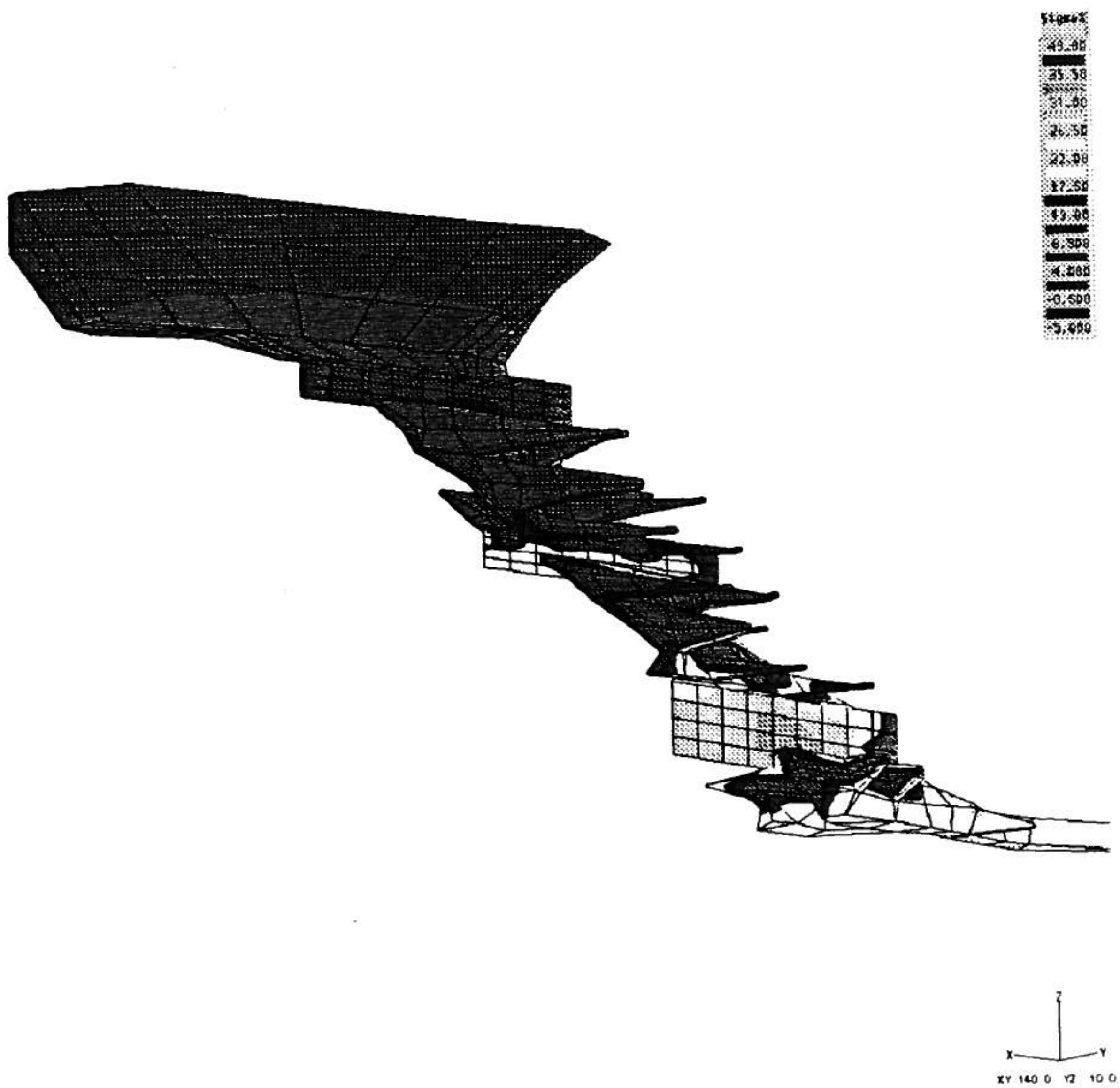
(c) Negative tractions representing effects of excavation

(d) Fictitious forces and stresses on elements of imaginary surface on infinite plane

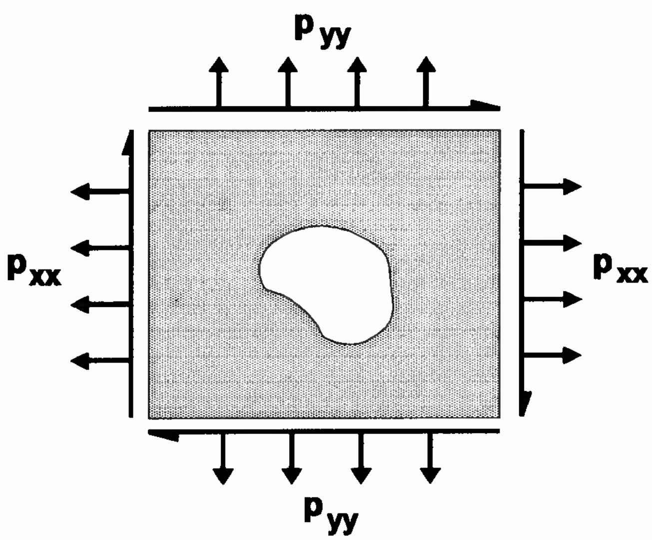
THE BOUNDARY ELEMENT METHOD - 2D (after Hoek and Brown, 1980)	FIGURE 3.24



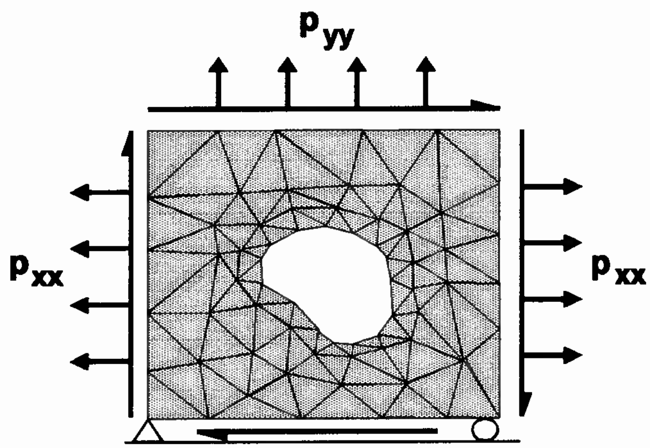
EXAMINE-2D OUTPUT - DETOUR LAKE MINE	FIGURE 3.25



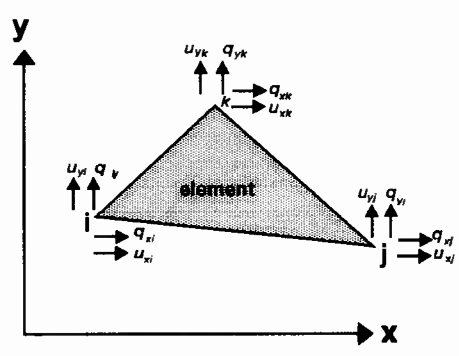
BEAP-3D OUTPUT - DETOUR LAKE MINE	FIGURE 3.26



(a) Excavation to be modelled with field stresses



(b) Discretization of model domain with approrlate boundary conditions

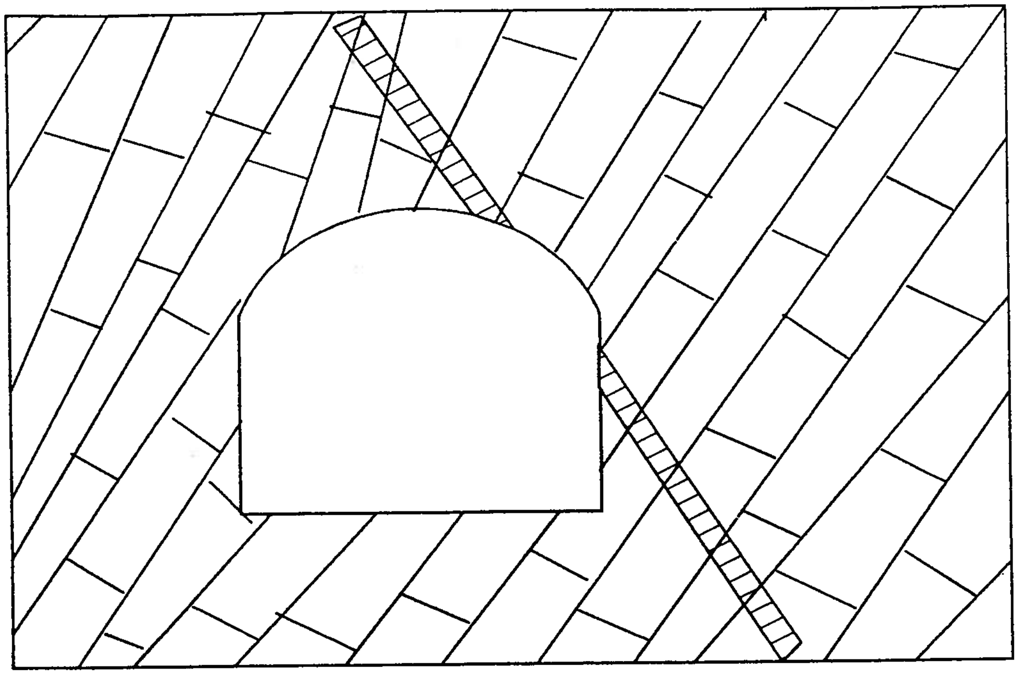


(c) Simple triangular element with six degrees of freedom

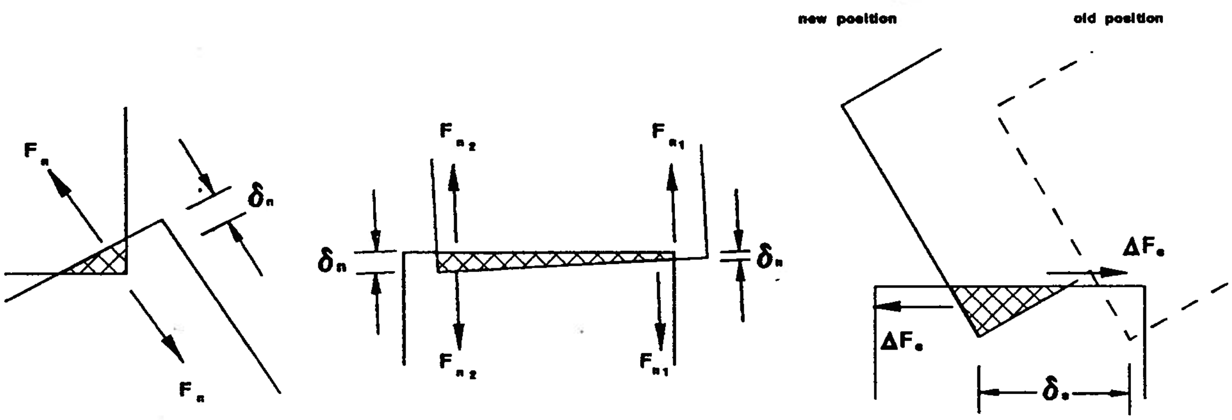
(after Brady et al., 1985)

THE FINITE ELEMENT METHOD - 2D

FIGURE 3.27



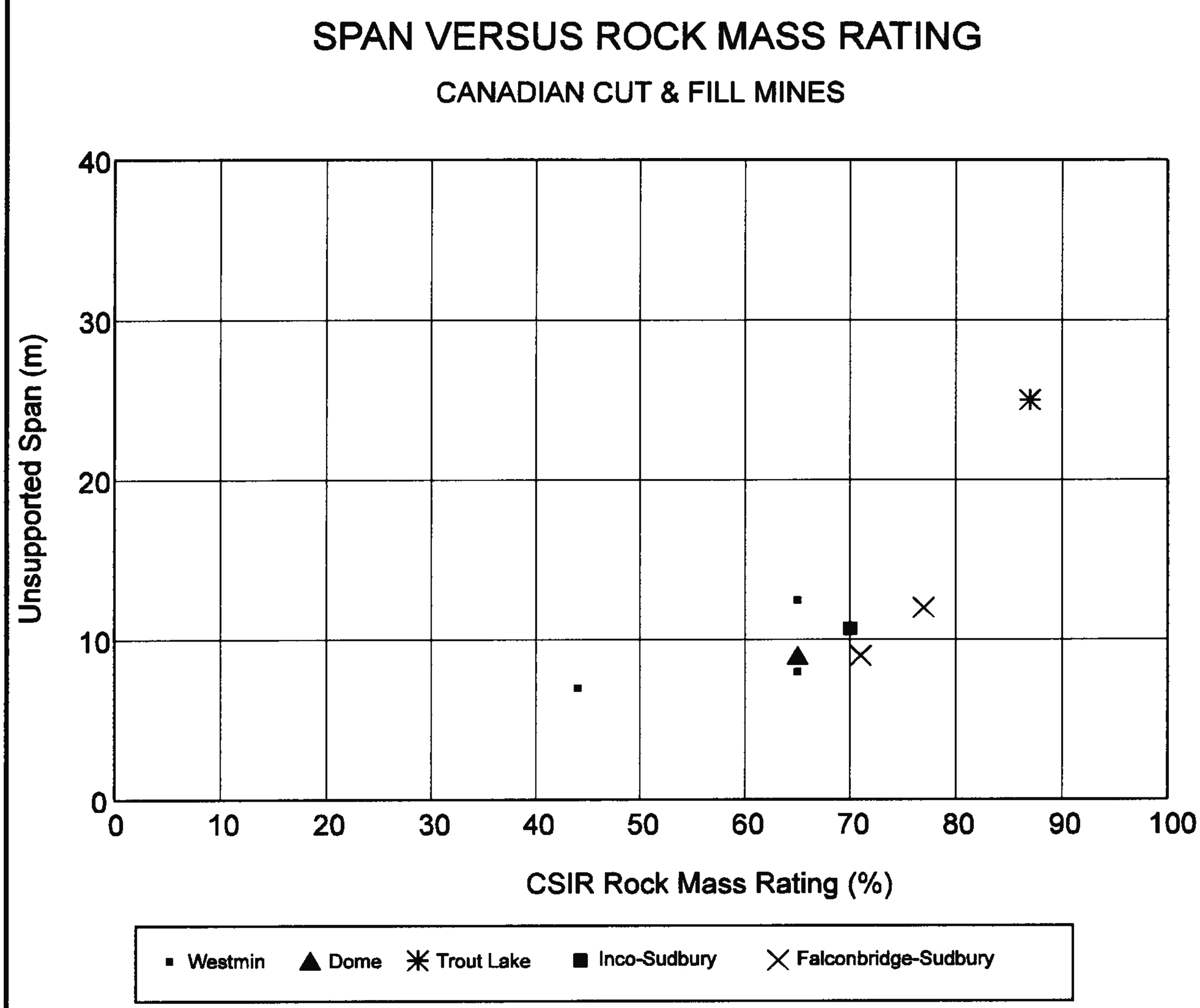
a)Schematic representation of an opening in which the stability is controlled by discrete rock blocks and therefore amenable to modelling by the distinct element method



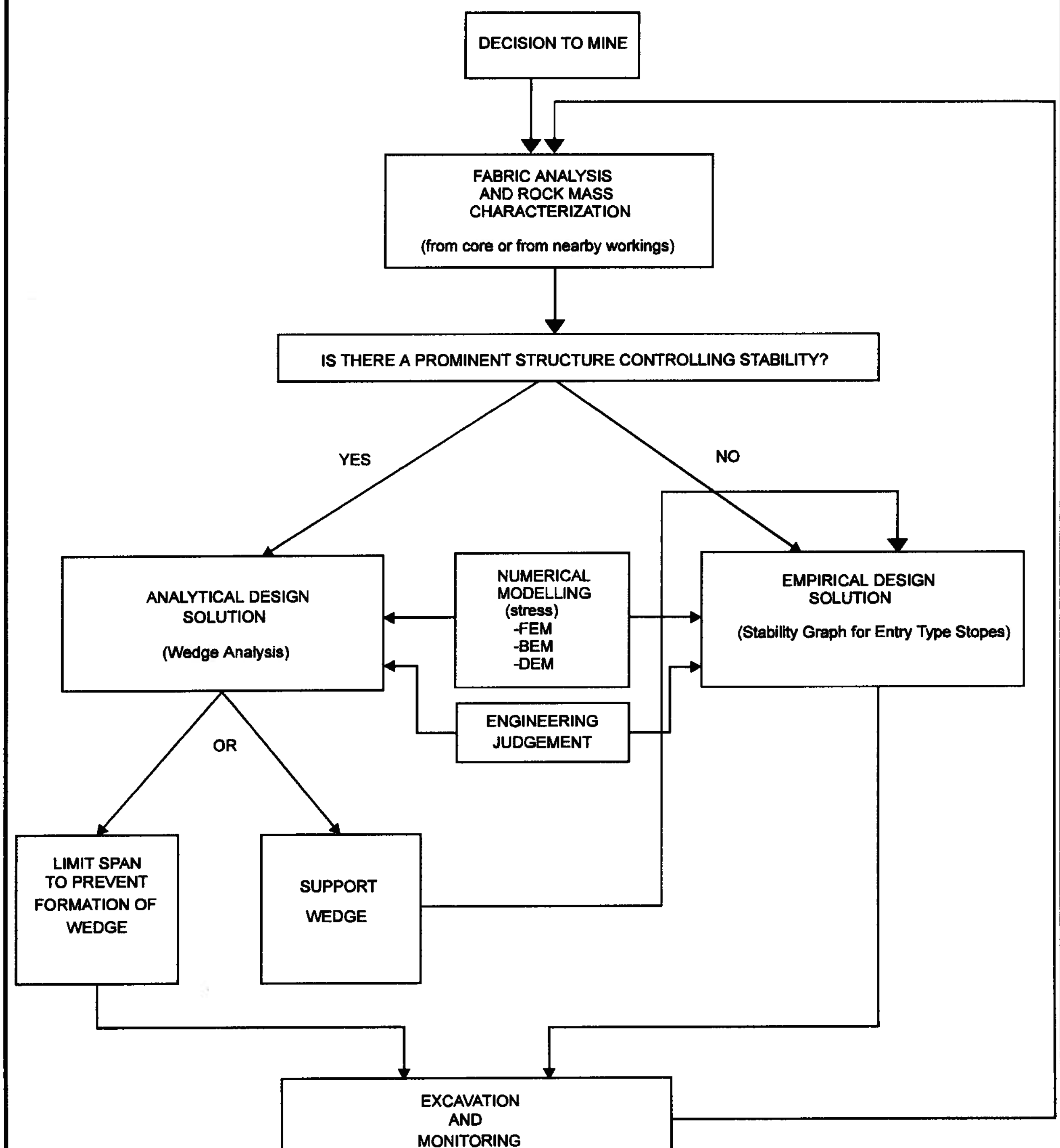
b) Normal and shear modes of interaction between distinct elements

(after Brady and Brown, 1985)

THE DISTINCT ELEMENT METHOD -2D	FIGURE 3.28



SPAN VERSUS ROCK MASS RATING AT OTHER CANADIAN OPERATIONS	FIGURE 3.29



SPAN DESIGN METHODOLOGY AT DETOUR LAKE MINE

FIGURE 3.30

4. DATABASE

4.1 INTACT STRENGTH PROPERTIES

An estimate of the intact rock strength parameters is required for rock mass classification and for input into stress modeling programs. Prior to this study, direct unconfined compressive strength test data of Detour Lake Mine rock had been conducted by:

- CANMET (March, 1985);
- Terraprobe (July, 1984); and,
- Smith Engineering (October, 1984).

The results of these tests are summarized in Table 4.1. It must be recognized that the above testing was conducted prior to the commencement of underground mining. Once underground access was available, indirect UCS measurements were obtained from point load strength tests on representative samples from 120 Level, 360 Level, and 560 Level. These results are presented in Figure 4.1 and 4.2. There was a large variation between all of these strengths so further UCS testing was carried out at University of British Columbia.

Representative AQ drill core (26 mm diameter) was obtained from the mine for testing. All samples failed through intact rock. The results are given in Figure 4.3 and 4.4. Unconfined compressive strength test results from UBC for each of the three zones are compared to results of other researchers in Figure 4.5. Testing by UBC indicated a UCS for the mafic hangingwall rock of 165 MPa. This compares well to a weighted average of 162 MPa for all other direct tests. The UCS of the MZ rock was very similar to that of the hangingwall at 166 MPa. This is to be expected since the lithology of the HW and the MZ is similar except for the presence of veining in the MZ. The UBC result is similar to the value obtained by CANMET (169 MPa) but significantly higher than the Terraprobe value (91 MPa).

UBC testing of the Talc Zone rock indicated a significantly lower average UCS of 28 MPa as compared to the average of other researchers (93 MPa). Previous test work had been carried out on near surface rock which may account for the difference. The UBC results are supported by recent tests carried out by Voest-Alpine (1991) in their recent assessment of the suitability of a roadheader for mining of the Talc Zones. Strengths were obtained between 22 and 38 MPa with an average of 29 MPa based on four samples taken from the 460 Talc Zones. Another series of tests were carried out by UBC to confirm these results. Rock samples were obtained from attack drifts in the 360 T20 #3 and 460 T40 #6 stopes. The rock samples were subsequently cored to 52 mm diameter and tested. The results are given in Table 4.2. The unconfined compressive strength was higher at 48 MPa. The samples from these tests appeared to be more siliceous than previous samples tested which may have contributed to the higher strength. These results can be considered to represent an upper bound on the unconfined compressive strength of the Talc Zone rock.

Given that the UBC values were averaged from 10 tests and were taken from the most recently mined areas, the values were used as design strengths for the purposes of this study. These and other rock properties which have been measured at Detour Lake Mine are summarized in Table 4.3.

4.2 FABRIC ANALYSIS

As part of this study, detailed fabric mapping was carried out in the following areas of the Main Zone and Talc Zones:

260 MZ #12 Lift	360 T20 #4
260 T40 #1	560 MZ # 1
360 MZ #12 Lift	460 T40 #6
360 T40 #3, #4	460 T40 #5
360 MZ # 5 Lift	

Detailed line mapping was carried out and features were recorded on a standard geotechnical mapping sheet (Figure 4.6). The following information was recorded: location, distance, rock type, structure type, number of features having similar orientation, spacing, rock and joint infill hardness, strike, dip, aperture width, planarity, roughness, continuity, and water. Approximately 350 structures were mapped in the Main Zone and 419 in the Talc Zones.

4.2.1 Joint Orientation

The orientations of the structures mapped during this study have been plotted on lower hemisphere equal area stereonets (Figures 4.7 to 4.8). In addition to the structures mapped as part of the detailed line mapping, a compilation was made of structural mapping recorded by the DLM geology department on previously mined lifts. Figures 4.9 to 4.12 show stereonets which record structural data for each lift studied. These stereonets suggest a north-south joint set and an east-west joint set both dipping vertical to subvertical. Random jointing is also present. Figure 4.13 shows stereonets for the Main Zone and for the Talc Zone with structures from all lifts plotted. Two joint sets are predominant:

Joint Set A: Mean Orientation: Strike: 096° Dip: 90°
Joint Set B: Mean Orientation: Strike: 353° Dip: 83°

The same two joint sets are predominant in the Talc Zones with only slightly different orientations:

Joint Set A: Mean Orientation: Strike: 093° Dip: 90°

Joint Set B: Mean Orientation: Strike: 352° Dip: 86°

4.2.2 Joint Roughness

Joint Roughness has been measured by comparing the profile of each joint to 10 standard profiles shown in Figure 4.14. The joint roughness for the MZ and TZ have been plotted on a frequency histogram (Figure 4.15(a)). In the case of the mafics, the highest percentage of joints have a Joint Roughness Coefficient (JRC) of 2-4 indicating at smooth, planar surface. Most other joints have a JRC of 4 to 10. In the Talc Zones, the highest percentage of joints have a JRC of 6-8 which is a rough stepped surface.

4.2.3 Rock Strength

The rock strength was established by field index testing described in Table 4.4. The rock hardness is placed in 5 categories, R1 being the weakest to R5 being the hardest. The frequency histogram for rock hardness is given in Figure 4.15(b). The majority of Main Zone rocks tested have a hardness of R4 indicating a UCS of 100 - 200 MPa. All other rocks in the Main Zone had an R5 hardness. Approximately 75% of the Talc Zone rocks had an R3 hardness (50 - 100 MPa) with the remainder being R2 in hardness.

4.2.4 Joint Aperture

Joint aperture was classified into 5 categories ranging from very tight to moderately wide (Table 4.5). Figure 4.16(a) is a frequency histogram indicating the joint aperture for talc and mafic rocks. Most mafics and talc rocks have joints which are tight to very tight.

4.2.5 Joint Spacing

The joint spacing is classified into the three categories used by the CSIR rock mass rating classification system (Figure 4.16(b)). Most main zone joints have a spacing between 5 cm and 1 metre. A smaller percentage of joints have a spacing of 1-3 metres.

4.2.6 Joint Continuity

Joint continuity has been classified into three categories, 0 - 3 m, 3 - 5 m, and 5 - 10 m. This data has been plotted on a frequency histogram shown in Figure 4.17(a). Talc Zone joints occur in all categories. Most joints in the mafics occur in the 3 - 5 m and 5 - 10 metre categories. In some cases, it may not be possible to establish the full length of the joint if it continues beyond the wall of the excavation. Figure 4.17(b) indicates the number of joint ends which are visible. The Talc zone joints are generally less continuous with 1 or 2 ends visible. Most mafics have 0 of 1 end visible. Zero ends visible indicates that the joint is longer than the dimensions of the excavation. The openings in the mafics are wider so this would indicate that joints are generally longer in the MZ than the TZ.

4.3 ROCK MASS CHARACTERIZATION

All work areas including those mined prior to the study were assessed a CSIR rock mass rating. The rating was performed using a standard rock classification form shown in Figure 4.18. The CSIR rock mass classification system was chosen for use at DLM for the following reasons.

- Consistency of results among those performing the rating;
- It relatively quick to use and understand; and,
- The percentage scale is easier to get a "feel" for than the logarithmic NGI - Q rating.

The RMR can be converted to the NGI - Q using Equation 3.33. This relationship was checked periodically and proven to be valid for the accuracy required.

4.3.1 Main Zone

The rock mass ratings for the MZ are compiled in Table 4.6. The average RMR in the Main Zone is 73 with a standard deviation of 7.8. The lower ratings generally occur on the footwall side of ore body where the joint spacing is closer and the orientation of the joints is more random. A typical CSIR rock mass rating, based on the fabric analysis presented previously, is provided in Table 4.8.

4.3.2 Talc Zone

CSIR rock mass ratings for all of the Talc Zone workings analyzed are provided in Table 4.7. The mean RMR for talc is 49 with a standard deviation of 11. The rock quality generally decreases with increasing distance from the chert horizon. From the fabric analysis presented above, a typical CSIR - RMR for the TZ is given in Table 4.8.

4.3.3 Hangingwall Rock

Rock mass ratings were not routinely performed in the hangingwall mafic rock since the rock quality is consistently high, (averaging approximately 80%) and the development headings are generally not wider than 5 m. Rock mass ratings were sometimes carried out in larger excavations such as maintenance shops and lunchrooms.

4.4 STRESS DETERMINATION

The principal stresses at Detour Lake Mine were determined in 1985 by CANMET using overcoring techniques (Arjang et al. 1985). The measured stresses are comparable in direction and magnitude to other measurements made at shallow depth in northeastern Ontario and Quebec (Figure 4.19). The vertical stress (σ_3) has a magnitude of 0.029 MPa/metre depth. The major principal stress acts in a ENE-WSW direction. For modeling purposes, the major principal stress (σ_1) is assumed to act horizontally, parallel to the strike of the orebody and with a magnitude of 2.6 times the vertical stress. The intermediate principal stress (σ_2) is assumed to act perpendicular to the strike of the orebody with a magnitude of 1.3 times the vertical stress.

The major principal stress is favourably oriented in approximately the same direction as the strike of the orebody (Figure 2.3). The stress conditions in the sill pillars will therefore be influenced mainly by the intermediate stress, σ_2.

Prior to filling the undercut of the three active stopes, five vibrating wire stress meters were installed in each sill pillar at approximately 30 metre intervals along strike. The purpose of this instrumentation is to determine actual stress changes in the sill pillar and to use the information to calibrate numerical modeling estimates of the stress changes.

The stress meters are normally read on a monthly basis but are read on a weekly basis when the sill pillar thickness becomes 30 metres or less. Figures 4.20 to 4.22 show the results of the stress monitoring for the 260, 360, 460, 560 sill pillars as well as the surface crown pillar. All but one of the original stress metres in the 260 pillar are no longer functional. It is believed that the cable has been severed within the fill. Three additional stress meters were installed in January, 1991.

4.5 MINING HISTORY

A historical record of the mining at Detour Lake Mine has been compiled such that recorded stress changes can be related to mining activity Figure 4.23 indicates the time span during which mining of each lift took place.

Mining began on the 260 and 360 stopes in 1987 and proceeded at a rate of approximately 2 months per lift. Due to the greater ore widths, mining on the 460 Level proceeded at a rate of approximately 4 months per lift. Mechanized cut and fill stoping recently began on the 560 Level and is progressing at a rate of 2 months per lift. Mechanized cut and fill mining of the 260 stope was completed in October, 1991, leaving a 26 metre thick crown pillar between the pit bottom and the back of the 260 stope. There are no plans to recover this pillar until the end of the mine life. Mechanized cut and fill mining of the 360 stope was completed in January, 1992, leaving a 16 metre thick sill pillar.

Table 4.1 Summary of Intact Rock Strength Characteristics

Rock Type	Specific Gravity	UCS (MPa)	Tensile Strength (MPa)	Modulus of Elasticity (GPa)	Poisson's Ratio	Friction Angle	Cohesion (MPa)
MAFIC- Hangingwall and Footwall							
CANMET	2.9±0.02 (12)	270±42 (5)	20±1 (8)	88±4 (6)	0.25± 0.01 (6)	50 (4)	60 (4)
TERRAPROBE	*	91±17 (5)		26±5 (5)	0.29± 0.07 (5)		
J. SMITH		147±31 (12)	19± (12)	57±13 (12)	0.1±0.02 (11)		
MAIN ZONE							
CANMET							
Silicified	3.0±0.02 (9)	191±1 (3)	20±1 (6)	108±8 (3)	0.24± 0.03 (3)	51 (4)	44 (4)
Main	2.9±0.01 (19)	169±26 (10)	19±1 (9)	89±6 (10)	0.26± 0.02 (10)	43 (4)	40 (4)
Average	3.0±0.06 (28)	174±69 (13)	20±2 (15)	93±18 (13)	0.26± 0.05 (13)	47 (8)	42 (8)
TERRAPROBE	*	93±24 (5)		27±4 (4)	0.43± 0.12 (4)		
TALC ZONE							
CANMET	3.0±0.004 (9)	92±2 (3)	10±1 (7)	51±5 (3)	0.34± 0.02 (3)	42 (4)	20 (4)
TERRAPROBE		93±14 (5)		31±4 (5)	0.53**± 0.11 (5)		

* failed along existing discontinuities

** reported value, however this is higher than the possible range for rock

( ) number of samples tested

Table 4.2 UBC Unconfined Compressive Strength Test Results

	Talc Zone	Number of Tests
Unconfined Compressive Strength	48 ± 5 MPa (9 tests)	9
Modulus of Deformation	32 ± 2 GPa (5 tests)	5
Poisson's Ratio	0.27 ± 0.06 (5 tests)	5
Specific Gravity	3.0	

Table 4.3 Detour Lake Mine - Rock Properties

Main Zone	
Unconfined Compressive Strength	165 MPa (avg), 190 MPa(upper)
Modulus of Deformation	93 GPa
Poisson's Ratio	0.26
Specific Gravity	3.0
Talc Zone	
Unconfined Compressive Strength	28 MPa (lower), 48 MPa (upper)
Modulus of Deformation	32 GPa
Poisson's Ratio	0.22
Specific Gravity	3.0

Table 4.4 Approximate Classification of Rock Based on Strength (after Brown, 1981)

No.	Description	Unconfined Compressive Strength			Examples
		lb/in^2	kg/cm^2	MPa	
R1	**VERY WEAK ROCK** -Crumbles under sharp blows with geological pick point, can be cut with a pocket knife.	150-3500	10-250	1-25	chalk, rock salt
R2	**MODERATELY WEAK ROCK** -Shallow cuts or scraping with pocket knife with difficulty, pick point indents deeply with firm blow.	3500-7500	250-500	25-50	coal, schist siltstone
R3	**MODERATELY STRONG ROCK** -Knife cannot be used to scrape or peel surface, shallow indentations under firm blow from pick.	7500-15000	500-1000	50-100	sandstone, slate
R4	**STRONG ROCK** -hand held sample breaks with one firm blow from hammer end of geological picks.	15000-30000	1000-2000	100-200	marble, granite gneiss
R5	**VERY STRONG ROCK** -requires many blows from geological pick to break intact sample.	>30000	>2000	>200	quartzite dolerite,gabbro, basalt

Table 4.5 Joint Aperture Classification

Very Tight	<0.1 mm
Tight	0.1-0.5 mm
Moderately Open	0.5-2.5 mm
Open	2.5-10 mm
Very Wide	10-25 mm

Table 4.6 Rock Mass Classification - Detour Lake Mine Main Zone

CASE NO.	DATE RECORDED	STOPE LOCATION	RMR (%)	Q
1	FEB/1990	230M4#11	85	95.2
2	FEB/1990	230M3#11	87	118.8
3	FEB/1990	330M4#10	67	12.9
4	FEB/1990	330M3#11	77	39.1
5A	FEB/1990	430M3#5	78	43.7
5B	FEB/1990	430M3#5	78	43.7
9	MAR/1990	230M4#11	85	95.2
10	MAR/1990	230M3#11	87	118.8
11A	MAR/1990	330M4#10	73	25.1
11B	MAR/1990	330M4#10	63	8.3
12A	MAR/1990	430M3#5	78	43.7
12B	MAR/1990	430M3#5	78	43.7
14	APR/1990	200m5#12	73	25.1
15	APR/1990	230M4#11	85	95.2
17A	APR/1990	330M4#11	73	25.1
17B	APR/1990	330M4#11	63	25.1
18	APR/1990	300M5#12	77	39.1
22	MAY/1990	200M5#12	73	25.1
23	MAY/1990	200M6#12	77	39.1
25A	MAY/1990	300M5#12W	77	39.1
25B	MAY/1990	300M5#12E	67	12.9
27A	MAY/1990	430M6#6	78	43.7
27B	MAY/1990	430M6#6	78	43.7
33	NOV/1989	230M3#10	77	39.1
34	NOV/1989	230M3#10	87	118.8
35	NOV/1989	230M4#10	85	95.2
36	NOV/1989	330M3#10	77	39.1
37	NOV/1989	330M4#10E	63	8.3
38A	NOV/1989	430M3#5	78	43.7
38B	NOV/1989	430M3#5	78	43.7
39	JAN12/90	230M3#11	87	118.8
40	JAN12/90	240M4#10	85	95.2
41	JAN12/90	330M4#10	63	8.3
42	JAN12/90	330M3#11	77	39.1
46A	FEB13/89	460M1#3	72	22.4
46B	FEB13/89	460M1#3	53	2.7
51	DEC12/87	360M2#2	60	5.9
54	OCT3/89	460M2#4	66	11.5
55	SEP17/88	360M2#4	55	3.4
56	OCT/87	360M1#1	69	16.1
57	SEP/87	260M1#1	69	16.1
58	JUNE/90	200M5#13	64	9.2
59	JUNE/90	200M6#12	70	18.0
60	JUNE/90	300M5#12	77	39.1
62	JUNE/90	430M3#5	78	43.7
62B	JUNE/90	430M3#5	78	43.7
63	JULY/90	200M6#13	80	54.6
64	JULY/90	300M5#13	66	11.5
66A	JULY/90	430M3#6	64	9.2
66B	JULY/90	430M3#6	64	9.2
67	AUG/90	200M5#12	80	54.6
68	AUG/90	200M6#13	80	54.6
69	AUG/90	300M5#13	66	11.5
70	AUG/90	300M6#13	79	48.9
72A	AUG/90	430M3#7	79	48.9
72B	AUG/90	430M3#7W	64	9.2
73	AUG/90	430M4#7	77	39.1
75	AUG/90	460LONGHOLE	67	12.9
76	SEP/90	200M5#14	80	54.6
77	SEP/90	300M6#13	79	48.9
79	SEP/90	430M4#7	79	48.9
80	SEP/90	430M3#7	77	39.1
82	SEP/90	460LONGHO	67	12.9
85	OCT/90	200M5#14	80	54.6
86	OCT/90	200M6#14	80	54.6
87	OCT/90	300M5#14	73	25.1
89	OCT/90	430M4#8	77	39.1
90	OCT/90	430M3#7	77	39.1
92	NOV/90	200M6#15	79	48.9
93	NOV/90	300M6#14	78	43.7
95	NOV/90	430M4#8	77	39.1
96	NOV/90	460LONGHO	67	12.9
97	FEB/91	200M6#16	70	18.0
98	FEB/91	200M5#16	82	68.2
99	FEB/91	300M5#15	68	14.4
101A	FEB/91	430M4#8	78	43.7
101B	FEB/91	430M4#8	78	43.7
102	FEB/91	430M3#8	68	14.4
103	MARCH/91	200M6#16	70	18.0
104	MARCH/91	300M5#15	68	14.4
105	MARCH/91	300M6#16	76	35.0
107	MARCH/91	430M4#9	78	43.7
108	MARCH/91	430M3#8	68	14.4
109	APRIL/91	200M6#17	70	18.0
110	APRIL/91	200M5#17	67	12.9
111	APRIL/91	300M5#16	68	14.4
112	APRIL/91	300M6#16	76	35.0
115	APRIL/91	430M4#9	78	43.7
116	JULY/91	200M7#18	79	48.9
117	JULY/91	200M8#19	77	39.1
118	JULY/91	300M5#19	69	16.1
119	JULY/91	300M6#16	64	9.2
121	JULY/91	430M4#10	69	16.1
122	SEP/91	200M7#19	77	39.1
123	SEP/91	200M8#19	83	76.2
124	SEP/91	300M5#17	63	8.3
125	SEP/91	300M5#17	63	8.3
130	SEP/91	430M4#10	69	16.1
131	OCT/91	200M7#19	78	43.7
132	OCT/91	200M8#20	79	48.9
133	OCT/91	300M5#18	65	10.3
134	OCT/91	300M5#18	65	10.3
135	OCT/91	300M6#17	75	31.3
138	OCT/91	430M3#10	80	54.6
139	OCT/91	430M4#11	81	61.0
140	NOV/91	200M7/M8#20	78	43.7
141	NOV/91	300M5M6 #18	63	8.3
142	NOV/91	300M5M6 #18	63	8.3
143	NOV/91	400M5#13	75	31.3
144	NOV/91	430M4#11	79	48.9
145	DEC/91	200M7/M8#20	78	43.7
146	DEC/91	300M5M6 #18	63	8.3
147	DEC/91	300M5M6 #18	63	8.3
148	DEC/91	400M5#13	75	31.3
149	DEC/91	560M2#2	54	3.0
150	DEC/91	660M2#2	75	31.3
151	JAN/92	300M5M6 #18	63	8.3
152	MARCH/92	300M5M6 #18	63	8.3
153	MARCH/92	560M1#2	81	61.0
154	MARCH/92	560M2#2	54	3.0
155	MARCH/92	560M2#2	54	3.0
156	MARCH/92	575SLR	70	18.0
157	MARCH/92	590SLR	72	22.4

Mean: 73.0
Standard Deviation: 7.8
Note : Q is calculated using RMR=9lnQ+44

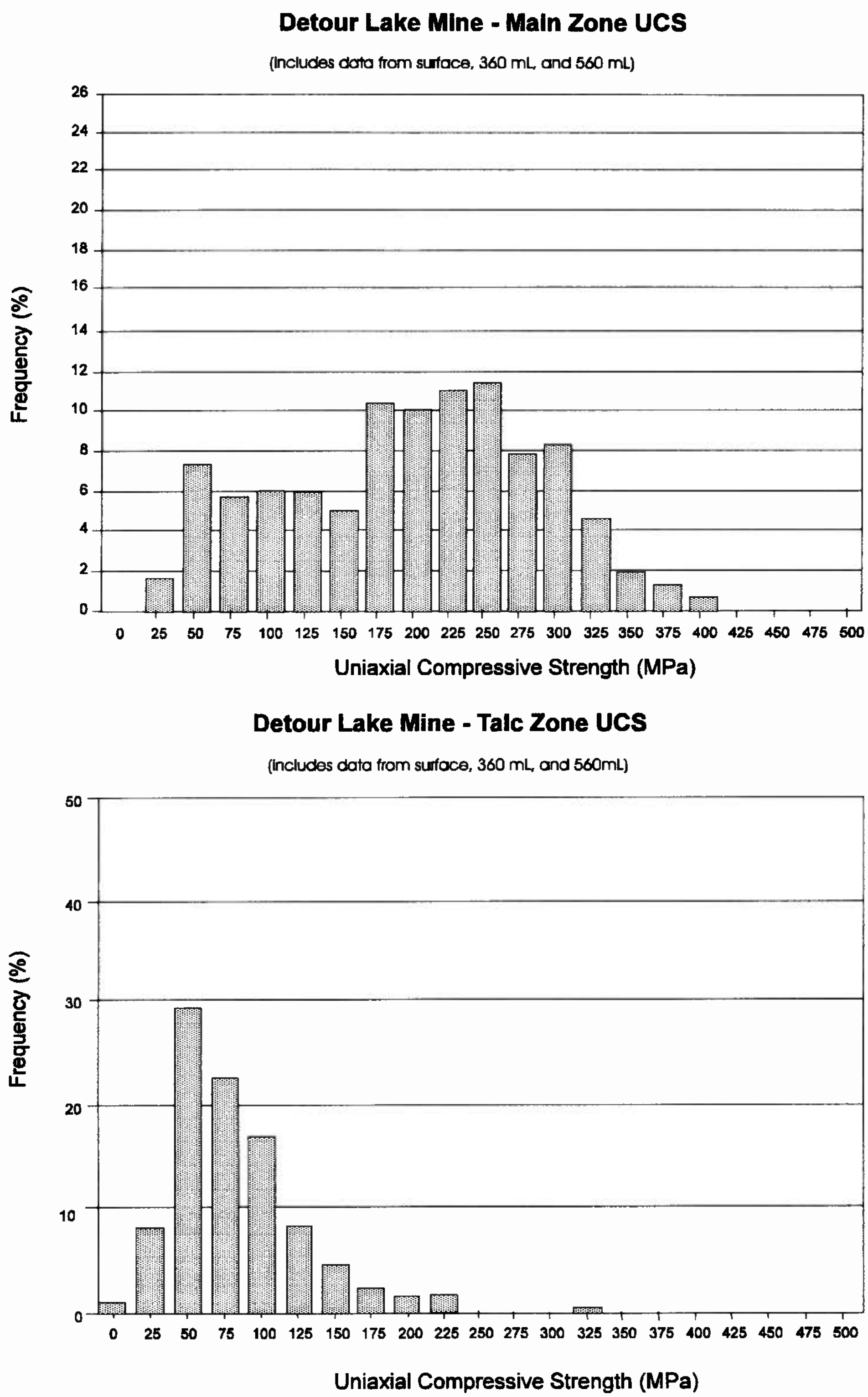
Table 4.7 Rock Mass Classification - Detour Lake Mine Talc Zone

CASE NO.	DATE RECORDED	STOPE LOCATION	RMR (%)	Q
6	FEB/1990	430T40#5	51	2.2
7	FEB/1990	430T5#5	67	12.9
8	FEB/1990	430T60#5	50	1.9
13	MAR/1990	430T60#5	50	1.9
16	APR/1990	260T70ACCESS	42	0.8
19	APR/1990	430T5#6	62	7.4
20	APR/1990	430T40#6	49	1.7
21	APR/1990	430T60#6	52	2.4
24	MAY/1990	260T70ACCESS	42	0.8
26	MAY/1990	360T20#4	58	4.7
28	MAY/1990	430T40#6	49	1.7
29	MAY/1990	430T20#6	61	6.6
30	MAY/1990	430T60#6	42	0.8
31A	FEB/1990	360T40#3/4	48	1.6
31B	FEB/1990	360T40#3/4	48	1.6
32	FEB/1990	360T20#3	58	4.7
43	JAN12/90	430T5#5	67	12.9
44	JAN12/90	430T40#5	55	3.4
45	JAN12/90	430T60#5	50	1.9
47	FEB13/89	230T40#5	48	1.6
48	NOV1/88	230T40#4	48	1.6
49	FEB13/90	460T60#3	25	0.1
50	JAN19/89	360T40#3	48	1.6
52	FEB28/89	360T60#2	28	0.2
53	FEB28/89	360T60#1	28	0.2
61	JUNE/90	360T20#40	58	4.7
65	JULY/90	360T40#10	48	1.6
71	AUG/90	360T40#10	40	0.6
74	AUG/90	430T5#7	67	12.9
78	SEP/90	360T40#10	40	0.6
81	SEP/90	430T60#7	60	5.9
83	JULY/90	560T15#1	43	0.9
84	JULY/90	560TACCESS	25	0.1
88	OCT/90	360T40#11]	55	3.4
94	NOV/90	360T60#11	38	0.5
100	FEB/91	360T60#13	38	0.5
106	MARCH/91	360T60#13	38	0.5
113	APRIL/91	360T60#14	38	0.5
114	APRIL/91	360T60#14	38	0.5
114	APRIL/91	360T20#10	65	10.3
120A	JULY/91	360T40#17	45	1.1
120B	JULY/91	360T40#17	45	1.1
126	SEP/91	360T40#18	43	0.9
127	SEP/91	360T40#18	43	0.9
128	SEP/91	360T20#14	56	3.8
129	SEP/91	360T20#14	56	3.8
136	OCT/91	360T40#18	45	1.1
137	OCT/91	360T40#18	45	1.1

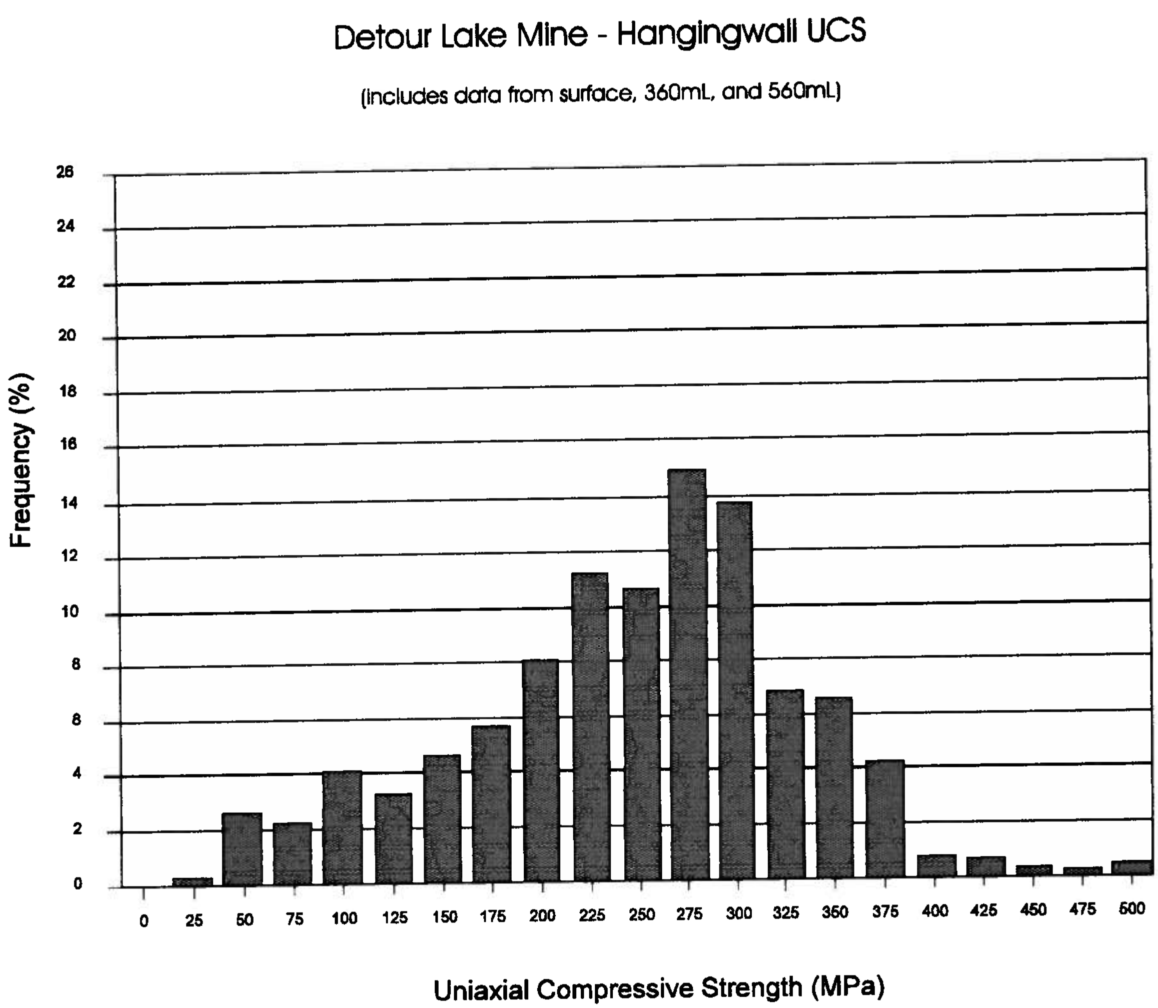
Mean: 48.9
Standard Deviation: 11.2
Note: Q is calculated using RMR=9lnQ+44

Table 4.8 Typical CSIR Rock Mass Rating at Detour Lake Mine

Category	MAIN ZONE		TALC ZONE	
	Description	Rating	Description	Rating
Strength	160-180 MPa	13	35-50 MPa	4
RQD	90%	17	80%	16
Joint Spacing	0.4 m	16	0.3 m	9
Joint Condition	smooth, hard, tight	17	smooth surfaces, soft	10
Groundwater	none	10	none	10
Joint Orientation		0		0
TOTAL		73		49

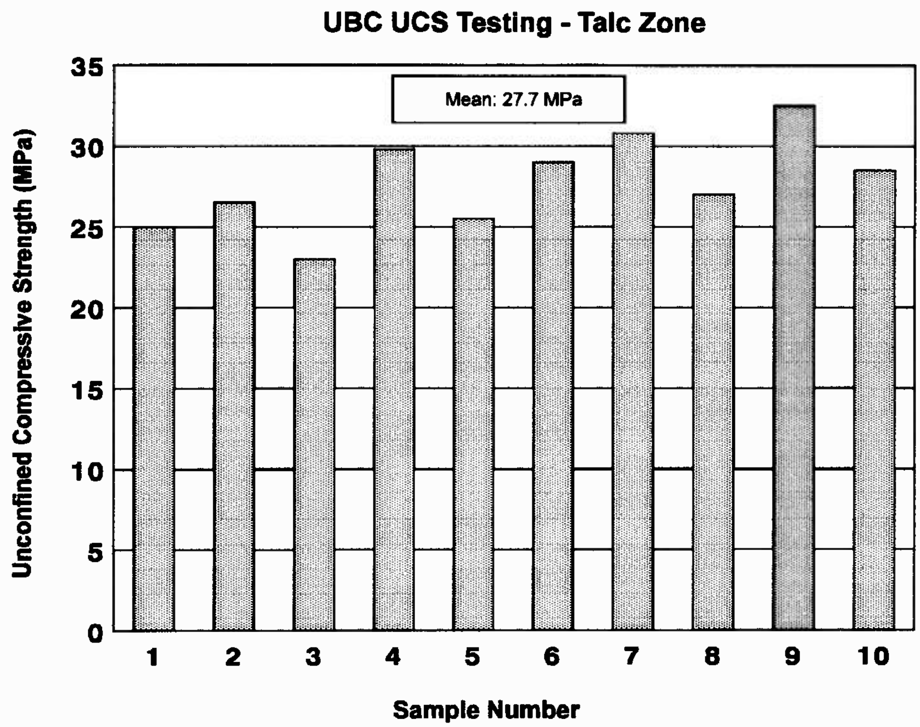


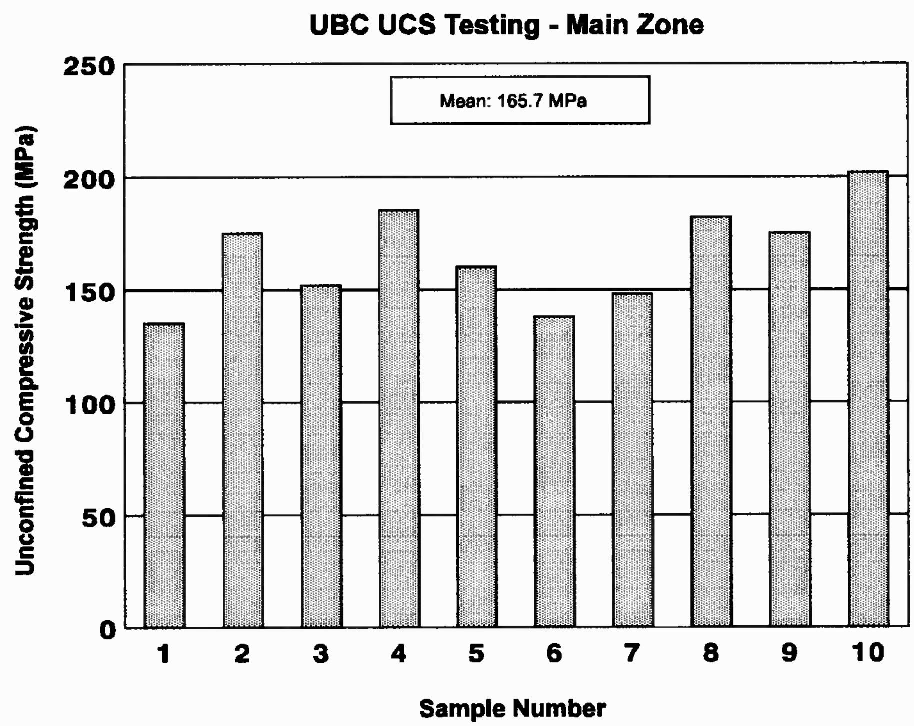
DETOUR LAKE MINE UCS TESTING MAIN ZONE AND TALC ZONE	FIGURE 4.1



Average UCS = 235.4 MPa

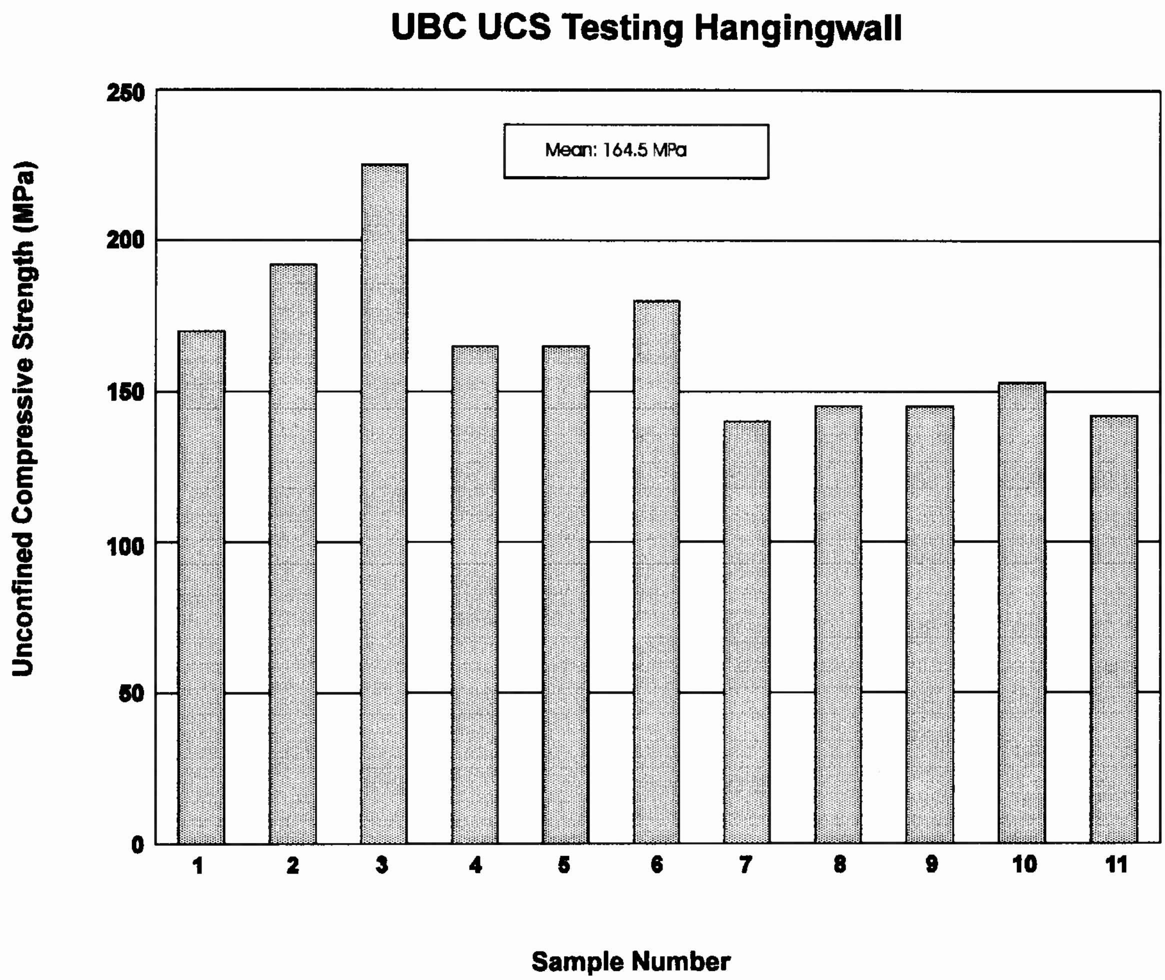
DETOUR LAKE MINE - UCS TESTING HANGINGWALL MAFIC ROCK	FIGURE 4.2





DETOUR LAKE MINE - UCS TESTING BY UBC OF TALC ZONE AND MAIN ZONE

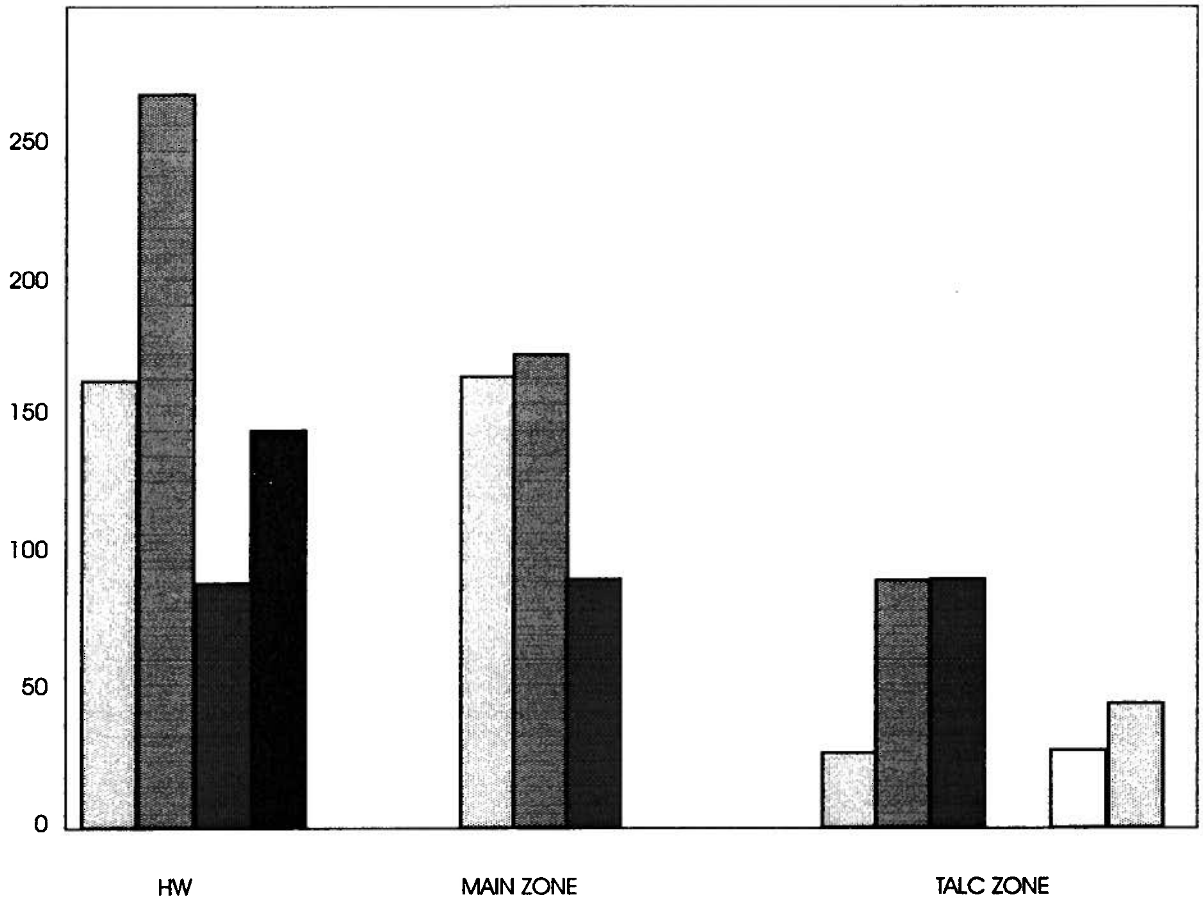
FIGURE 4.3



DETOUR LAKE MINE - UCS TESTING BY UBC OF HANGINGWALL MAFIC ROCK

FIGURE 4.4

UCS Testing Summary



Zone	Mean UCS	Standard Deviation	No. of Samples
HW	165	27	11
Main	166	22	10
Talc (lower)	28	3	10
Talc (upper)	48	5	9

DETOUR LAKE MINE
UCS TESTING SUMMARY

FIGURE 4.5

Date: ________ Mapped by: ________ Page ____

Project: Bench, Drift, Ramp: Traverse: Ref. Point Coord. Survey Direction Length:

Distance	Wall	Rock Type	Feature	No. of Features	Spacing	Rock Hardness	Strike	Dip	Dip Direct.	Openness	Planarity	Roughness	Infilling	Ends Visible	Length	Water	Comments

NGI Classification

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

RQD	Joints/m	______	J_a	Joint Alteration	______
J_n	No. of joint sets	______	J_w	Presence of Water	______
J_r	Joint Roughness	______	SRF	Stress Condition	______

CSIR Classification

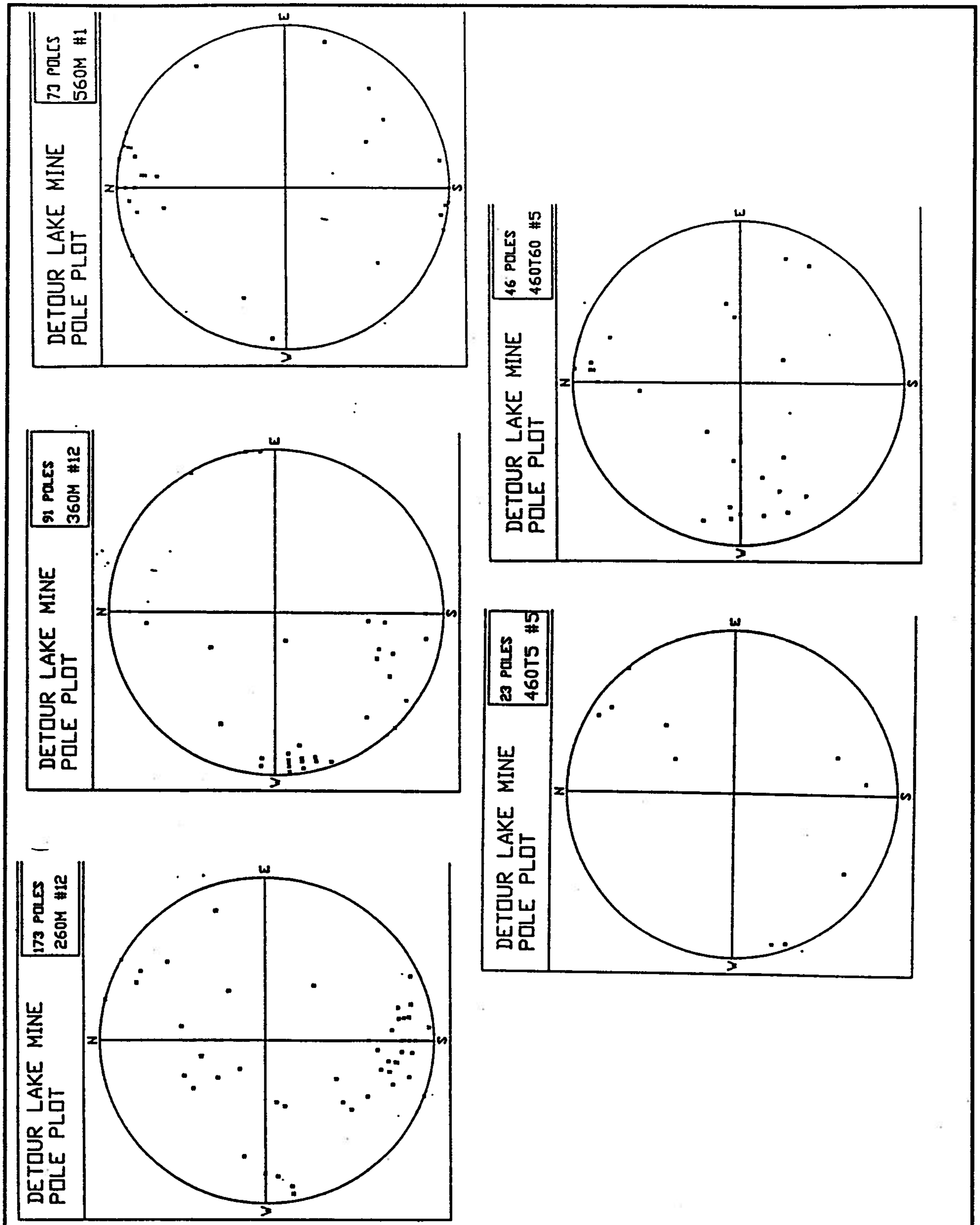
RMR = A + B - C + D + E - F = ______

	Description	Rating
A. Strength		
B. RQD Joints/m		
C. Joint Spacing		

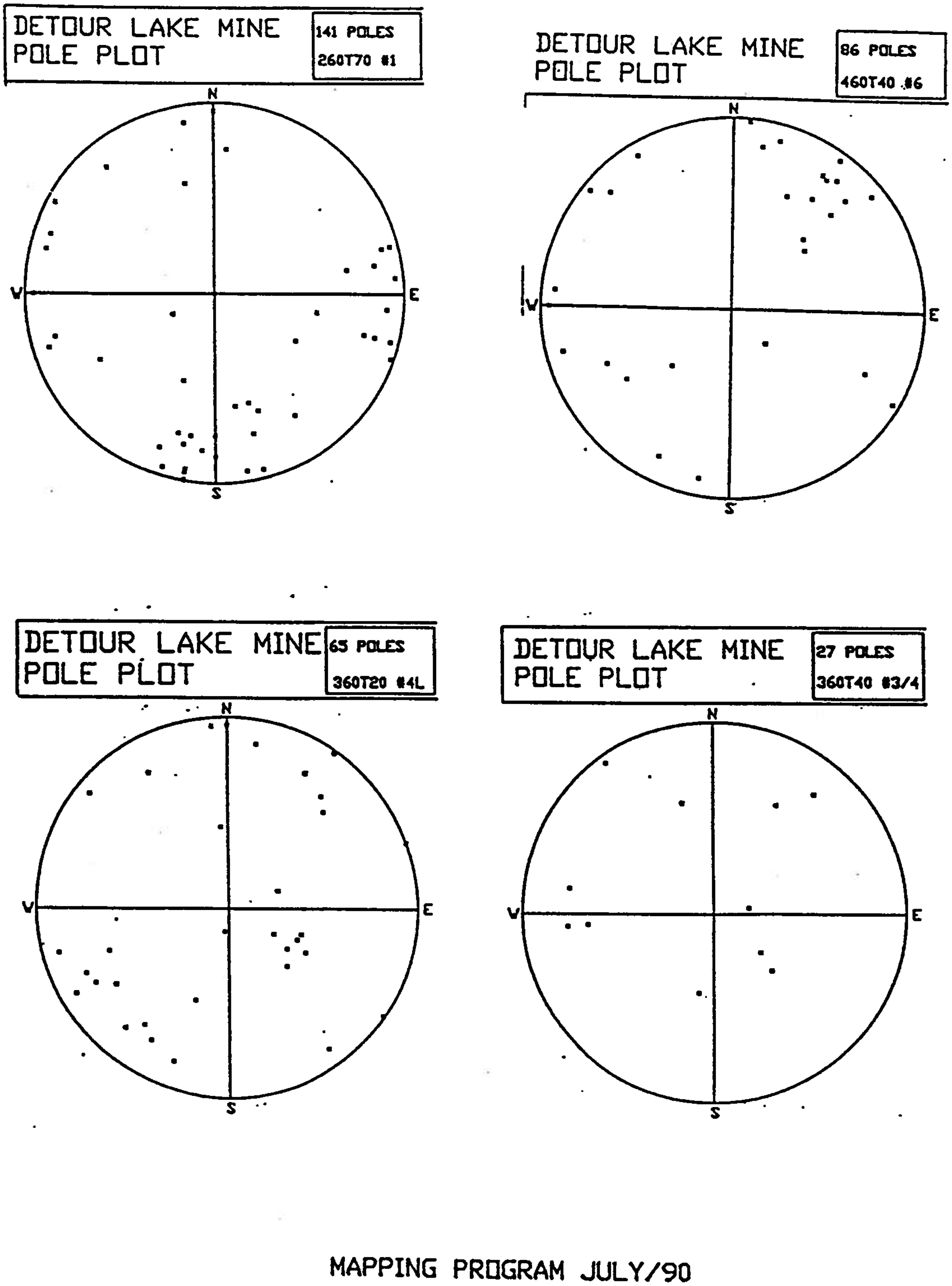
	Description	Rating
D. Joint Condition		
E. Groundwater		
F. Joint Orientation		

GEOTECHNICAL MAPPING FORM

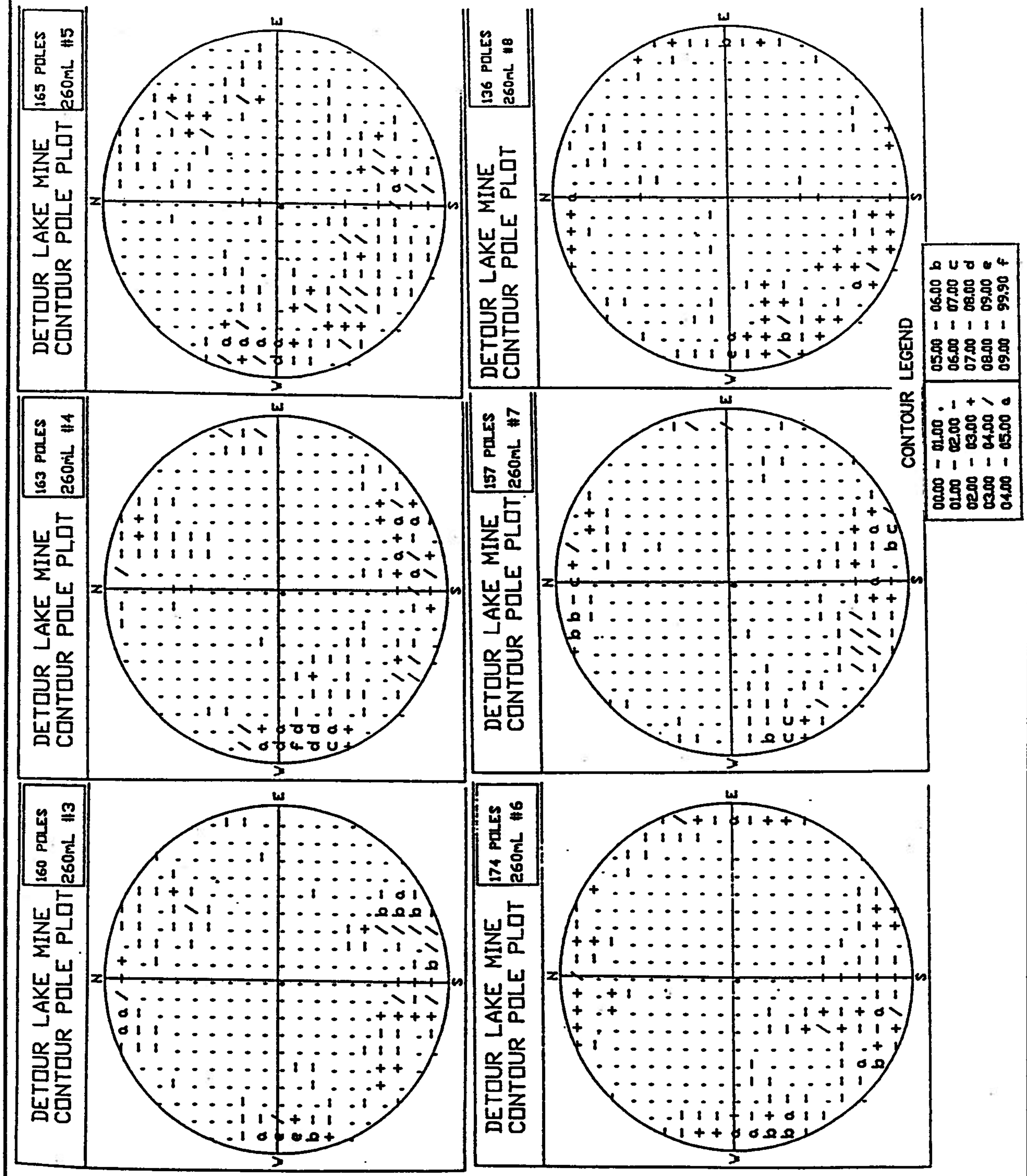
FIGURE 4.6



POLE PLOTS FROM FABRIC MAPPING	FIGURE 4.7

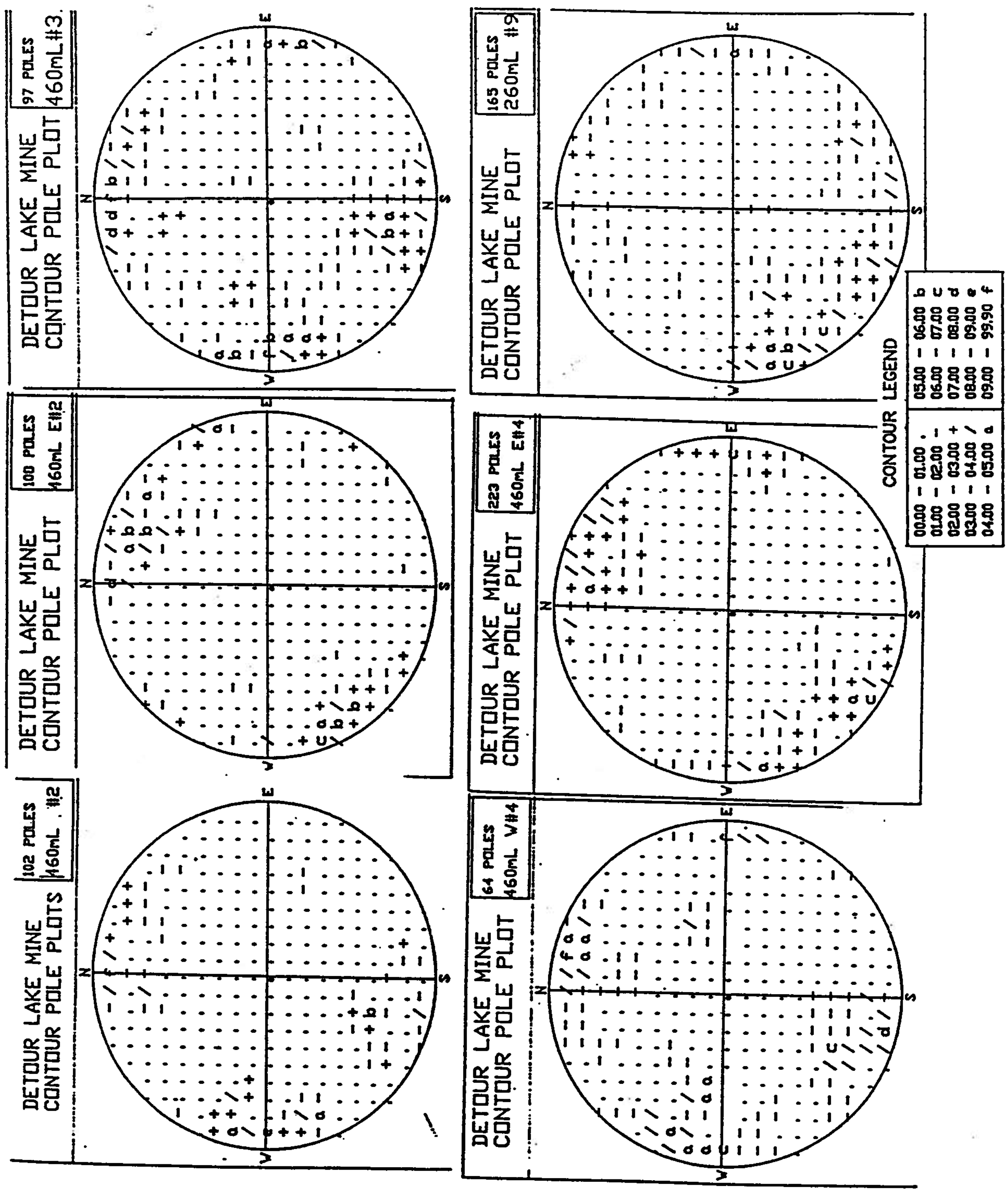


POLE PLOTS FROM FABRIC MAPPING	FIGURE 4.8



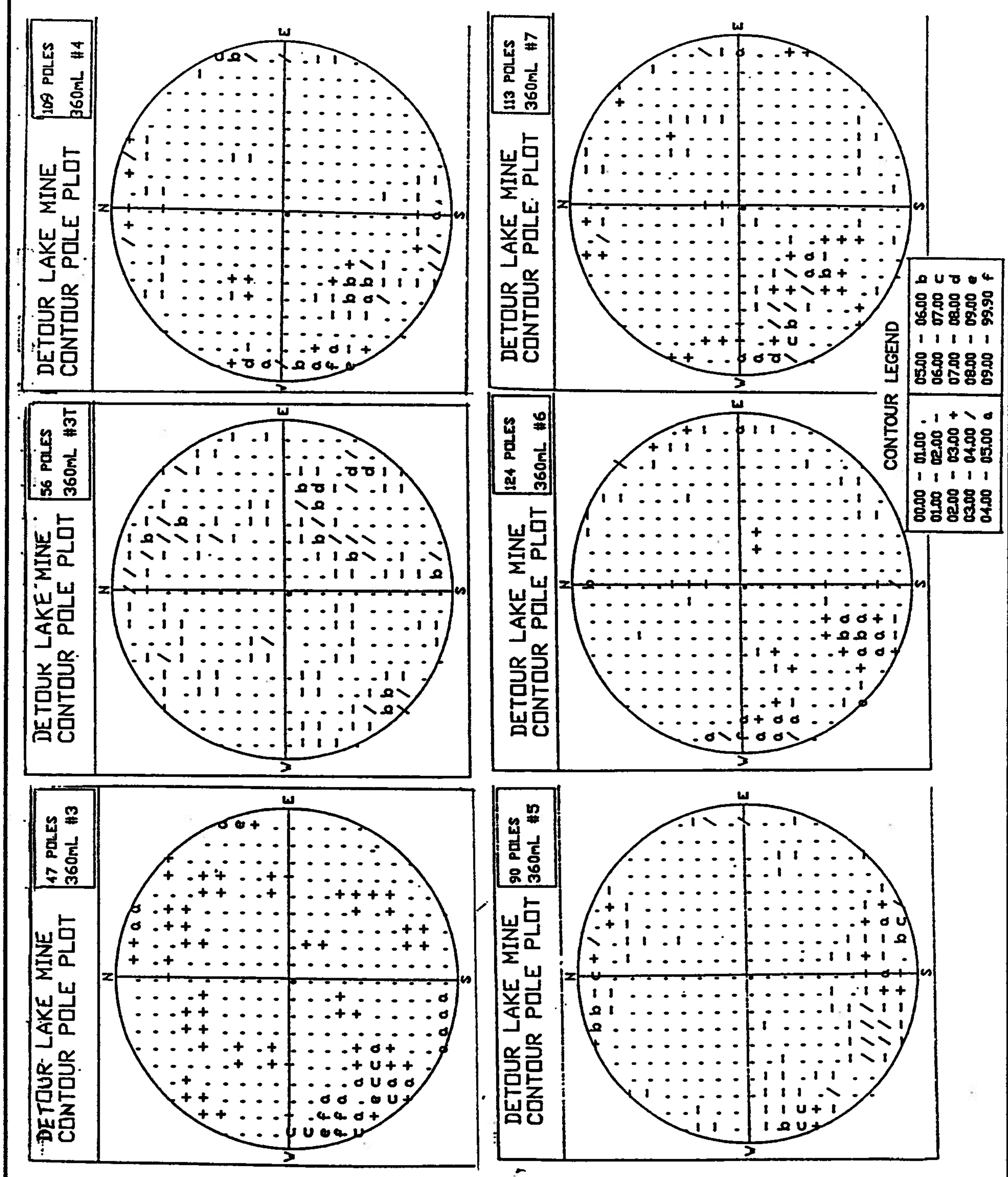
CONTOURS OF POLE PLOTS - DLM MAPPING

FIGURE 4.9

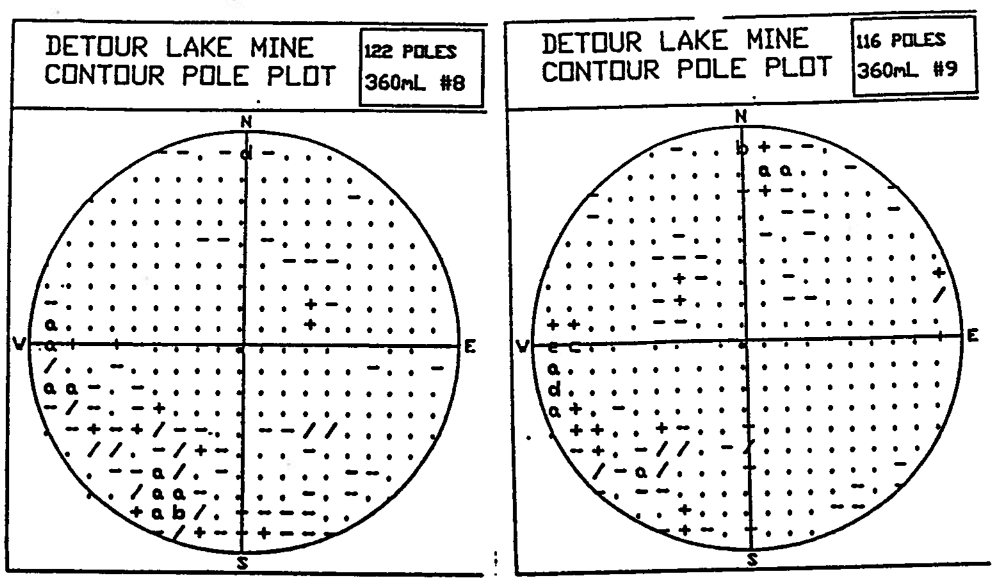


CONTOURS OF POLE PLOTS - DLM MAPPING

FIGURE 4.10



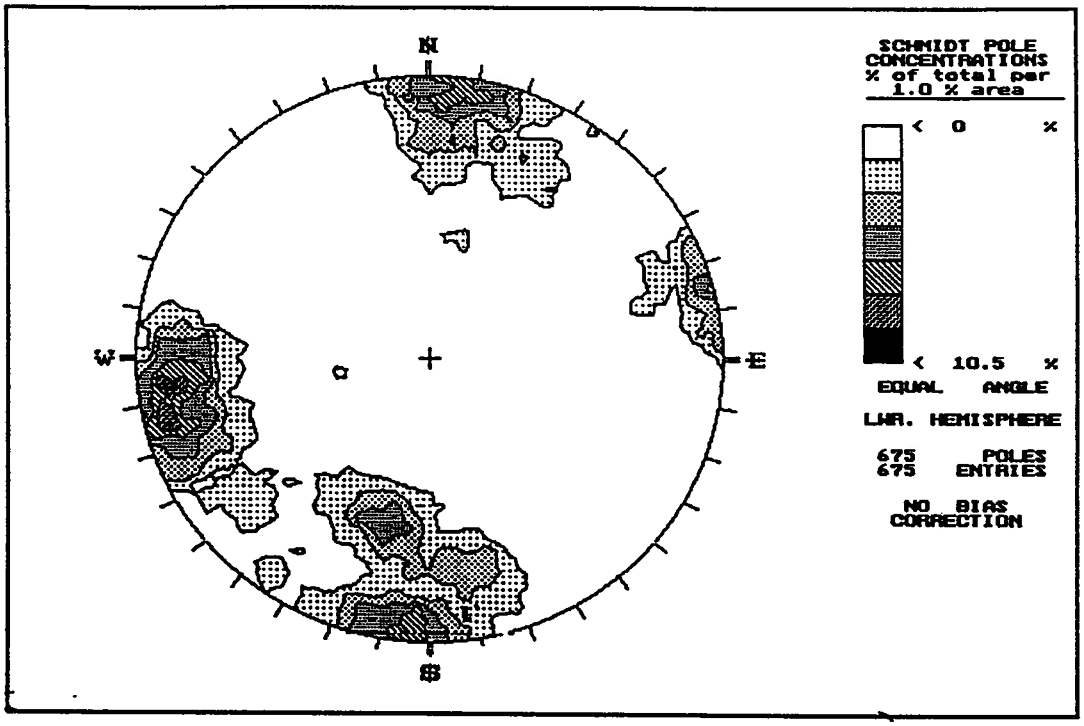
CONTOURS OF POLE PLOTS - DLM MAPPING	FIGURE 4.11



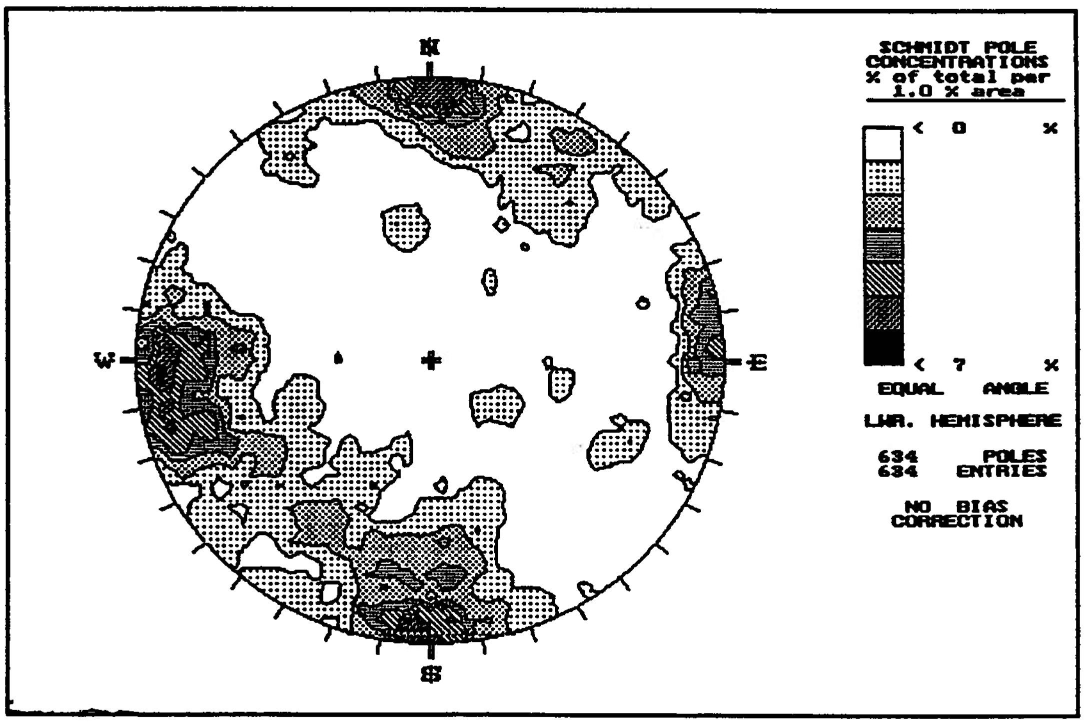
CONTOUR LEGEND	
00.00 – 01.00 .	05.00 – 06.00 b
01.00 – 02.00 –	06.00 – 07.00 c
02.00 – 03.00 +	07.00 – 08.00 d
03.00 – 04.00 /	08.00 – 09.00 e
04.00 – 05.00 a	09.00 – 99.90 f

CONTOURS OF POLE PLOTS - DLM MAPPING

FIGURE 4.12

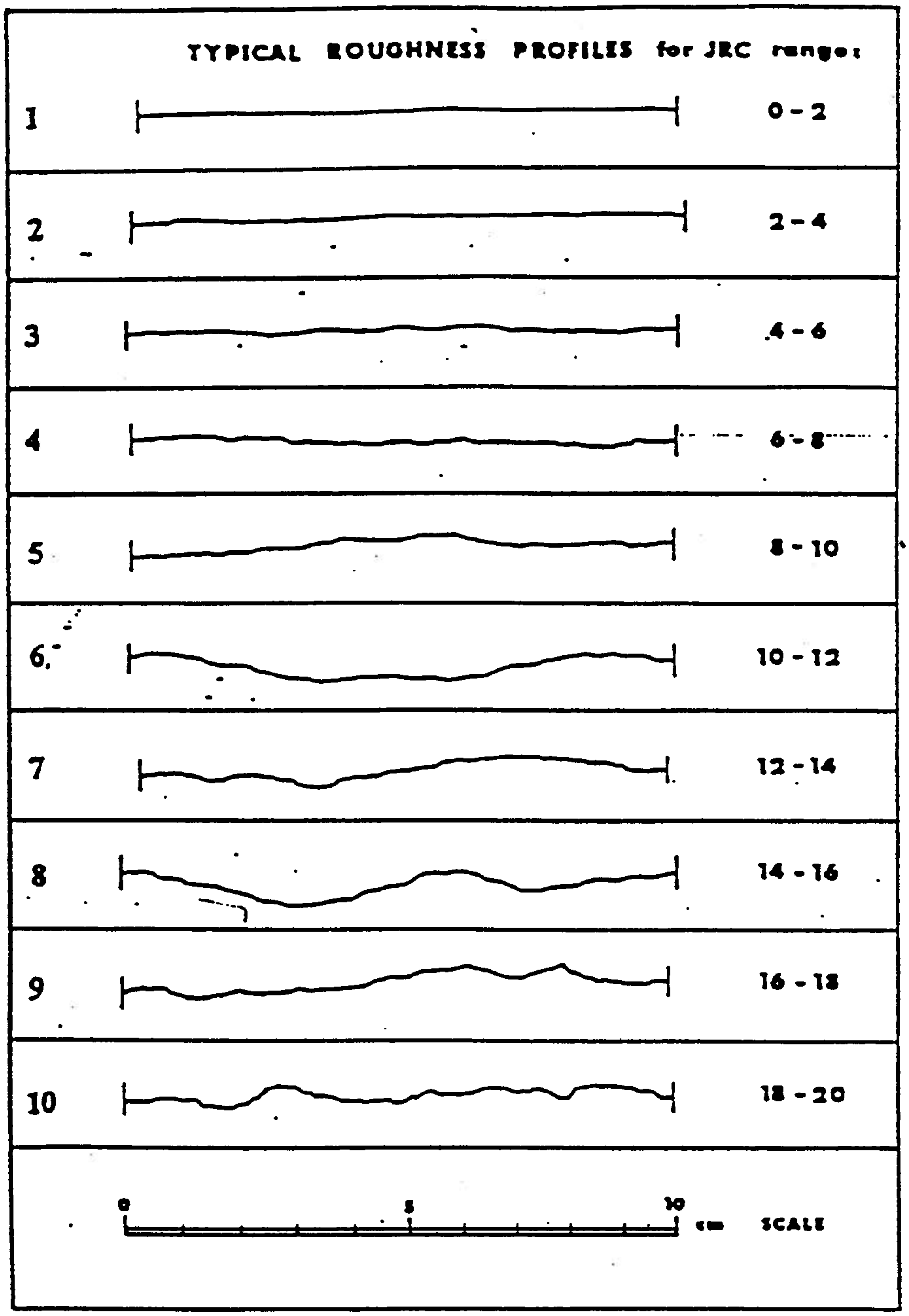


MAIN ZONE



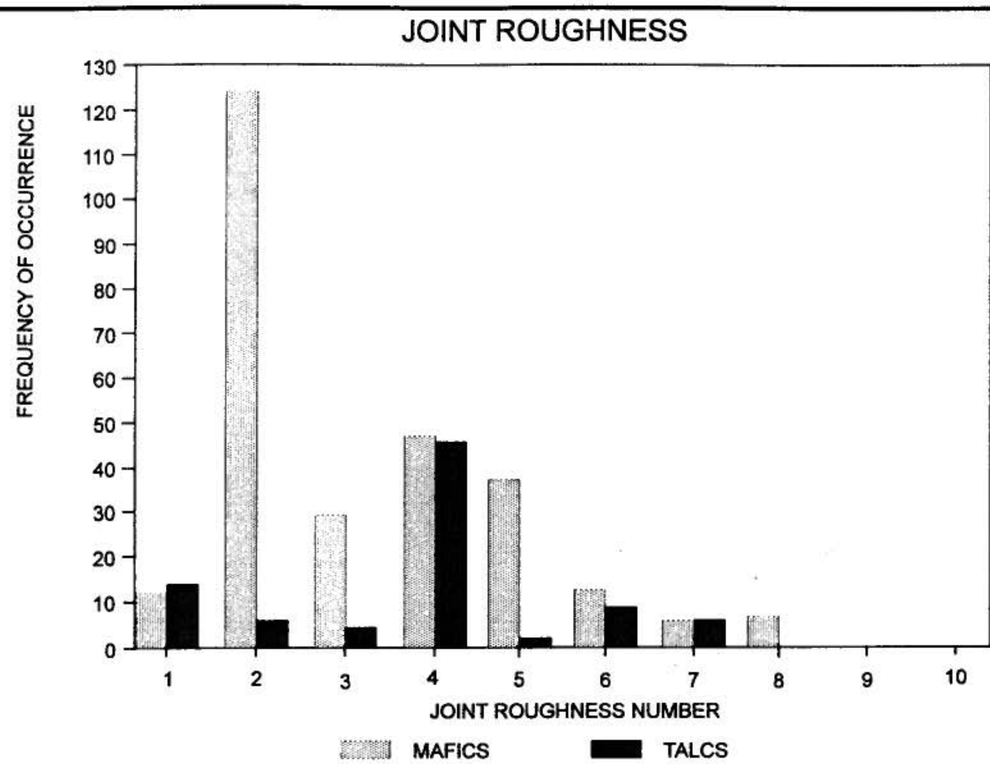
TALC ZONE

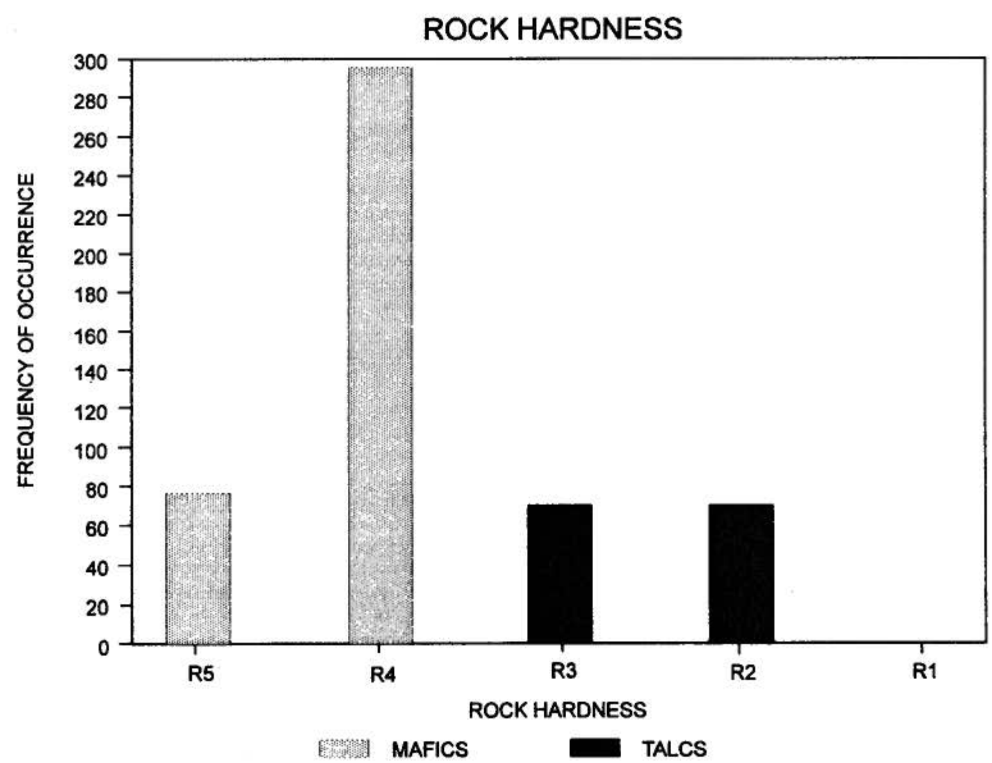
CONTOURS OF POLE PLOTS - ALL STRUCTURES IN MAIN ZONE AND TALC ZONE	FIGURE 4.13



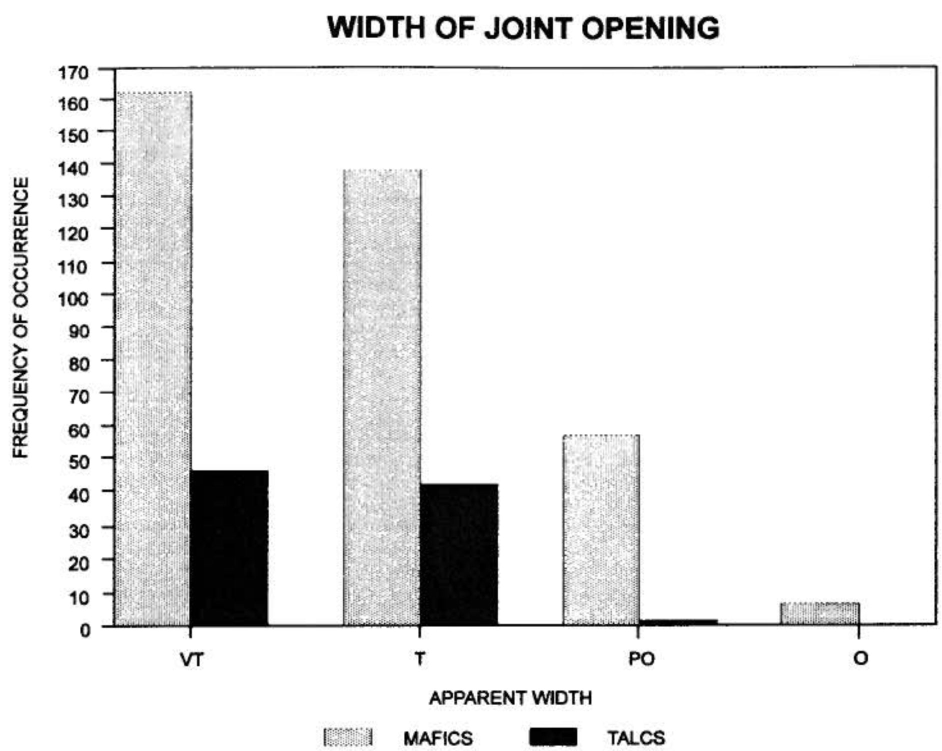
TYPICAL ROUGHNESS PROFILES FOR JRC RANGES
(after Barton et al., 1977)

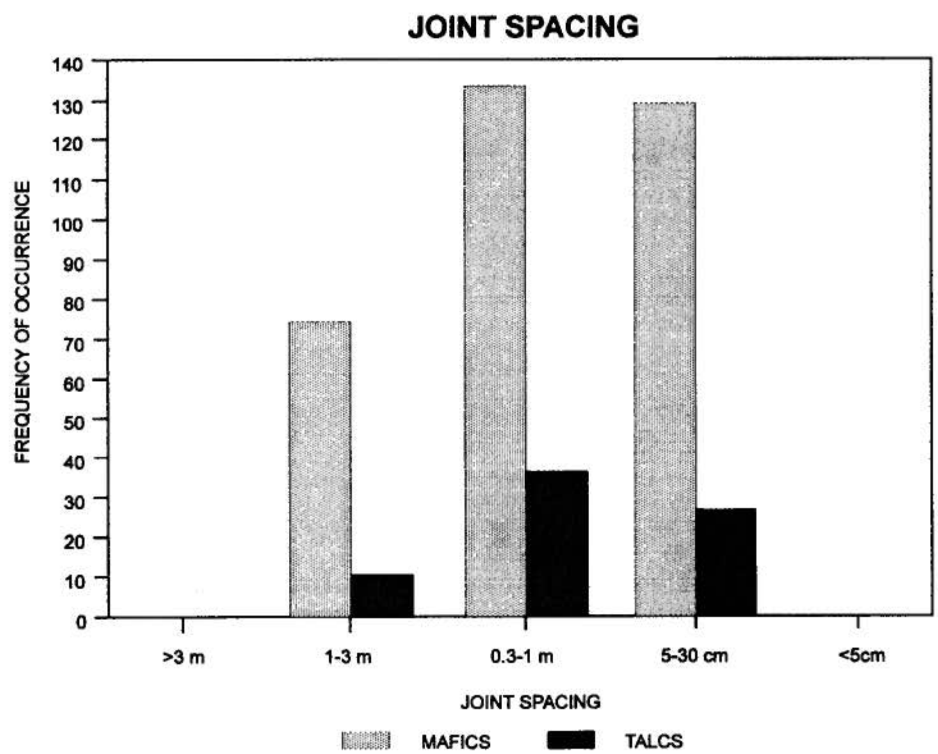
FIGURE 4.14





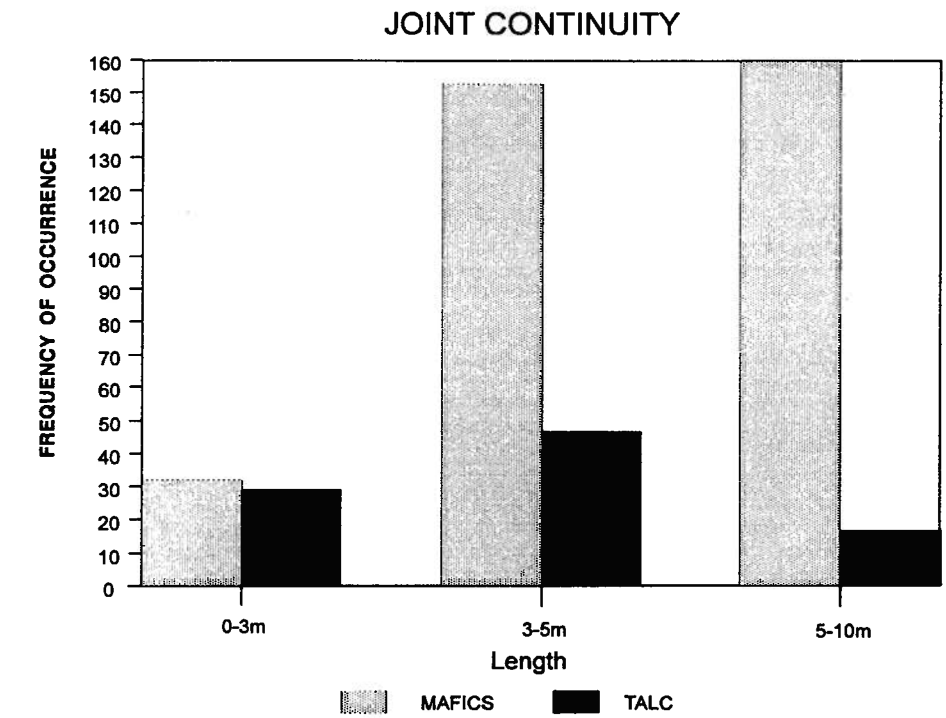
DETOUR LAKE MINE FABRIC MAPPING JOINT ROUGHNESS AND ROCK HARDNESS FREQUENCY	FIGURE 4.15

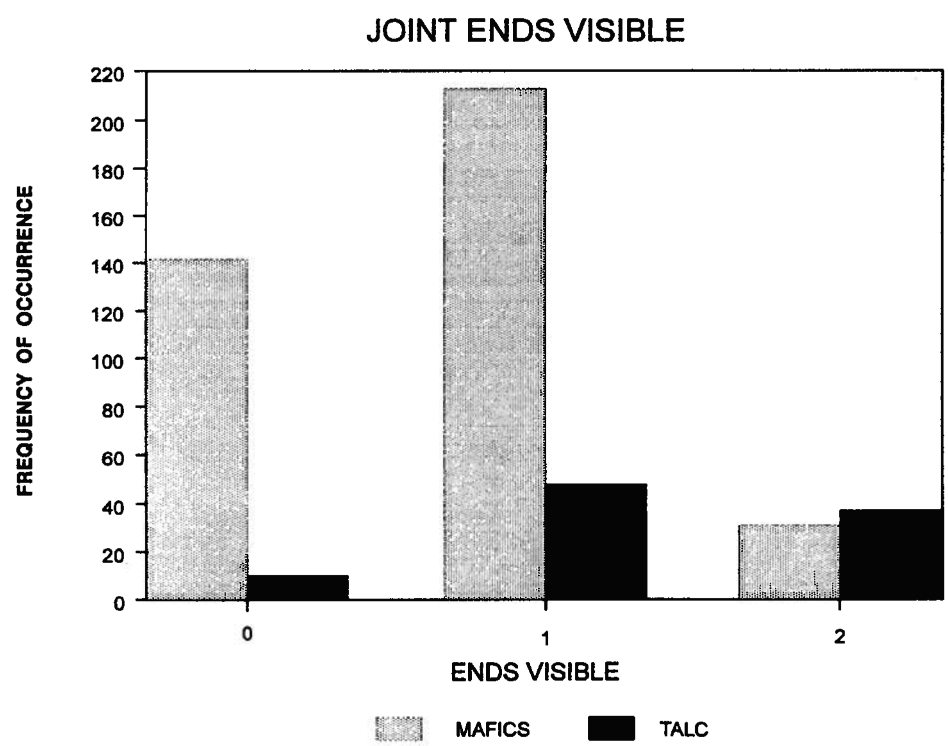




DETOUR LAKE MINE FABRIC MAPPING
JOINT APERTURE AND SPACING

FIGURE 4.16





DETOUR LAKE MINE FABRIC MAPPING JOINT CONTINUITY AND ENDS VISIBLE	FIGURE 4.17

ROCK MASS CLASSIFICATION

DATE: ____________ LOCATION: ____________ NAME: ____________

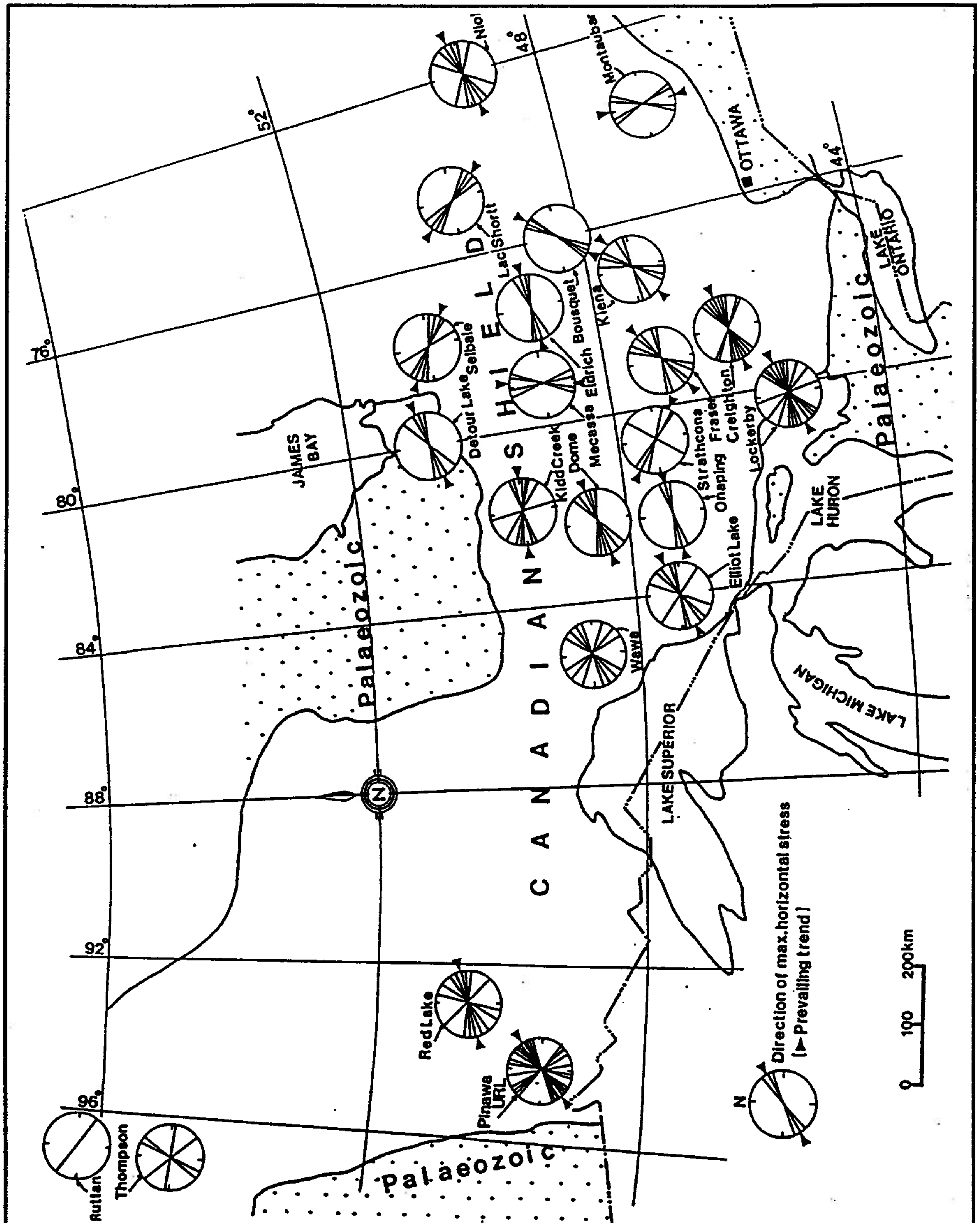
	PARAMETER		RANGE OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4-8 MPa	2-4 MPa	1-2 MPa	For this low range uniaxial compressive test is preferred		
		Uniaxial Compressive Strength	>200 MPa	100-200 MPa	50-100 MPa	25-50 MPa	10-25 MPa	3-10 MPa	1-3 MPa
	Rating		15	12	7	4	2	2	0
2	Drill Core Quality (RQD)		90%–100%	75%-90%	50%-75%	25%-50%	<25%		
	Rating		20	17	13	8	3		
3	Spacing of Joints		>3m	1-3m	0.3-1m	50-300 mm	<50 mm		
	Rating		30	25	20	10	5		
4	Condition of Joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation< 1mm Hard joint wall rock	Slightly rough surfaces Separation<1mm Soft Joint Wall Rock	Gouge <5 mm thick or Slickensided Surfaces or Joints open 1-5 mm Continuous joints	Soft gouge>5 mm thick or Joints open >5 mm Continuous Joints		
	Rating		25	20	12	6	0		
5	Ground water	Inflow per 10 m tunnel length	None		<25 litres/min	25-125 litres/min	>125 litres/min		
		Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	0		0.0-0.02	0.2-0.5	>0.5		
		General conditions	Completely dry		Moist only (interstitial water	Water under moderate pressure	Severe water problems		
	Rating		10		7	4	0		

	DESCRIPTION	VALUE
Strength		
RQD (joints/m³)		
Joint Spacing		
Condition of Joints		
Groundwater		
Jont Orientation		
	TOTAL	

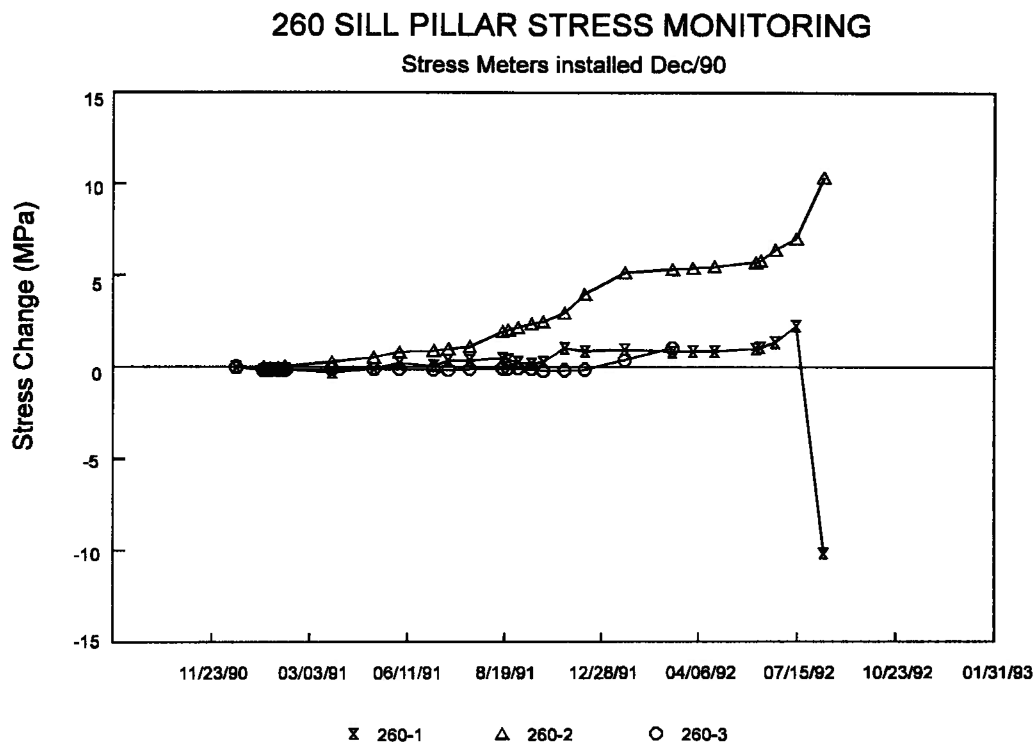
JOINT SETS

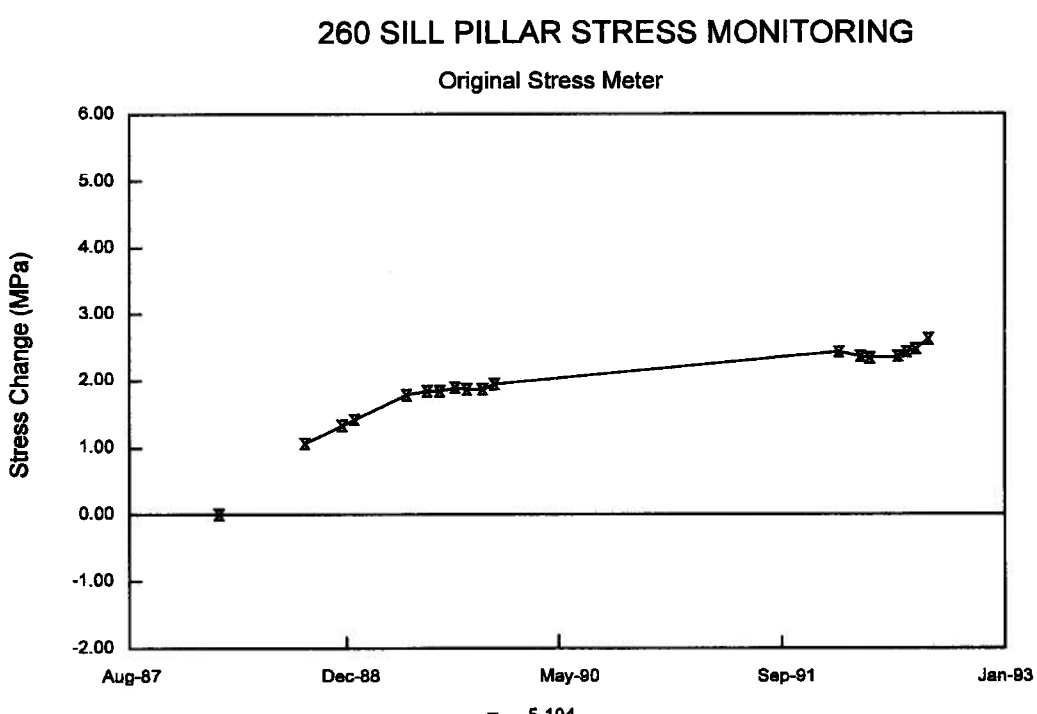
	Strike	Dip	Spacing
Joint Set 1			
Joint Set 2			
Joint Set 3			

ROCK MASS CLASSIFICATION FORM — FIGURE 4.18

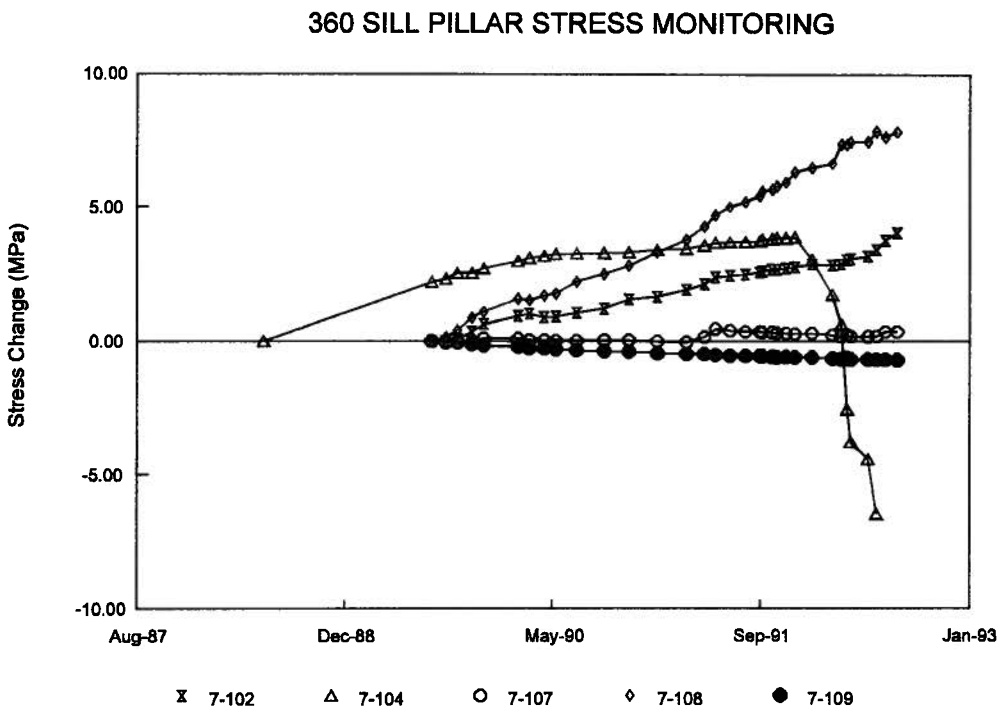


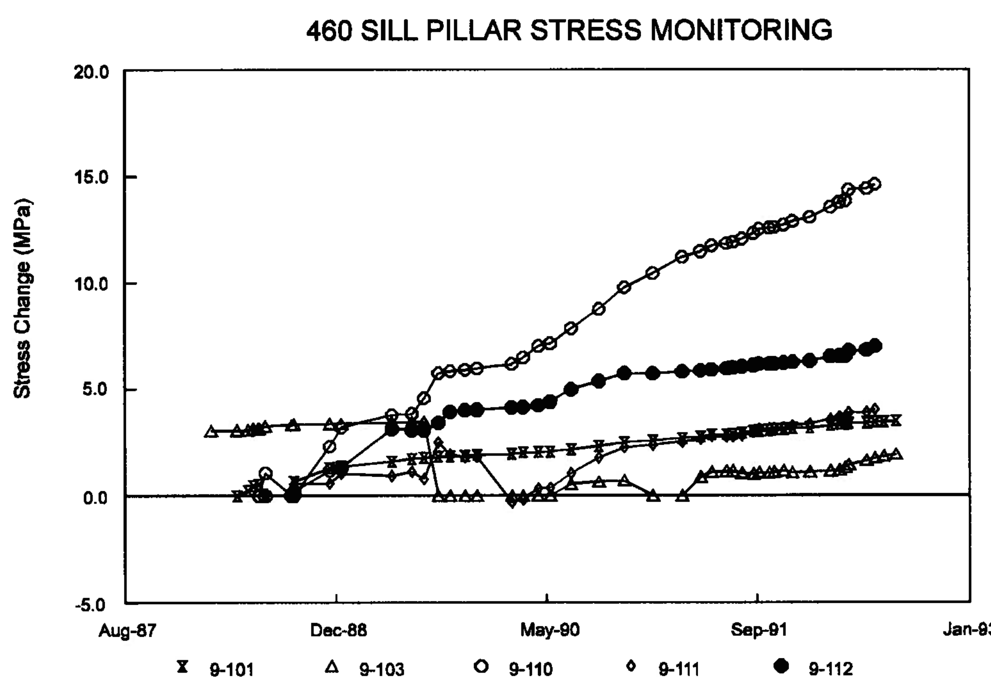
IN-SITU STRESSES IN THE CANADIAN SHIELD (after Herget, 1990)	FIGURE 4.19



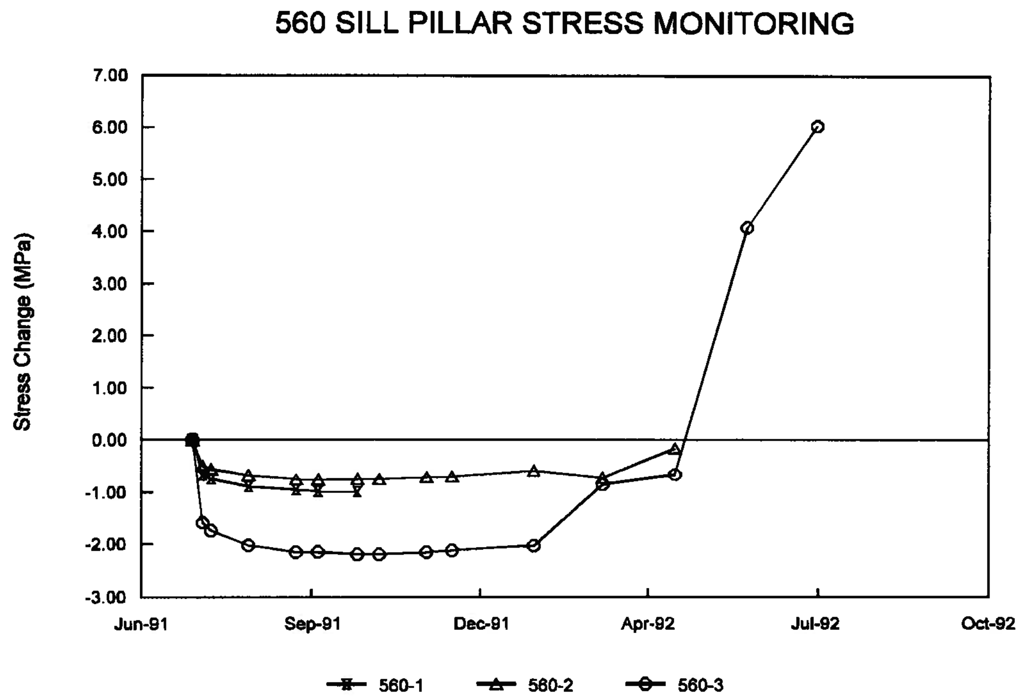


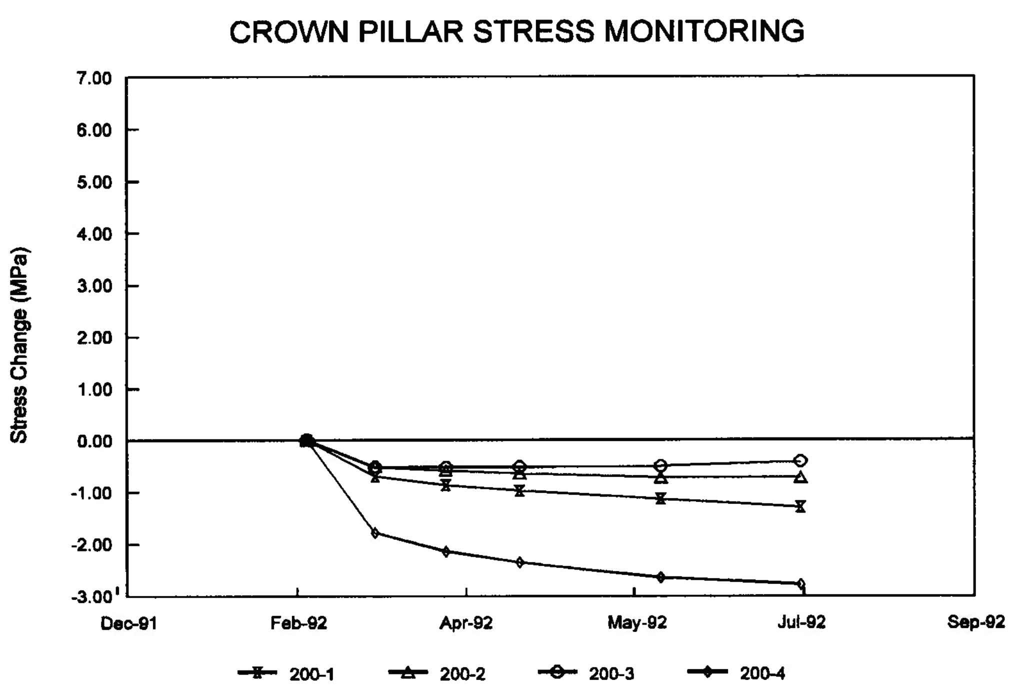
260 SILL PILLAR STRESS MONITORING	FIGURE 4.20



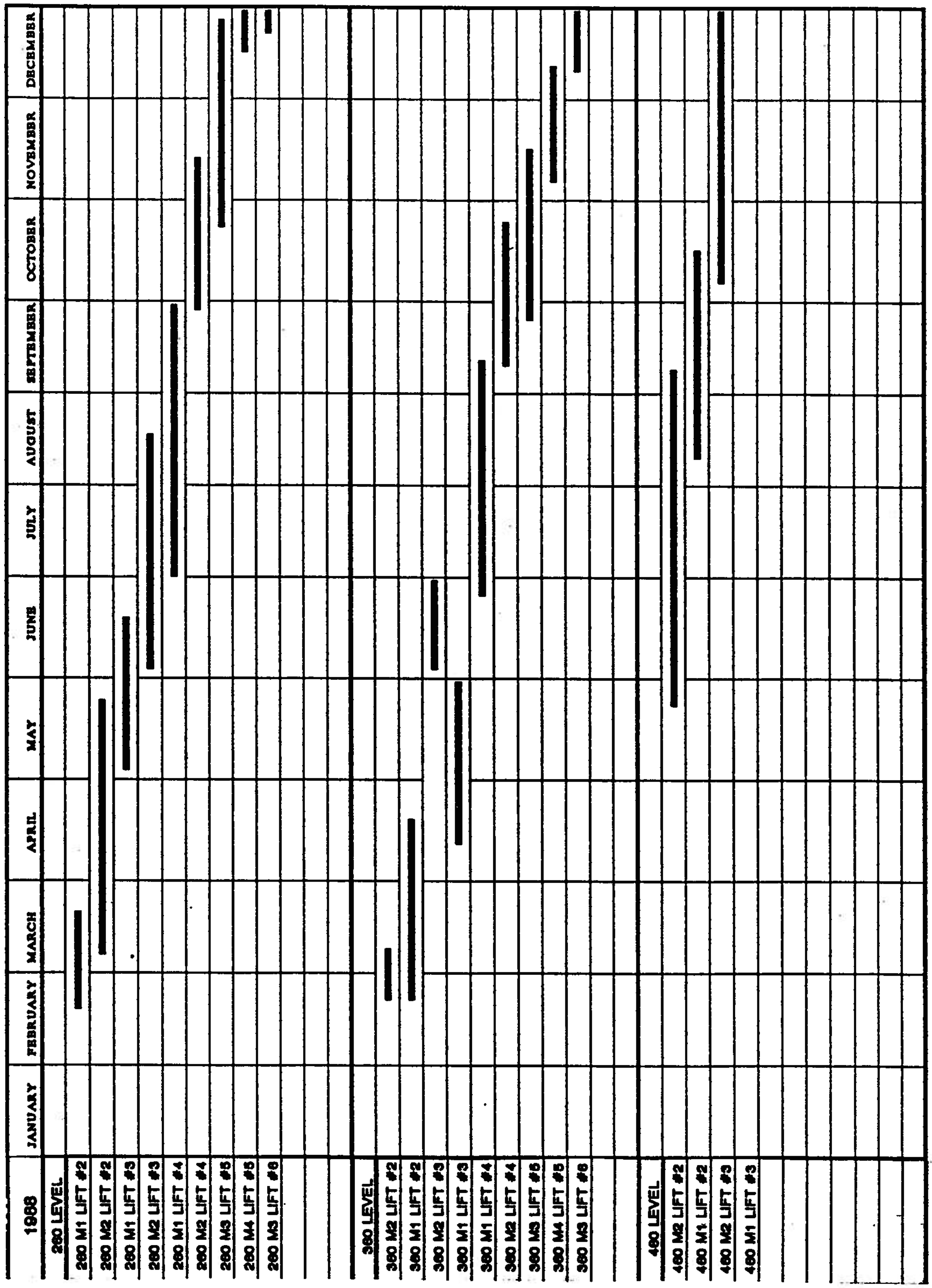


360 AND 460 SILL PILLAR STRESS MONITORING	FIGURE 4.21



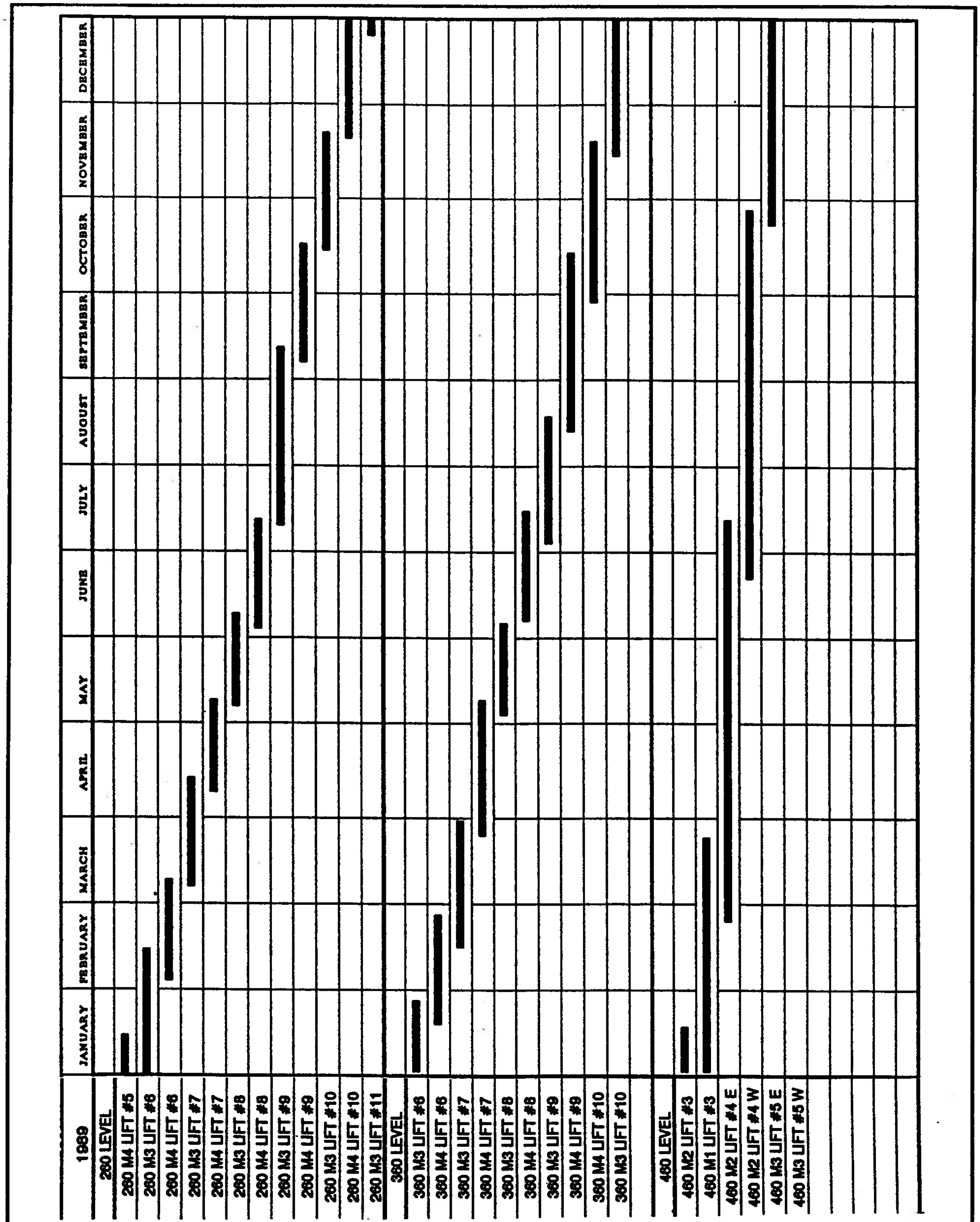


SURFACE CROWN PILLAR AND 560 SILL PILLAR STRESS MONITORING	FIGURE 4.22



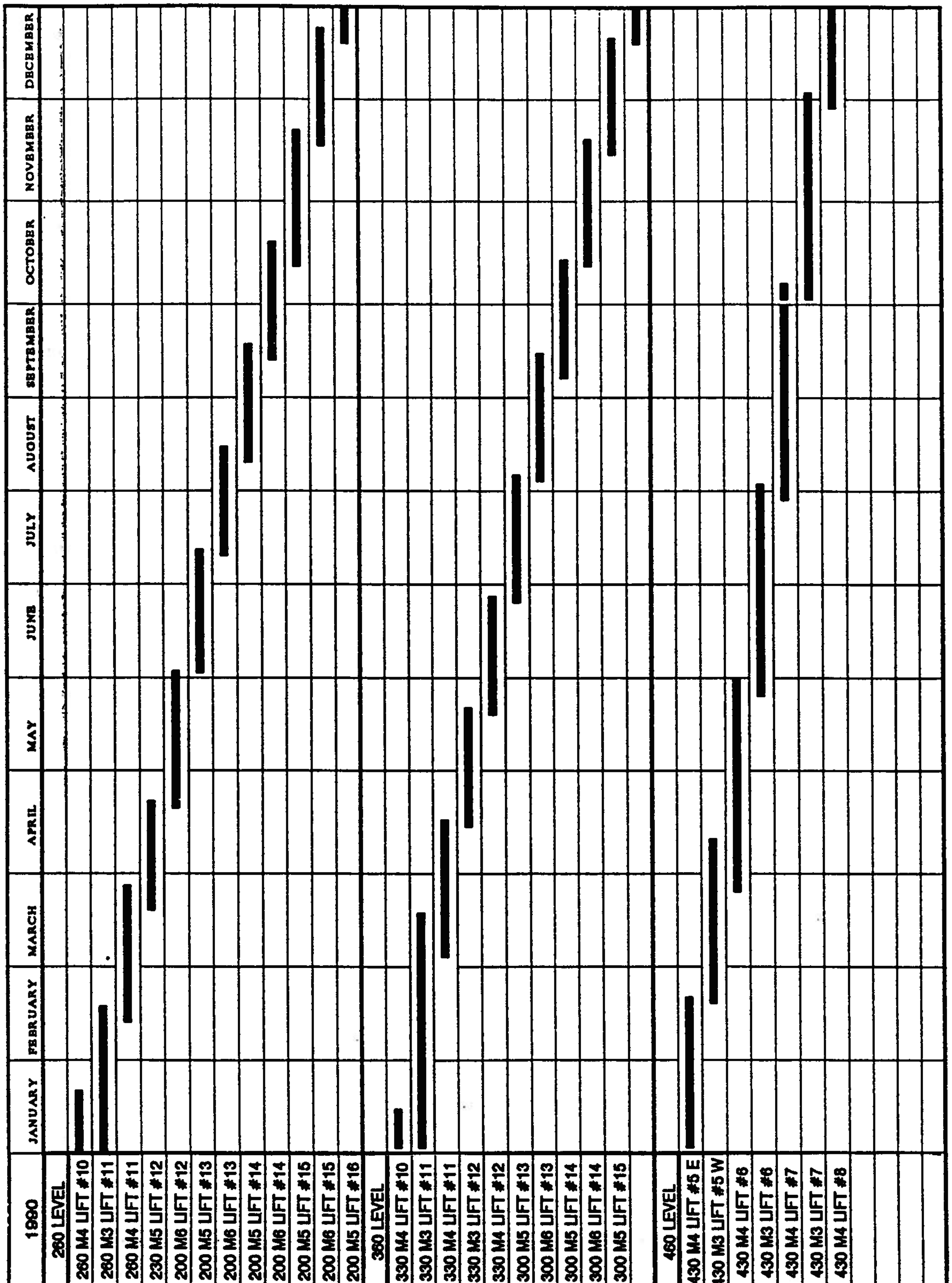
DETOUR LAKE MINE PRODUCTION SCHEDULE

FIGURE 4.23

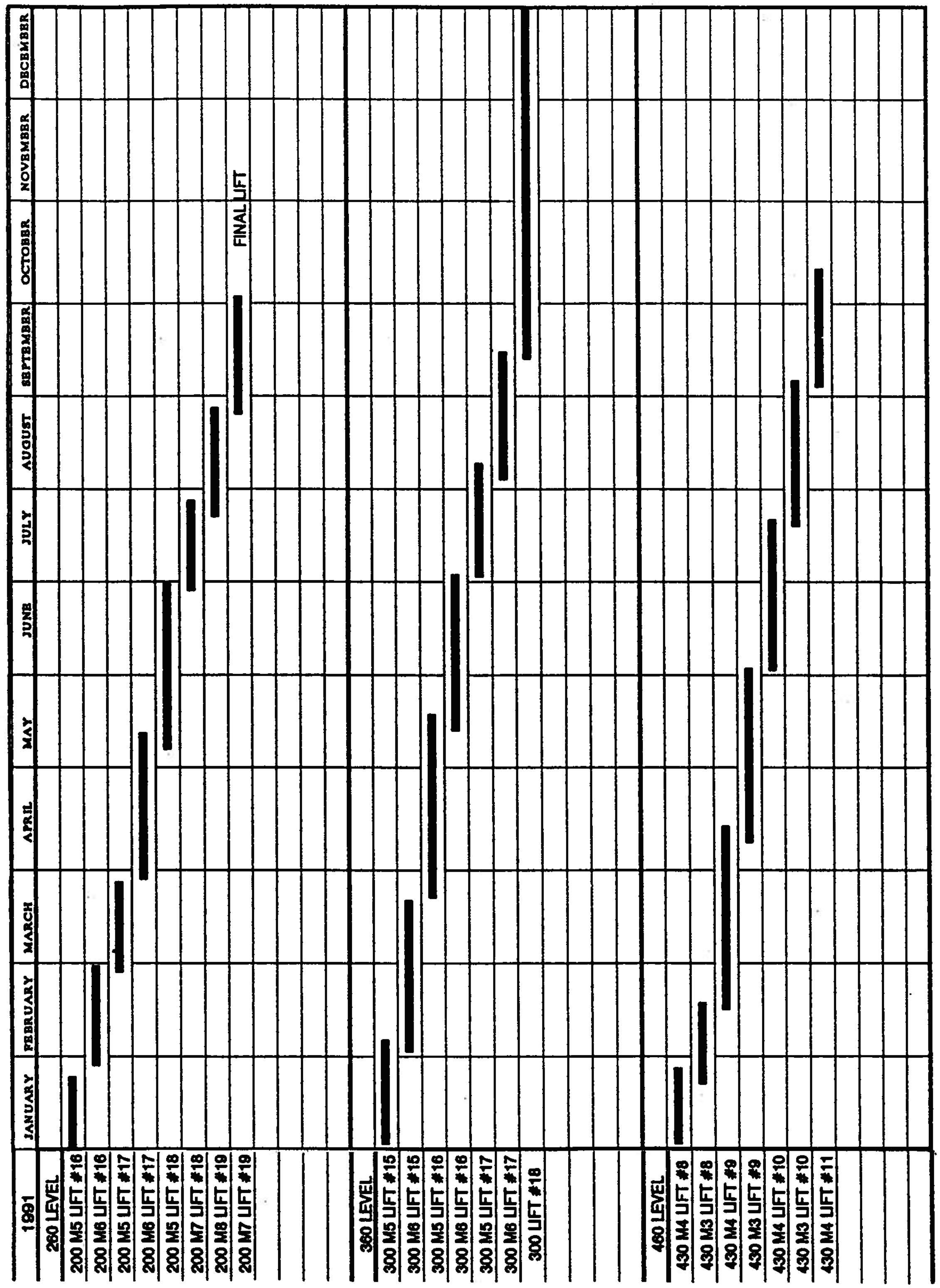


DETOUR LAKE MINE PRODUCTION SCHEDULE

FIGURE 4.23 (continued)

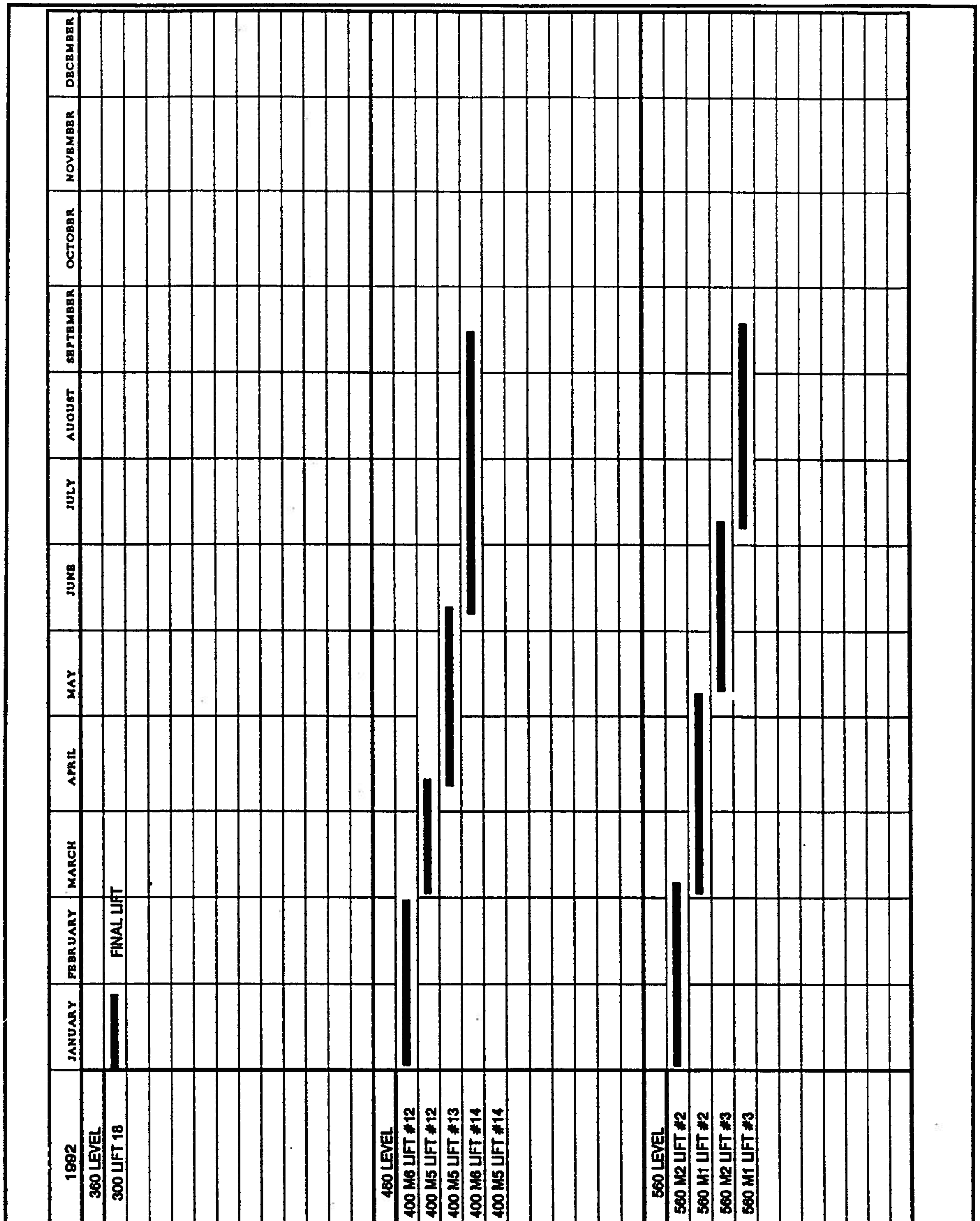


DETOUR LAKE MINE PRODUCTION SCHEDULE	FIGURE 4.23 (continued)



DETOUR LAKE MINE PRODUCTION SCHEDULE

FIGURE 4.23
(continued)



DETOUR LAKE MINE PRODUCTION SCHEDULE

FIGURE 4.23 (continued)

5. NUMERICAL MODELING

5.1 THREE-DIMENSIONAL BEAP

The shape of many underground excavations and the influence of neighbouring excavations make 2-D plane strain analysis inappropriate for many practical mining problems. In such cases, a three-dimensional analysis may be required to achieve a reasonable degree of accuracy.

Three-dimensional boundary element modeling has many practical applications for rock mechanics engineers including:

- pillar design;
- open stope span design/dilution studies;
- shaft and service tunnel layouts;
- analyses of complex excavation geometries;
- stope sequencing studies; and
- parametric design studies

BEAP-3D or Boundary Element Applications Package has been developed by CANMET and was used in the course of this project to carry out three dimensional stress analysis. BEAP-3D is a powerful numerical modeling package designed specifically for modeling three-dimensional underground openings. The version used was capable of modeling up to 1000 elements. The program utilizes a graphical pre-processor called *Mine Designer* (CANMET, 1991) for creating a model geometry file as well as a graphical post-processor called *ViewBeap* (CANMET, 1991) for viewing stress contours.

As with any numerical modeling procedure for determining stresses, the accuracy of the *BEAP-3D* analysis depends on the accuracy of three main input parameters:

1. the stress-strain relation(s) of the material(s);
2. the pre-mining stress conditions; and
3. the model geometry.

The *BEAP-3D* analysis assumes the materials to be linear elastic, homogeneous, and isotropic. This assumption is considered accurate for laboratory scale specimens of intact rock but usually does not represent the stress-strain relationship of the rock mass since the strength is controlled by the discontinuities. Inhomogeneity of the rock mass can be modeled by using different material parameters for different groups of elements in the model.

The pre-mining stress conditions must be determined as input for the analysis. Both the magnitude and direction of the principal stresses are required. These values are normally determined from in-situ stress measurements.

Three dimensional modeling permits better accuracy in modeling the actual mine geometry. The detail of the modeling is limited by the number of elements the program can handle. Large models containing 800-1000 elements require approximately 15 hours to run on a Sparc workstation. As with any numerical modeling program, the results obtained should be carefully scrutinized and applied with a good degree of engineering judgment, recognizing the simplifying assumptions contained in the model.

5.2 NUMERICAL MODELS

There were three main objectives to be achieved through the 3D numerical modeling:

- Post Pillar - to estimate the stress conditions within a post pillar;
- Sill Pillar - To estimate the stress conditions in the sill pillar and to determine whether horizontal stress was contributing to observed cases of instability; and,
- Overall Mine - To locate areas of high stress concentration not already predicted and which may contribute to instability.

5.2.1 Post Pillar Modeling

Post pillars are used at Detour Lake Mine as a means of reducing the exposed span in wide cut and fill stopes. The use of post pillars as a means of support is discussed in greater detail in Section 7 of this report. Post pillars are started on the footwall side of cut and fill stopes. With each lift mined, the stope boundaries shifts southeast due the plunge and dip of the ore. The vertical post pillar migrates towards the centre of the span. These pillars are typically 5 metres square and up to 25 metres in height. It is intended that the pillars will yield as the width to height ratio decreases, however experience at Detour Lake Mine and other operations demonstrate that the post-yield strength of the pillar is still capable of providing support to the immediate back.

A 25 metre high post pillar has been modeled with *BEAP-3D* employing 489 elements. The model assumes an opening 40 by 50 metre excavation, 25 metres high, plunging 45° to the west and dipping at 56° to the north (Figure 5.1). The post pillar extends from the top centre of the modeled stope to the east side of the stope which is commonly the case at Detour Lake Mine. The excavation and post pillar are placed at a depth of between 420 and 460 metres below surface to simulate conditions on Pillar 941 of the 460 Stope at DLM. This pillar was the subject of a support trial whereby the back was pre-supported with cable bolts and the pillar removed. This support trial is described in further detail in Section 7 of this report.

In the computer model, the uncemented backfill has been assumed to be incapable of carrying load from the pillar since the elastic modulus of the fill is much lower than that of the pillar. In addition, the active confining stress provided by the fill is assumed to be zero. In this sense, the model can be considered conservative since the fill does indeed provide confinement to loose blocks which would normally spall off the pillar. This confinement allows strength to be maintained in the pillar core.

The major and minor principal stresses through the pillar are shown in Figures 5.2(a) and 5.2(b). The stresses are shown on a horizontal plane through the centre of the pillar. The higher stresses are shown to be on the east side of the pillar due to the pillar being "grown" from the east wall. The weighted average pillar stress across the east-west centre line of the pillar has been calculated to be 19.8 MPa. The maximum σ_1 of about 40 MPa occurs on the northeast and southeast corners of the pillar. Contours of the horizontal confining stress, σ_3, are shown in Figure 5.2(b) on the same horizontal plane. The confining stress is shown to be less than zero across the entire cross section. The average pillar stress computed by *BEAP-3D* is much lower than what would be predicted using other estimation methods such as tributary theory:

$$\begin{aligned} \sigma_p &= \gamma g h \frac{\text{rock column area}}{\text{pillar area}} \\ \sigma_p &= \frac{(2700)(9.81)(435)(475)}{25} \\ \sigma_p &= 219 \text{ MPa} \end{aligned} \tag{5.1}$$

Despite the low average pillar strength relative to the intact uniaxial compressive strength of the rock (165 MPa) the pillar can be expected to yield due to the existing structure and the lack of confining stress. Using the modified Hoek and Brown failure criterion for jointed rock masses the maximum principal stress at failure is given by (Hoek et al., 1992):

$$\sigma_1 = \sigma_3 + \sigma_c \left(m_b \frac{\sigma_3}{\sigma_c} \right)^a \tag{5.2}$$

where,

m_b and a are parameters which depend on the quality of the rock mass

The failure constants m_b and a have been estimated to be 3.4 and 0.45 respectively, consistent with a good quality, very blocky, fine grained basalt. Clearly, the use of this failure criterion predicts zero strength when there is no confining stress. This confirms the expectation that the pillar has yielded.

Despite having yielded, post pillars at Detour Lake Mine do have residual, post-yield strength and continue to provide support to the immediate back. This will be documented in Section 6 of this report where it will be shown that unstable back conditions develop where spans exceed approximately 20 metres at Detour Lake Mine. Where post pillars are used with a span of 20 metres between pillars, overall span can be increased to the full width of the orebody (35-40 metres). It can therefore be concluded that a post pillar provides support to the stope back making use of its post-yield strength.

The confinement provided by the fill makes a significant contribution to the post-yield strength of the pillar. As the pillar yields, it dilates and compresses the fill so the fill approaches the passive Rankine state. Assuming a passive Rankine earth pressure coefficient, Kp=3.5, and a depth of fill of 7.5 m to the centre of the 25 m tall pillar, the horizontal confining stress is calculated to be 0.5 MPa. From equation 5.2, the post-yield pillar strength is then be estimated to be 22 MPa.

Additional *BEAP-3D* runs were carried out for pillar heights of 20, 15, 10, and 5 metres. The major and minor principal stresses were found to increase at the mid-height of the pillar as the pillar height decreased. Figure 5.3 shows the increased σ_1 and σ_3 stresses for a pillar height of 15 metres. The relationship between pillar height and pillar stress as determined by the modeling is given in Figure 5.4. This relationship cannot be used for general design purposes since pillar stress is also a function of the excavation geometry and the pre-mining stress field.

5.2.2 Sill Pillar Modeling

The sill pillar modeling was carried out to assess whether high horizontal stresses were developing in the stope back as the sill pillar thickness decreased and whether these stresses contributed to an increased frequency of instability in the stope back. Previous boundary element modeling using the 2D BEM program *EXAMINE-2D*, was believed to be overestimating the stress level since a composite vertical section had to be used which modeled all stopes on the same vertical section. In reality this does not occur because the stope is plunging to the west at 45°. In addition, the influence of the backslashed attack ramps on the induced stress around the stopes could not be handled with a 2D model. The backslashed attack ramps were driven approximately perpendicular to the maximum principal stress so high stresses could be expected to develop between attacks driven from different levels.

The entire Detour Lake Mine was modeled from the 460 Level to surface. Major components of the model include the open pit, 260 Stope, 360 Stope, 460 Stope, Quartz Zone longhole stopes, and the 360 Talc stopes. The components of the model are shown in Figure 5.5.

The major principal stress, σ_1 was approximated as acting horizontally in an east-west direction with a magnitude of 0.075 MPa per metre depth. The intermediate principal stress was approximated as acting north-south with a magnitude of 0.038 MPa per metre depth. The minor principal stress, σ_3 was assumed to be vertical and having a magnitude of 0.029 MPa per metre. The rock mass was assumed to be homogeneous, isotropic, and linear elastic with a Young's Modulus of 30 GPa and Poisson's Ratio of 0.26.

The main model was run to simulate the mine geometry as it existed in February, 1992 with pillars having the following thickness:

	Thickness (m)
Crown Pillar	24.1
260 Sill Pillar	13.8
360 Sill Pillar	53.0

Figure 5.6 is a view of the entire mine showing the major and minor principal stresses at the surfaces of the stopes as well as in the sill pillars. Figure 5.7 is a close up view showing stresses in the crown pillar on a subvertical plane through the pillar. The magnitudes for maximum and average stresses in the pillar are provided in Table 5.1. The highest stresses occur in the lower centre part of the pillar between the top attack ramps of the 260 stope. The average pillar stress along the midline of the pillar is 14.9 MPa with an average confining stress of 1.2 MPa. Applying the modified Hoek-Brown failure criterion, the factor of safety for the pillar (defined as the average pillar strength divided by the average pillar stress) is 2.17. A factor of safety of 1.5 is generally considered necessary for permanent support in underground mines (Hoek and Brown, 1980). A maximum horizontal stress of 17.2 MPa occurs in the immediate back of the 260 stope which is only one tenth the uniaxial compressive strength. It can therefore be concluded that high stresses were not a contributing factor for cases of instability recorded in the 260 stope. Rather, the lack of confining stress in the immediate back results in relaxation of the rock into the opening which can lead to structural failure and general rock mass failure.

Figure 5.8 shows maximum and minimum principal stresses in the 260 sill pillar. The highest pillar stress occurs in the centre of the pillar as an elongated band between the two attack ramps. High stresses can also be observed at the top of the attack ramps indicating that they are likely contributing to high stresses in the pillar. The average σ_1 pillar stress measured on a vertical line through the middle of the pillar is 35.6 MPa. The average confining stress, σ_3, is 5.4 MPa. Applying the modified Hoek-Brown failure criterion, the factor of safety is estimated to be 1.91. In the immediate back of the 360 Stope the horizontal stress is in the range of 22 to 26 MPa. Again, this suggests that high stresses did not contribute to cases of instability recorded in this stope.

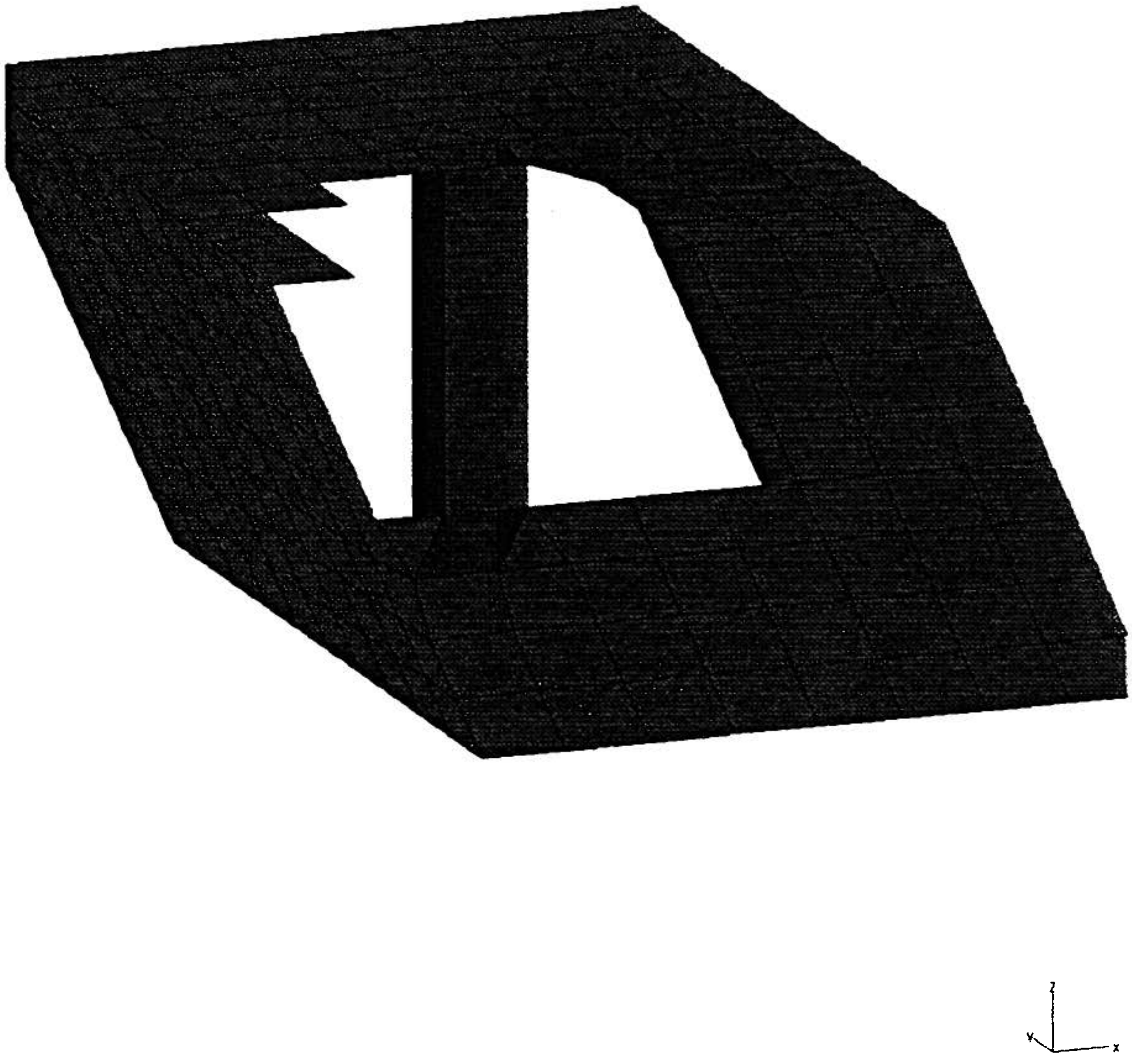
Figure 5.9 shows the major and minor principal stresses for the 360 sill pillar. Horizontal stress is being concentrated above the east attack ramp of the 460 stope and below the west ramp of the 360 stope. As the pillar decreases in thickness as mining proceeds, these two attacks will further concentrate stress in the pillar at this location. The average major principal stress in the pillar is 33.5 MPa and the average minor principal stress, σ_3 is 7.0 MPa. This yields a factor of safety of 2.3 for the pillar.

5.2.3 Other Areas

Another area of stress concentration is between attack ramps driven approximately one above the other. Since the in-situ major principal stress acts roughly perpendicular to these attack ramps, high stresses can be expected to be easily generated. An example is shown in Figure 5.10 is the area between the #3 and #5 attack ramps of the 360 Stope. High σ_1 stress in the order of 50 MPa is observed in this area. Stress is particularly concentrated at the intersection of the attacks and the stope. These high stress areas do not generally pose a threat to stability since they develop in the floor of the backslashed attack and away from active workings.

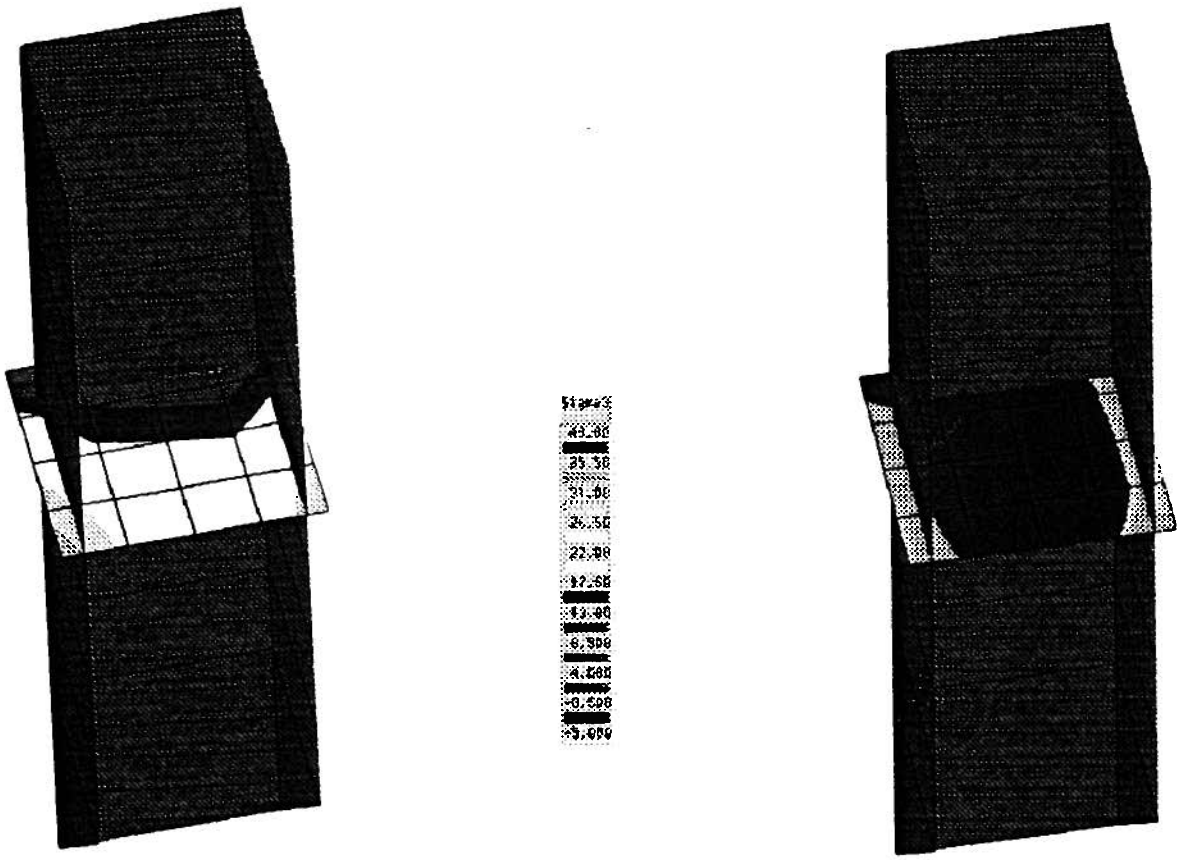
Table 5.1 Major and Minor Principal Stresses in Crown and Sill Pillars

	Average Pillar Stress (MPa)		Maximum Pillar Stress (MPa)		Minimum Pillar Stress (MPa)		Pillar Centre (MPa)	
	σ_1	σ_3	σ_1	σ_3	σ_1	σ_3	σ_1	σ_3
Crown Pillar	14.9	1.2	17.2	2.2	13.2	-0.91	14.5	2.1
260 Sill Pillar	35.6	5.4	51.3	23.2	6.0	-23.4	36.2	6.3
360 Sill Pillar	33.5	7.0	38.3	11.3	30.8	1.3	32.3	8.8



POST PILLAR 941
BEAP-3D MODEL GEOMETRY

FIGURE 5.1

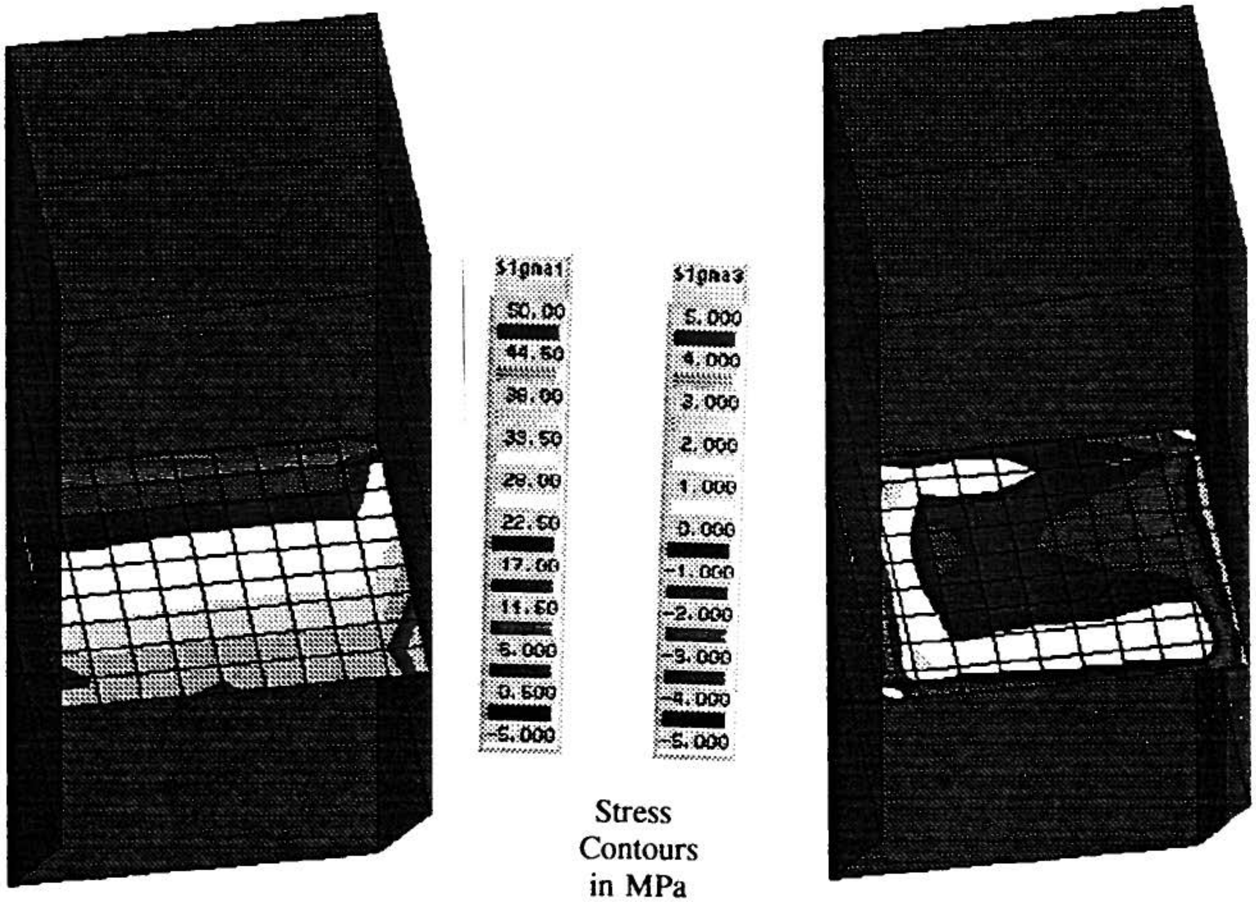


Stress Contours
in MPa

(a) σ_1 Stress

(b) σ_3 Stress

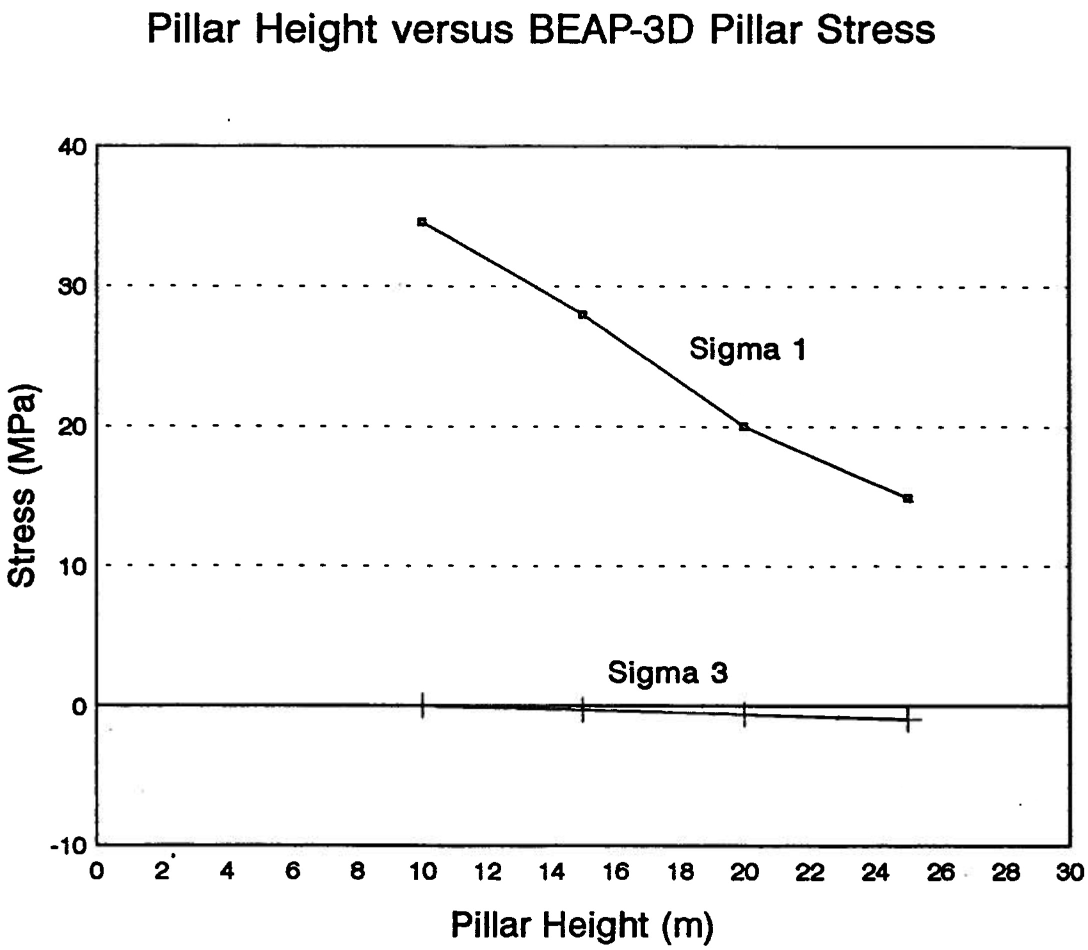
POST PILLAR 941 - MAJOR AND MINOR PRINCIPAL STRESSES AT MID HEIGHT OF PILLAR	FIGURE 5.2



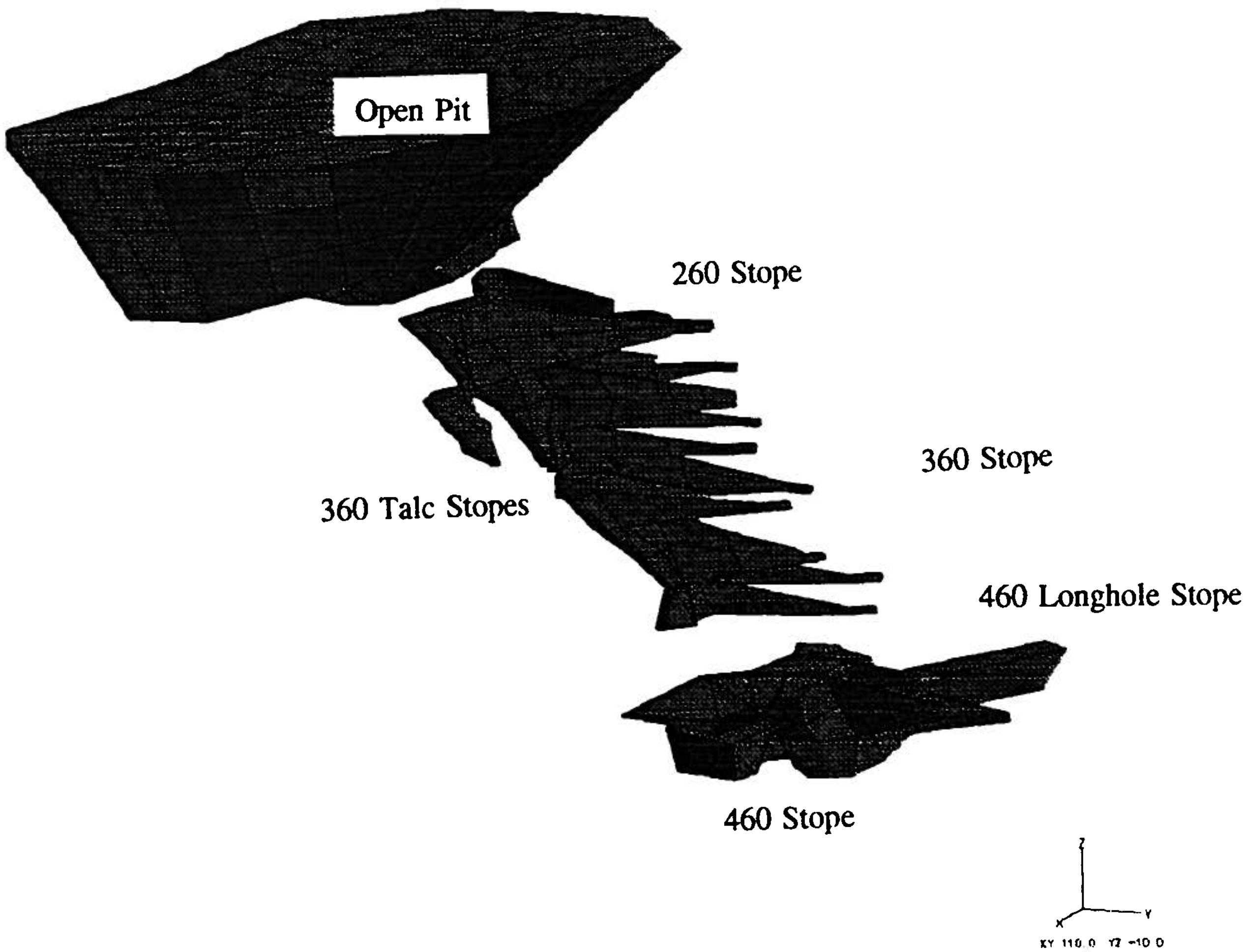
(a) σ_1 Stress

(b) σ_3 Stress

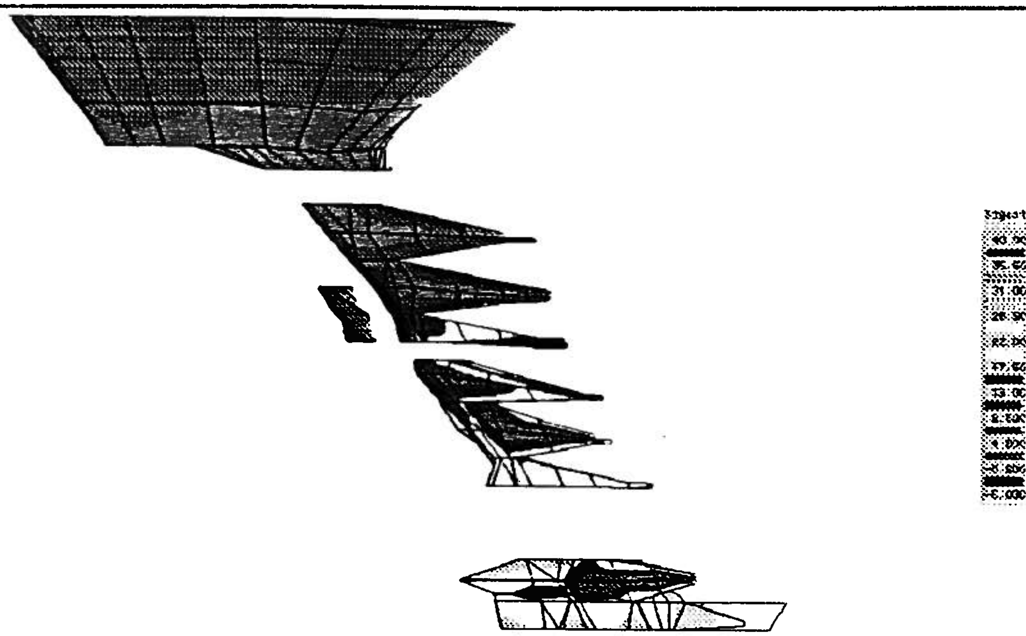
POST PILLAR 941 - 15 M HEIGHT MAJOR AND MINOR PRINCIPAL STRESSES AT MID-HEIGHT	FIGURE 5.3



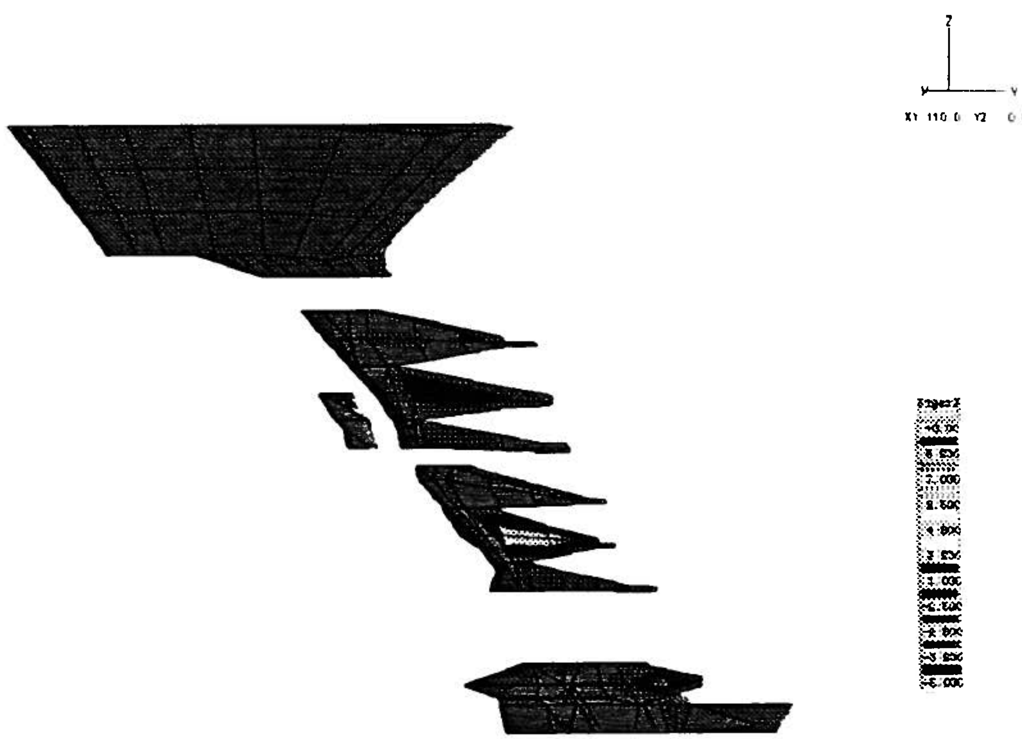
POST PILLAR 941 BEAP-3D PILLAR STRESS VERSUS PILLAR HEIGHT	FIGURE 5.4



DETOUR LAKE MINE BEAP-3D MODEL GEOMETRY	FIGURE 5.5

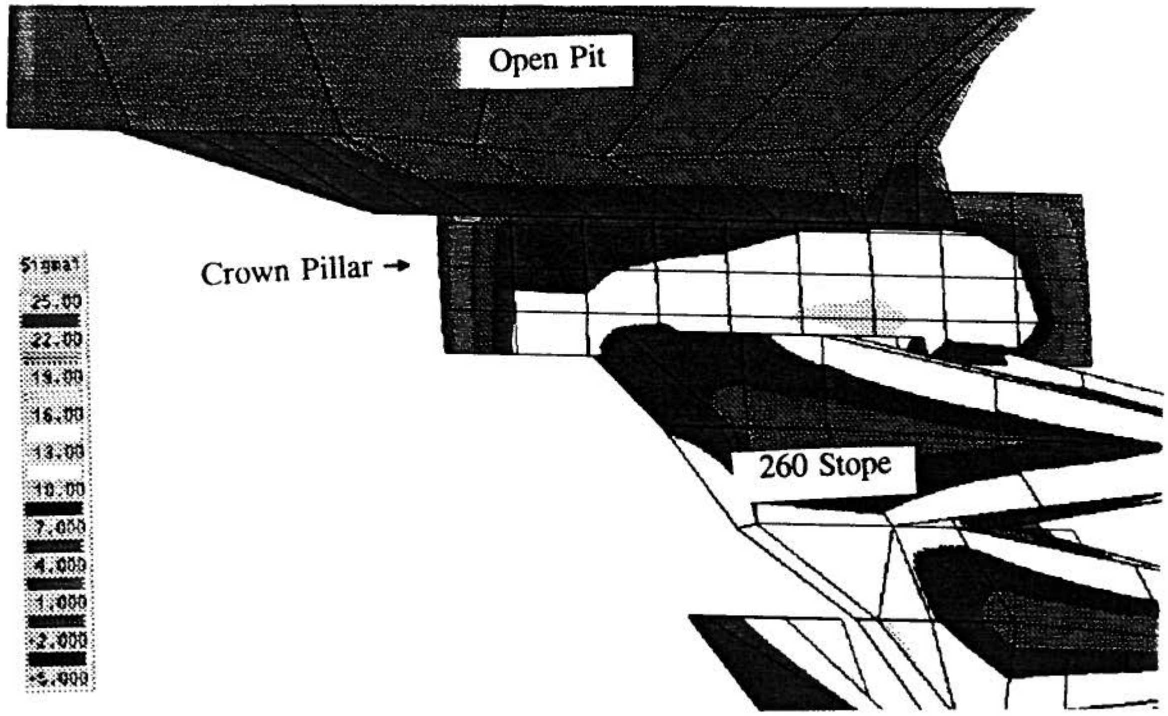


(a) σ_1 Stress on Excavation Boundaries

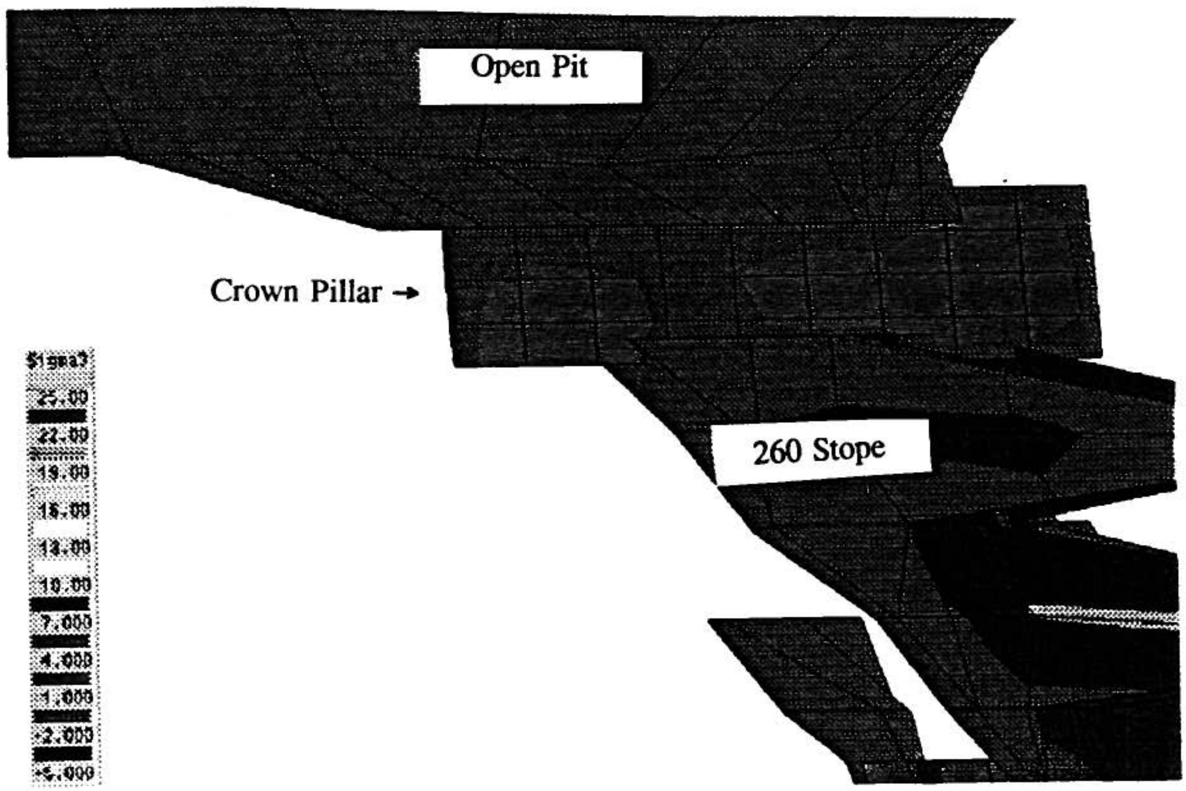


(b) σ_2 Stress on Excavation Boundaries

PRINCIPAL STRESSES ON EXCAVATION BOUNDARIES DETOUR LAKE MINE	FIGURE 5.6

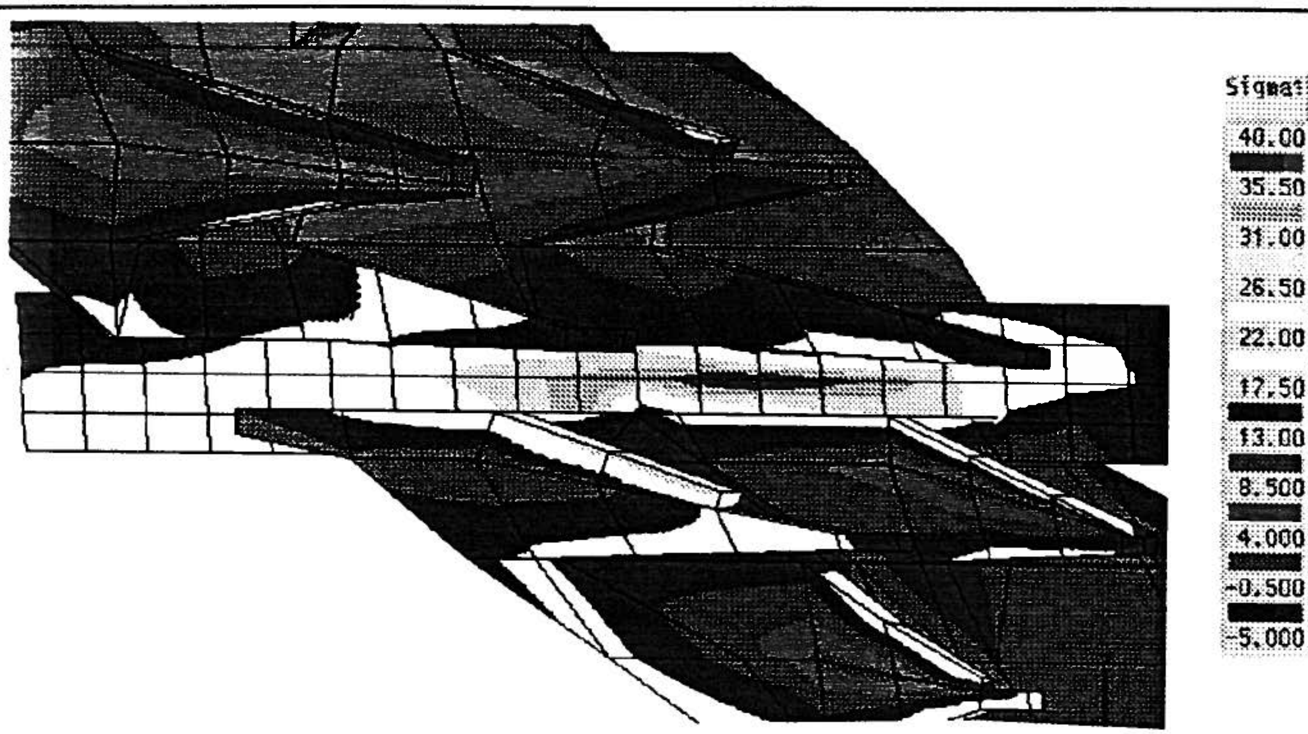


(a) σ_1 Stress

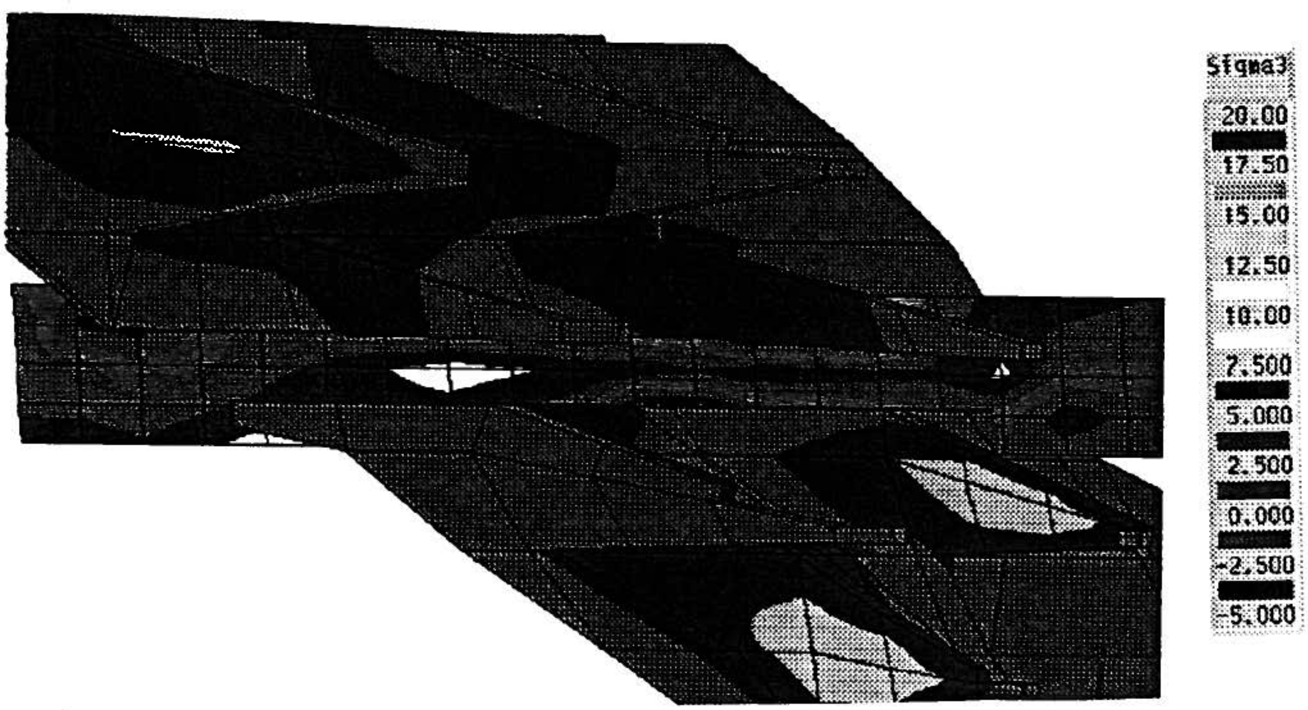


(b) σ_2 Stress

PRINCIPAL STRESSES IN CROWN PILLAR DETOUR LAKE MINE	FIGURE 5.7



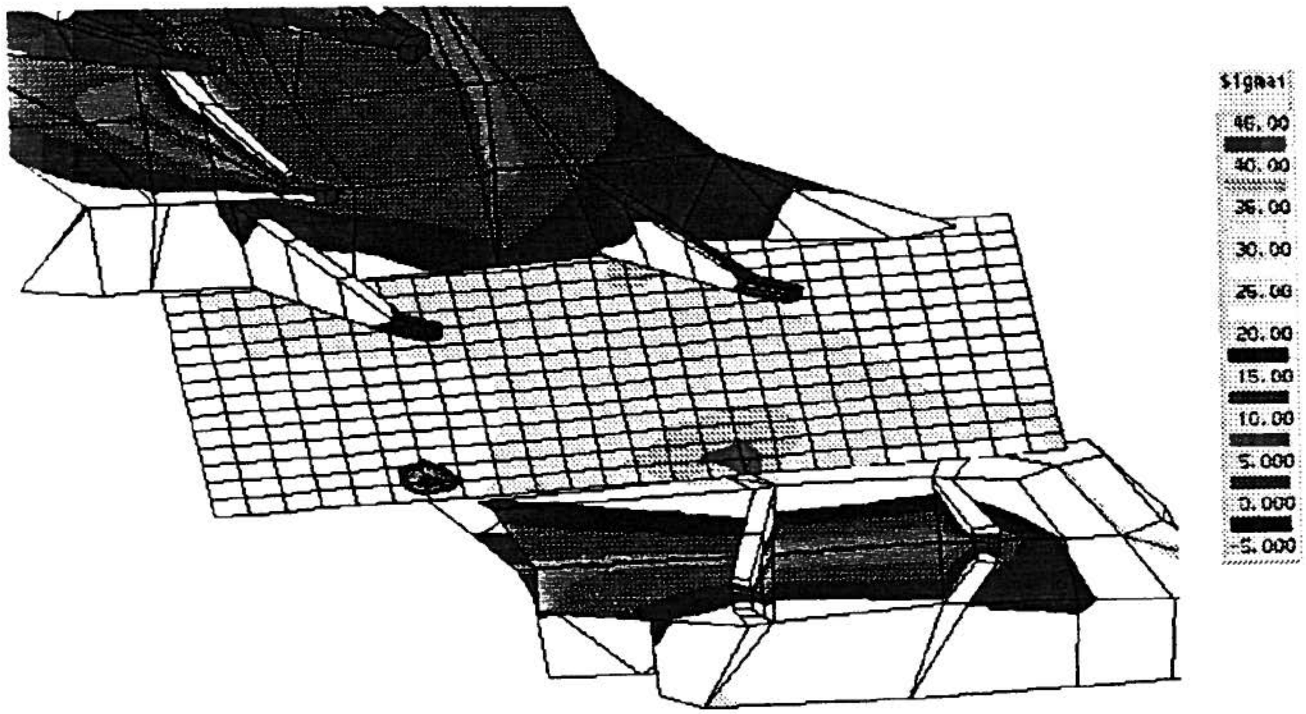
(a) σ_1 Stress



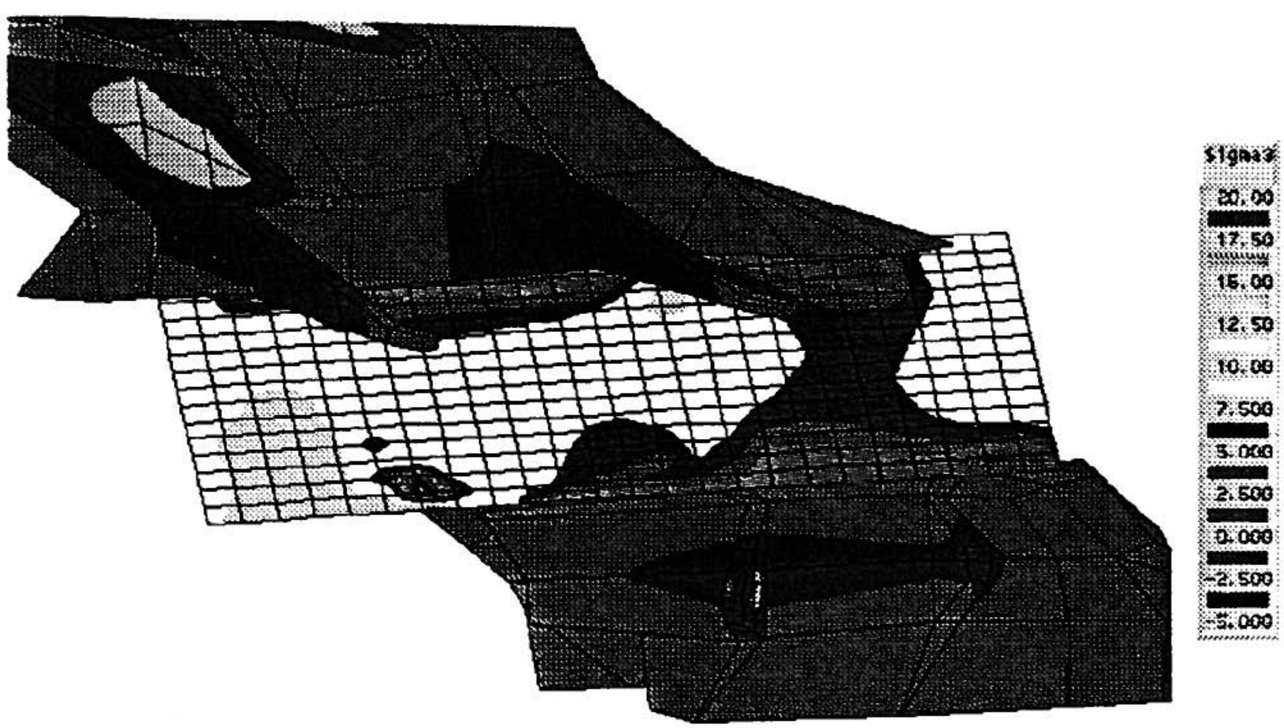
(b) σ_2 Stress

PRINCIPAL STRESSES IN 260 SILL PILLAR OF DETOUR LAKE MINE

FIGURE 5.8

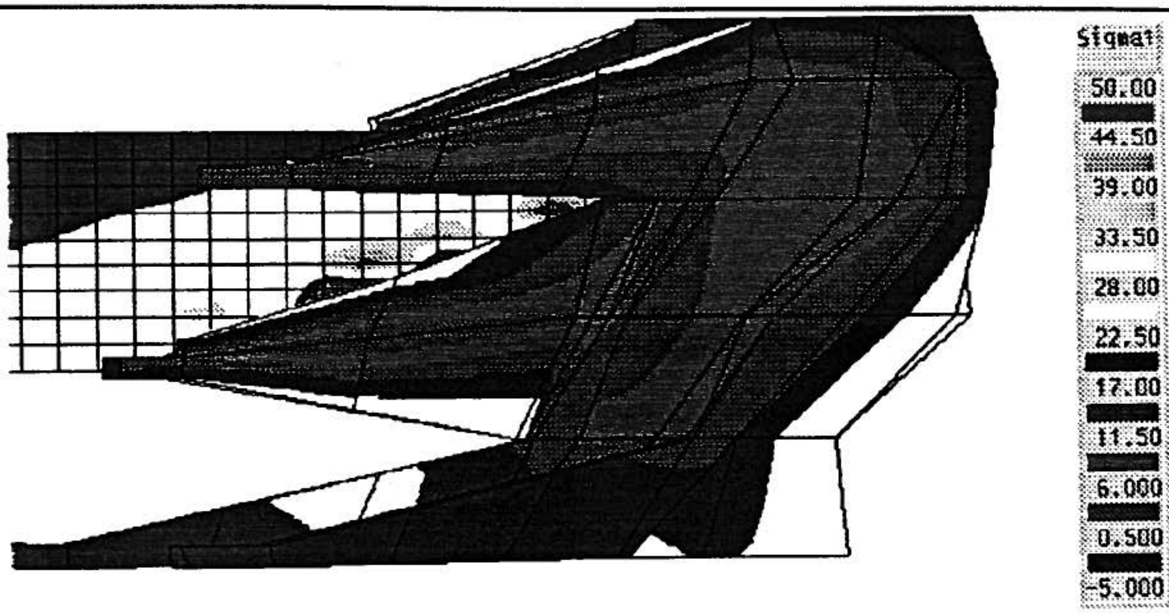


(a) σ_1 Stress

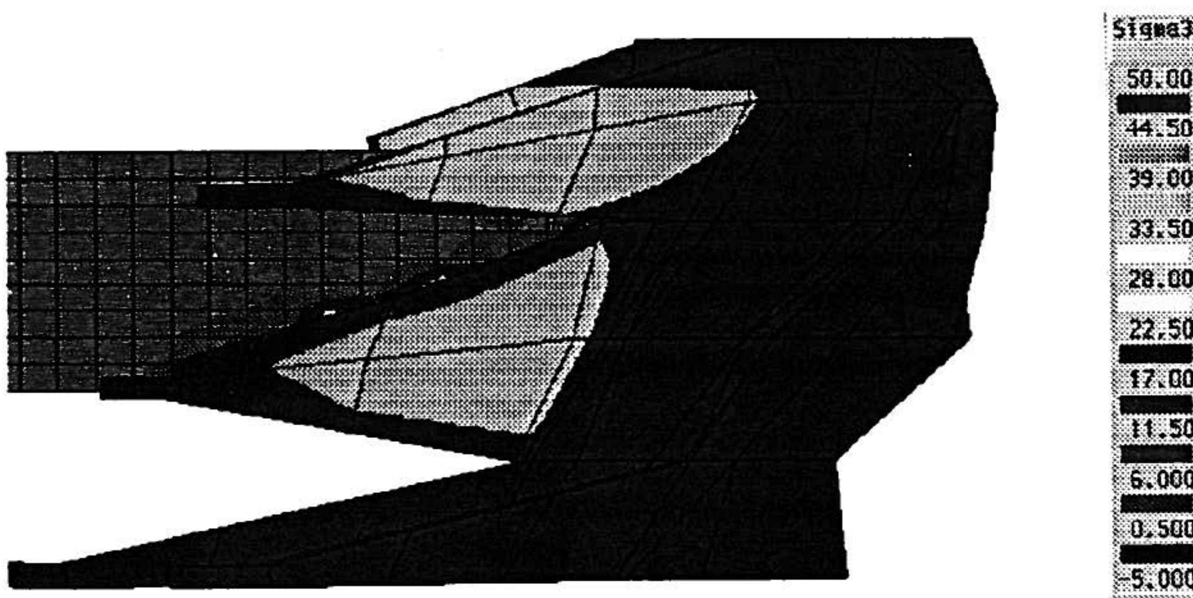


(b) σ_2 Stress

PRINCIPAL STRESSES IN 360 SILL PILLAR OF DETOUR LAKE MINE	FIGURE 5.9



(a) σ_1 Stress



(b) σ_2 Stress

PRINCIPAL STRESSES BETWEEN ATTACK RAMPS OF 360 STOPE	FIGURE 5.10

6. SPAN DESIGN

6.1 DATABASE

The Detour Lake Mine database consists of 172 records of observations made from the commencement of underground mining in 1987 until March, 1992. Each record consists of the location, Geomechanics Rock Mass Rating (RMR), stability, span, support, and a note of the existence of major structures or flat jointing. The Geomechanics Rock Mass Rating has been described in Chapter 3. The terms span, stability, and support require further definition.

6.1.1 Definition of Span

For the purposes of this study, the term ***span*** refers to the diameter of the largest circle which can be drawn within the boundaries of the excavation as viewed in plan (Figure 6.1a). This definition has been adopted for two reasons. First, entry-type stopes can commonly have width (FW to HW) to length (along strike) ratios greater than 0.5, making plane strain analogies invalid. (Other analytical methods define span in a way which assumes plane strain conditions.) Secondly, post pillars and irregular stope boundaries make the calculation of hydraulic radius (used by other researchers) difficult and misleading. In cut and fill stopes, the span should also include the overhang material on the hangingwall, which is not contained by the fill (Figure 6.1b).

The term ***unsupported span*** refers to spans with no support or spans with pattern rock bolting (commonly 1.8 m long bolts on 1.2 m x 1.2 m pattern). It does not include spans which are supported by more intensive ground support such as timber sets, shotcrete, cable bolts, or post pillars. The pattern rock bolting is designed to control loose close to the surface of the excavation, which may develop after scaling as a result of nearby blasting vibrations or stress redistribution caused by subsequent mining activity.

6.1.2 Definition of Stability

Bieniawski (1984) has shown that there is a relationship between span, rock mass rating, and stand-up time. Larger spans will result in lower stand-up times for a given rock mass rating. In this study, most of the data has been obtained from cut and fill stopes where the required stand-up time is approximately three months. Therefore, a stable excavation is defined as one which has remained stable for at least three months.

An excavation's stability is classified into three categories: stable, potentially unstable, and unstable.

6.1.2.1 Stable Excavations

Stable Excavations are characterized by the following:

- There have been no uncontrolled falls of ground;
- If instrumentation has been installed, there has not been any movement of the back which would warrant concern; and,
- There were no extraordinary support measures implemented.

6.1.2.2 Potentially Unstable Excavations

Potentially unstable excavations are usually not difficult for experienced personnel, familiar with ground conditions at their operation, to identify. The openings may exhibit the following characteristics:

- The opening may exhibit strong slips or faults with orientations forming potential wedges in the back;
- Extra ground support may have been installed to prevent potential falls of ground;
- Instrumentation installed in the back has recorded continuing movement of the back; or,
- There may be an increased frequency of popping and snapping indicating working of the ground.

6.1.2.3 Unstable Excavations

Unstable openings are simply those where an uncontrolled fall of ground has occurred. A fall of ground would usually involve failure through existing support; or, in the case of no support, the extent of the failure would be large enough to cause damage to pattern rock bolt support if it were installed. Uncontrolled falls of ground are distinguished from "loose incidents" which occur close to the face prior, to scaling and before rock bolts have been installed.

6.1.3 Detour Lake Mine Observations

The raw data is provided in Table 6.1 and plotted in Figure 6.2. Figure 6.3 and Table 6.2 employ the same data except that the RMR has been reduced by 10% where unfavourable structural orientation is recorded. At Detour Lake Mine, this correction is applied where:

- faults or long, continuous, open joints are observed in the back (0-60 degree dip); and,
- flat jointing is observed in back (0-30 degree dip).

It has been found that the combination of the flat structure combined with the dominant vertical EW and NS joint sets are the greatest source of instability at Detour Lake Mine. It can be seen in Table 6.3 that the use of this correction results in a lower standard deviation of the corrected unstable data and virtually unchanged standard deviations for the stable and potentially unstable data sets. The 10%

correction has been suggested by Bieniawski (1984) as a correction factor applied for unfavourable joint orientation.

6.2 STATISTICAL ANALYSIS

6.2.1 Objective

The objective of the following statistical analysis is to define three zones on the span versus RMR graph; stable, unstable and potentially unstable, which can be used as a guide for the design of cut and fill stopes. Table 6.3 summarizes the main statistics for each of these three groups. The large variance in each of the three groups results in the boundaries of the groups being ill-defined. Figure 6.4 is a plot of the data with bivariate ellipses for each of the three groups. These ellipses illustrate the overlapping of data which occurs between groups. All three groups show parallel positive correlations and the group centroids are shown to lie approximately along the same line. This feature will prove useful in the statistical analysis which follows.

Figure 6.5 is a density plot of the data for each of the three groups. The contours indicate that most of the data has been collected from a range of rock qualities from 40 to 85 and spans from 5 to 30 metres. The choice of data collection sites was random in the sense that one or two observations were made from each lift of each stope in the mine. It is not of interest, however what the probability is of finding a certain span at a certain rock mass rating. Rather, it is necessary to determine the probability of a stope being stable, potentially unstable, or unstable given the span and RMR. For this reason, the ideal sample would have a uniform density over the span-RMR domain. This was not possible however, given actual stoping conditions. The actual density plot for all data points is given in Figure 6.6. The concentration of data in some areas and the lack of data in others, is an unavoidable source of error in the analysis.

6.2.2 Group Classification

In order to define the boundaries of the stable, potentially unstable, and unstable zones, conventional partitioned cluster analysis (Romesburg, 1990) was attempted but the technique tended to create compact clusters rather than linear ones, which could reasonably be expected given the parallel trending bivariate ellipses in Figure 6.4. A form of discriminant analysis proved to be more successful in defining groups. The goal of discriminant analysis is to find the linear combination of variables that maximizes the variance between groups relative to the variance within groups. The technique employed uses the generalized Mahalanobis distance, D_j (Sneath and Sokal, 1973) to classify data points into the three groups. The distances D_1, D_2, and D_3 are computed for each case. The case can then be classified according to the minimum Mahalanobis distance.

There are two main criteria which must be satisfied in order for the method to be valid. The first requirement is for multivariate normality. This can be assessed using a normal distribution probability plot for each of the three groups (Figure 6.7). The closer the points approximate a straight line, the more normal the distribution. From the graphs, it can be seen that the expected values approximate a straight line. The second requirement is for homoscedasticity of the sample dispersion matrices. That is, the variances between groups must be similar for each variable. This can be assessed using dot plots for each variable (Figure 6.8). The vertical length of the band of data in each column should be similar to achieve homoscedasticity. This is approximately the case for this data despite a low number of samples in the potentially unstable range. Although the data does not show near perfect bivariate normality and homoscedasticity, the Mahalanobis classification technique has shown considerable robustness to violations of these requirements (Sneath, Sokal, 1973).

The Mahalanobis distance classification technique is carried out as follows:

$$\text{Let } \mathrm{x}_1 \ldots x_n = \text{all pairs of stable data (span, RMR) i.e. } \mathrm{x}_1 = \begin{bmatrix} 15 \\ 85 \end{bmatrix}$$

$$\text{Let } \mathrm{y}_1 \ldots y_n = \text{all pairs of potentially stable data (span, RMR) i.e. } \mathrm{y}_1 = \begin{bmatrix} 13 \\ 61 \end{bmatrix}$$

$$\text{Let } \mathrm{z}_1 \ldots z_n = \text{all pairs of unstable data (span, RMR) i.e. } \mathrm{z}_1 = \begin{bmatrix} 20 \\ 77 \end{bmatrix}$$

then the mean sample vectors for each group are given by (Johnson and Wichern, 1988)

$$\overline{\mathrm{x}} = \frac{1}{\mathrm{n}_1} \sum_{\mathrm{i}=1}^{\mathrm{n}_1} x_i ,$$

$$\overline{\mathrm{y}} = \frac{1}{\mathrm{n}_2} \sum_{\mathrm{i}=1}^{\mathrm{n}_2} y_i ,$$

$$\overline{\mathrm{z}} = \frac{1}{\mathrm{n}_3} \sum_{\mathrm{i}=1}^{\mathrm{n}_3} z_i ,$$

The sample covariance matrices can be evaluated for each group using the following relationships (Johnson and Wichern, 1988):

Stable Group: $$S_1 = \frac{1}{n_1 - 1}\sum_{i=1}^{n_1}(x_i - \bar{x})(x_i - \bar{x})^T$$

Potentially Unstable Group: $$S_2 = \frac{1}{n_2 - 1}\sum_{i=1}^{n_2}(y_i - \bar{y})(y_i - \bar{y})^T$$

Unstable Group: $$S_3 = \frac{1}{n_3 - 1}\sum_{i=1}^{n_3}(z_i - \bar{z})(z_i - \bar{z})^T$$

If the groups can be classified as multivariate normal distributions with the same variance, then a pooled sample covariance matrix S_{pooled} (Johnson and Wichern, 1988) can be formed where:

$$S_{pooled} = \frac{(n_1 - 1)S_1 + (n_2 - 1)S_2 + (n_3 - 1)S_3}{n_1 + n_2 + n_3 - 3}$$

Figure 6.9a contains three fictional groups (say stable, unstable, and potentially unstable) in a two dimensional domain. For any new point $l=(l_1,l_2)$ it is possible to classify it into one of the groups using the Mahalanobis Distance (Johnson and Wichern, 1988). The minimum Mahalanobis Distance defines the group to which the new case belongs.

The Mahalanobis Distances, D_j are given by:

$$D_1 = (x_1 - \bar{x})^T [S]^{-1} (x_1 - \bar{x})$$
$$D_2 = (y_1 - \bar{y})^T [S]^{-1} (y_1 - \bar{y})$$
$$D_3 = (z_1 - \bar{z})^T [S]^{-1} (z_1 - \bar{z})$$

There will be points on the graph where the distance $D_1=D_2$, $D_2=D_3$, and $D_1=D_3$. These points will define boundaries of the stable-potentially unstable, unstable-potentially unstable, and stable-unstable zones respectively (Figure 6.9b). The equations of these linear boundaries can be derived as follows:

For any new point, ℓ, in the SPAN - RMR domain, where $\ell = \begin{bmatrix} \text{SPAN} \\ \text{RMR} \end{bmatrix}$

Set $D_1 = D_2$

$$(\ell - \bar{x})^T S^{-1} (\ell - \bar{x}) = (\ell - \bar{y})^T S^{-1} (\ell - \bar{y})$$

$$\ell^T S^{-1} \ell - \bar{x}^T S^{-1} \ell - \ell^T S^{-1} \bar{x} + \bar{x}^T S^{-1} \bar{x} = \ell^T S^{-1} \ell - \bar{y}^T S^{-1} \ell - \ell^T S^{-1} \bar{y} + \bar{y}^T S^{-1} \bar{y}$$

Eliminating terms to obtain:

$$(\bar{y}^T - \bar{x}^T) S^{-1} \ell + \ell^T S^{-1} [\bar{y} - \bar{x}] + \bar{x}^T S^{-1} \bar{x} - \bar{y}^T S^{-1} \bar{y} = 0$$

This is the equation for a straight line,

$$\ell^T 2S^{-1} [\bar{y} - \bar{x}] + [c] = 0$$

where,

$$a_1 \ell_1 + a_2 \ell_2 + c = 0 = a_1 SPAN + a_2 RMR + c$$

$$\begin{bmatrix} a_1 \\ a_2 \end{bmatrix} = [a] = 2[S]^{-1} [[\bar{y}] - [\bar{x}]]$$

$$c = [\bar{x}]^T [S]^{-1} [\bar{x}] - [\bar{y}]^T [S]^{-1} [\bar{y}]$$

Similarly for $D_1 = D_2$ and $D_1 = D_3$.

Using the corrected database, the variance covariance matrix has been computed :

$$[S] = \begin{bmatrix} 37.31 & 44.28 \\ 44.28 & 135.7 \end{bmatrix} \quad \text{then,} \quad [S]^{-1} = \begin{bmatrix} 0.0438 & -0.0143 \\ -0.0143 & 0.0120 \end{bmatrix}$$

Setting $D_1 = D_2 \Rightarrow$ Span $= 0.8858(\text{RMR}) - 43.15$

Setting $D_1 = D_3 \Rightarrow$ Span $= 0.7556(\text{RMR}) - 31.35$

Setting $D_2 = D_3 \Rightarrow$ Span $= 0.6496(\text{RMR}) - 21.77$

The three lines corresponding to D_1=D_2, D_2=D_3, and D_1=D_3 have been plotted in Figure 6.10. The band width of the zone is wide at low RMR and converges to a point at RMR = 90. This feature is a function of the statistical technique used and the fact that all three group centroids do not lie in a perfectly straight line. The band width at the centre of the data is taken to be the most accurate since the highest concentration of data is located there. If the centroids of the three groups formed a perfect line, all three lines would be parallel. For these reasons, the band width at the point where the centroid trend line (Figure

6.4) crosses the $D_1 = D_3$ line has been applied along the length of line $D_1 = D_3$ (Figure 6.11). The centre line defining points equidistant from the stable and unstable zones centroids is redundant and has been removed since we wish to define the boundaries of the potentially unstable zone. Finally, the graph should only be applied to design of stopes which fall within the limits of the database used to create the graph. The potentially unstable zone has been shaded and cutoffs applied at the ends where insufficient data exists (Figure 6.12). This graph will be referred to as the *Stability Graph for Entry-Type Stopes* and is recommended for design of spans in cut and fill stopes, shrinkage stopes, undercuts of longhole stopes and other temporary entry-type mine workings.

The Mahalanobis distances have been computed for each data point and are provided in Table 6.4 along with the probabilities that a point belongs to either of the three groups. The probabilities are simply a ratio of the Mahalanobis distances. A contour plot of these probabilities is given in Figure 6.13. It gives the user of the Stability Graph for Entry-Type Excavations a measure of the confidence that a given design will fall within each of the groups. For example, the probability of a design with a span of 20 metres and an RMR of 75, is stable is 0.6. The probability it is potentially unstable is 0.3. The probability it is unstable is 0.1.

6.3 LIMITATIONS OF THE EMPIRICAL DESIGN METHOD

6.3.1 Structure

It is important to emphasize that the lower span design line shown in Figure 6.12 represents the maximum span for a given rock quality above which the potential for instability is high. The design span should always be below the shaded region. The most important factors controlling the degree to which the design span falls beneath the potentially unstable zone is how effectively the ground conditions are being monitored and whether more intensive support can be installed if required.

6.3.2 State of Stress

The empirical span design method presented above does not directly account for factors such as the state of stress in the back and the influence of fill. One mine reported in the industry survey that these factors also influence their span design. In most rock types, changes in stress may be recognized by a corresponding change in rock mass quality. In such cases, stress conditions are being accounted for indirectly using rock mass ratings. Laubscher (1990) has suggested a rating factor which increases the rock mass rating for increasing confinement since frictional resistance to sliding along joints will be increased. This is the case for material away from the surface of the excavation however lack of confinement at excavation surface will allow for movement along joint surfaces. In non-bursting ground this will be manifested as a lower rock mass rating. Therefore, proposed design curves remain valid since the Rock Mass Rating is a dynamic variable.

The reduction of RMR due to bursting is one aspect of another CANMET sponsored research study currently underway at Dickenson Mines in Ontario. Following a rockburst, the opening joints surrounding an excavation could cause the rock mass rating to decrease by as much as 20%. Modeling of stresses around the cut and fill stopes carried out in Chapter 5 indicated low stresses immediately surrounding the excavation so the influence of high stresses could not be assessed in any greater detail for this study.

6.4 COMPARISON WITH OTHER EMPIRICAL SPAN DESIGN METHODS

Other empirical span design methods have been described in Chapter 3 of this report. Direct comparisons of the data and design curves are not possible due to the differences in each researcher's definitions of terms such as unsupported span and stability. Other researchers such as Potvin (1988) and Laubscher have used hydraulic radius rather than span. Some initial comparisons can be made to the work of Bieniawski (1974) and Barton *et al.* (1974).

6.4.1 Comparison to RMR Data

Bieniawski's original failure case histories presented in Figure 3.15 have been superimposed on the three stability zones established above (Figure 6.14). Note that the data does not include any stable case histories. The lower bound of the failure band represents the point below which failure will never occur. This is roughly comparable to the lower bound of the potentially unstable zone of the cut and fill stability graph. The upper bound of Bieniawski's failure band is defined as the line above which instability will occur immediately. Below the line are unstable cases where instability occurs after a period of time. Using the previous definition of stable (stable for at least 3 months), all points below 67% RMR would be classed as unstable since their stand-up times are less than 3 months. The upper bound on the band is valid for RMR greater than 73% (stand-up time greater than 3 months). A new curve based on this short-term stability requirement is given in Figure 6.15. In general, the Bieniawski curve is not a good predictor of short term stability in entry-type excavations. The method predicts 37% of the stable DLM cases in the stable zone compared with 77% using the Stability Graph for Entry-Type Excavations

6.4.2 Comparison to NGI Data

Figure 3.12 presents the relationship between the equivalent dimension, D_e and the NGI Tunneling Index, Q. Assuming the recommended excavation support ratios of 3.0 and 5.0 for mining excavations, upper and lower bounds to the potentially unstable zone can be established. These bounds have been plotted using the rock mass rating and span on linear scales for comparison purposes (Figure 6.16). The NGI index Q was converted to RMR using Equation 3.33.

Despite that only two case histories were used to determine the ESR's of the curves in Figure 6.16, the results are remarkably close to the potentially unstable zone of the Stability Graph for Entry-Type Excavations over the range 40-85 RMR. Although Barton's curve is shown to extend below 40% RMR there are too few case histories to justify it.

6.4.3 Comparison to Golder Crown Pillar Data

The Golder crown pillar case histories (Carter, 1990), described in Section 3, are plotted on the Stability Graph for Entry-Type Excavations in Figure 6.17. For this database, the boundaries of the potentially unstable zone are somewhat conservative. Only one unstable point plots in the stable zone while 19 stable points plot in the unstable zone. This result reflects a more conservative definition of "unstable" used for entry-type stopes in the Detour Lake Mine database.

6.5 INSTRUMENTATION AND OBSERVATION

6.5.1 Instrumentation to Determine Instability

Ground Movement Monitors (GMM's) have proven to be the most effective type of instrumentation for regular monitoring of stability in stope backs at Detour Lake Mine. The GMM consists of a sliding linear potentiometer which is attached to a threaded-both-ends rock bolt. The rock bolt can be anchored at any depth depending on where the discontinuity is expected (Figure 6.18). At DLM they are installed at a depth of 3.8 metres since this is the limit of the drilling equipment and groundfalls greater than three metres deep have never occurred. A plate on the GMM is glued to the collar of the hole and as the back moves relative to the anchor, a resistance change can be measured, which is then converted to distance. The cost of the GMM is approximately CDN $450.00 however they are durable and recoverable. GMM's are used at DLM wherever unfavourable structure is encountered or where the rock mass and span are such that the stope plots in the potential unstable zone of the Stability Graph for Non-Entry Excavations.

Figure 6.19 shows the typical GMM response that could be expected in a stope which is being progressively enlarged, creating a wider span. For the first day, or until after the first nearby blast, slight fluctuations in the readings can be expected due to adjustment of the anchor glue setting and other factors. The readings should then stabilize with a slow steady movement rate or no movement at all. At Detour Lake Mine, an area is considered potentially unstable if the rate of movement accelerates to 1 mm or more in a 24 hour period. This rate has been established only through experience. The normal monitoring interval is once per day and this increases to once or twice per shift if the rate of movement is high. When the movement exceeds 1 mm per day, production from the area is halted while monitoring continues. If the movement subsides, the stope will be reactivated and additional support will usually be installed.

Other movement detectors such as Mine Spiders (Figure 6.20) have been used at Detour Lake Mine but were not successful because blasting would set them off prematurely. These instruments consist of spring loaded reflective canisters which are attached to a rock bolt. Four arms extend from the canister to the back and if the back moves, a fluorescent indicator will pop down indicating movement has taken place. Another disadvantage of the Mine Spiders is that it is not possible to record the magnitude of the movement which has taken place.

Multi-wire extensometers can provide the same information as a GMM however they are expensive and non-recoverable. Readings must be taken by technicians whereas GMM readings can be taken by supervisors and stope leaders. For these reasons, extensometers are only used in special circumstances.

6.5.2 Visual Monitoring of Ground Conditions

At DLM, design spans very closely approach the lower boundary of the potentially unstable zone. This is possible because ground conditions are closely monitored by geological personnel trained in geotechnical mapping. Underground supervisors and crew leaders are also trained in recognizing and responding to changes in ground conditions. An effective reporting system must be in place to ensure that changes in ground conditions are promptly communicated to the engineering department so design changes can be made. The more confidence a mine has in its ability to quickly recognize and adjust to changes in rock mass quality, the closer its design spans can safely approach the potentially unstable zone.

6.6 CONCLUSIONS

The Stability Graph for Entry-Type Excavations derived above is a significant improvement over existing methods of predicting stable spans in cut and fill stopes. The graph recognizes the real world uncertainty which exists between stable and unstable excavations.

Users of the graph must always be aware of the limits of the database which control its applicability. These are:

- Span is determined by the diameter of the largest circle drawn between pillars and stope boundaries;
- The term span refers to spans with keyblock support only;
- The term stable refers to short term stability (approximately 3 months);

- The graph is considered applicable over the Geomechanics rock mass rating of 40 to 85;
- High horizontal stresses are not assumed to be a factor controlling stability; and
- The graph applies to horizontal design surfaces.

Table 6.1 Raw Data - Detour Lake Mine

Case No.	Date Recorded	Stope Location	RMR (%)	Q	Span (m)	Cond-ition	STABILITY S" = STABLE, "?" = POTENTIALLY UNSTABLE "U" =UNSTABLE, "*"STABLE WITH SUPPORT
1	FEB/1990	230M4#11	85	95.2	15	S	STABLE
2	FEB/1990	230M3#11	87	118.8	25	S	STABLE
3	FEB/1990	330M4#10	77	39.1	20	?	FLAT OPEN JOINTS(0-20 DEG) - SMALL GROUND FALL 25T
4	FEB/1990	330M3#11	77	39.1	16	S	STABLE
5A	FEB/1990	430M3#5	78	43.7	19	S	POST PILLAR OF 5m
5B	FEB/1990	430M3#5	78	43.7	28	P	28m INCLUDING POST PILLAR
6	FEB/1990	430T40#5	61	6.6	13	U	WEDGE 55 DEGREE+DYKE - COLLAPSE
7	FEB/1990	430T5#5	67	12.9	9	S	
8	FEB/1990	430T60#5	50	1.9	9	S	BACK STABLE, WALL UNSTABLE
9	MAR/1990	230M4#11	85	95.2	15	S	
10	MAR/1990	230M3#11	87	118.8	15	S	
11A	MAR/1990	330M4#10	73	25.1	20	S	
11B	MAR/1990	330M4#10	73	25.1	20	U	BREAST FLAT + RELEASE
12A	MAR/1990	430M3#5	78	43.7	19	S	POST PILLAR OF 5m
12B	MAR/1990	430M3#5	78	43.7	28	P	28m INCLUDING POST PILLAR
13	MAR/1990	430T60#5	50	1.9	9	S	BACK STABLE,WALL UNSTABLE
14	APR/1990	200m5#12	73	25.1	20	S	
15	APR/1990	230M4#11	85	95.2	15	S	
16	APR/1990	260T70ACCESS	42	0.8	5	S	
17A	APR/1990	330M4#11	73	25.1	20	S	BACK
17B	APR/1990	330M4#11	73	25.1	20	U	BREAST FLAT + RELEASE
18	APR/1990	300M5#12	77	39.1	18	S	
19	APR/1990	430T5#6	62	7.4	12	S	STEEP STRUCTURE - 4m SUPER SWELLEX
20	APR/1990	430T40#6	49	1.7	8	?	?
21	APR/1990	430T60#6	52	2.4	6	S	s
22	MAY/1990	200M5#12	73	25.1	20	S	s
23	MAY/1990	200M6#12	77	39.1	25	S	s
24	MAY/1990	260T70ACCESS	42	0.8	6	S	s
25A	MAY/1990	300M5#12W	77	39.1	20	S	s
25B	MAY/1990	300M5#12E	77	39.1	23	U	WEDGE
26	MAY/1990	360T20#4	58	4.7	10	S	s
27A	MAY/1990	430M6#6	78	43.7	20	S	POST PILLAR
27B	MAY/1990	430M6#6	78	43.7	39	P	30m INCLUDING POST PILLAR
28	MAY/1990	430T40#6	49	1.7	12	?	5m ADVANCE + SLASH RETREAT
29	MAY/1990	430T20#6	61	6.6	10	S	s
30	MAY/1990	430T60#6	52	2.4	6	?	STRUCTURE
31A	FEB/1990	360T40#3/4	58	4.7	15	U	WEDGE STRUCTURE - NO CABLE BOLTS
31B	FEB/1990	360T40#3/4	58	4.7	15	S	WEDGE STRUCTURE + CABLE BOLTS
32	FEB/1990	360T20#3	58	4.7	13	S	s
33	NOV/1989	230M3#10	87	118.8	22	S	FLAT JOINT
34	NOV/1989	230M3#10	87	118.8	19	S	s
35	NOV/1989	230M4#10	85	95.2	14	S	s
36	NOV/1989	330M3#10	77	39.1	20	S	s
37	NOV/1989	330M4#10E	73	25.1	17	S	FLAT JOINT
38A	NOV/1989	430M3#5	78	43.7	18	S	POST PILLAR BETWEEN PILLARS
38B	NOV/1989	430M3#5	78	43.7	28	S	28m INCLUDING POST PILLAR
39	JAN12/1990	230M3#11	87	118.8	15	S	
40	JAN12/1990	240M4#10	85	95.2	16	?	
41	JAN12/1990	330M4#10	73	25.1	17	?	FAULT/STRUCTURE

Table 6.1 Raw Data - Detour Lake Mine (continued)

Case No.	Date Recorded	Stope Location	RMR (%)	Q	Span (m)	CONDITION	STABILITY S" = STABLE, "?" = POTENTIALLY UNSTABLE "U" =UNSTABLE, "*"STABLE WITH SUPPORT
42	JAN12/1990	330M3#11	77	39.1	12	S	
43	JAN12/1990	430T5#5	67	12.9	2	S	
44	JAN12/1990	430t40#5	55	3.4	5	S	DRILLING CAUSE UNSTABLE
45	JAN12/1990	430T60#5	50	1.9	5	S	
46A	FEB13/1989	460M1#3	72	22.4	16	S	POST PILLAR SPANS IN MAFICS
46B	FEB13/1989	460M1#3	53	2.7	10	S	POST PILLAR SPANS IN TALC
47	FEB13/1989	230T40#5	48	1.6	8	S	
48	NOV 1/1988	230T40#4	48	1.6	20	U	COLLAPSE RMR FROM LIFT #5
49	FEB13/1990	460T60#3	35	0.4	13	U	FLAT STRUCTURE+WEDGE-COLLAPSE
50	JAN19/1989	360T40#3	58	4.7	15	U	WEDGE+FLAT-COLLAPSE
51	DEC12/1987	360M2#2	70	18.0	22	U	WEDGE+FLAT-COLLAPSE
52	FEB28/1989	360T60#2	38	0.5	14	U	FAULT+FLAT-MONITORED+SEQUENCE CHANGED
53	FEB28/1989	360T60#1	38	0.5	15	U	
54	OCT3/1989	460M2#4	76	35.0	22	U	
55	SEPT17/1988	360M2#4	65	10.3	12	?	
56	OCT/1987	360M1#1	69	16.1	25	S	
57	SEPT/1987	260M1#1	69	16.1	20	S	
58	JUNE/1990	200M5#13	64	9.2	16	S	
59	JUNE/1990	200M6#12	70	18.0	20	S	
60	JUNE/1990	300M5#12	77	39.1	23	S	
61	JUNE/1990	360T20#4	58	4.7	10	S	
62	JUNE/1990	430M3#5	78	43.7	20	S	STABLE WITH POST PILLAR
62B	JUNE/1990	430M3#5	78	43.7	39	*	STABLE INCLUDES POST PILLAR
63	JULY/1990	200M6#13	80	54.6	12	S	
64	JULY/1990	300M5#13	76	35.0	17	?	WEDGE FLAT FAULT -RMR
65	JULY/1990	360T40#10	48	1.6	7	S	
66A	JULY/1990	430M3#6	74	28.0	35	S	INCLUDES POST PILLAR STRUCT 45-60 DEG
66B	JULY/1990	430M3#6	74	28.0	15	S	BETWEEN PILLAR STRUCTURE 45-60 DEG
67	AUG/1990	200M5#12	80	54.6	6	S	
68	AUG/1990	200M6#13	80	54.6	12	S	
69	AUG/1990	300M5#13	76	35.0	17	?	FLAT STRUCTURE MODERATELY STABLE 4M SWELLEX
70	AUG/1990	300M6#13	79	48.9	18	S	
71	AUG/1990	360T40#10	40	0.6	7	S	
72A	AUG/1990	430M3#7	79	48.9	25	S	
72B	AUG/1990	430M3#7W	64	9.2	25	S	
73	AUG/1990	430M4#7	77	39.1	35	*	INCLUDES POST PILLAR
74	AUG/1990	430T5#7	67	12.9	9	S	
75	AUG/1990	460 LONGHOLE	67	12.9	25	U	
76	SEPT/1990	200M5#14	80	54.6	19	S	
77	SEPT/1990	300M6#13	79	48.9	18	S	
78	SEPT/1990	360T40#10	40	0.6	7	S	
79	SEPT/1990	430M4#7	79	48.9	25	S	
80	SEPT/1990	430M3#7	77	39.1	20	S	
81	SEP/90	430T60#7	60	5.9	5	S	
82	SEP/90	460LONGHO	77	39.1	25	U	*****-10 DUE TO FLAT STRUCTURE(DLM)
83	JULY/90	560T15#1	53	2.7	6	?	*******-10 DUE TO FLAT STRUCTURE
84	JULY/90	560TACCESS	35	0.4	6	U	HEADING STOPPED-FAULT
85	OCT/90	200M5#14	80	54.6	18	S	
86	OCT/90	200M6#14	80	54.6	20	S	

Table 6.1 Raw Data - Detour Lake Mine (continued)

Case No.	Date Recorded	Stope Location	RMR (%)	Q	Span (m)	COND-ITION	STABILITY S" = STABLE, "?" = POTENTIALLY UNSTABLE "U" =UNSTABLE, "*"STABLE WITH SUPPORT
87	OCT/90	300M5#14	73	25.1	25	S	
88	OCT/90	360T40#11]	55	3.4	7	S	
89	OCT/90	430M4#8	77	39.1	25	S	
90	OCT/90	430M3#7	77	39.1	35	S	
91	OCT/90	460LONGHO	77	39.1	25	U	
92	NOV/90	200M6#15	79	48.9	14	S	
93	NOV/90	300M6#14	78	43.7	26	S	
94	NOV/90	360T60#11	38	0.5	5	S	
95	NOV/90	430M4#8	77	39.1	20	S	
96	NOV/90	460LONGHO	77	39.1	25	U	FLAT JOINTS
97	FEB/91	200M6#16	70	18.0	11	S	
98	FEB/91	200M5#16	82	68.2	14	S	
99	FEB/91	300M5#15	78	43.7	21	U	FLAT JOINTS
100	FEB/91	360T60#13	38	0.5	5	U	WEDGE>45
101A	FEB/91	430M4#8	78	43.7	25	S	POST PILLAR
101B	FEB/91	430M4#8	78	43.7	35	*	SUPPORT NO POST
102	FEB/91	430M3#8	78	43.7	20	?	DYKE 40-60
103	MARCH/91	200M6#16	70	18.0	11	S	
104	MARCH/91	300M5#15	78	43.7	21	U	FLAT JOINTS
105	MARCH/91	300M6#16	76	35.0	24	S	
106	MARCH/91	360T60#13	38	0.5	5	*	1.8m SWELLEX ON 1mX1m PATERN
107	MARCH/91	430M4#9	78	43.7	35	*	SUPPORT CABLE
108	MARCH/91	430M3#8	78	43.7	20	?	DYKE 40-60
109	APRIL/91	200M6#17	70	18.0	11	S	
110	APRIL/91	200M5#17	77	39.1	18	S	FLAT JOINTS
111	APRIL/91	300M5#16	78	43.7	16	U	FLAT JOINTS(STABLE ONLY WITH BIRDCAGE)
112	APRIL/91	300M6#16	76	35.0	24	S	BREAST FAILING NOT BACK
113	APRIL/91	360T60#14	38	0.5	5	U	STABLE ONLY IF SUPPORTED
114	APRIL/91	360T20#10	65	10.3	5	S	
115	APRIL/91	430M4#9	78	43.7	35	*	CABLE SUPPORT PILLAR
116	JULY/91	200M7#18	79	48.9	20	S	
117	JULY/91	200M8#19	77	39.1	16	S	
118	JULY/91	300M5#19	79	48.9	15	U	FLAT JOINTS/STABLE WITH SUPPORT
119	JULY/91	300M6#16	74	28.0	17	S	FLAT JOINTS
120A	JULY/91	360T40#17	45	1.1	7	U	STABLE WITH SUPPORT ONLY
120B	JULY/91	360T40#17	45	1.1	7	*	STABLE WITH SUPPORT ONLY
121	JULY/91	430M4#10	69	16.1	25	*	STABLE WITH SUPPORT ONLY
122	SEP/91	200M7#19	77	39.1	12	S	
123	SEP/91	200M8#19	83	76.2	15	S	
124	SEP/91	300M5#17	73	25.1	24	U	FLAT JOINTS
125	SEP/91	300M5#17	73	25.1	24	*	FLAT JOINTS/STABLE WITH SUPPORT
126	SEP/91	360T40#18	43	0.9	7	U	
127	SEP/91	360T40#18	43	0.9	7	*	STABLE WITH SUPPORT
128	SEP/91	360T20#14	56	3.8	5	U	STABLE WITH SUPPORT
129	SEP/91	360T20#14	56	3.8	5	*	STABLE WITH SUPPORT
130	SEP/91	430M4#10	69	16.1	25	*	STABLE WITH SUPPORT
131	OCT/91	200M7#19	78	43.7	18	S	
132	OCT/91	200M8#20	79	48.9	17	S	
133	OCT/91	300M5#18	75	31.3	18	U	FLAT JOINTS

Table 6.1 Raw Data - Detour Lake Mine (continued)

Case No.	Date Recorded	Stope Location	RMR (%)	Q	Span (m)	COND-ITION	STABILITY S" = STABLE, "?" = POTENTIALLY UNSTABLE "U" =UNSTABLE, "*"STABLE WITH SUPPORT
134	OCT/91	300M5#18	75	31.3	18	*	STABLE WITH SUPPORT/FLAT JOINTS
135	OCT/91	300M6#17	75	31.3	21	*	STABLE WITH BIRDCAGE
136	OCT/91	360T40#18	45	1.1	7	U	
137	OCT/91	360T40#18	45	1.1	7	*	STABLE WITH SWELLEX
138	OCT/91	430M3#10	80	54.6	20	S	CAVED IN FEB/92
139	OCT/91	430M4#11	81	61.0	23	*	STABLE WITH BIRDCAGE
140	NOV/91	200M7/M8#20	78	43.7	15	S	
141	NOV/91	300M5M6 #18	73	25.1	24	U	FLAT JOINTS
142	NOV/91	300M5M6 #18	73	25.1	24	*	FLAT JOINTS BIRCAGE IS STABLE
143	NOV/91	400M5#13	75	31.3	26	*	STRUCTURE STABLE WITH CABLE
144	NOV/91	430M4#11	79	48.9	24	*	CABLE PILLAR
145	DEC/91	200M7/M8#20	78	43.7	15	S	
146	DEC/91	300M5M6 #18	73	25.1	24	U	FLAT JOINTS
147	DEC/91	300M5M6 #18	73	25.1	24	*	FLAT JOINTS
148	DEC/91	400M5#13	75	31.3	26	?	CAVED JAN/92 WAS MOVING
149	DEC/91	560M2#2	54	3.0	10	*	STABLE WITH SWELLEX/TALC
150	DEC/91	660M2#2	75	31.3	5	S	
151	JAN/92	300M5M6 #18	73	25.1	24	*	FLAT JOINTS/STABLE WITH CABLES
152	MARCH/92	300M5M6 #18	73	25.1	24	*	FLAT JOINTS/STABLE WITH CABLES
153	MARCH/92	560M1#2	81	61.0	9	S	
154	MARCH/92	560M2#2	54	3.0	10	U	
155	MARCH/92	560M2#2	54	3.0	10	*	STABLE WITH SWELLEX
156	MARCH/92	575SLR	70	18.0	5	S	
157	MARCH/92	590SLR	72	22.4	5	S	

Table 6.2 Corrected Data Detour Lake Mine

Case No.	Date Recorded	Stope Location	RMR (%)	Q	Span (m)	COND-ITION	STABILITY S" = STABLE, "?" = POTENTIALLY UNSTABLE "U" =UNSTABLE, "*"STABLE WITH SUPPORT
1	FEB/1990	230M4#11	85	95.2	15	S	STABLE
2	FEB/1990	230M3#11	87	118.8	25	S	STABLE
3	FEB/1990	330M4#10	67	12.9	20	?	FLAT OPEN JOINTS(0-20 DEG) - SMALL GROUND FALL 25T
4	FEB/1990	330M3#11	77	39.1	16	S	STABLE
5A	FEB/1990	430M3#5	78	43.7	19	S	POST PILLAR OF 5m
5B	FEB/1990	430M3#5	78	43.7	28	P	28m INCLUDING POST PILLAR
6	FEB/1990	430T40#5	51	2.2	13	U	WEDGE 55 DEGREE+DYKE - COLLAPSE
7	FEB/1990	430T5#5	67	12.9	9	S	
8	FEB/1990	430T60#5	50	1.9	9	S	BACK STABLE, WALL UNSTABLE
9	MAR/1990	230M4#11	85	95.2	15	S	
10	MAR/1990	230M3#11	87	118.8	15	S	
11A	MAR/1990	330M4#10	73	25.1	20	S	
11B	MAR/1990	330M4#10	63	8.3	20	U	BREAST FLAT + RELEASE
12A	MAR/1990	430M3#5	78	43.7	19	S	POST PILLAR OF 5m
12B	MAR/1990	430M3#5	78	43.7	28	P	28m INCLUDING POST PILLAR
13	MAR/1990	430T60#5	50	1.9	9	S	BACK STABLE,WALL UNSTABLE
14	APR/1990	200m5#12	73	25.1	20	S	
15	APR/1990	230M4#11	85	95.2	15	S	
16	APR/1990	260T70ACCESS	42	0.8	5	S	
17A	APR/1990	330M4#11	73	25.1	20	S	BSCK
17B	APR/1990	330M4#11	63	25.1	20	U	BREAST FLAT + RELEASE
18	APR/1990	300M5#12	77	39.1	18	S	
19	APR/1990	430T5#6	62	7.4	12	S	STEEP STRUCTURE - 4m SUPERSWELLEX
20	APR/1990	430T40#6	49	1.7	8	?	?
21	APR/1990	430T60#6	52	2.4	6	S	
22	MAY/1990	200M5#12	73	25.1	20	S	
23	MAY/1990	200M6#12	77	39.1	25	S	
24	MAY/1990	260T70ACCESS	42	0.8	6	S	
25A	MAY/1990	300M5#12W	77	39.1	20	S	
25B	MAY/1990	300M5#12E	67	12.9	23	U	WEDGE
26	MAY/1990	360T20#4	58	4.7	10	S	
27A	MAY/1990	430M6#6	78	43.7	20	S	
27B	MAY/1990	430M6#6	78	43.7	39	P	30m INCLUDING POST PILLAR
28	MAY/1990	430T40#6	49	1.7	12	?	5m ADVANCE + SLASH RETREAT
29	MAY/1990	430T20#6	61	6.6	10	S	
30	MAY/1990	430T60#6	42	0.8	6	?	STRUCTURE
31A	FEB/1990	360T40#3/4	48	1.6	15	U	WEDGE STRUCTURE - NO CABLE BOLTS
31B	FEB/1990	360T40#3/4	48	1.6	15	S	WEDGE STRUCTURE + CABLE BOLTS
32	FEB/1990	360T20#3	58	4.7	13	S	
33	NOV/1989	230M3#10	77	39.1	22	S	FLAT JOINT
34	NOV/1989	230M3#10	87	118.8	19	S	
35	NOV/1989	230M4#10	85	95.2	14	S	
36	NOV/1989	330M3#10	77	39.1	20	S	
37	NOV/1989	330M4#10E	63	8.3	17	S	FLAT JOINT
38A	NOV/1989	430M3#5	78	43.7	18	S	
38B	NOV/1989	430M3#5	78	43.7	28	S	28m INCLUDING POST PILLAR
39	JAN12/90	230M3#11	87	118.8	15	S	
40	JAN12/90	240M4#10	85	95.2	16	?	STRESS

Table 6.2 (cont.) Corrected Data - Detour Lake Mine

Case No.	Date Recorded	Stope Location	RMR (%)	Q	Span (m)	COND-ITION	STABILITY S" = STABLE, "?" = POTENTIALLY UNSTABLE "U" =UNSTABLE, "*"STABLE WITH SUPPORT
41	JAN12/90	330M4#10	63	8.3	17	?	FAULT/STRUCTURE
42	JAN12/90	330M3#11	77	39.1	12	S	
43	JAN12/90	430T5#5	67	12.9	2	S	
44	JAN12/90	430T40#5	55	3.4	5	S	DRILLING CAUSE UNSTABLE
45	JAN12/90	430T60#5	50	1.9	5	S	
46A	FEB13/89	460M1#3	72	22.4	16	S	
46B	FEB13/89	460M1#3	53	2.7	10	S	
47	FEB13/89	230T40#5	48	1.6	8	S	
48	NOV1/88	230T40#4	48	1.6	20	U	COLLAPSE FROM LIFT#5
49	FEB13/90	460T60#3	25	0.1	13	U	FLAT STRUCTURE+WEDGE-COLLAPSE
50	JAN19/89	360T40#3	48	1.6	15	U	WEDGE+FLAT-COLLAPSE
51	DEC12/87	360M2#2	60	5.9	22	U	WEDGE+FLAT-COLLAPSE
52	FEB28/89	360T60#2	28	0.2	14	U	FLAT STRUCTURE-COLLAPSE
53	FEB28/89	360T60#1	28	0.2	15	U	WEDGE+FLAT-COLLAPSE
54	OCT3/89	460M2#4	66	11.5	22	U	WEDGE+FLAT-COLLAPSE
55	SEP17/88	360M2#4	55	3.4	12	?	FAULT+FLAT-MONITORED
56	OCT/87	360M1#1	69	16.1	25	S	
57	SEP/87	260M1#1	69	16.1	20	S	
58	JUNE/90	200M5#13	64	9.2	16	S	
59	JUNE/90	200M6#12	70	18.0	20	S	
60	JUNE/90	300M5#12	77	39.1	23	S	
61	JUNE/90	360T20#40	58	4.7	10	S	
62	JUNE/90	430M3#5	78	43.7	20	S	STABLE WITH POST PILLAR
62B	JUNE/90	430M3#5	78	43.7	39	*	STABLE INCLUDES POST PILLAR
63	JULY/90	200M6#13	80	54.6	12	S	
64	JULY/90	300M5#13	66	11.5	17	?	WEDGE FLAT FAULT RMR
65	JULY/90	360T40#10	48	1.6	7	S	
66A	JULY/90	430M3#6	64	9.2	35	*	INCLUDES POST PILLAR STRUCTURE 45-60 DEG
66B	JULY/90	430M3#6	64	9.2	15	S	BETWEEN PILLARS STRUCTURE 45-60 DEG
67	AUG/90	200M5#12	80	54.6	6	S	
68	AUG/90	200M6#13	80	54.6	12	S	
69	AUG/90	300M5#13	66	11.5	17	?	FLAT STRUCTURE MOD.STABLE
70	AUG/90	300M6#13	79	48.9	18	S	
71	AUG/90	360T40#10	40	0.6	7	S	
72A	AUG/90	430M3#7	79	48.9	25	S	
72B	AUG/90	430M3#7W	64	9.2	25	S	
73	AUG/90	430M4#7	77	39.1	35	*	INCLUDES POST PILLAR
74	AUG/90	430T5#7	67	12.9	9	S	
75	AUG/90	460LONGHOLE	67	12.9	25	U	
76	SEP/90	200M5#14	80	54.6	19	S	
77	SEP/90	300M6#13	79	48.9	18	S	
78	SEP/90	360T40#10	40	0.6	7	S	
79	SEP/90	430M4#7	79	48.9	25	S	
80	SEP/90	430M3#7	77	39.1	20	S	
81	SEP/90	430T60#7	60	5.9	5	S	
82	SEP/90	460LONGHO	67	12.9	25	U	*****-10 DUE TO FLAT STRUCTURE(DLM)
83	JULY/90	560T15#1	43	0.9	6	?	*******-10 DUE TO FLAT STRUCTURE
84	JULY/90	560TACCESS	25	0.1	6	U	HEADING STOPPED-FAULT
85	OCT/90	200M5#14	80	54.6	18	S	

Table 6.2 (cont.) Corrected Data - Detour Lake Mine

Case No.	Date Recorded	Stope Location	RMR (%)	Q	Span (m)	COND-ITION	STABILITY S" = STABLE, "?" = POTENTIALLY UNSTABLE "U" =UNSTABLE, "*"STABLE WITH SUPPORT
86	OCT/90	200M6#14	80	54.6	20	S	
87	OCT/90	300M5#14	73	25.1	25	S	
88	OCT/90	360T40#11]	55	3.4	7	S	
89	OCT/90	430M4#8	77	39.1	25	S	
90	OCT/90	430M3#7	77	39.1	35	S	
91	OCT/90	460LONGHO	77	39.1	25	U	
92	NOV/90	200M6#15	79	48.9	14	S	
93	NOV/90	300M6#14	78	43.7	26	S	
94	NOV/90	360T60#11	38	0.5	5	S	
95	NOV/90	430M4#8	77	39.1	20	S	
96	NOV/90	460LONGHO	67	12.9	25	U	FLAT JOINTS
97	FEB/91	200M6#16	70	18.0	11	S	
98	FEB/91	200M5#16	82	68.2	14	S	
99	FEB/91	300M5#15	68	14.4	21	U	FLAT JOINTS
100	FEB/91	360T60#13	38	0.5	5	U	WEDGE>45
101A	FEB/91	430M4#8	78	43.7	25	S	POST PILLAR
101B	FEB/91	430M4#8	78	43.7	35	*	SUPPORT NO POST
102	FEB/91	430M3#8	68	14.4	20	?	DYKE 40-60
103	MARCH/91	200M6#16	70	18.0	11	S	
104	MARCH/91	300M5#15	68	14.4	21	U	FLAT JOINTS
105	MARCH/91	300M6#16	76	35.0	24	S	
106	MARCH/91	360T60#13	38	0.5	5	*	1.8m SWELLEX ON 1mX1m PATERN
107	MARCH/91	430M4#9	78	43.7	35	*	SUPPORT CABLE
108	MARCH/91	430M3#8	68	14.4	20	?	DYKE 40-60
109	APRIL/91	200M6#17	70	18.0	11	S	
110	APRIL/91	200M5#17	67	12.9	18	S	FLAT JOINTS
111	APRIL/91	300M5#16	68	14.4	16	U	FLAT JOINTS(STABLE ONLY WITH BIRDCAGE)
112	APRIL/91	300M6#16	76	35.0	24	S	BREAST FAILING NOT BACK
113	APRIL/91	360T60#14	38	0.5	5	U	STABLE ONLY IF SUPPORTED
114	APRIL/91	360T60#14	38	0.5	5	*	STABLE ONLY IF SUPPORTED
114	APRIL/91	360T20#10	65	10.3	5	S	
115	APRIL/91	430M4#9	78	43.7	35	*	CABLE SUPPORT PILLAR
116	JULY/91	200M7#18	79	48.9	20	S	
117	JULY/91	200M8#19	77	39.1	16	S	
118	JULY/91	300M5#19	69	16.1	15	U	FLAT JOINTS/STABLE WITH SUPPORT
119	JULY/91	300M6#16	64	9.2	17	S	FLAT JOINTS
120A	JULY/91	360T40#17	45	1.1	7	U	STABLE WITH SUPPORT ONLY
120B	JULY/91	360T40#17	45	1.1	7	*	STABLE WITH SUPPORT ONLY
121	JULY/91	430M4#10	69	16.1	25	*	STABLE WITH SUPPORT ONLY
122	SEP/91	200M7#19	77	39.1	12	S	
123	SEP/91	200M8#19	83	76.2	15	S	
124	SEP/91	300M5#17	63	8.3	24	U	FLAT JOINTS
125	SEP/91	300M5#17	63	8.3	24	*	FLAT JOINTS/STABLE WITH SUPPORT
126	SEP/91	360T40#18	43	0.9	7	U	
127	SEP/91	360T40#18	43	0.9	7	*	STABLE WITH SUPPORT
128	SEP/91	360T20#14	56	3.8	5	U	STABLE WITH SUPPORT
129	SEP/91	360T20#14	56	3.8	5	*	STABLE WITH SUPPORT
130	SEP/91	430M4#10	69	16.1	25	*	STABLE WITH SUPPORT
131	OCT/91	200M7#19	78	43.7	18	S	

Table 6.2 (cont.) Corrected Data - Detour Lake Mine

Case No.	Date Recorded	Stope Location	RMR (%)	Q	Span (m)	COND-ITION	STABILITY S" = STABLE, "?" = POTENTIALLY UNSTABLE "U" =UNSTABLE, "*"STABLE WITH SUPPORT
132	OCT/91	200M8#20	79	48.9	17	S	
133	OCT/91	300M5#18	65	10.3	18	U	FLAT JOINTS
134	OCT/91	300M5#18	65	10.3	18	*	STABLE WITH SUPPORT/FLAT JOINTS
135	OCT/91	300M6#17	75	31.3	21	*	STABLE WITH BIRDCAGE
136	OCT/91	360T40#18	45	1.1	7	U	
137	OCT/91	360T40#18	45	1.1	7	*	STABLE WITH SWELLEX
138	OCT/91	430M3#10	80	54.6	20	S	CAVED IN FEB/92
139	OCT/91	430M4#11	81	61.0	23	*	STABLE WITH BIRDCAGE
140	NOV/91	200M7/M8#20	78	43.7	15	S	
141	NOV/91	300M5M6 #18	63	8.3	24	U	FLAT JOINTS
142	NOV/91	300M5M6 #18	63	8.3	24	*	FLAT JOINTS BIRCAGE IS STABLE
143	NOV/91	400M5#13	75	31.3	26	*	STRUCTURE STABLE WITH CABLE
144	NOV/91	430M4#11	79	48.9	24	*	CABLE PILLAR
145	DEC/91	200M7/M8#20	78	43.7	15	S	
146	DEC/91	300M5M6 #18	63	8.3	24	U	FLAT JOINTS
147	DEC/91	300M5M6 #18	63	8.3	24	*	FLAT JOINTS
148	DEC/91	400M5#13	75	31.3	26	?	CAVED JAN/92 WAS MOVING
149	DEC/91	560M2#2	54	3.0	10	*	STABLE WITH SWELLEX/TALC
150	DEC/91	660M2#2	75	31.3	5	S	
151	JAN/92	300M5M6 #18	63	8.3	24	*	FLAT JOINTS/STABLE WITH CABLES
152	MARCH/92	300M5M6 #18	63	8.3	24	*	FLAT JOINTS/STABLE WITH CABLES
153	MARCH/92	560M1#2	81	61.0	9	S	
154	MARCH/92	560M2#2	54	3.0	10	U	
155	MARCH/92	560M2#2	54	3.0	10	*	STABLE WITH SWELLEX
156	MARCH/92	575SLR	70	18.0	5	S	
157	MARCH/92	590SLR	72	22.4	5	S	

Table 6.3(a) Statistical Summary of Raw Data

	Stable Cases		Potentially Unstable Cases		Unstable Cases	
	SPAN	RMR	SPAN	RMR	SPAN	RMR
No. of Cases	98	98	13	13	32	32
Minimum	2	38	6	49	5	35
Maximum	35	87	26	85	25	79
Range	33	49	20	36	20	44
Mean	15.388	71.296	15.154	68.154	16.469	61.375
Variance	45.003	144.664	36.474	165.308	48.386	259.145
Std. Dev.	6.708	12.028	6.039	12.857	6.956	16.098
Std. Error	0.678	1.215	1.675	3.566	1.23	2.846
C.V.	0.436	0.169	0.399	0.189	0.422	0.262
Median	15.5	77	17	75	17	68.5

Table 6.3(b) Statistical Summary of Corrected Data

	Stable Case		Potentially Unstable Cases		Unstable Cases	
	SPAN	RMR	SPAN	RMR	SPAN	RMR
No. of Cases	98	98	13	13	32	32
Minimum	2	38	6	42	5	25
Maximum	35	87	26	85	25	77
Range	33	49	20	43	20	52
Mean	15.388	70.684	15.154	61.231	16.469	54.5
Variance	45.003	147.332	36.474	164.026	48.386	215.032
Std. Dev.	6.708	12.138	6.039	12.807	6.956	14.664
Std. Error	0.678	1.226	1.675	3.552	1.23	2.592
C.V.	0.436	0.172	0.399	0.209	0.422	0.269
Median	15.5	77	17	66	17	61.5

Table 6.4 Mahalanobis Distance and Group Classification Probabilities

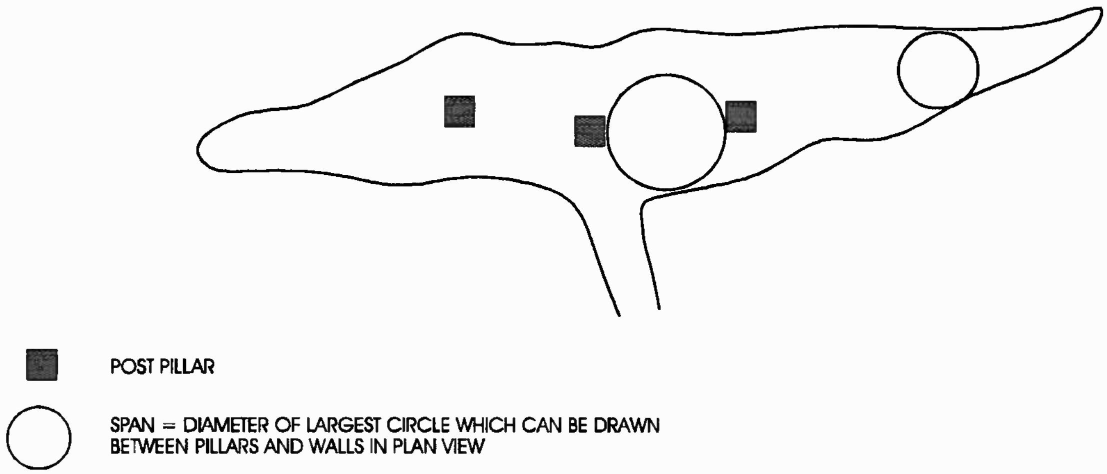
CASE No.	RMR (%)	SPAN (m)	DISTANCE 1	DISTANCE 2	DISTANCE 3	PROB. 1	PROB. 2	PROB. 3	ORIGINAL GROUP	PREDICTED GROUP	MINIMUM PROBABILITY
1	85	15	1.464	2.423	3.271	0.856	0.133	0.012	1	1	0.856
2	87	25	1.528	2.041	2.589	0.661	0.265	0.074	1	1	0.661
3	67	20	1.198	0.726	0.989	0.261	0.411	0.328	2	2	0.411
4	77	16	0.544	1.498	2.334	0.688	0.260	0.052	1	1	0.688
5A	78	19	0.622	1.357	2.104	0.619	0.299	0.082	1	1	0.619
6	50	5	1.763	1.525	1.959	0.315	0.466	0.219	1	2	0.466
6	51	13	1.765	0.838	0.524	0.118	0.394	0.488	3	3	0.488
7	67	9	1.010	1.620	2.444	0.653	0.293	0.055	1	1	0.653
8	50	9	1.642	1.001	1.208	0.193	0.450	0.358	1	2	0.450
9	85	15	1.464	2.423	3.271	0.856	0.133	0.012	1	1	0.856
10	87	15	1.666	2.626	3.473	0.879	0.112	0.008	1	1	0.879
11A	73	20	0.796	0.943	1.536	0.435	0.382	0.183	1	1	0.435
11B	63	20	1.543	0.834	0.682	0.169	0.392	0.440	3	3	0.440
12A	78	19	0.622	1.357	2.104	0.619	0.299	0.082	1	1	0.619
13	50	9	1.642	1.001	1.208	0.193	0.450	0.358	1	2	0.450
14	73	20	0.796	0.943	1.536	0.435	0.382	0.183	1	1	0.435
	85	15	1.464	2.423	3.271	0.856	0.133	0.012	1	1	0.856
16	42	5	2.271	1.680	1.724	0.139	0.447	0.414	1	2	0.447
17A	73	20	0.796	0.943	1.536	0.435	0.382	0.183	1	1	0.435
17B	63	20	1.543	0.834	0.682	0.169	0.392	0.440	3	3	0.440
18	77	18	0.503	1.320	2.103	0.625	0.297	0.078	1	1	0.625
19	62	12	0.698	0.660	1.464	0.406	0.417	0.177	1	2	0.417
20	49	8	1.716	1.134	1.355	0.199	0.455	0.346	2	2	0.455
21	52	6	1.592	1.384	1.871	0.335	0.458	0.207	1	2	0.458
22	73	20	0.796	0.943	1.536	0.435	0.382	0.183	1	1	0.435
23	77	25	1.564	1.529	1.781	0.363	0.384	0.253	1	2	0.384
24	42	6	2.267	1.605	1.569	0.119	0.428	0.453	1	3	0.453
25A	77	20	0.712	1.241	1.921	0.556	0.331	0.113	1	1	0.556
25B	67	23	1.765	1.233	1.086	0.171	0.379	0.450	3	3	0.450
26	58	10	1.028	0.829	1.497	0.363	0.437	0.201	1	2	0.437
27A	78	20	0.726	1.324	2.019	0.584	0.316	0.099	1	1	0.584
28	49	12	1.878	0.976	0.670	0.108	0.390	0.502	2	3	0.502
29	61	10	0.865	0.980	1.739	0.450	0.405	0.144	1	1	0.450
30	42	6	2.267	1.605	1.569	0.119	0.428	0.453	2	3	0.453
31A	48	15	2.280	1.320	0.527	0.055	0.307	0.638	3	3	0.638
32	58	13	1.079	0.329	0.934	0.260	0.440	0.300	1	2	0.440
33	77	22	1.029	1.279	1.805	0.480	0.360	0.160	1	1	0.480
34	87	19	1.306	2.216	3.004	0.815	0.164	0.021	1	1	0.815
35	85	14	1.596	2.551	3.406	0.870	0.120	0.009	1	1	0.870
36	77	20	0.712	1.241	1.921	0.556	0.331	0.113	1	1	0.556
37	63	17	1.038	0.280	0.799	0.257	0.423	0.320	1	2	0.423
38A	78	18	0.567	1.416	2.203	0.652	0.281	0.068	1	1	0.652
39	87	15	1.666	2.626	3.473	0.879	0.112	0.008	1	1	0.879
40	85	16	1.347	2.305	3.143	0.839	0.146	0.015	2	1	0.839
41	63	17	1.038	0.280	0.799	0.257	0.423	0.320	2	2	0.423
42	77	12	1.130	2.034	2.898	0.789	0.189	0.022	1	1	0.789
43	67	2	2.334	2.941	3.721	0.822	0.166	0.012	1	1	0.822
44	55	5	1.579	1.637	2.246	0.457	0.416	0.128	1	1	0.457
46A	72	16	0.114	0.995	1.829	0.555	0.340	0.105	1	1	0.555
46B	53	10	1.407	0.800	1.159	0.231	0.451	0.318	1	2	0.451
47	48	8	1.796	1.170	1.323	0.178	0.450	0.372	1	2	0.450
48	48	20	2.973	2.060	1.209	0.020	0.195	0.785	3	3	0.785
49	25	13	4.376	3.419	2.615	0.002	0.081	0.917	3	3	0.917
50	48	15	2.280	1.320	0.527	0.055	0.307	0.638	3	3	0.638

Table 6.4 (cont.) Mahalanobis Distance and Group Classification Probabilities

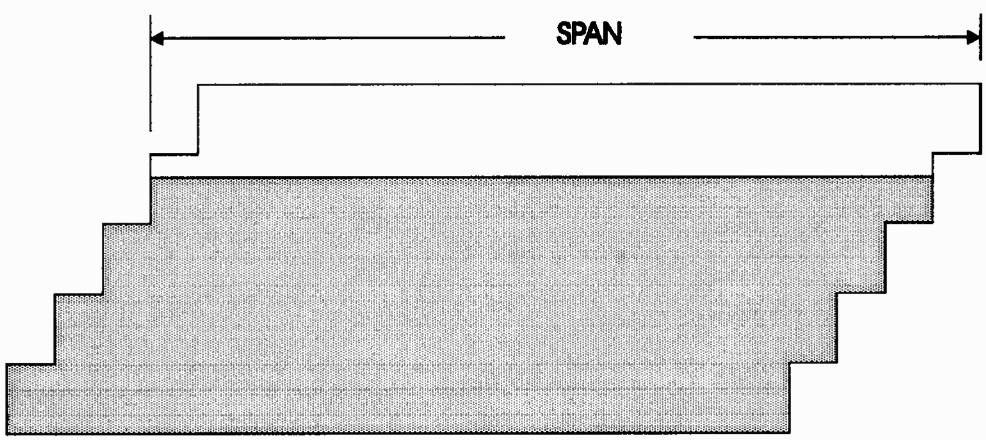
CASE No.	RMR (%)	SPAN (m)	DISTANCE 1	DISTANCE 2	DISTANCE 3	PROB. 1	PROB. 2	PROB. 3	ORIGINAL GROUP	PREDICTED GROUP	MINIMUM PROBABILITY
51	60	22	2.166	1.403	0.837	0.082	0.318	0.600	3	3	0.600
52	28	14	4.185	3.226	2.409	0.003	0.091	0.907	3	3	0.907
53	28	15	4.303	3.343	2.512	0.002	0.081	0.917	3	3	0.917
54	66	22	1.654	1.084	0.964	0.177	0.386	0.437	3	3	0.437
55	55	12	1.302	0.533	0.895	0.218	0.441	0.341	2	2	0.441
56	69	25	2.002	1.532	1.349	0.159	0.365	0.476	1	3	0.476
57	69	20	1.042	0.753	1.165	0.315	0.409	0.276	1	2	0.409
58	64	16	0.790	0.218	1.020	0.318	0.424	0.258	1	2	0.424
59	70	20	0.971	0.785	1.256	0.344	0.405	0.251	1	2	0.405
60	77	23	1.203	1.340	1.776	0.441	0.371	0.188	1	1	0.441
61	58	10	1.028	0.829	1.497	0.363	0.437	0.201	1	2	0.437
62	78	20	0.726	1.324	2.019	0.584	0.316	0.099	1	1	0.584
63	80	12	1.410	2.331	3.194	0.837	0.150	0.014	1	1	0.837
64	66	17	0.748	0.378	1.102	0.339	0.417	0.244	2	2	0.417
65	48	7	1.800	1.271	1.503	0.205	0.461	0.334	1	2	0.461
66B	64	15	0.662	0.300	1.161	0.354	0.421	0.225	1	2	0.421
67	80	6	2.477	3.309	4.160	0.914	0.082	0.003	1	1	0.914
68	80	12	1.410	2.331	3.194	0.837	0.150	0.014	1	1	0.837
69	66	17	0.748	0.378	1.102	0.339	0.417	0.244	2	2	0.417
70	79	18	0.641	1.513	2.304	0.677	0.265	0.059	1	1	0.677
71	40	7	2.452	1.683	1.454	0.077	0.379	0.543	1	3	0.543
72A	79	25	1.503	1.593	1.927	0.425	0.370	0.206	1	1	0.425
72B	64	25	2.377	1.737	1.281	0.082	0.307	0.611	1	3	0.611
73	77	35	3.451	3.085	2.778	0.080	0.266	0.654	1	3	0.654
74	67	9	1.010	1.620	2.444	0.653	0.293	0.055	1	1	0.653
75	67	25	2.146	1.598	1.299	0.124	0.345	0.532	3	3	0.532
76	80	19	0.733	1.543	2.303	0.671	0.267	0.062	1	1	0.671
77	79	18	0.641	1.513	2.304	0.677	0.265	0.059	1	1	0.677
78	40	7	2.452	1.683	1.454	0.077	0.379	0.543	1	3	0.543
79	79	25	1.503	1.593	1.927	0.425	0.370	0.206	1	1	0.425
80	77	20	0.712	1.241	1.921	0.556	0.331	0.113	1	1	0.556
81	60	5	1.545	1.884	2.600	0.598	0.335	0.067	1	1	0.598
82	67	25	2.146	1.598	1.299	0.124	0.345	0.532	3	3	0.532
83	43	6	2.188	1.555	1.573	0.134	0.439	0.427	2	2	0.439
84	25	6	3.776	2.898	2.322	0.010	0.180	0.810	3	3	0.810
85	80	18	0.721	1.610	2.404	0.701	0.249	0.050	1	1	0.701
86	80	20	0.792	1.498	2.215	0.639	0.285	0.075	1	1	0.639
87	73	25	1.750	1.476	1.527	0.250	0.389	0.361	1	2	0.389
88	55	7	1.375	1.274	1.860	0.385	0.440	0.176	1	2	0.440
89	77	25	1.564	1.529	1.781	0.363	0.384	0.253	1	2	0.384
91	77	25	1.564	1.529	1.781	0.363	0.384	0.253	3	2	0.384
92	79	14	0.996	1.946	2.803	0.781	0.193	0.025	1	1	0.781
93	78	26	1.712	1.669	1.878	0.355	0.381	0.263	1	2	0.381
94	38	5	2.583	1.887	1.741	0.084	0.398	0.518	1	3	0.518
95	77	20	0.712	1.241	1.921	0.556	0.331	0.113	1	1	0.556
96	67	25	2.146	1.598	1.299	0.124	0.345	0.532	3	3	0.532
97	70	11	0.771	1.525	2.378	0.666	0.281	0.053	1	1	0.666
98	82	14	1.296	2.248	3.104	0.831	0.154	0.016	1	1	0.831
99	68	21	1.308	0.877	1.061	0.254	0.406	0.340	3	2	0.406
100	38	5	2.583	1.887	1.741	0.084	0.398	0.518	3	3	0.518
101A	78	25	1.531	1.558	1.852	0.394	0.378	0.229	1	1	0.394
102	68	20	1.118	0.733	1.076	0.288	0.411	0.301	2	2	0.411
103	70	11	0.771	1.525	2.378	0.666	0.281	0.053	1	1	0.666
104	68	21	1.308	0.877	1.061	0.254	0.406	0.340	3	2	0.406
105	76	24	1.417	1.391	1.691	0.372	0.386	0.243	1	2	0.386
108	68	20	1.118	0.733	1.076	0.288	0.411	0.301	2	2	0.411

Table 6.4 (cont.) Mahalanobis Distance and Group Classification Probabilities

CASE No.	RMR (%)	SPAN (m)	DISTANCE 1	DISTANCE 2	DISTANCE 3	PROB. 1	PROB. 2	PROB. 3	ORIGINAL GROUP	PREDICTED GROUP	MINIMUM PROBABILITY
109	70	11	0.771	1.525	2.378	0.666	0.281	0.053	1	1	0.666
110	67	18	0.830	0.488	1.103	0.331	0.415	0.254	1	2	0.415
111	68	16	0.394	0.596	1.424	0.435	0.394	0.171	3	1	0.435
112	76	24	1.417	1.391	1.691	0.372	0.386	0.243	1	2	0.386
113	38	5	2.583	1.887	1.741	0.084	0.398	0.518	3	3	0.518
114	65	5	1.672	2.220	2.998	0.720	0.248	0.033	1	1	0.720
116	79	20	0.753	1.410	2.116	0.612	0.301	0.087	1	1	0.612
117	77	16	0.544	1.498	2.334	0.688	0.260	0.052	1	1	0.688
118	69	15	0.160	0.805	1.660	0.503	0.369	0.128	3	1	0.503
119	64	17	0.941	0.282	0.900	0.283	0.423	0.294	1	2	0.423
120A	45	7	2.031	1.385	1.432	0.146	0.441	0.413	3	2	0.441
122	77	12	1.130	2.034	2.898	0.789	0.189	0.022	1	1	0.789
123	83	15	1.262	2.221	3.070	0.828	0.156	0.017	1	1	0.828
124	63	24	2.272	1.601	1.130	0.086	0.315	0.599	3	3	0.599
126	43	7	2.195	1.490	1.419	0.115	0.420	0.466	3	3	0.466
128	56	5	1.559	1.677	2.312	0.486	0.401	0.113	3	1	0.486
131	78	18	0.567	1.416	2.203	0.652	0.281	0.068	1	1	0.652
132	79	17	0.667	1.597	2.415	0.706	0.246	0.048	1	1	0.706
133	65	18	1.011	0.428	0.906	0.276	0.419	0.305	3	2	0.419
136	45	7	2.031	1.385	1.432	0.146	0.441	0.413	3	2	0.441
138	80	20	0.792	1.498	2.215	0.639	0.285	0.075	1	1	0.639
140	78	15	0.757	1.715	2.565	0.738	0.226	0.037	1	1	0.738
141	63	24	2.272	1.601	1.130	0.086	0.315	0.599	3	3	0.599
145	78	15	0.757	1.715	2.565	0.738	0.226	0.037	1	1	0.738
146	63	24	2.272	1.601	1.130	0.086	0.315	0.599	3	3	0.599
148	75	26	1.837	1.627	1.697	0.269	0.387	0.345	2	2	0.387
150	75	5	2.269	3.036	3.871	0.879	0.115	0.006	1	1	0.879
153	81	9	2.020	2.899	3.759	0.891	0.103	0.006	1	1	0.891
154	54	10	1.325	0.780	1.217	0.255	0.453	0.292	3	2	0.453
156	70	5	1.928	2.611	3.425	0.813	0.172	0.015	1	1	0.813
157	72	5	2.056	2.778	3.601	0.842	0.147	0.011	1	1	0.842



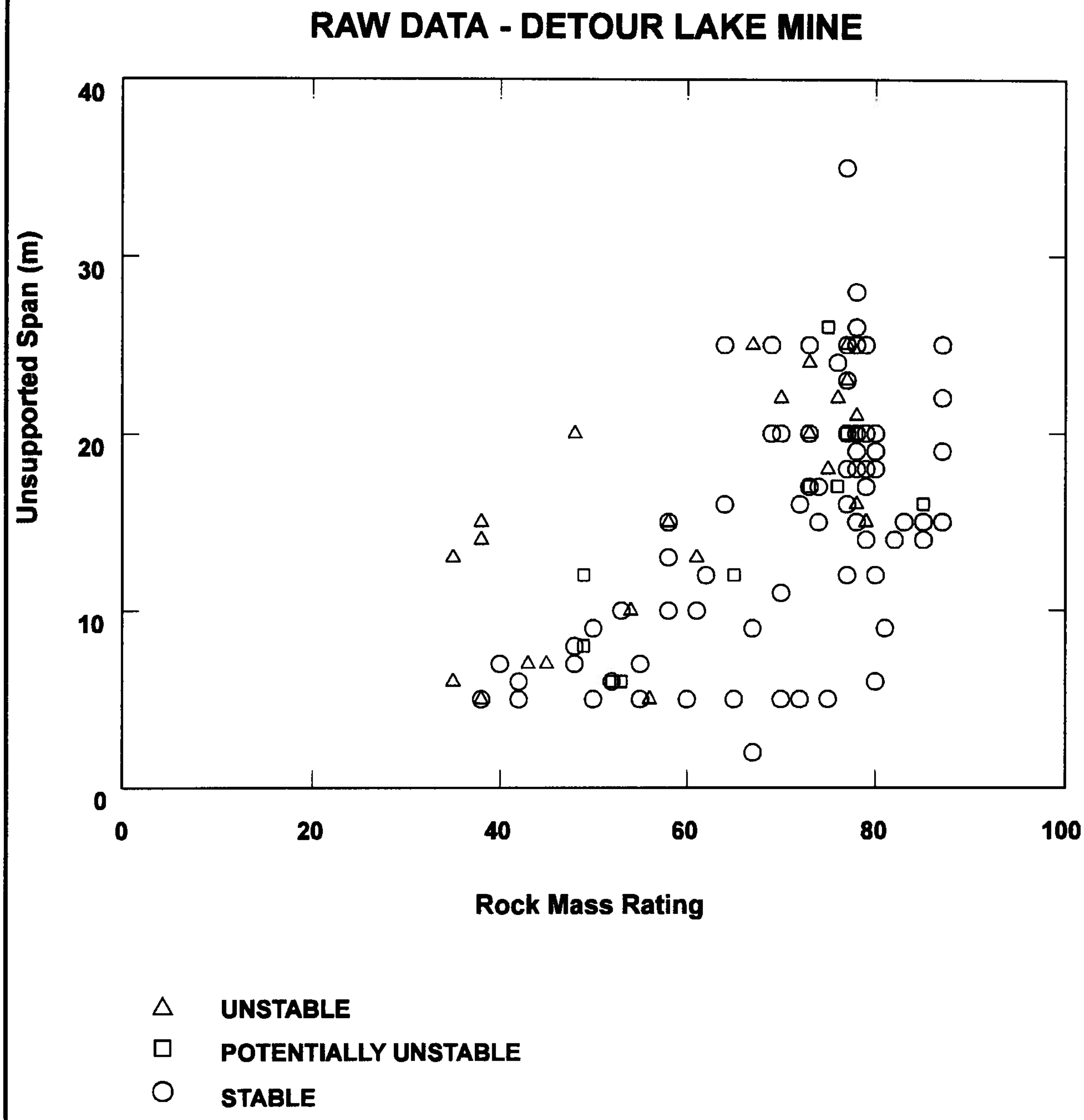
(a) Span Definition in Plan View



SPAN INCLUDES PORTION OF HANGINGWALL OVERHANG NOT TIGHT FILLED

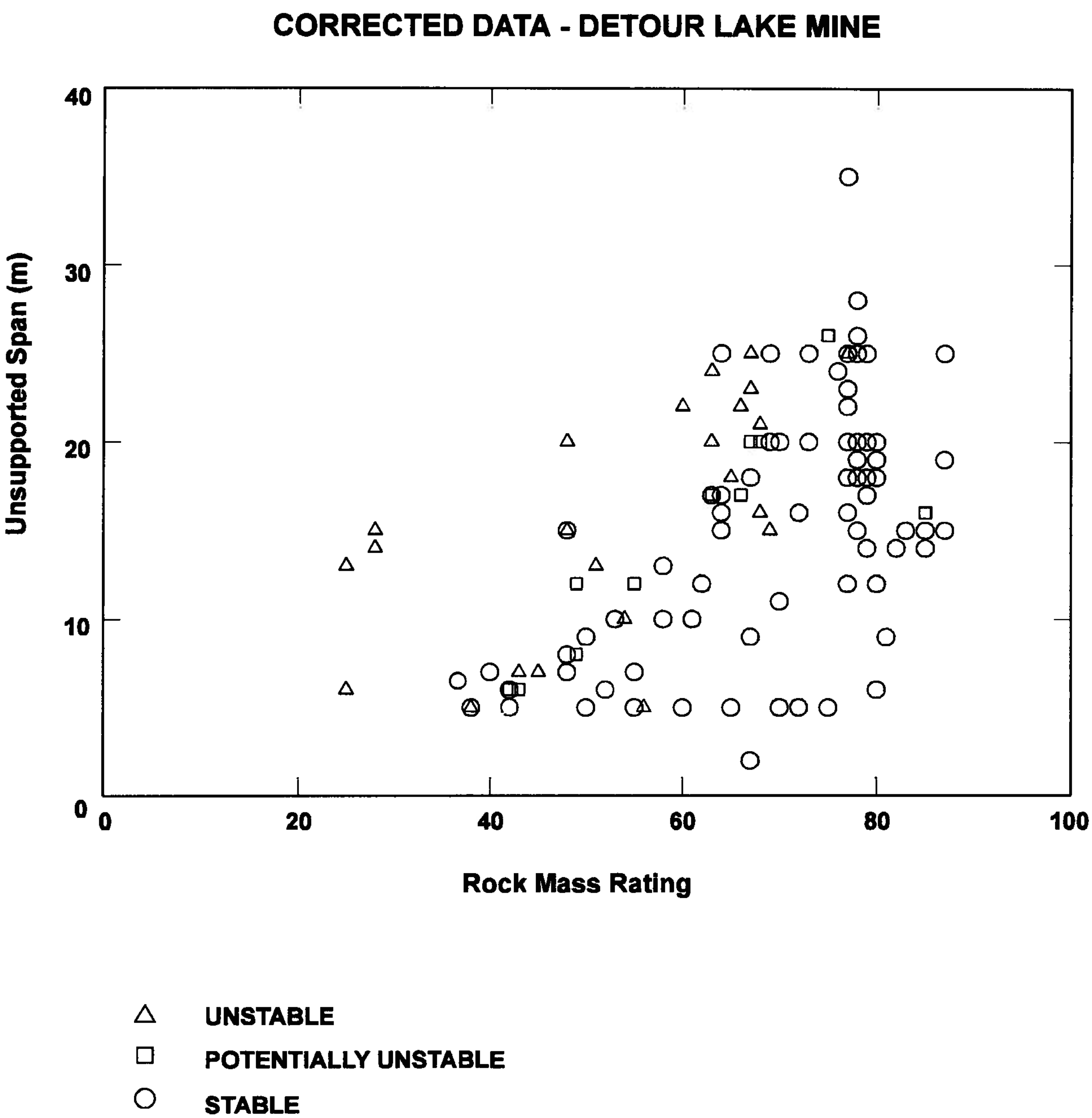
(b) Span Definition in Section View

SPAN DEFINITION | **FIGURE 6.1**



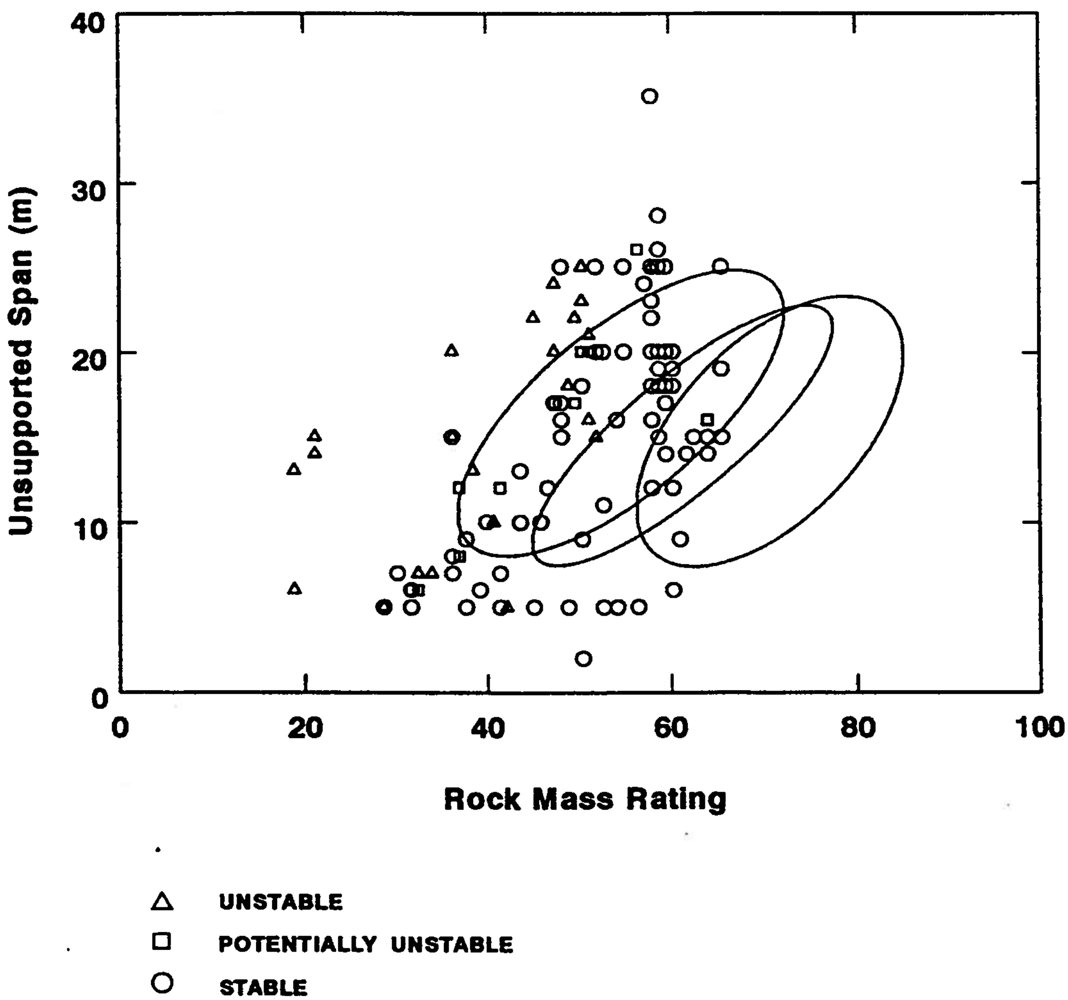
RAW DATABASE - DETOUR LAKE MINE

FIGURE 6.2

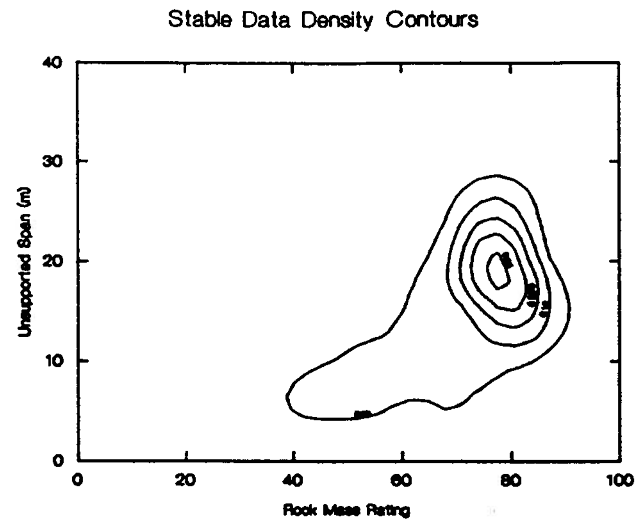


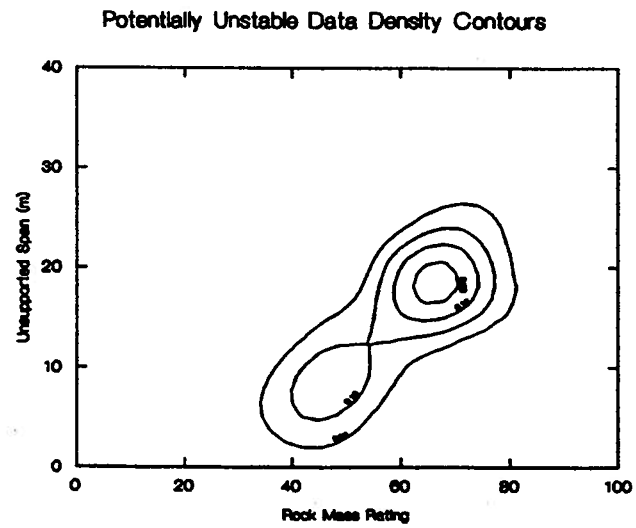
CORRECTED DATABASE - DETOUR LAKE MINE	FIGURE 6.3

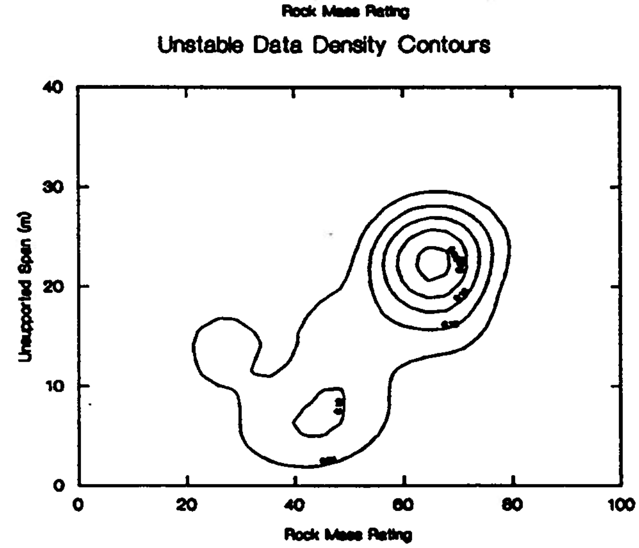
BIVARIATE ELLIPSES ON CORRECTED DATA



BIVARIATE ELLIPSES ON CORRECTED DATA	FIGURE 6.4

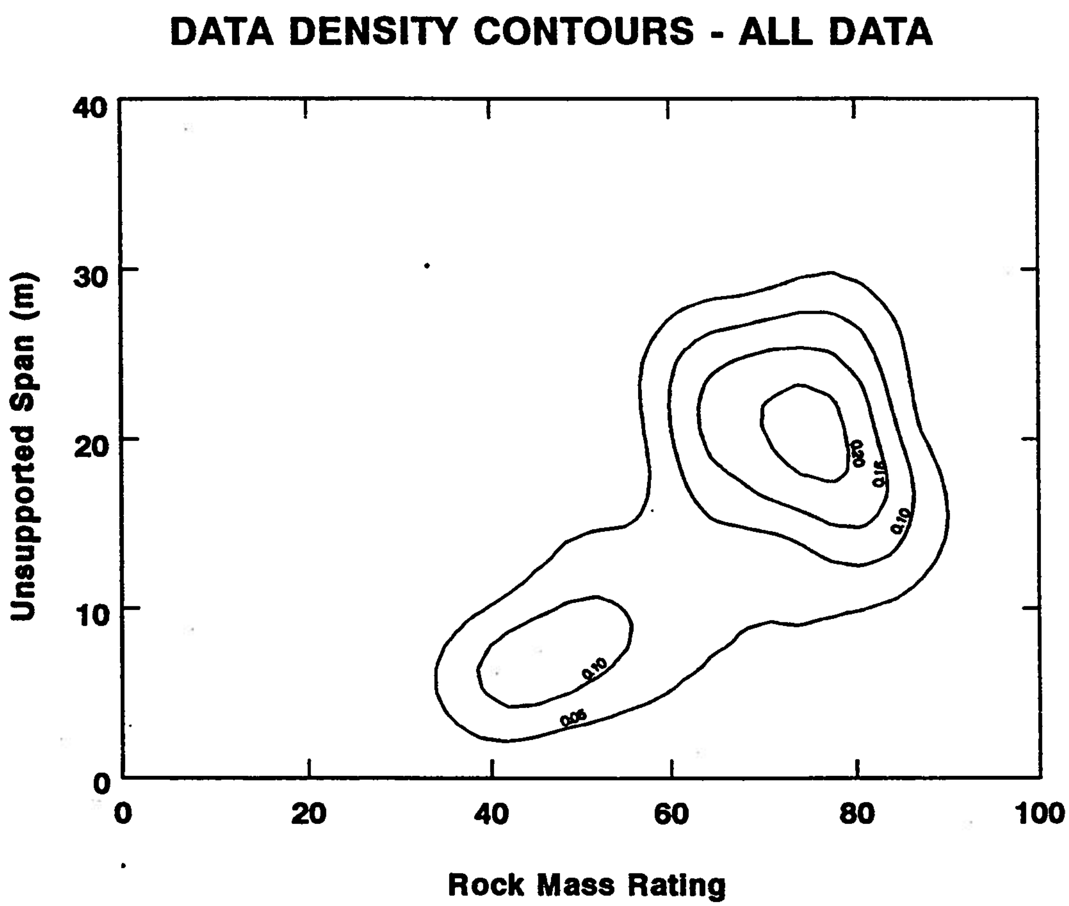




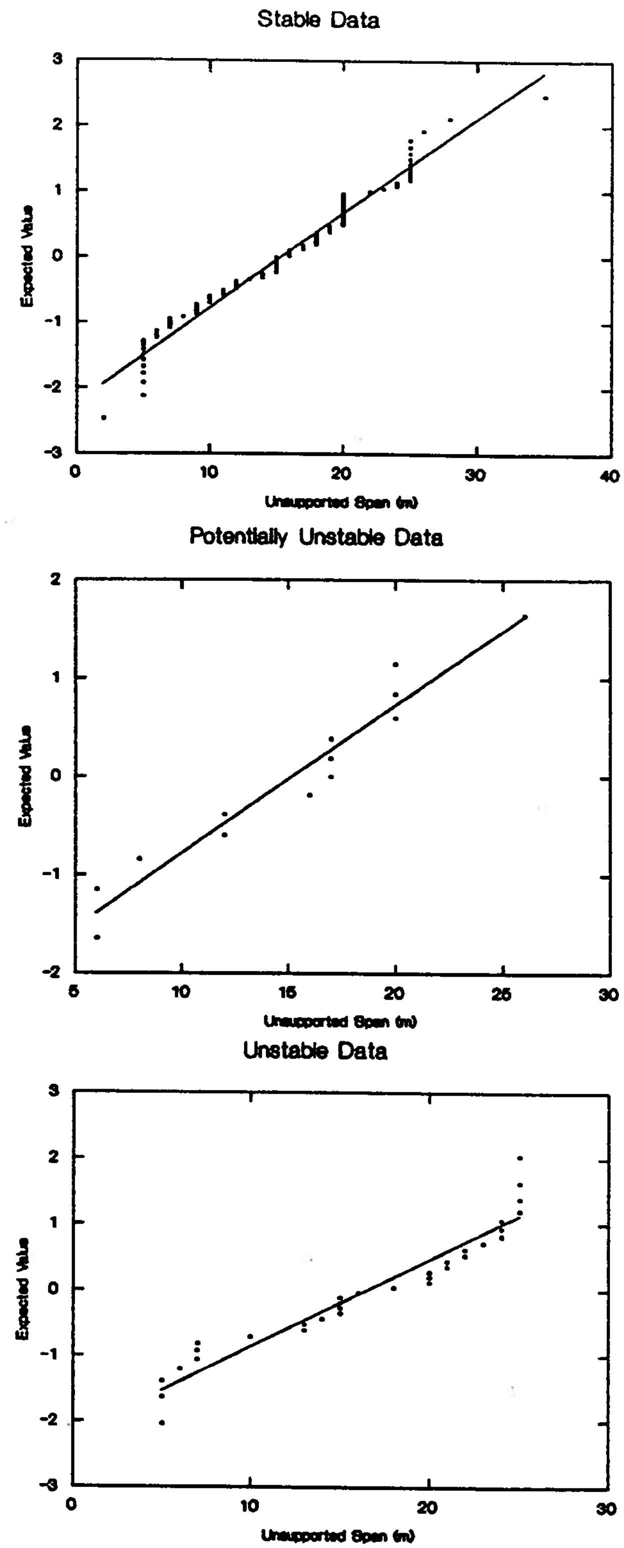


DATA DENSITY CONTOURS BY GROUP

FIGURE 6.5

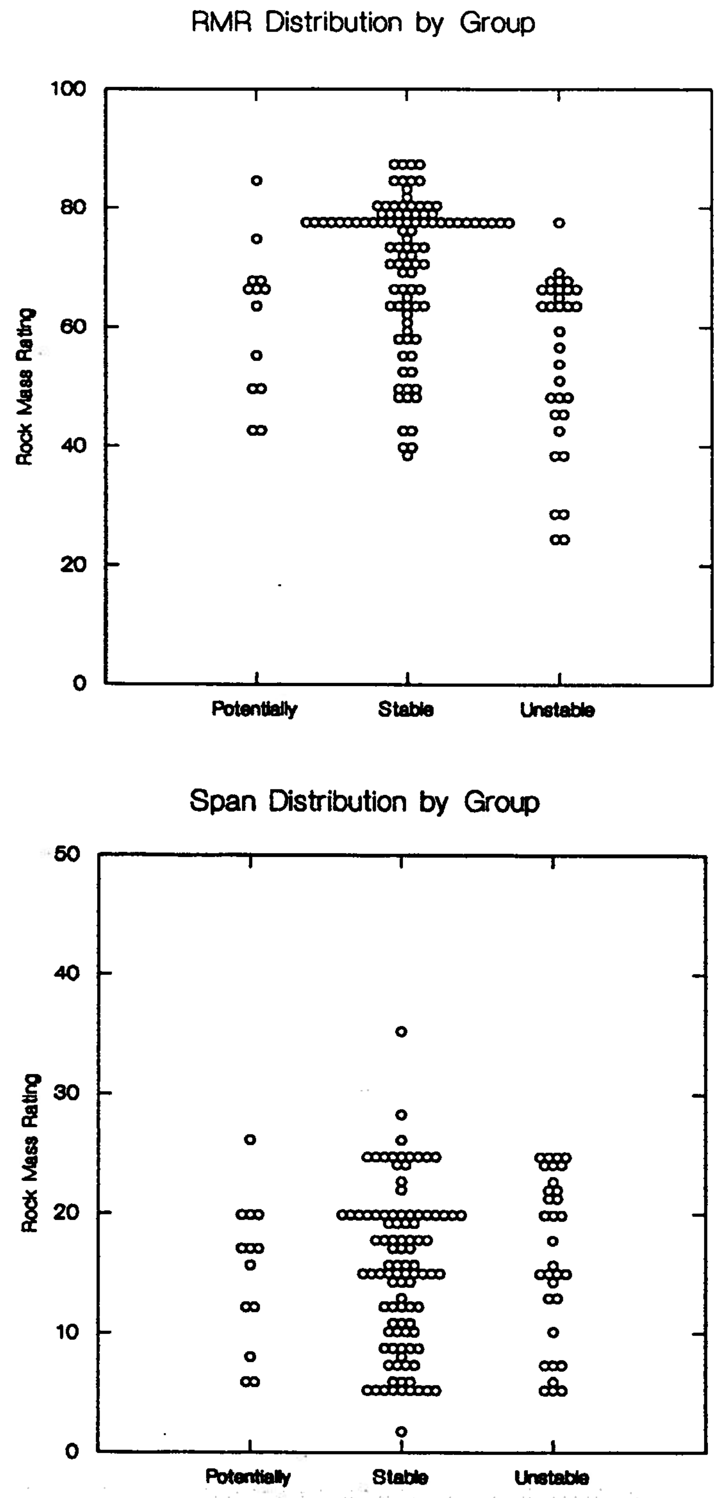


DATA DENSITY CONTOURS OF ALL DATA | FIGURE 6.6



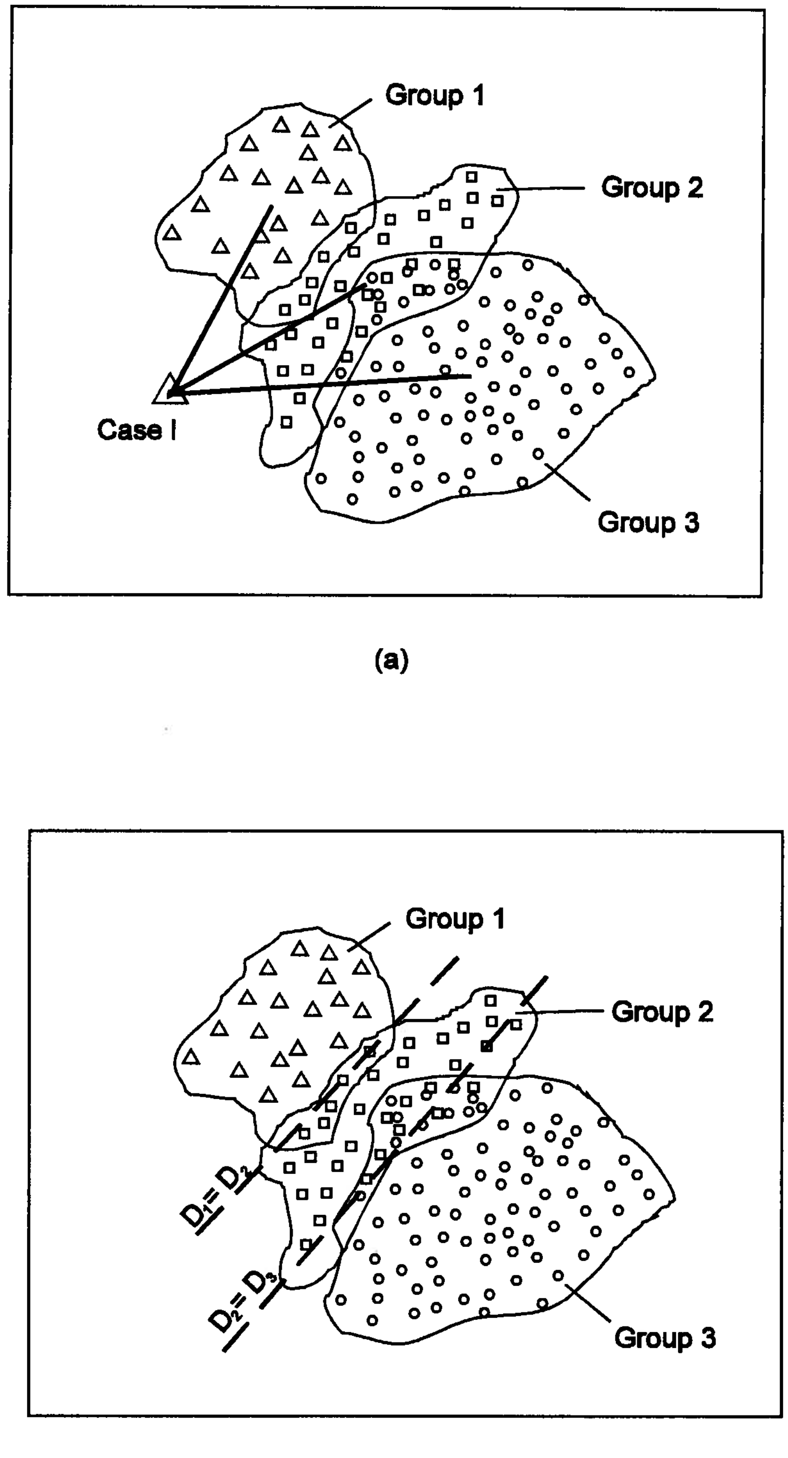
DATA DISTRIBUTION PROBABILITY PLOTS BY GROUP

FIGURE 6.7

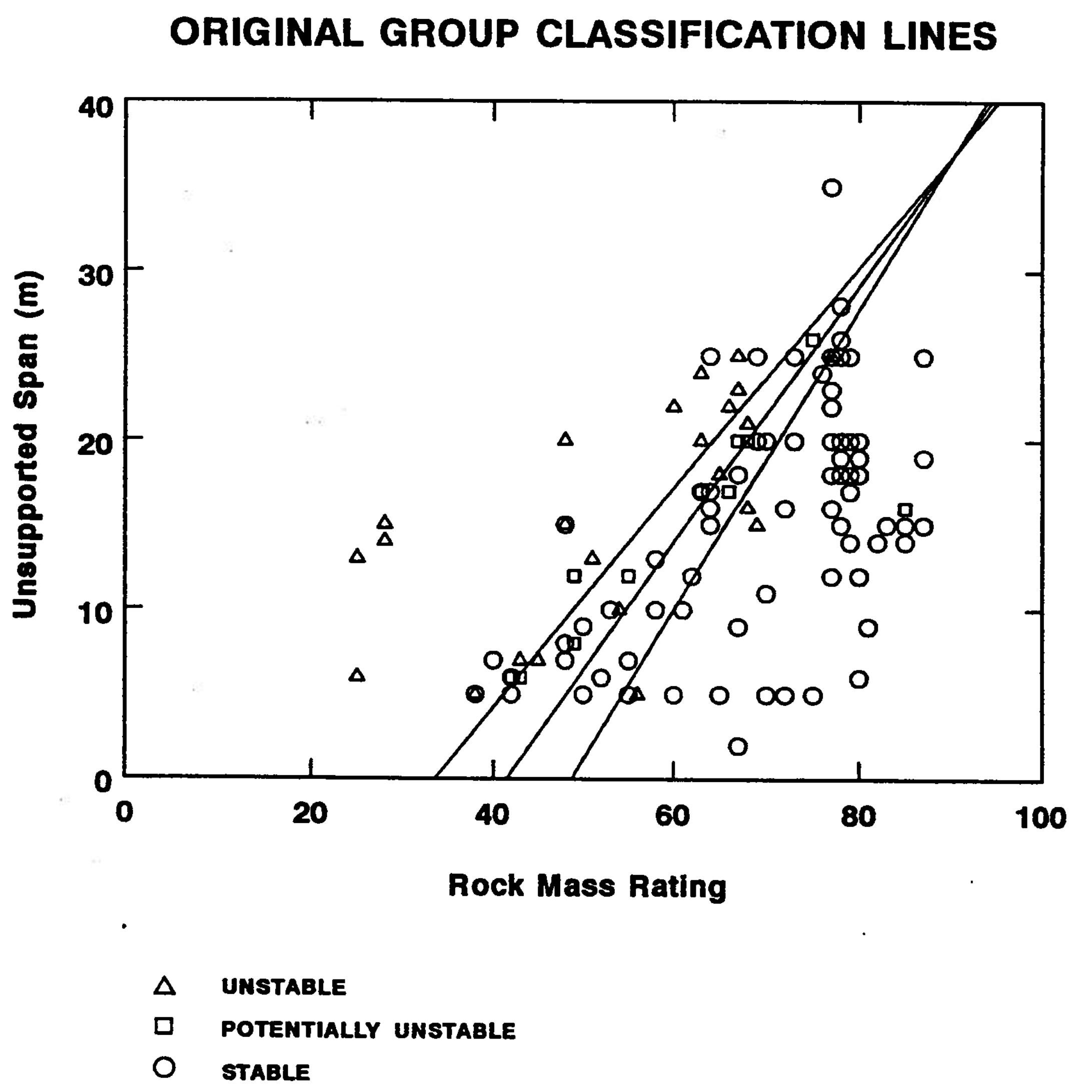


DATA DISTRIBUTION DOT PLOTS BY GROUP

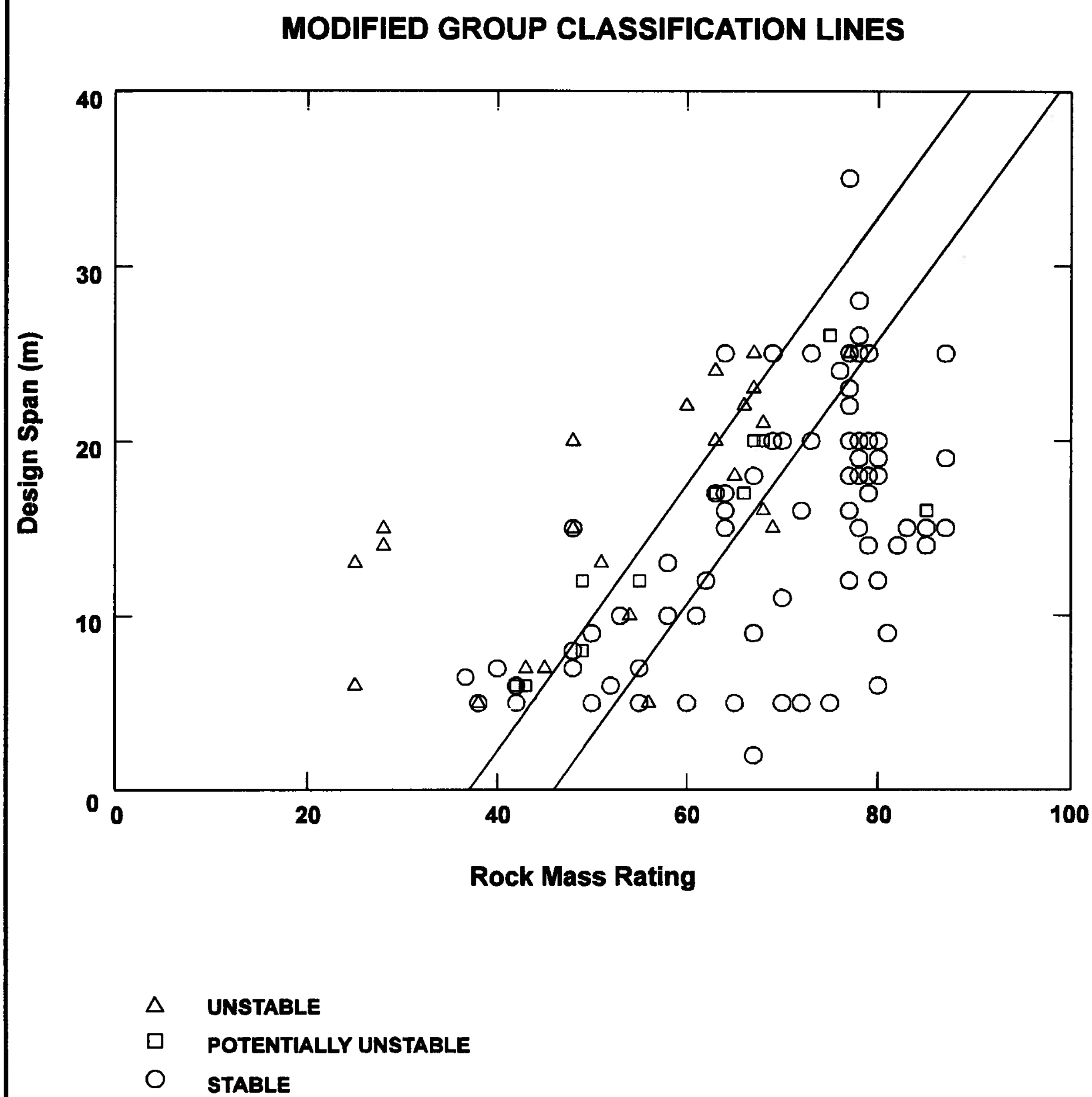
FIGURE 6.8



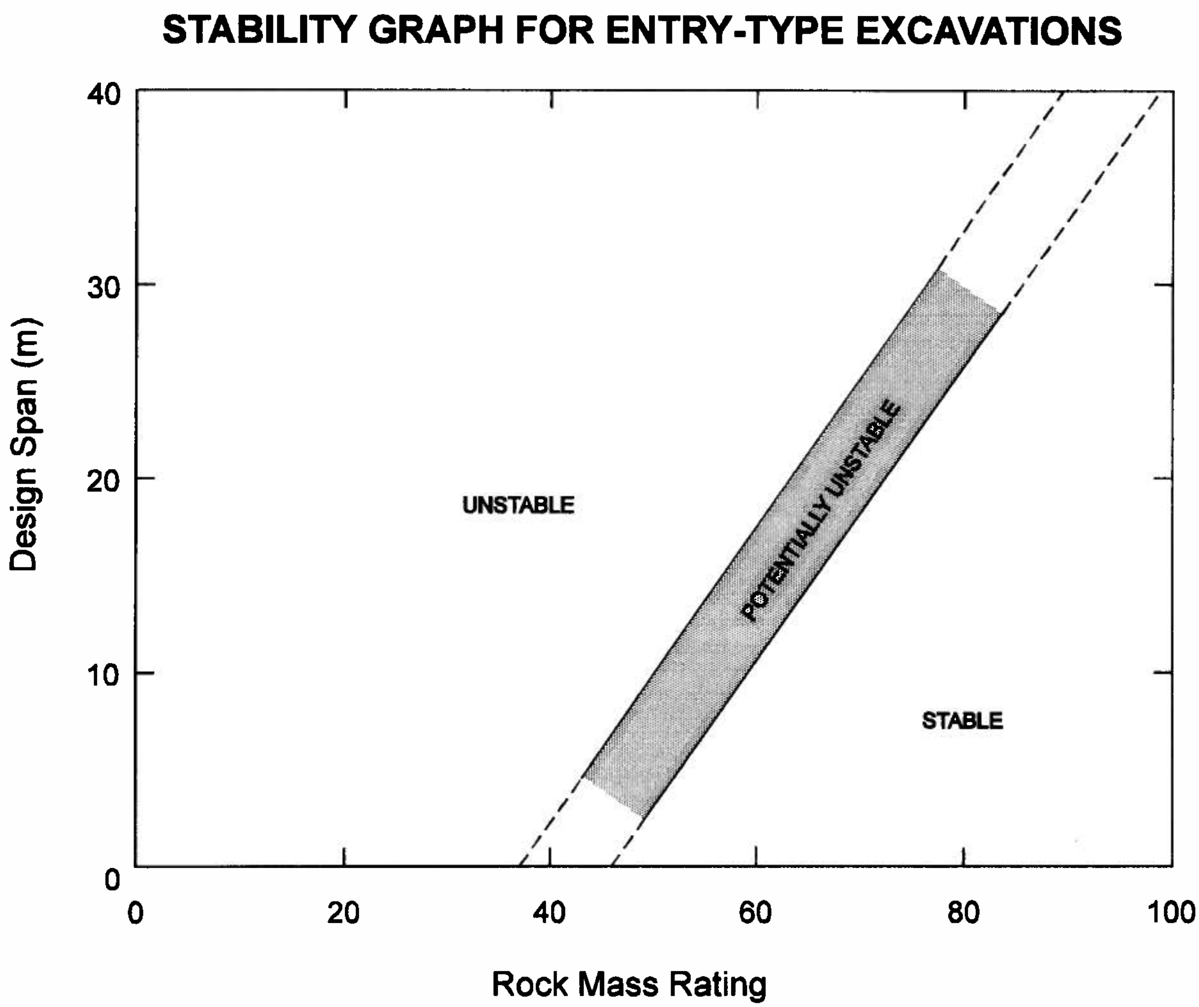
STATISTICAL CLASSIFICATION OF DATA INTO THREE POPULATIONS | FIGURE 6.9



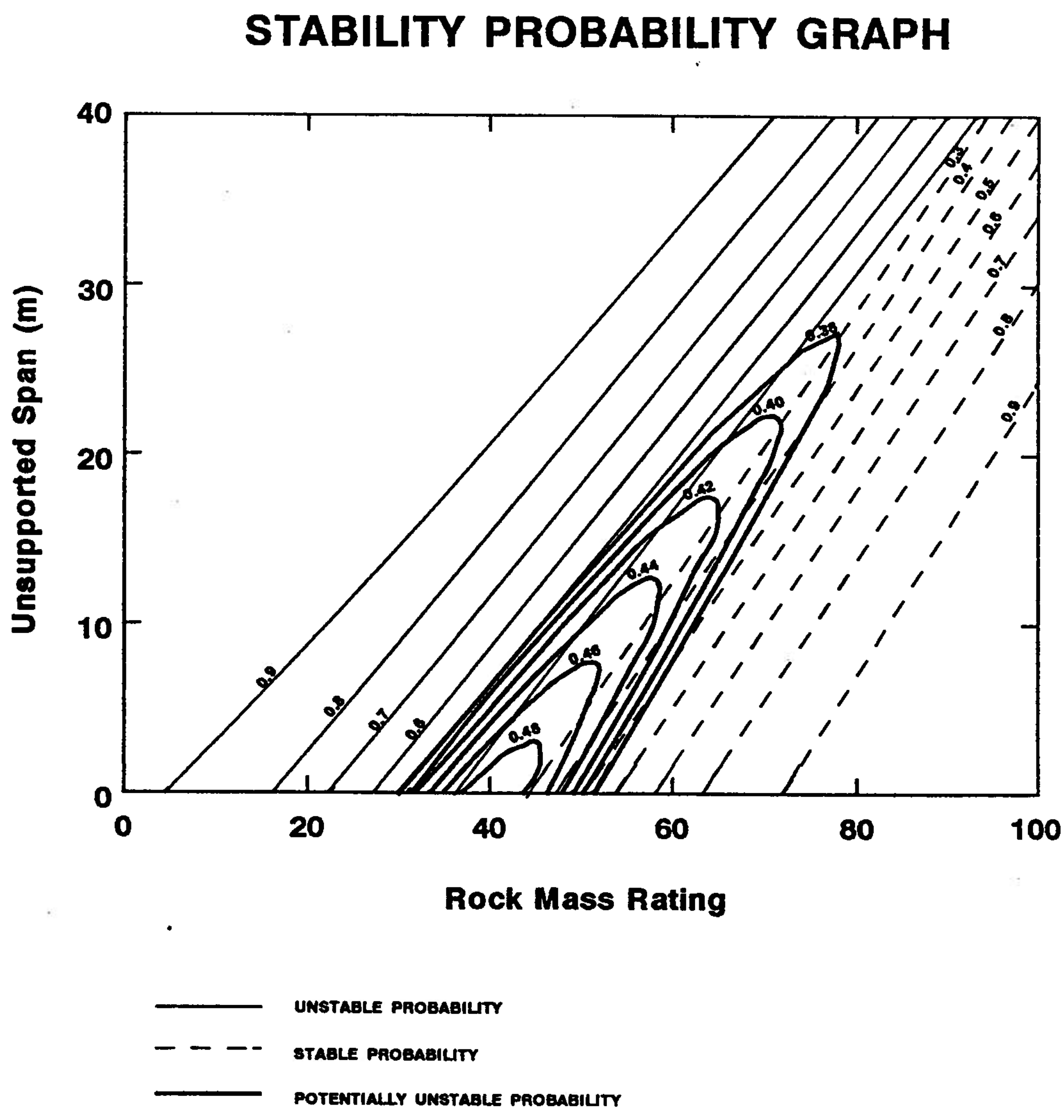
ORIGINAL GROUP CLASSIFICATION LINES	FIGURE 6.10



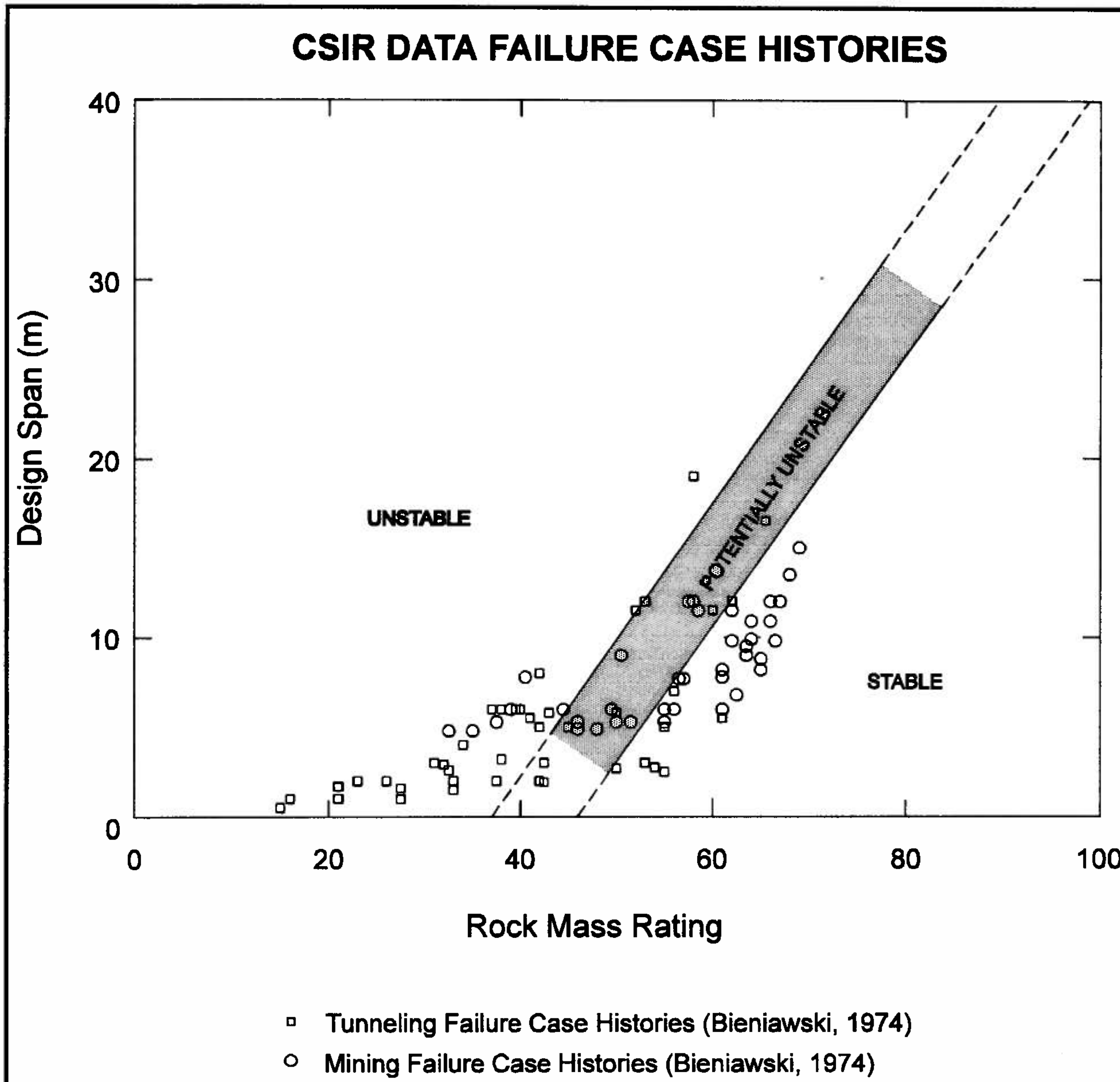
MODIFIED GROUP CLASSIFICATION LINES	FIGURE 6.11



STABILITY GRAPH FOR ENTRY-TYPE EXCAVATIONS	FIGURE 6.12

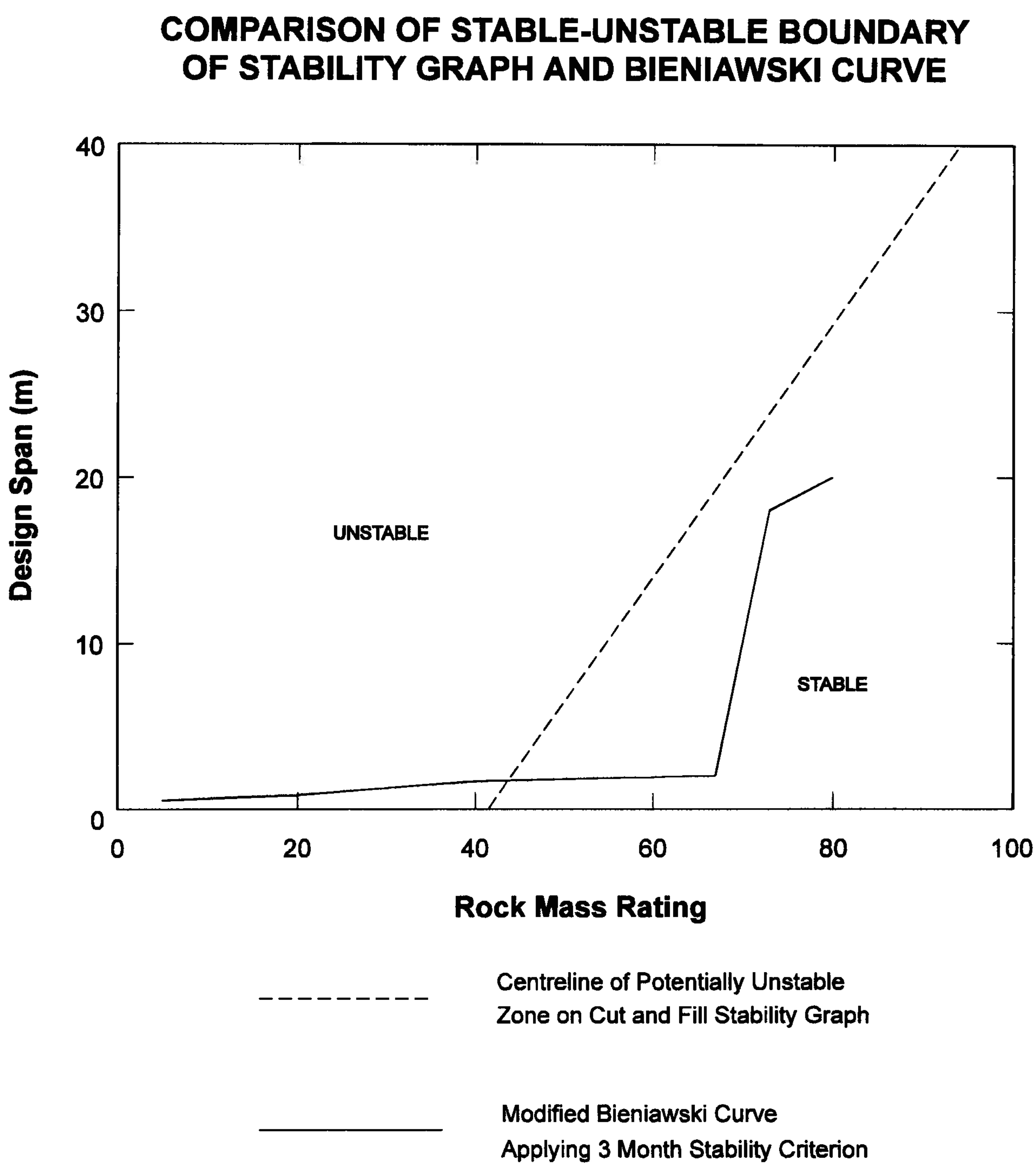


STABILITY PROBABILITY GRAPH	FIGURE 6.13

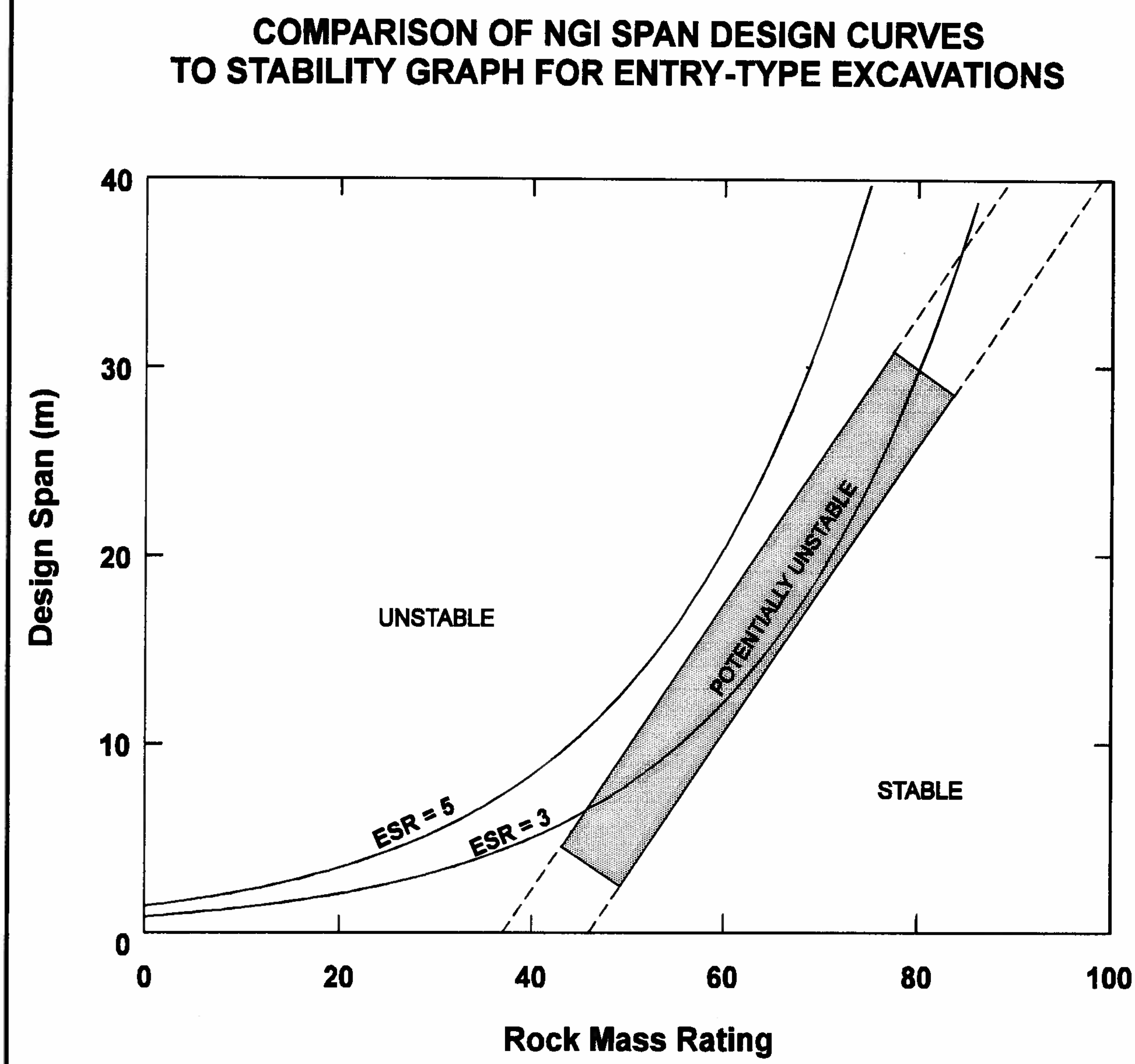


COMPARISON OF BIENIAWSKI DATA TO STABILITY GRAPH FOR ENTRY-TYPE EXCAVATIONS

FIGURE 6.14

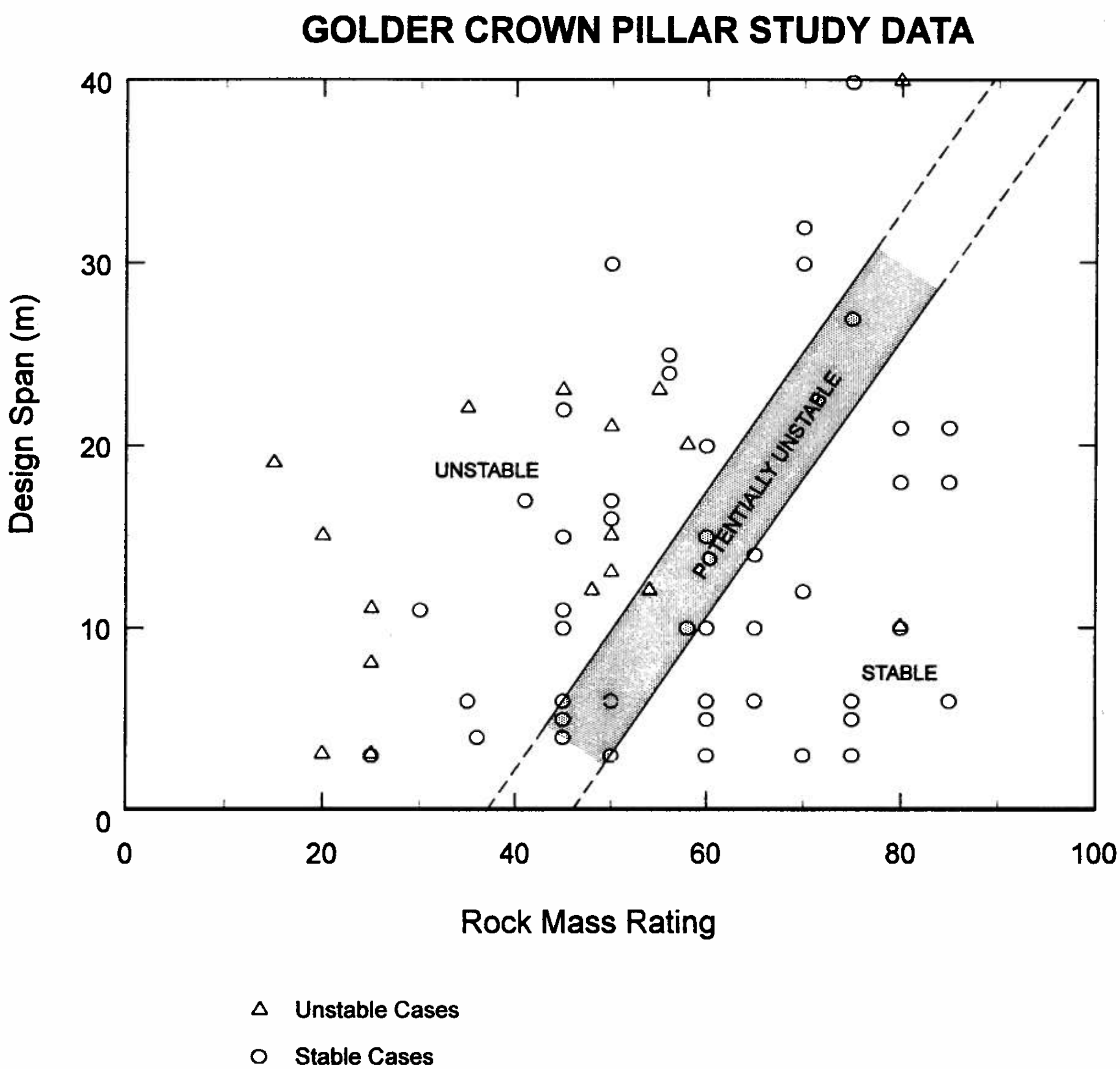


COMPARISON OF STABLE-UNSTABLE BOUNDARY OF STABILITY GRAPH TO MODIFIED BIENIAWSKI CURVE	FIGURE 6.15

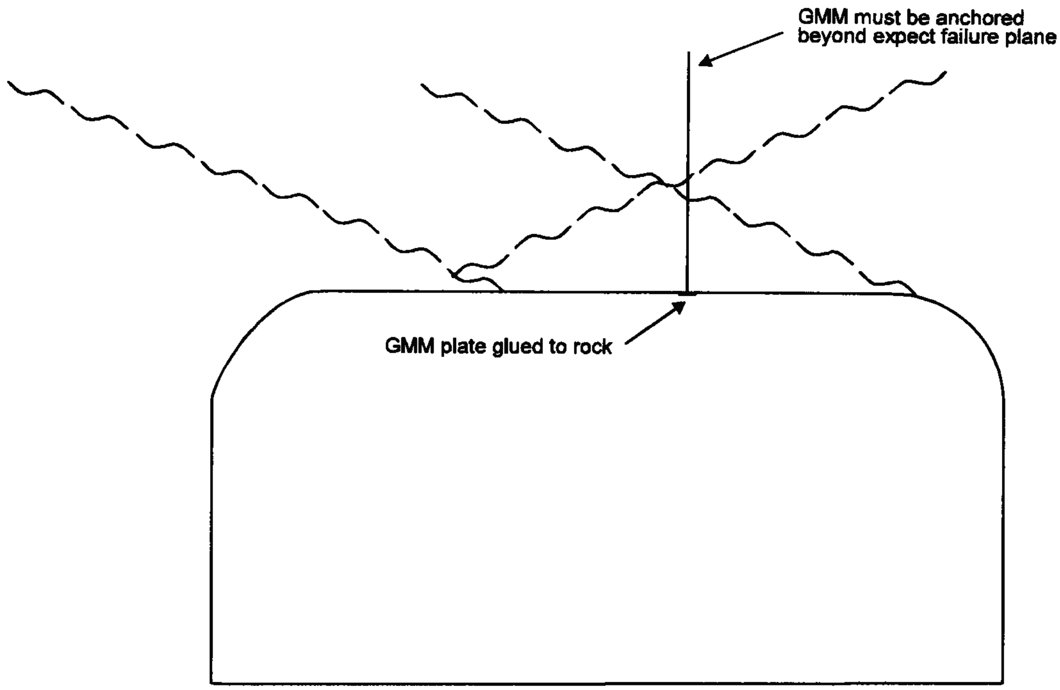


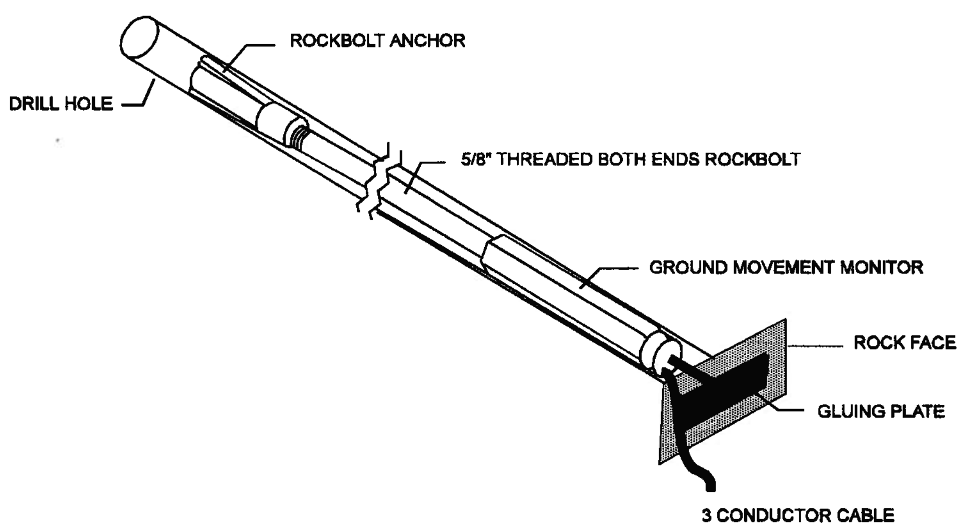
COMPARISON OF NGI SPAN DESIGN CURVES TO STABILITY GRAPH FOR ENTRY-TYPE EXCAVATIONS

FIGURE 6.16

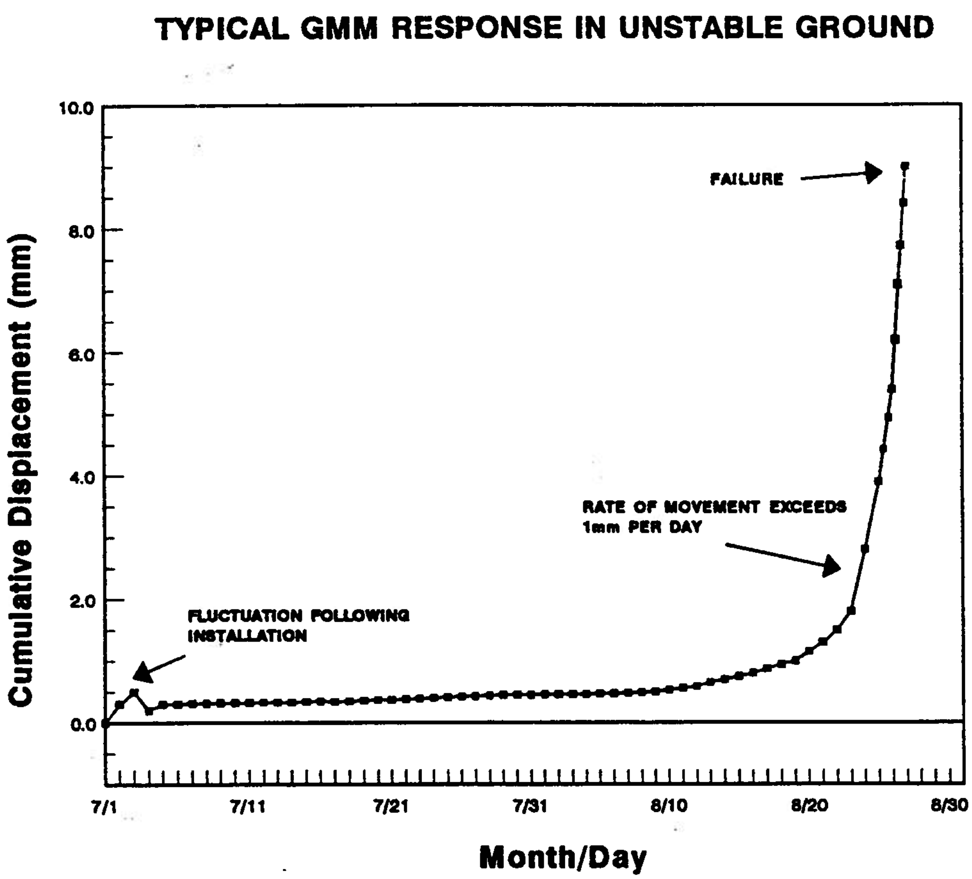


COMPARISON OF GOLDER CROWN PILLAR DATA TO STABILITY GRAPH FOR ENTRY-TYPE EXCAVATIONS	FIGURE 6.17

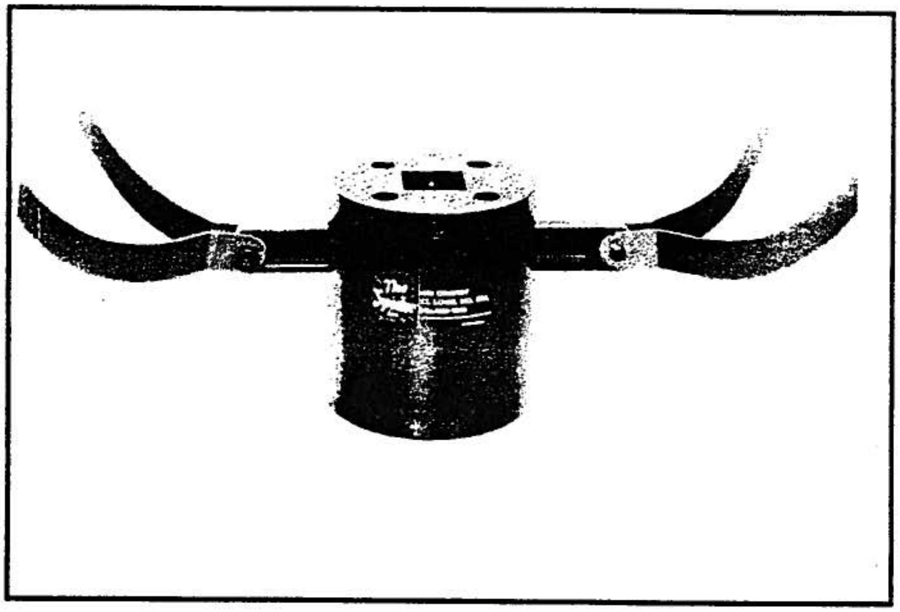




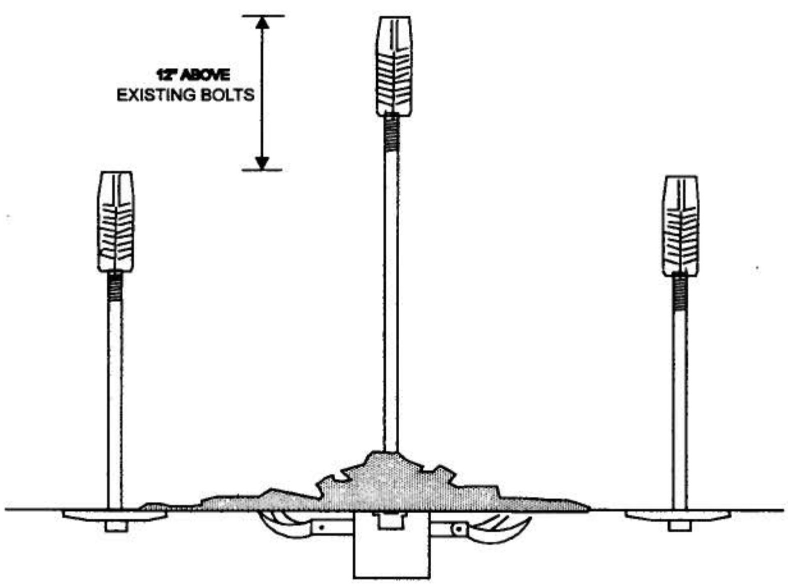
GROUND MOVEMENT MONITOR	FIGURE 6.18



TYPICAL GMM RESPONSE IN UNSTABLE GROUND	FIGURE 6.19



MINE SPIDER MONITORING DEVICE



MINE SPIDER GROUND MONITORING DEVICE	FIGURE 6.20

7. INFLUENCE OF GROUND SUPPORT ON SPAN DESIGN

7.1 INTRODUCTION

Support of cut and fill stopes is commonly provided by key-block support (rockbolts and friction stabilizers), cable bolting, post pillars and backfill. The Stability Graph for Entry-Type Excavations demonstrates the increased stable span which is achieved with improved rock quality. If artificial support is viewed as acting to reinforce the rock mass, support will have the effect of increasing the stable span.

Bawden *et al.*(1989) have estimated the effect of support on rock quality by studying the improvement in the Modified NGI -Q Rating that is achieved for varying concentrations of cable bolt support. Figure 7.1 shows the relationship between the Q'-supported and Q'-unsupported for Bolt Factors ranging from 1 to 8. The Bolt Factor is defined as the length of cable per square metre of face supported. Another relationship which attempts to quantify the improvement in rock quality achieved with support was developed using the Rock Mass Rating System (Milne *et al*, 1987). Figure 7.2 shows the expected increase in RMR due to a Bolt Factor of 1 for cable bolts. This work is an initial attempt to quantify the effect of support on rock quality. Although the approach appears promising, a more extensive database over a wide range of rock conditions and support concentrations is required before the validity of these relations can be established.

7.2 KEY-BLOCK SUPPORT

The objective of key-block support is to reinforce the rock mass and make it self-supporting by holding in place key blocks at the immediate surface of the excavation. These blocks in turn provide geometric support to the surrounding rock (Figure 7.3). Key block support is normally provided by short rock bolts, grouted dowels, or friction stabilizers. In hard, blocky ground, mechanical bolts will usually suffice whereas in softer, weaker ground, friction stabilizers such as Swellex or Split Sets may be used.

Most key-block support design procedures rely on empirical relationships between bolt length, joint spacing and bolt spacing (Lang, 1961) and (U.S. Army Corps of Engineers, 1980). Other methods are based on the reinforced arch concept whereby tensioned rockbolts are used to create a reinforced arch which supports the loose rock above (Stillborg, 1986). Both methods have been used for civil engineering excavations but the author is not aware of any mining operations which routinely employ them. In Canadian mines, a minimum pattern is usually established from past experience and miners are often instructed to decrease the spacing or increase the length of bolts to accommodate locally poor conditions. A typical pattern in Canadian mines is 1.8 m bolts on a 1.2 m square pattern. In Canada, it is becoming standard practice to install key-block support for all excavations immediately after the face is mucked out.

Consequently, the case histories contained in the database presented in Chapter 6 all had key-block support installed. Therefore, the ability of key-block support to increase the stable span cannot be assessed using the Detour Lake Mine database.

7.3 CABLE BOLTING

Commencing approximately twenty years ago, a number of cut and fill mines developed systems to pre-reinforce the rock mass prior to excavation (Fuller, 1980). The main purpose of pre-reinforcement is to control the amount of dilation into the opening immediately following excavation, thereby maintaining the integrity of the original rock mass. Fuller, (1980) has suggested that, as with key block support, the role of cable bolts is to reinforce the rock mass rather than to directly support it. The most common type of pre-reinforcement is a cement grouted steel cable bolt. The technique involves drilling holes of sufficient length to pre-support at least two lifts (Figure 7.4). The holes can be angled to follow the dip and plunge of the ore or they can be angled to intersect the hangingwall. Cable bolts are grouted along the full length of the hole such that support to the back is immediately available when subsequent lifts are mined. Any cables left hanging after a blast can be cut with grinding saws or explosive cable cutting charges. In cut and fill mining, pre-reinforcement has the added benefit of supporting the breast face. Many cut and fill operations experience problems with the breast face collapsing onto the fill making drilling and blasting difficult and slow (Ng, 1990). Pre-support of the breast face with cable bolts can reduce these difficulties, thereby improving productivity.

7.3.1 Cable Bolts

The most common type of cable used for cable bolting is 16 mm diameter, seven strand cable with an ultimate tensile strength of 255 kN (25 tonnes). The cable is flexible allowing it to be installed in excavations with low head room. The grout normally consists of Portland cement and water mixed at 0.30 to 0.45 water to cement ratios by weight. Lowering the water content contributes significantly to higher grout strength (UCS) as shown in Figure 7.5a. The higher grout UCS in turn contributes to higher cable pull-out strengths (Figure 7.5b).

Installation of cable bolts in up-holes involves first drilling the hole (usually 45-57 mm diameter) to the desired length. Installations of 20 metres or more are common. A 6 mm ID plastic breather tube is taped to the top of the cable which is then pushed up the hole. An expansion shell or spring steel clips are attached to the top of the cable to prevent it from sliding down the hole before the cable is grouted. A 19 mm ID PVC grout tube is inserted about 0.5 metres into the bottom of the hole which is then sealed with rags, wedges, or resin (Figure 7.6). The grout is then pumped up the hole through the grout tube. The hole fills from the bottom up and air is allowed to escape through the breather tube at the top. When air stops flowing from the breather tube, the hole is full. The ends of the grout tube and breather tube are tied off to prevent leakage and these tubes remain part of the grouted bolt system.

A preferred but less common method involves inserting the grout tube to the top of the hole and pumping a thick grout (<0.35 W:C), withdrawing the grout tube as the hole fills. This may not be possible with some grout pumps or if the hole is too long.

7.3.2 Cable Bolt Modifications and Accessories

The objective of the cable bolt support system is to mobilize as much of the cable strength as possible by transferring the rock load through the grout to the cable. Therefore, the capacity of the system is governed by three components (Figure 7.7):

- Rock to grout bond;
- Grout to cable bond; and
- Strength of the cable.

It has been demonstrated in laboratory tests and from failure case histories that failure of the system normally occurs at the cable-grout interface. A number of accessories have been added to cable bolts in an effort to increase the cable grout bond strength by providing a perpendicular load bearing surface on the cable. These include ferrules or buttons hydraulically pressed on to the cable at regular intervals (Figure 7.8). The buttons are normally 25 mm to 32 mm in diameter and 37 mm to 44 mm long. Barrel and wedge type cable grips have also been used for this purpose. The spacing of the buttons on the cable should be less than the average joint spacing.

Birdcage cables are a modification of the conventional steel cable. The birdcage cable bolt is manufactured by destranding the cable at specified intervals. A common node spacing is about 20 cm. Birdcaging can be done on all or part of a cable. The destranded parts of the cable form anchors along the bolt where failure must occur by crushing and pulling through the grout. Laboratory pull tests conducted on birdcaged cables show increases in pull out strengths of between 36 and 79 percent. Figure 7.9 shows the effect on pull-out strength of birdcaging, buttons, and using two cables per hole.

The practice of post tensioning and plating of cable bolts is becoming more wide spread partly due to the availability of lightweight, simple to use tensioning jacks. Plates are used to prevent unraveling of the rock around the cable at the collar of the hole. Steel plates or wooden head blocks can be used for this purpose.

7.3.3 Cable Bolt Support Design

In cases where structural features have delineated a potential wedge failure, the cable bolt pattern must be designed to support the expected load. The computer program *UNWEDGE* program (Hoek, 1991) described in Chapter 3 is useful for calculating the size and weight of such a wedge knowing the orientation of the structures. Design charts such as Figure 7.5 can be used to obtain the pull-out strength. Dividing the rock load by the cable pull-out strength yields the number of cables required to support the dead load of the wedge.

Where a specific hazard has not been identified, the support design should consider the bolts as acting to reinforce the rock mass. The most tried and proven technique for this type of cable bolt support design is the empirical design method developed by Potvin (1988).

Potvin has developed a cable bolt density chart based on 96 case histories of stopes using cable bolt support (Figure 7.10). On the x-axis of the chart is plotted the average block volume represented by the RQD/J_n divided by the surface hydraulic radius. On the y-axis is plotted the cable bolt density expressed in bolts per square metre. As expected, an increase in the block volume or a decrease in the hydraulic radius will decrease the cable bolt density. The graph is divided into four zones for purposes of cable bolt support design. The shaded area represents conditions where block size is so small that cable bolting would not be effective or where the cable bolt density is insufficient. The zone delineated between lines 1 and 2 is the least conservative design zone and is suitable for non-entry mining methods. The zone between lines 2 and 3 is considered to be a conservative design zone for non-entry mining methods as well as being suitable for entry type mining methods such as cut and fill. The zone to the right of line 3 is considered to be the most conservative. Excavations requiring long term support such as civil engineering projects should have support which plots in this zone.

7.4 POST PILLARS

A variation of conventional overhand cut and fill for wider orebodies is post pillar cut and fill developed at Falconbridge Ltd.'s Strathcona Mine in the early 1970's (Cleland *et al.*, 1973) A typical post pillar stope layout is given in Figure 7.11. Progressive mining of several lifts creates a pattern of tall pillars with height to width ratios exceeding 2:1. Since lifts are filled before the next one is started above, only the tops of the pillars are visible. Cases histories reported in the literature indicate this mining method has been used to depths of 600 metres and no limit to the overall stope span has been encountered.

Post pillars are designed to gradually yield below the level of the fill. Support is provided by the post-yield strength of the pillar. Figure 7.12 shows vertical stress measurements from a post pillar at King Island Scheelite Mine in Australia. The graph shows the stress to be decreasing as pillar height increases.

Figure 7.13 shows the increase in vertical movement within a pillar as the pillar height increases. This combination of increasing strain with decreased stress is indicative of post-yield behaviour.

Post pillars are employed at Detour Lake Mine as a means of reducing the unsupported span in mechanized cut and fill stopes. Normally, 5 metre square post pillars are left with the maximum span between pillars or walls determined from the Stability Graph for Entry-Type Excavations (normally about 20 metres). Current practice is to begin growing post pillars on the footwall side of the stope (Figure 7.14). As the stope shifts south and east with each lift due to the plunge and dip of the orebody, the vertical pillar migrates from footwall to hangingwall.

The post pillars do not provide significant benefit when they are located close to the sides of the stope since the walls are also providing support. They would provide maximum benefit when located at the centre of the exposed span yet by the time they reach the centre of the span their width to height ratio is 4:1 to 5:1. At this height:width ratio, the pillar has minimal load bearing capacity. Hedley, (1975) has suggested that the effect of a post pillar is to provide support to the immediate back. With these considerations in mind, a cable bolt support trial was initiated at DLM to effectively eliminate the need for post pillars.

7.5 CABLE BOLT PILLAR

Cable bolting has a number of advantages over post pillars in cut and fill stopes:

- It is possible to calculate and monitor the strength of the cable bolt support whereas the strength of the post pillar is much harder to predict.
- The cables can be installed in the centre of the stope and angled to follow the plunge and dip of the stope thereby remaining in the centre of the stope providing maximum benefit;
- Greater ore extraction ratios can be achieved by mining the pillars; and,
- Greater mining efficiency achieved by mining full face and not having to mine around a pillar.

In order to compare the effectiveness of post pillars and cable bolting, a post pillar at Detour Lake Mine was replaced with an artificial cable bolt pillar in the back. Cable bolts were installed in the back adjacent to the pillar. Instrumentation was installed to measure the displacement of the back as well as the load taken up by the cables. Finally the post pillar was removed and the loads and displacements were monitored. This cable bolt support trial and the results of monitoring will be discussed below.

7.5.1 Location of Test

Post pillars are only employed in the 460 Stope of DLM where the orebody is up to 45 metres wide. It was determined that Pillar 941 on the 8th lift of the 430 M4 stope (Figure 7.15) would be the best location for the test for the following reasons:

- The pillar had a width:height ratio of 0.2:1 and was approaching the centre of the stope;
- There was an access 20 metres above the stope necessary for locating monitoring instrumentation;
- The cable bolting to be carried out adjacent to the pillar would be in a position which would not interfere with regular stoping operations; and
- Mine scheduling permitted time for cable bolt installation and pillar removal before the stope had to be filled.

7.5.2 Geometry of Post Pillar 941

Pillar 941 was initiated on the 4th lift of the 430 Stope and was 25 metres in height prior to removal. The cross sectional shape and area varies from lift to lift (Figure 7.16). The inconsistency in the size and shape of the pillar on each lift is due in part to the angle that the pillar was approached by the advancing breast face. The pillars are not trimmed to the design size after mining past them which also contributes to their larger than design size.

7.5.3 Estimate of Pillar Strength

An estimate of the post pillar strength was made in Chapter 5 using the modified Hoek-Brown failure criterion. The failure constants m_b and a have been estimated to be 3.4 and 0.45 respectively. The confining stress and therefore the pillar strength at the mid-height of the pillar was shown in Section 5 to be negligible using 3-D boundary element modeling. The above is based upon a rock mass rating of 80 for the undisturbed mafics (Table 7.2).

7.5.4 Estimate of State of Stress in Pillar

The stress on the post pillar can be initially estimated using tributary theory (Hoek *et al.*, 1980) where:

$$\sigma_p = \gamma z \left(\frac{\text{rock column area}}{\text{pillar area}} \right) \tag{7.1}$$

where,

γ = the unit weight of the rock

z = the depth of rock above pillar

The rock column area refers to the area supported by the post pillar which is assumed to be half the distance to the adjacent pillar. The minimum pillar area is 47.5 square metres which is the cross sectional area of the pillar on Lift #8. The rock column area is estimated from Figure 7.15 to be 420 square metres. Therefore, the average vertical pillar stress is estimated to be:

$$430\text{m} * 0.029\text{MPa} / \text{m} * (420 / 47.5) = 110\text{MPa} \tag{7.2}$$

Tributary theory severely overestimates the stress level in the pillar as compared to the results of 3-D boundary element modeling as shown in Chapter 5. This modeling has shown the average major principal stress at the mid-height of the pillar is approximately 20 MPa, sufficient to cause yielding of the pillar. The above evaluation corresponds well to visual observations whereby the rock mass rating was reduced from 80 to 58 at the final stages of extraction. It was concluded that the pillar had yielded but was maintaining support to the immediate back through its post-yield strength. The post-yield strength develops as the pillar dilates and compresses the confining fill.

7.5.5 Description of the Rock Mass

Visual inspection of Pillar 941 identified several vertical joints on the east side of the pillar which were open 2-5 centimetres. Several blocks were close to slabbing off the wall of the pillar. The pillar is located in an area between the Talc Zone and the Main Zone. A CSIR rock mass rating conducted on the north wall of the pillar yielded an RMR of 58 (Table 7.1). Rock to the north the pillar was typical Main Zone rock which had a measured RMR of 80 (Table 7.2). Rock to the south of the pillar was typical Talc ore which had a measured RMR of 65 (Table 7.3).

Figure 7.18 is a lower hemisphere equal angle stereonet plot of the structural data collected around the pillar during mining of lifts 4 to 8. The stereonet shows one major joint set and one minor one with the following orientations:

Joint Set A: Mean Orientation: Strike 297°, Dip 86°
Joint Set B: Mean Orientation: Strike 160°, Dip 90°

7.5.6 Cable Bolt Support Implementation

7.5.6.1 Support Design

To determine the number of cable bolts which would replace the post pillar, the maximum load being supported by the pillar was assumed to be a wedge 20 m square at the base and 3 metres high. Three metres is the maximum height of ground falls experienced at DLM to date. Using a specific gravity of 3.0

for DLM ore, the weight of this wedge would be 1800 tonnes. Thirty-five double, 5/8" diameter seven strand steel cable bolts were installed in a 1.5 m X 1.5 m square pattern adjacent to the pillar as shown in Figure 7.15. Each double cable bolt was assumed to have a strength of 54 tonnes. The bolts were installed in a concentrated pattern in the centre of the stope for three reasons:

- The cable bolts were being used to simulate the effect of the post pillar in reducing the unsupported span;
- To minimize the interference with normal mining activities; and
- To allow for monitoring from the 400 M5 Attack drift located above the stope.

7.5.6.2 Cable Bolt Installation

A 2 metre high pad was built up from waste rock below the area to be cable bolted. This was necessary for the longhole drill to reach the back. A Boart BCI-2 pneumatic longhole drill mounted on a rubber tired carrier was used to drill 52 mm diameter holes. The holes were all vertical and 19 metres long. Hole deviation was not measured but experience with drilling up-holes elsewhere in the mine with this rig indicated it to be 3 percent. Figure 7.19 is a section through Ring 1 showing the expected height of the drill holes and the future lift elevations. The bolts are intended to pre-support lifts 10 and 11.

The cable bolts were inserted up the holes using a cable bolt inserter mounted on a scissor lift vehicle. The end of each cable bolt had a hydraulically "pressed-on" end holding device. Two 7.5 cm long spring steel clips were fastened to the end holding device to prevent it from slipping down the hole during insertion. A 6 mm diameter (I.D.) breather tube was taped to the top of the bolt prior to insertion. After pushing the bolts up the holes, a 19 mm diameter (I.D.) grout tube was inserted roughly 0.5 m into the hole. The hole was then sealed with a combination of cloth, wedges, and resin grout.

Grouting was carried out with a Spedel Series 6000 grout pump and mixer. Type 30 high early strength cement was used in a 0.45 water cement ratio. A lower water cement ratio was desirable but not practical with the Spedel pump. The end of the breather tube was placed in a bucket of water and pumping continued until the air stopped coming out of the tube. Wooden squeeze blocks were attached to the cable bolts at the collar to contain any spalling around the hole collars and to contain small blocks with less than the required embedment length, in order to mobilize the full cable bolt strength (Figure 7.20(a)). The cement had 12 days to cure before the adjacent pillar was removed.

7.5.7 Monitoring Instrumentation

Four types of instrumentation were used to monitor the extraction of the post pillar. These include two multi-point extensometers, a ground movement monitor, 1 cable bolt mounted with three Tensmeg strain gauges, and one vibrating wire stress meter.

The two Wireflex extensometers were installed in holes drilled from the 400 M5 Attack drift (Figure 7.15). This was necessary in order to permit monitoring to continue until lift #10. Each extensometer had three anchors which were spaced to be 2 metres above the back elevations of the 8th, 9th, and 10th lifts (Figure 7.19). The extensometers were connected to an eight channel RST LE8200 Data Logger, which can take readings at programmable intervals and store the data for later retrieval.

A ground movement monitor (GMM) was installed in the back of the 430 M4 #8 Stope near the cable bolts as shown in Figure 7.15. The GMM was anchored at 3.6 metres into the back. There were no prominent joints in the vicinity which the GMM was intended to cross. It was deep enough however to record movement of a 3 metre high wedge which was the maximum height expected and for which the cable bolt pattern was designed. GMM readings were taken manually.

One cable bolt hole was drilled from the 400 M5 Attack. A single cable bolt was installed which had three Tensmeg strain gauges mounted on it to determine if the cables were taking any load after the pillar was removed. Like the extensometers, the strain gauges were mounted on the cable at intervals such that they would be 2 metres above the back of the 8th , 9th, and 10th lifts.

A Geokon vibrating wire stress meter was installed in a hole on the south wall of the stope which had not been undercut. The hole was drilled 3 metres deep with a percussion drill. The stress meter was oriented to measure the vertical stress change which would be expected to occur when the pillar was removed if the pillar was transmitting vertical stress. The stress meter was read manually before and after pillar removal.

7.5.8 Results of Monitoring

7.5.8.1 Visual Monitoring

As a safety precaution, mining of the 8th lift was completed before the pillar was removed so workers did not have to enter the area. Immediately prior to removing the pillar, a visual inspection of the back was made for subsequent comparison. The back was in good condition and had rock mass rating of 80, typical of the Main Zone ore. The jointing was observed to be tight in the back. Rock bolt plates were not showing any signs of taking load. The wooden squeeze blocks on the ends of the cable bolts were also not showing any squeezing.

The pillar was drilled off on January 30, 1991. There were no problems with "jammed steel" or other signs of a highly jointed rock reported by the driller. The holes were clean except for the first 0.6 metres where there were obvious signs of open joints. The pillar is shown in Figure 7.20(b) prior to being blasted. The pillar was blasted on January 31, 1991 at 4:08 a.m.. A 1m x 3m pillar remained standing after the blast but it was highly fractured and held up by the muckpile. It was recovered without additional blasting. The open span was increased to 33 metres with the removal of the post pillar. Another visual inspection was made after the blast and no changes were observed.

7.5.8.2 Instrumentation

The locations of the monitoring instruments are shown in Figure 7.15 and Figure 7.19. Figure 7.22 refers to the GMM in the centre of the span showing no significant movement (under 1 mm) before and after the pillar blast. Similar recordings were made on the 9th and 10th lifts indicating the backs of the lifts did not move This is verified visually as the area was classified as "stable".

Figure 7.23 shows the stress change versus time plot for the stress meter located in the footwall of the 8th lift. No significant load change was observed with the removal of the pillar. A pillar that was a load bearing element would be expected to show some load transfer to the adjacent pillar. This reinforces the original assumption that the post pillar was a minimal support member.

The Tensmeg monitoring data is shown in Figures 7.24. Figure 7.25 shows the same data over the first 24 hours after the blast. Strains on Anchor 1 of the Tensmeg located in the back of the 8th lift, indicated that load transfer onto the cables had occurred shortly after the removal of the post pillar. Loading on Anchor 2 above the 9th lift was minimal. The Tensmeg was calibrated to record maximum strains of 10,000 to -2,500 microstrain. 8000 microstrain corresponds to a load on the cable of 25 tons (Figure 7.26). Loading on Anchor 1 climbed to over 25 tons within 24 hours of the pillar blast which is sufficient load to cause failure.

Visual observations and ground movement monitor readings did not support the Tensmeg observations. When the 9th lift was mined through in March, the Tensmeg in the back did not show the same type of load increase. In fact the load decreased during this period. These results cannot be adequately explained unless damage had occurred to the instrumentation.

Movement within the extensometers is erratic as shown in Figure 7.27. These gauges are affected by the blast vibration and at this stage the displacement data is not considered to be reliable. The potentiometers are also believed to have been in contact with water which contributed to the poor quality of the data. The ground movement monitoring data is considered to be more reliable.

7.5.9 Summary of Cable Bolt Pillar Support Trial

Cable bolts were used to pre-reinforce 2 lifts. Upon extraction of the 9th and 10th lifts of the 430 M4 Stope, no movement of the ground movement monitors was observed. This was verified by visual observations as the back was characterized as being "stable". Extraction of the post pillar on the 8th lift did not result in deteriorating conditions, however, loading on the cables did increase. Difficulties were experienced with the instrumentation which prevented load magnitudes from being determined. The displacement which caused the load increase was not measured by the GMM's so it is expected that the magnitudes were small. It is concluded that the original post pillar was only providing support to the immediate back.

In Main Zone rock at Detour Lake Mine the maximum span which can be opened up before instability begins to occur is about 20 metres based on the Stability Graph for Entry-Type Excavations. Post pillars are used to reduce the span where the width of the ore exceeds the design span. Observations and analysis does indicate that the replacement of the post pillar with cables enabled spans on subsequent lifts to be increased up to 35 metres. The experience and confidence gained by this test has led to further cable bolting of stopes in this manner at DLM wherever post pillars are not practical.

It has been demonstrated that both cable bolts and post pillars can be used to increase the stable span. The supported span case histories obtained at Detour Lake Mine have been plotted on the Stability Graph for Entry-Type Excavations shown in Figure 7.28. It is difficult to assess the full potential benefit of the various support systems since there are no unstable supported case histories. A larger database of supported span case histories is required before such a graph could be used for design purposes however, this graph can be used to get an initial sense for the size of spans which can be designed for a given level of support.

Table 7.1 941 Post Pillar Geomechanics Rock Mass Rating

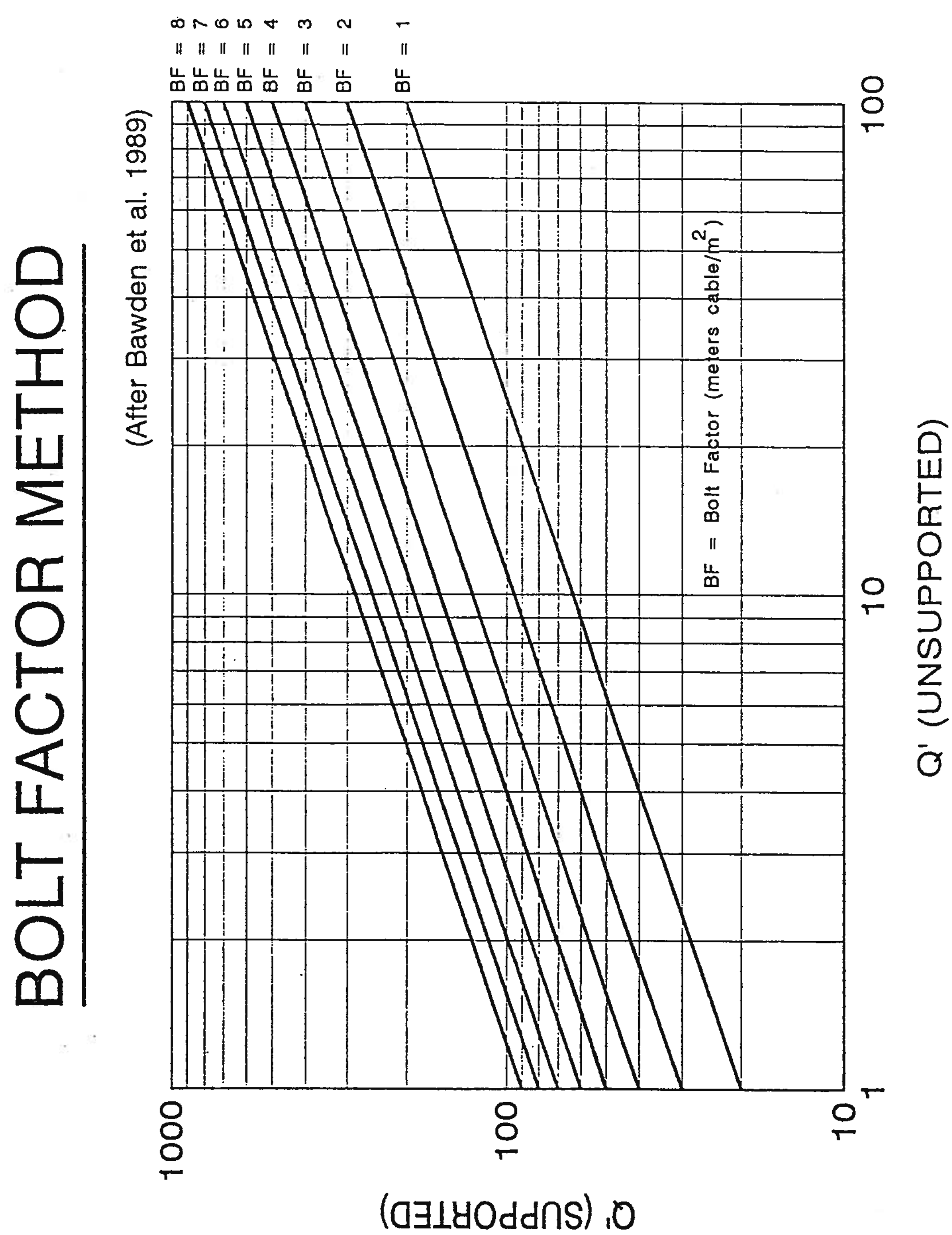
Category	Details	Rating (%)
Strength	170 MPa (R4)	14
RQD	65%	15
Joint Spacing	50-300 mm	10
Joint Condition	open joints	9
Groundwater	dry	10
TOTAL		**58**

Table 7.2 Main Zone Geomechanics Rock Mass Rating

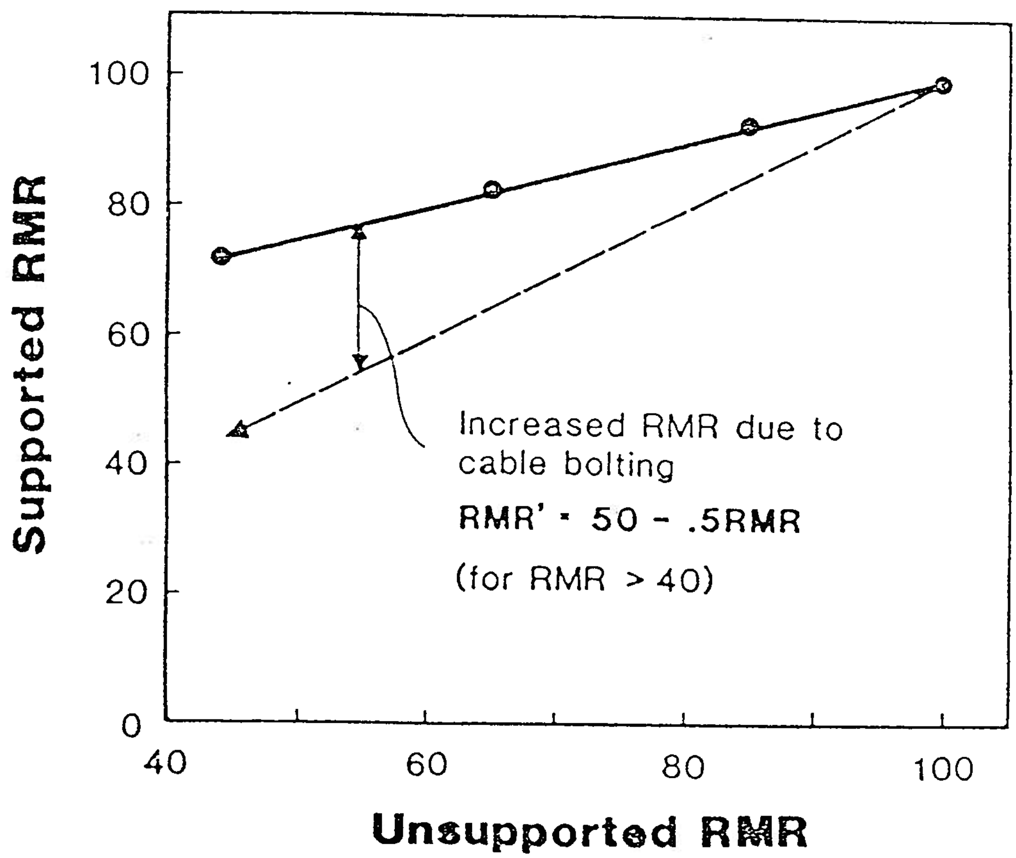
Category	Details	Rating (%)
Strength	170 MPa (R4)	14
RQD	90%	18
Joint Spacing	0.3-1m	20
Joint Condition	joints tight, slightly rough	18
Groundwater	dry	10
TOTAL		**80**

Table 7.3 Talc Zone Geomechanics Rock Mass Rating

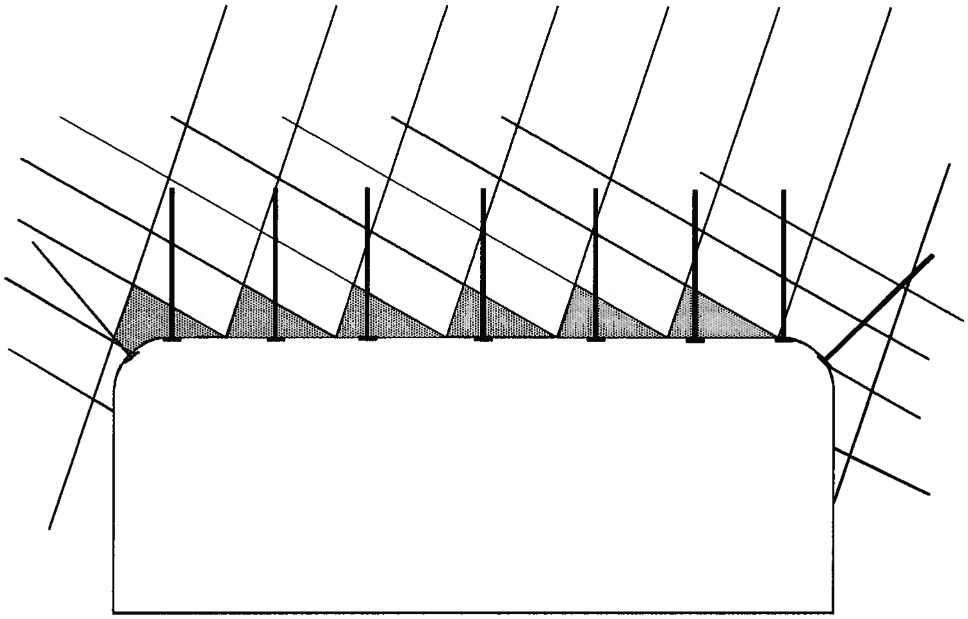
Category	Details	Rating (%)
Strength	40 MPa (R2)	5
RQD	90%	18
Joint Spacing	0.2-1m mm	16
Joint Condition	joints tight, soft wall rock	16
Groundwater	dry	10
TOTAL		**65**



BOLT FACTOR DESIGN CHART (After Bawden et al., 1989) | FIGURE 7.1

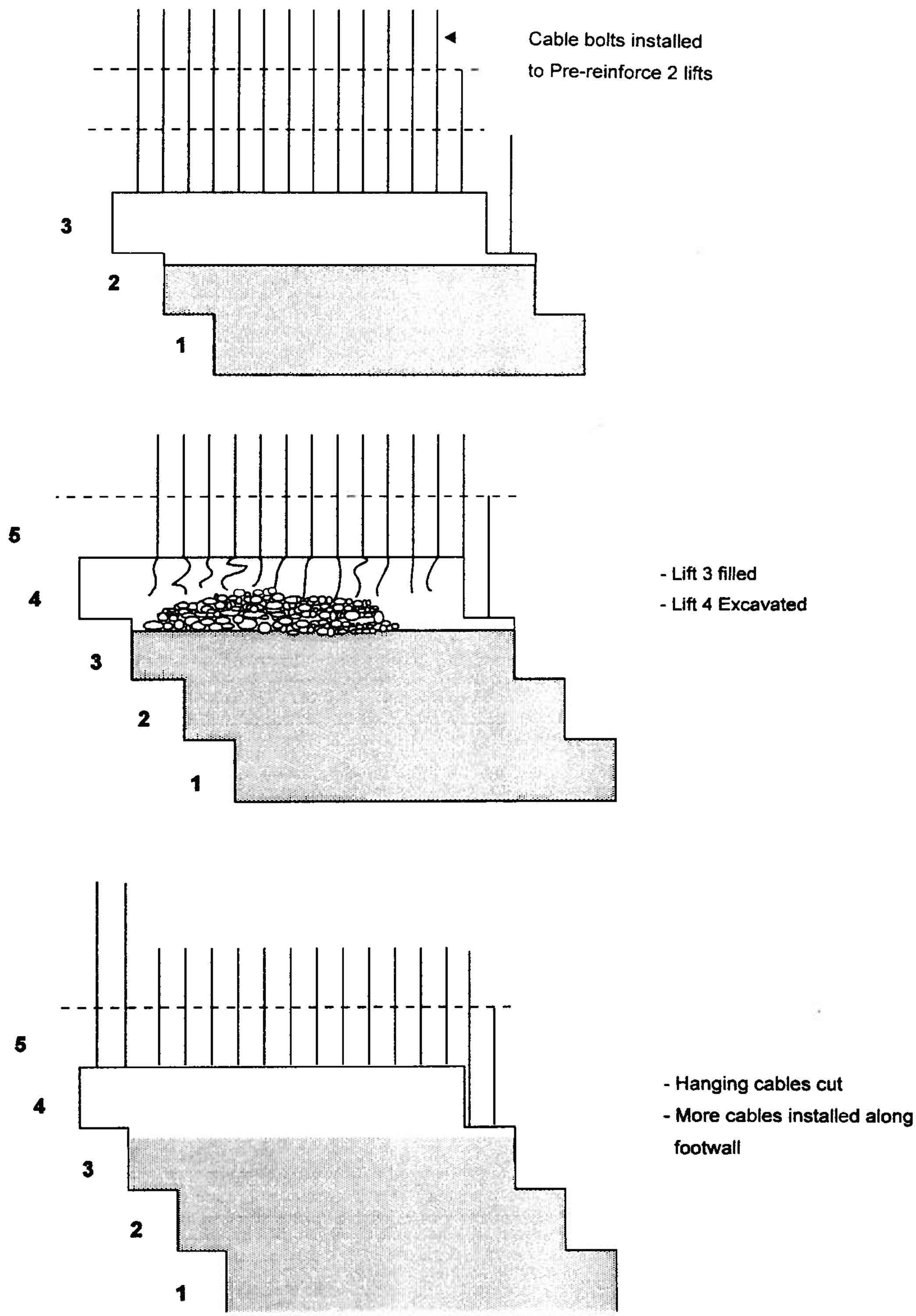


EFFECT OF SUPPORT ON THE RMR CLASSIFICATION VALUE	FIGURE 7.2



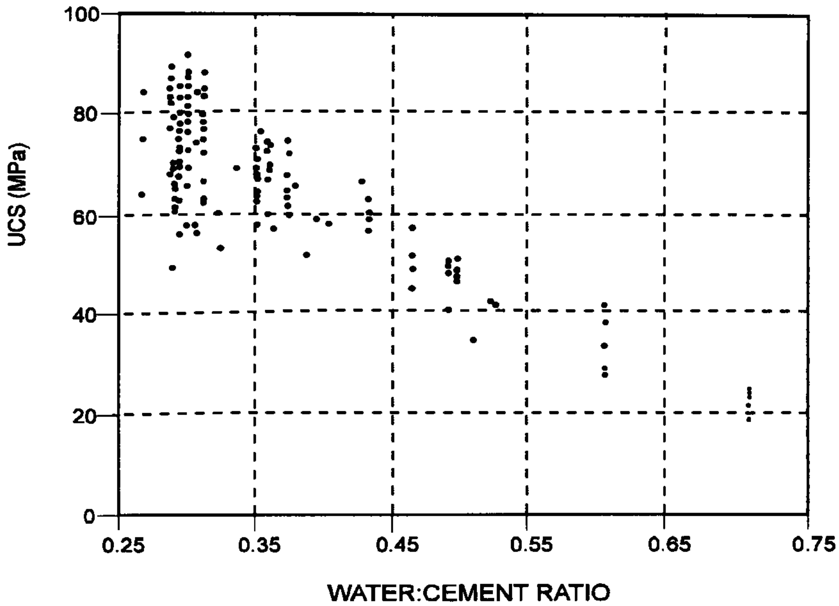
SUPPORT OF KEY BLOCKS PROVIDES
GEOMETRIC SUPPORT TO BLOCKS ABOVE

KEY BLOCK SUPPORT USING ROCKBOLTS	FIGURE 7.3



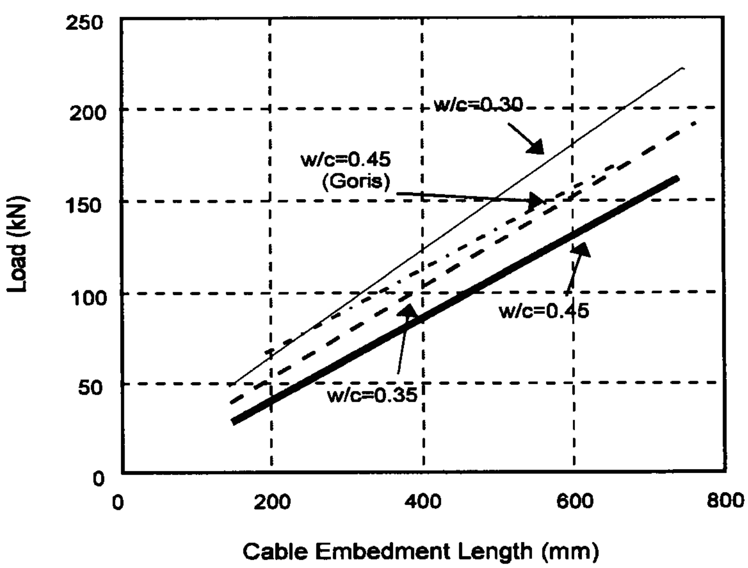
PRE-SUPPORT OF CUT AND FILL STOPES USING GROUTED CABLE BOLTS	FIGURE 7.4

(a)

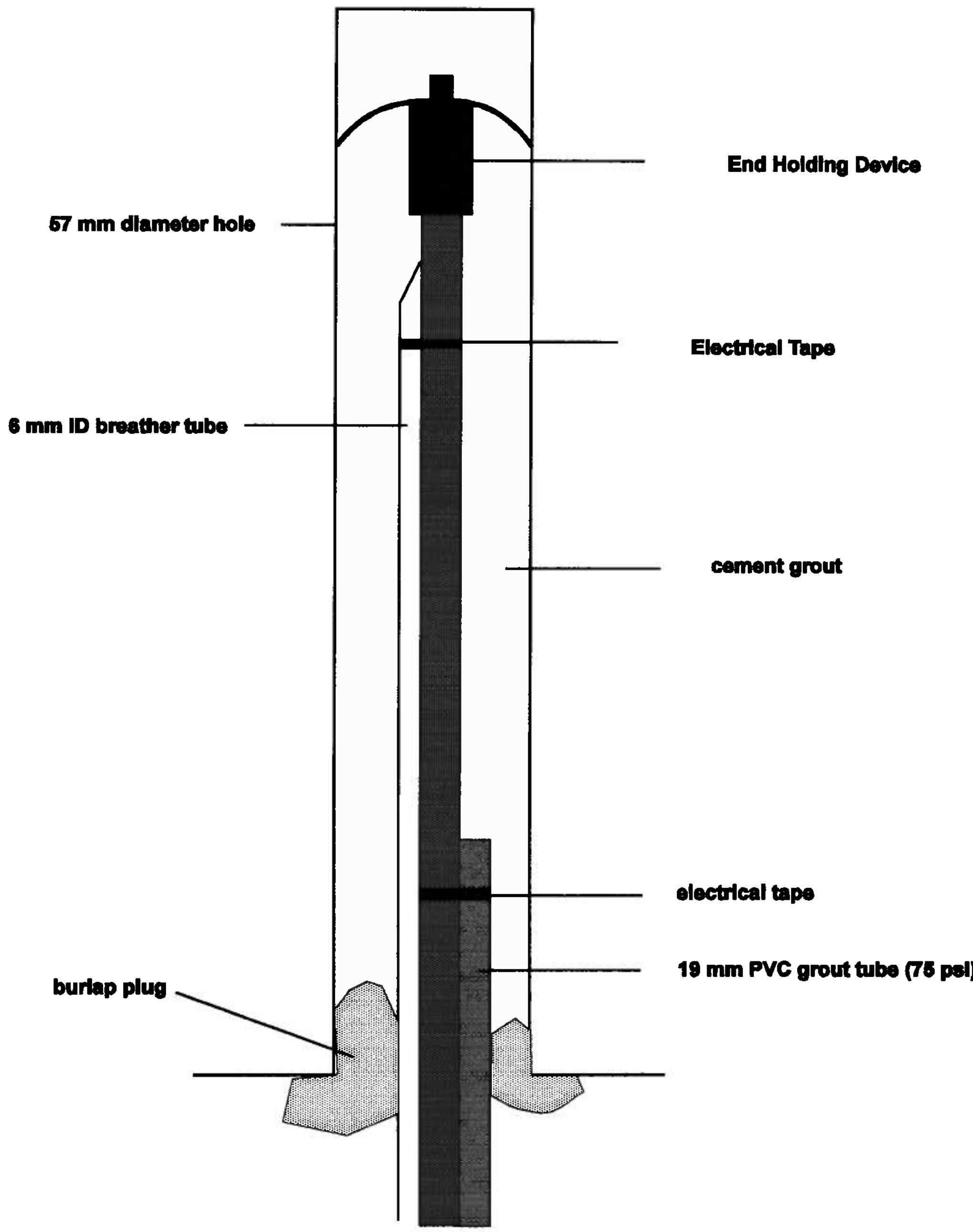


(after Hyett et al., 1992)

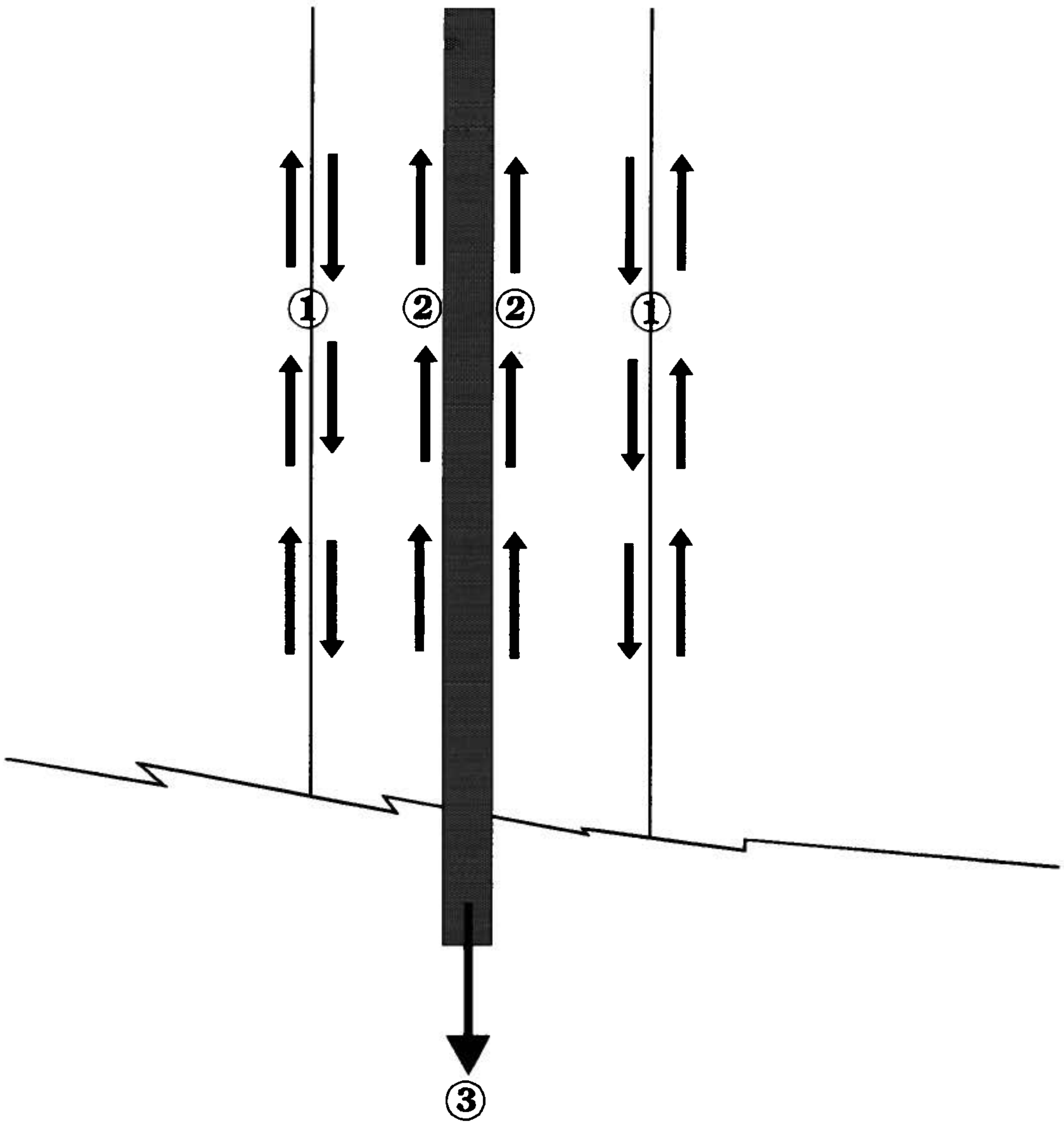
(b)



WATER:CEMENT RATIO VERSUS GROUT STRENGTH AND PULL-OUT STRENGTH	FIGURE 7.5



TYPICAL CABLE BOLT GROUTING ARRANGEMENT FOR UP-HOLES	FIGURE 7.6

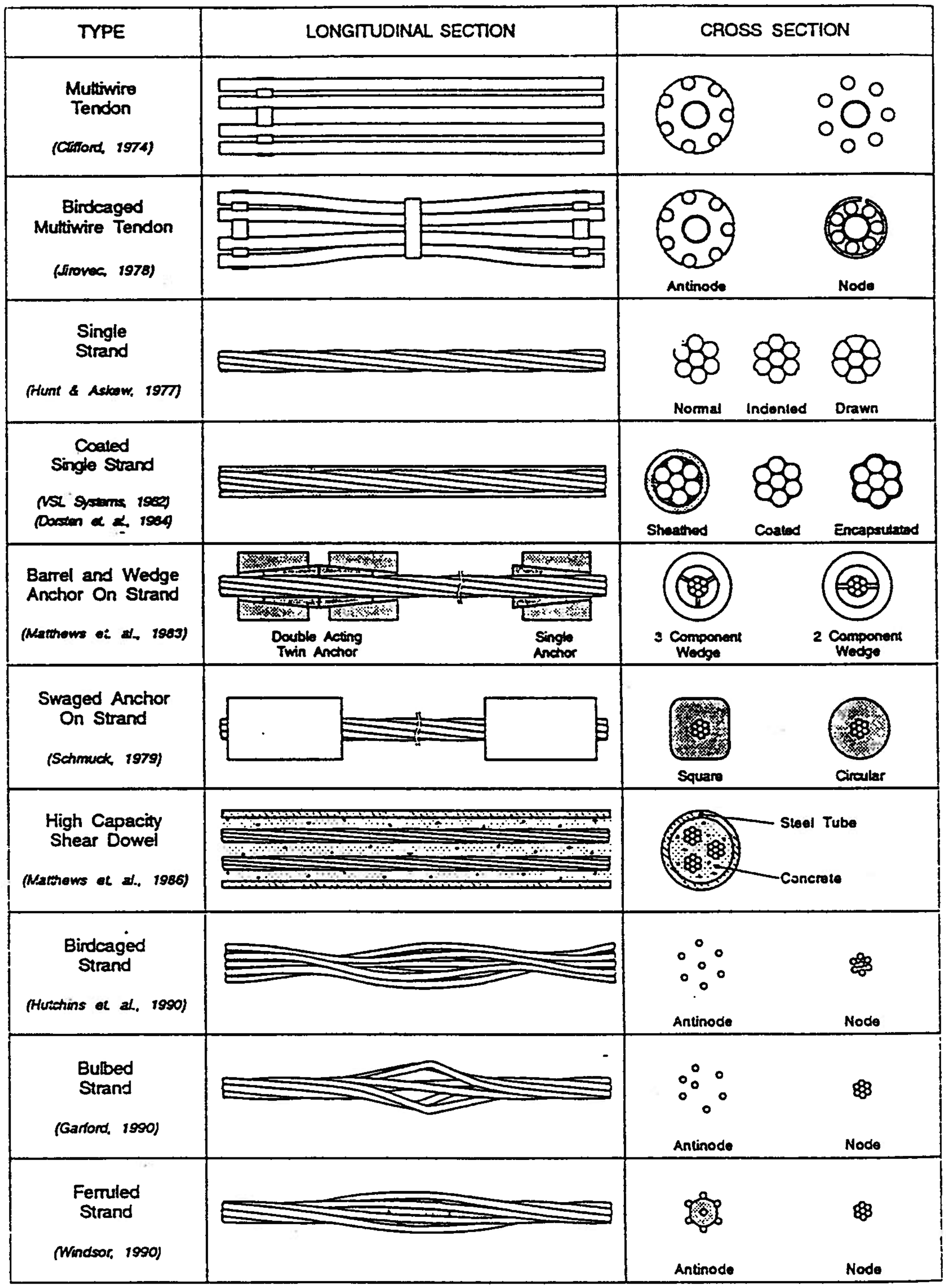


Strength Components Affecting Pull Out Strength
of a Cable Bolt

1. Rock Grout Bond Strength
2. Cable Grout Bond Strength
3. Tensile Strength of Cable

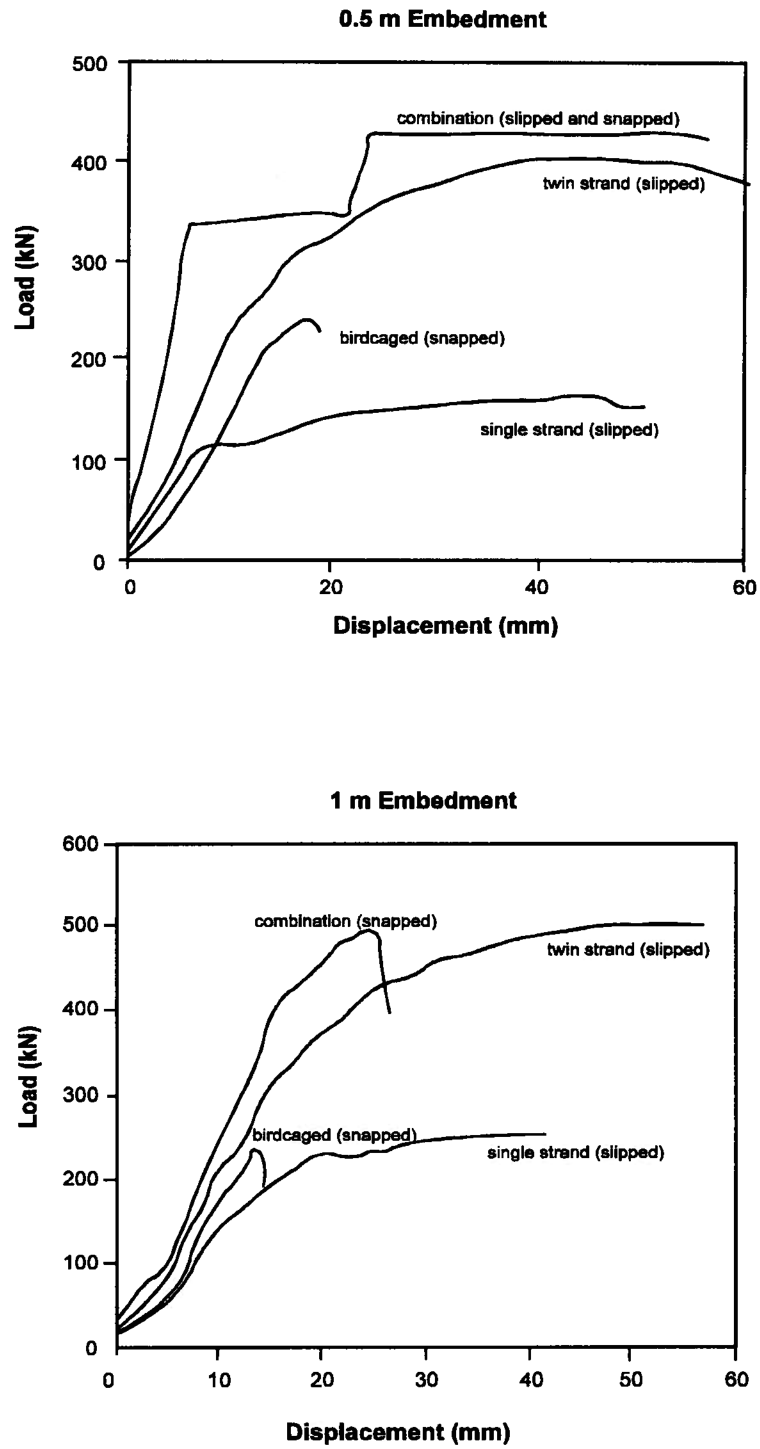
STRENGTH COMPONENTS FOR A GROUTED CABLE BOLT SYSTEM

FIGURE 7.7



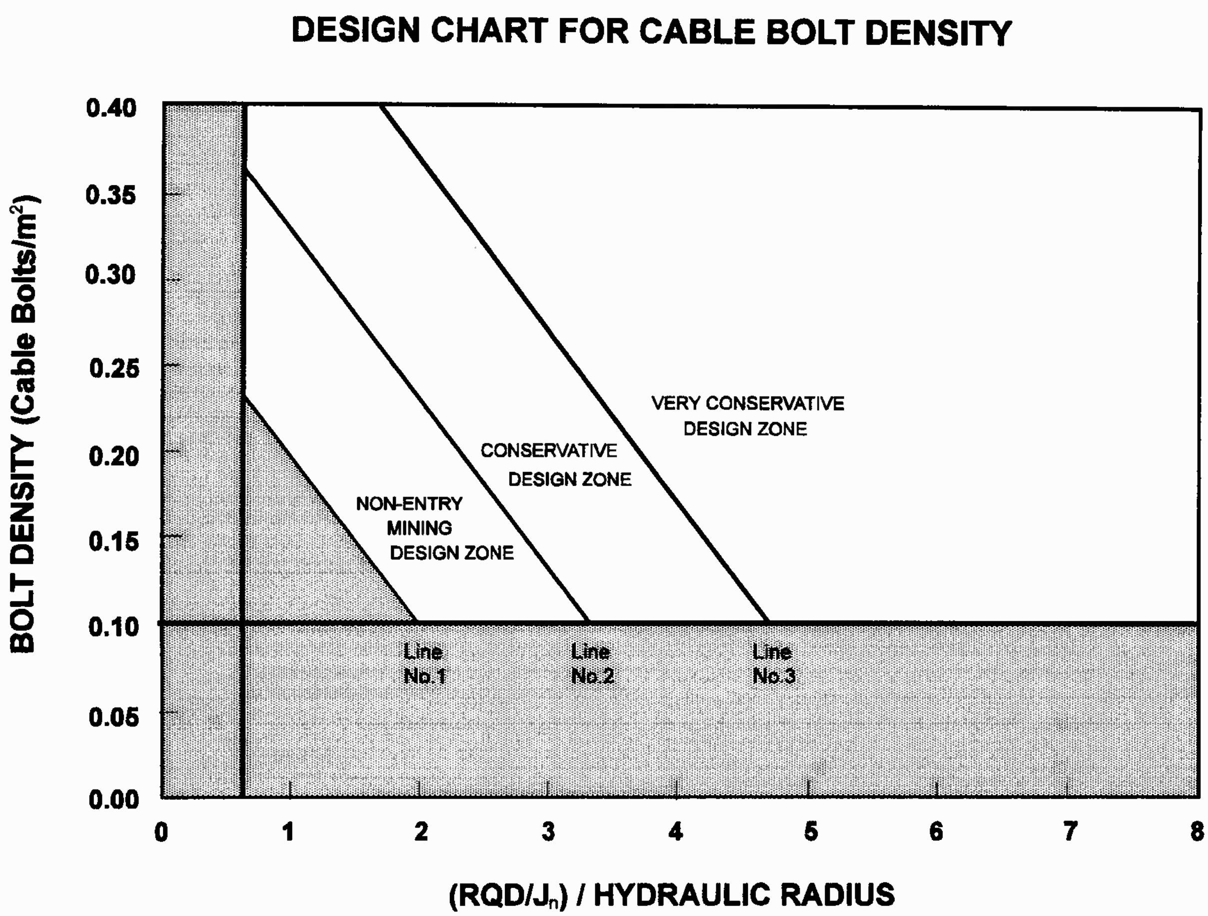
A SUMMARY OF THE DEVELOPMENT OF CABLE BOLT CONFIGURATIONS

FIGURE 7.8



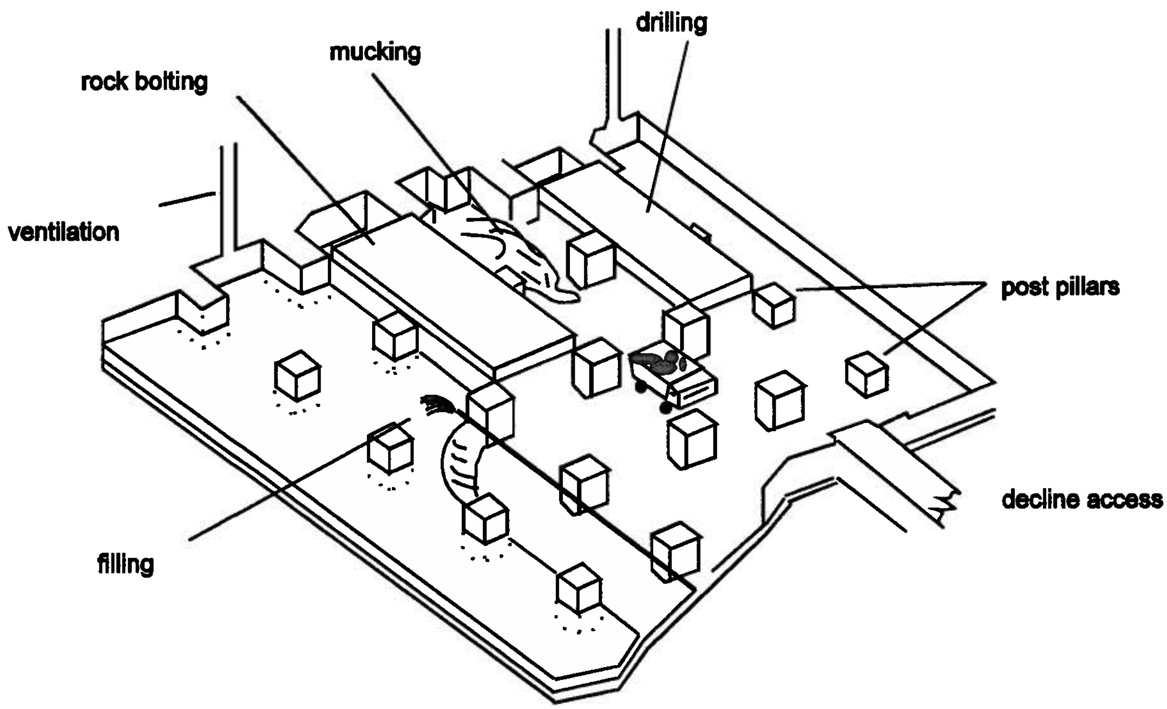
(after Villeasusa, 1992)

PULL OUT STRENGTHS FOR VARIOUS CABLE BOLT CONFIGURATIONS AND EMBEDMENT LENGTHS (after Villeasusa, 1992)	FIGURE 7.9

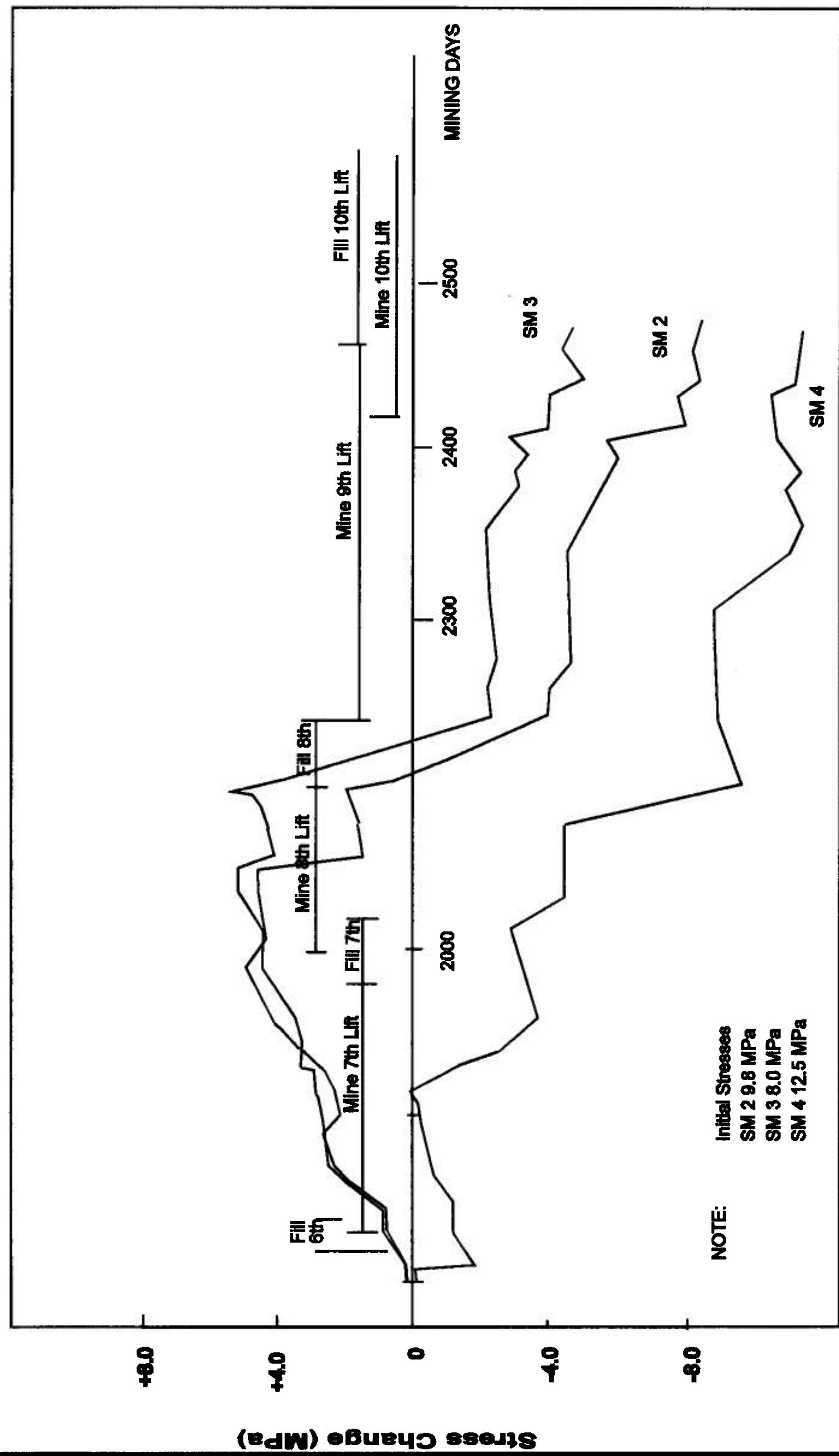


(after Potvin, 1992)

DESIGN CHART FOR CABLE BOLT DENSITY (after Potvin, 1992)	FIGURE 7.10

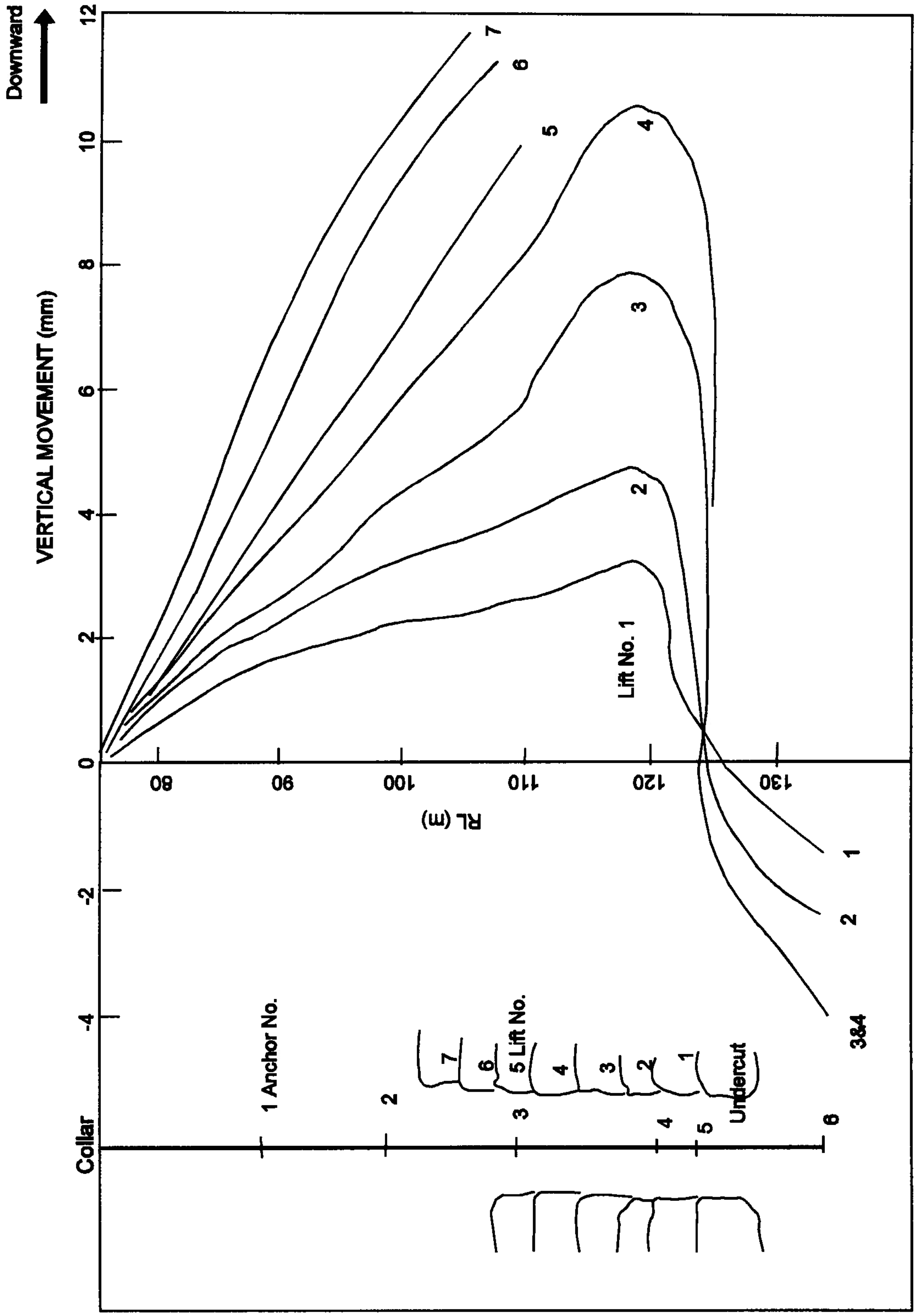


TYPICAL CUT AND FILL STOPE LAYOUT USING POST PILLARS (after Barrett et al., 1981)	FIGURE 7.11



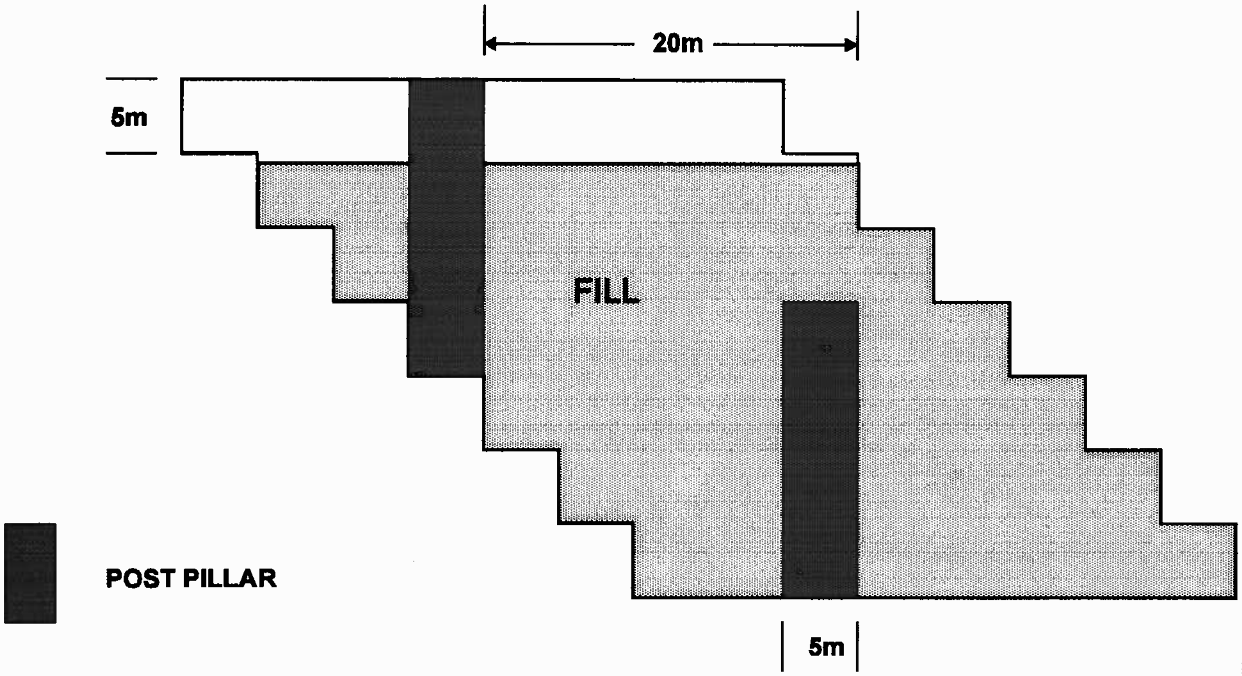
MEASUREMENT OF VERTICAL STRESS IN A POST PILLAR VERSUS PILLAR HEIGHT (after Barrett et al., 1981)

FIGURE 7.12



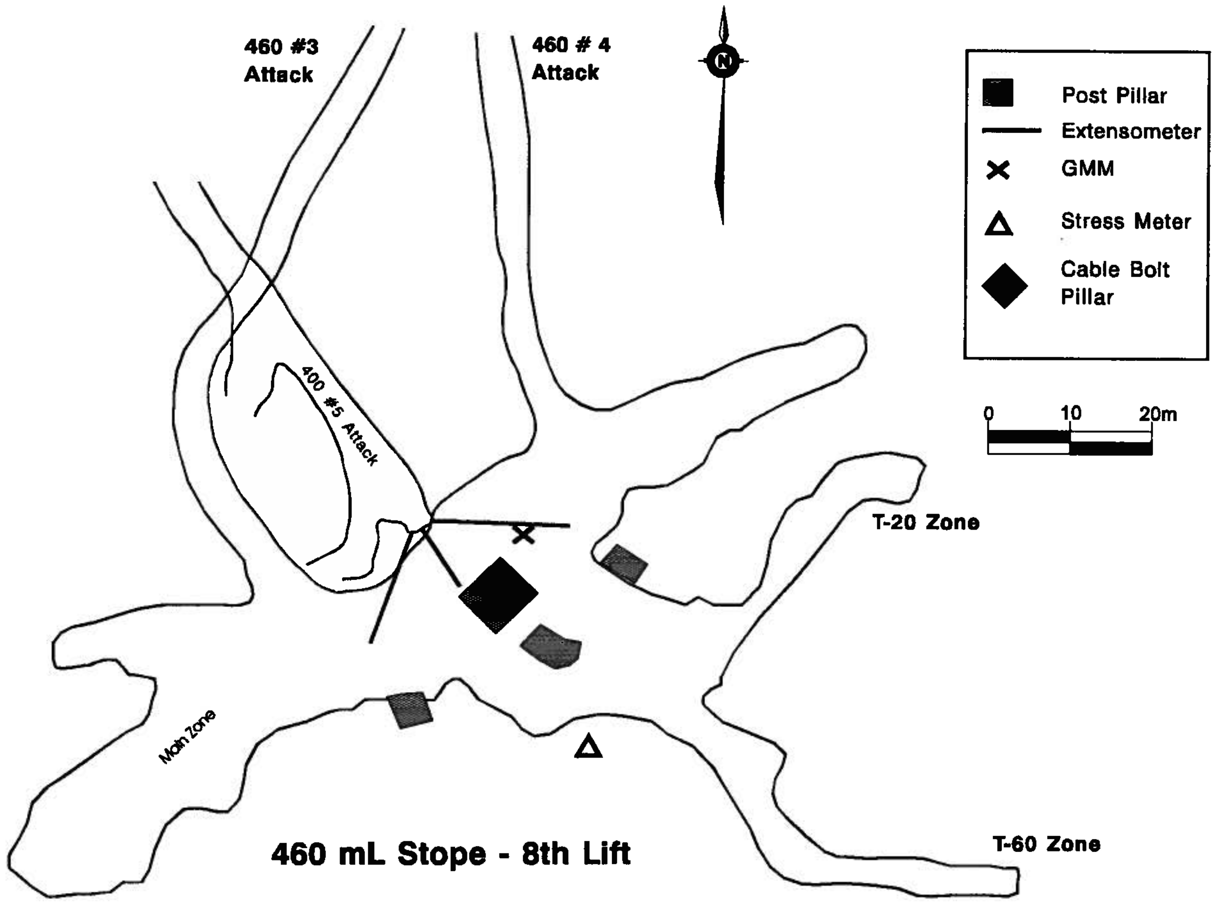
VERTICAL MOVEMENT IN A POST PILLAR VERSUS PILLAR HEIGHT (after Barrett et al., 1981)

FIGURE 7.13



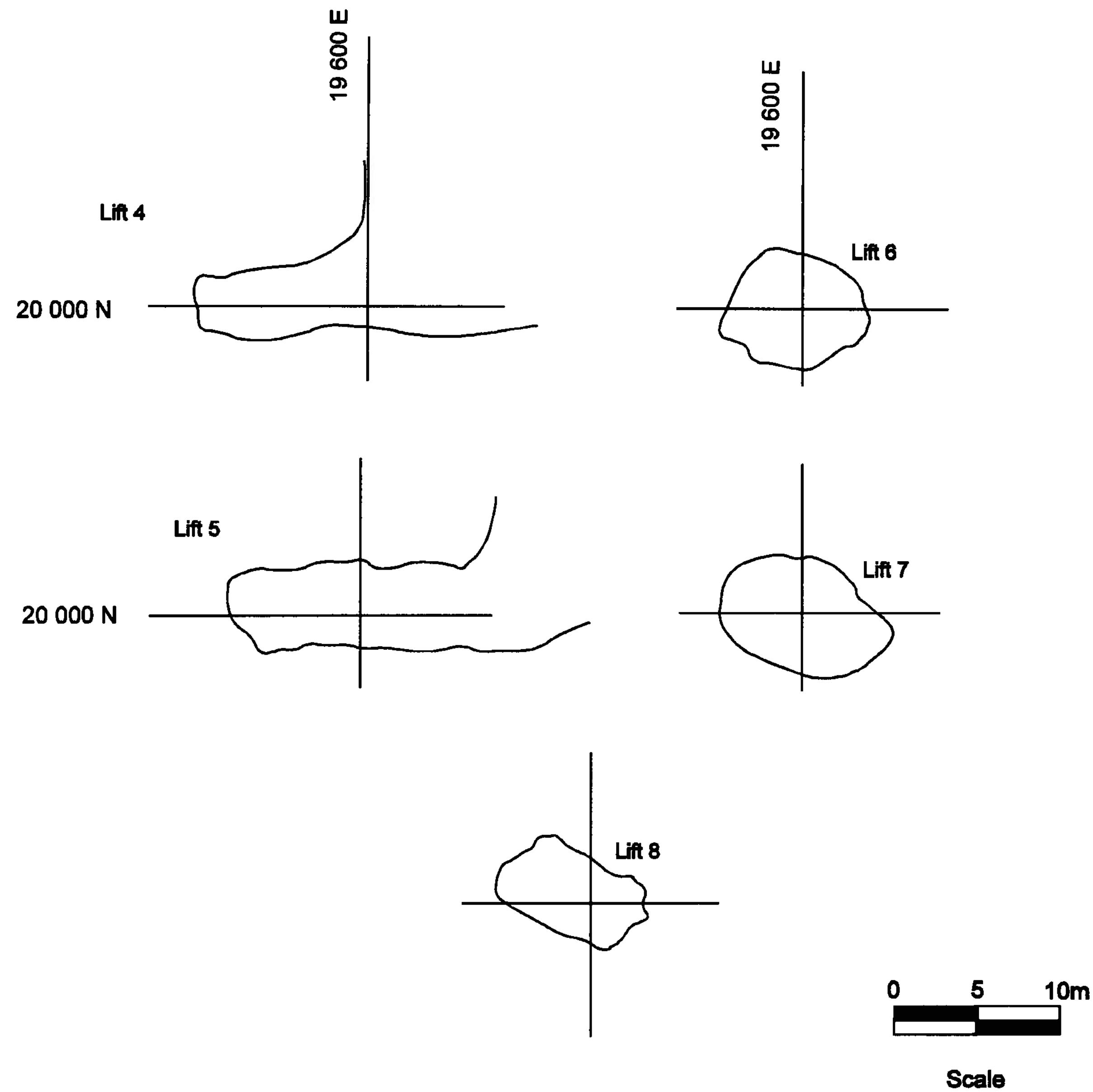
Schematic Cross Section of Post Pillars

SCHEMATIC SECTION OF A POST PILLAR CUT AND FILL STOPE AT DETOUR LAKE MINE	FIGURE 7.14

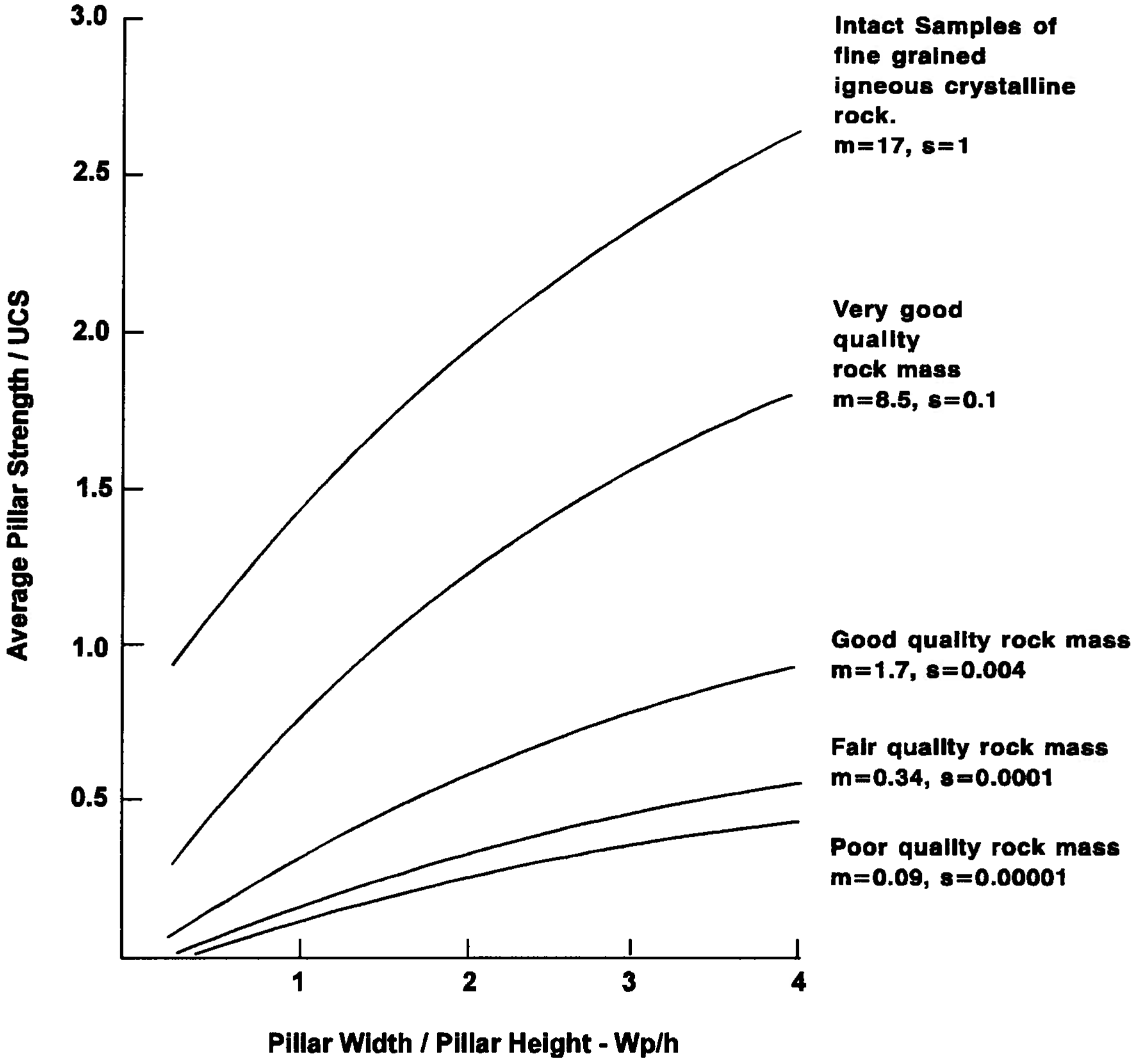


LOCATION OF CABLE BOLT SUPPORT TRIAL

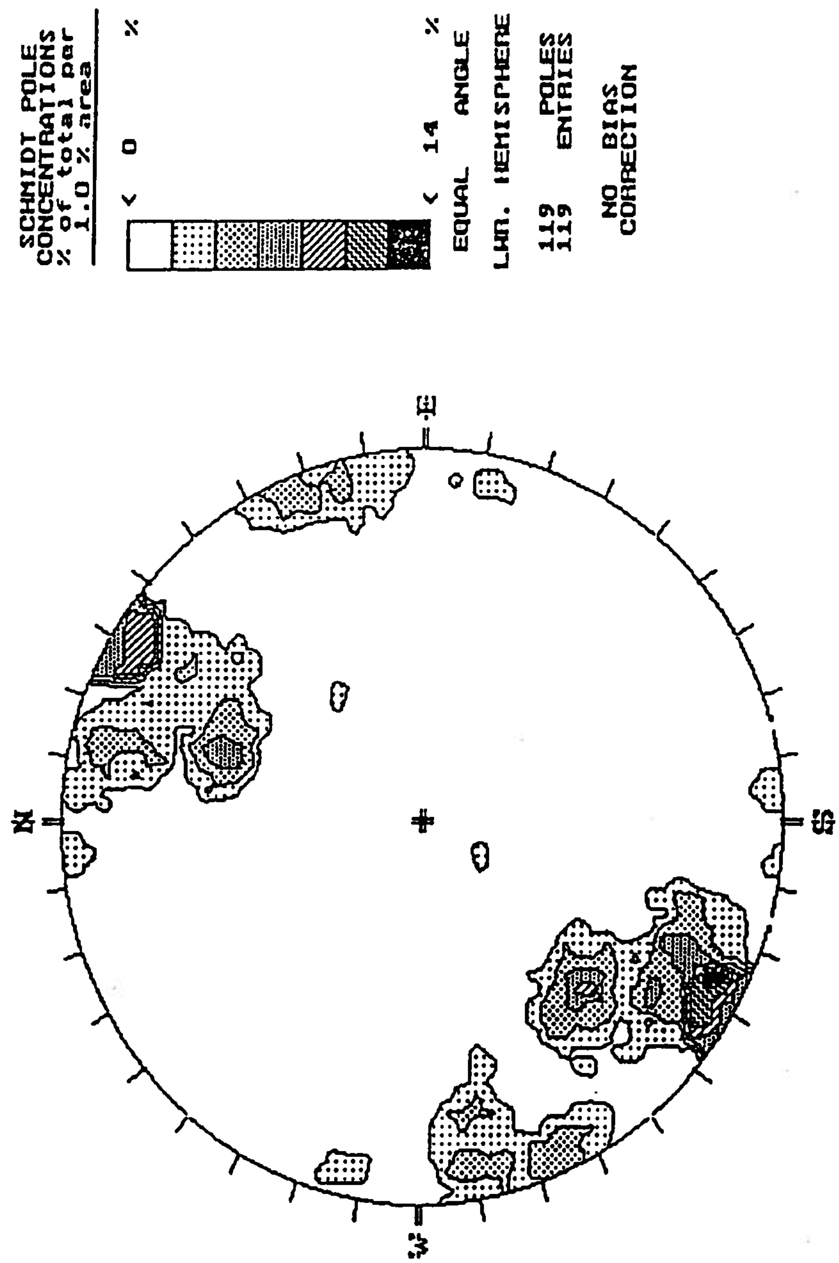
FIGURE 7.15



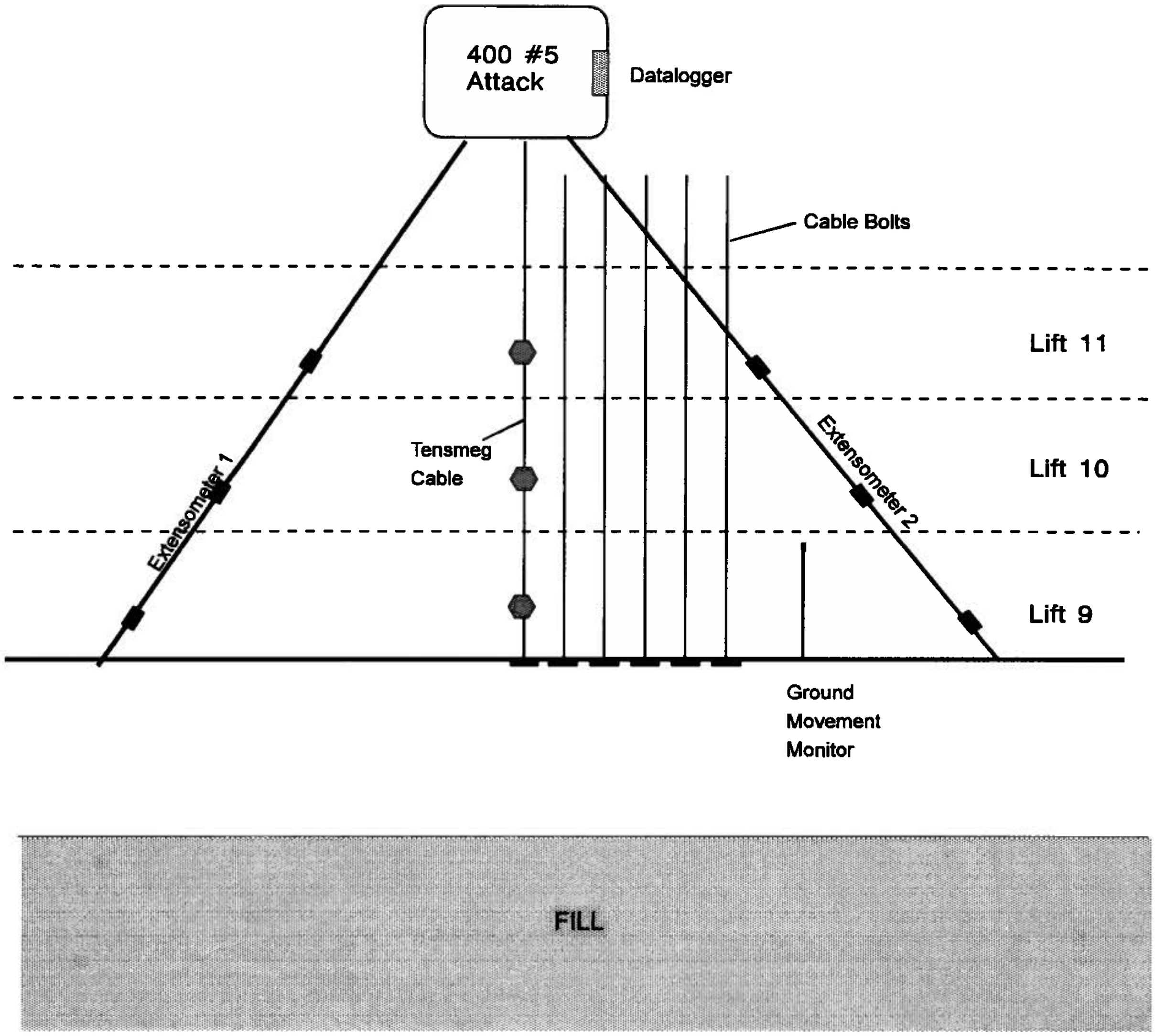
POST PILLAR 941 PLAN PROFILES	FIGURE 7.16



PILLAR STRENGTH VERSUS WIDTH:HEIGHT RATIO FOR DIFFERENT ROCK MASSES (after Hoek and Brown, 1980)	FIGURE 7.17

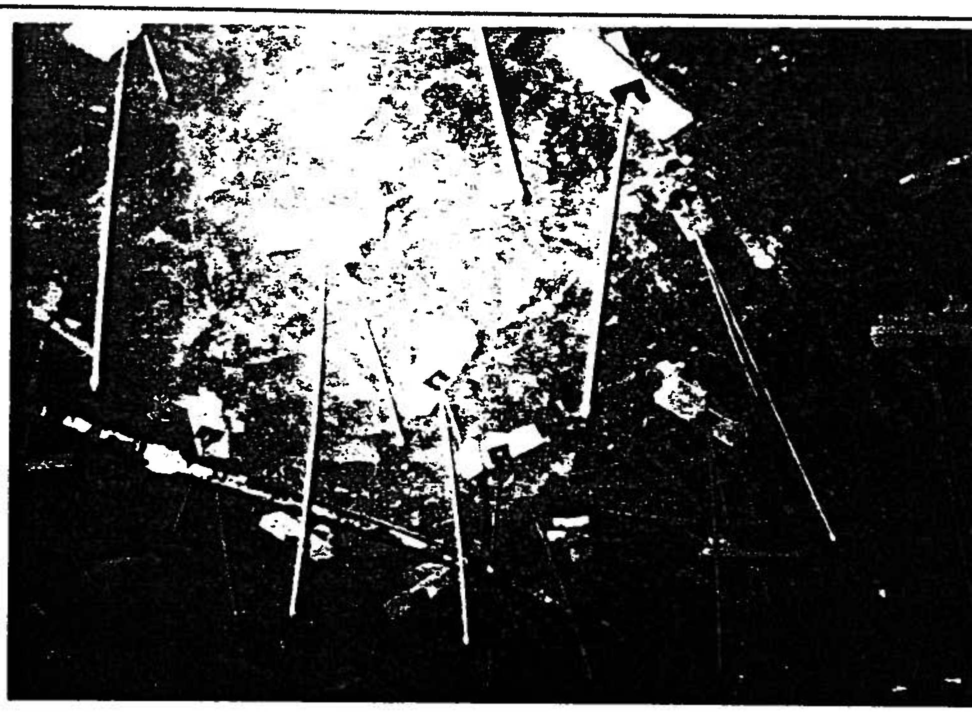


LOWER HEMISPHERE EQUAL AREA STEREONET PROJECTION OF STRUCTURE MAPPED AROUND PILLAR 941	FIGURE 7.18



SCHEMATIC SECTION THROUGH BOLTING RING #1
INDICATING LOCATION OF INSTRUMENTATION

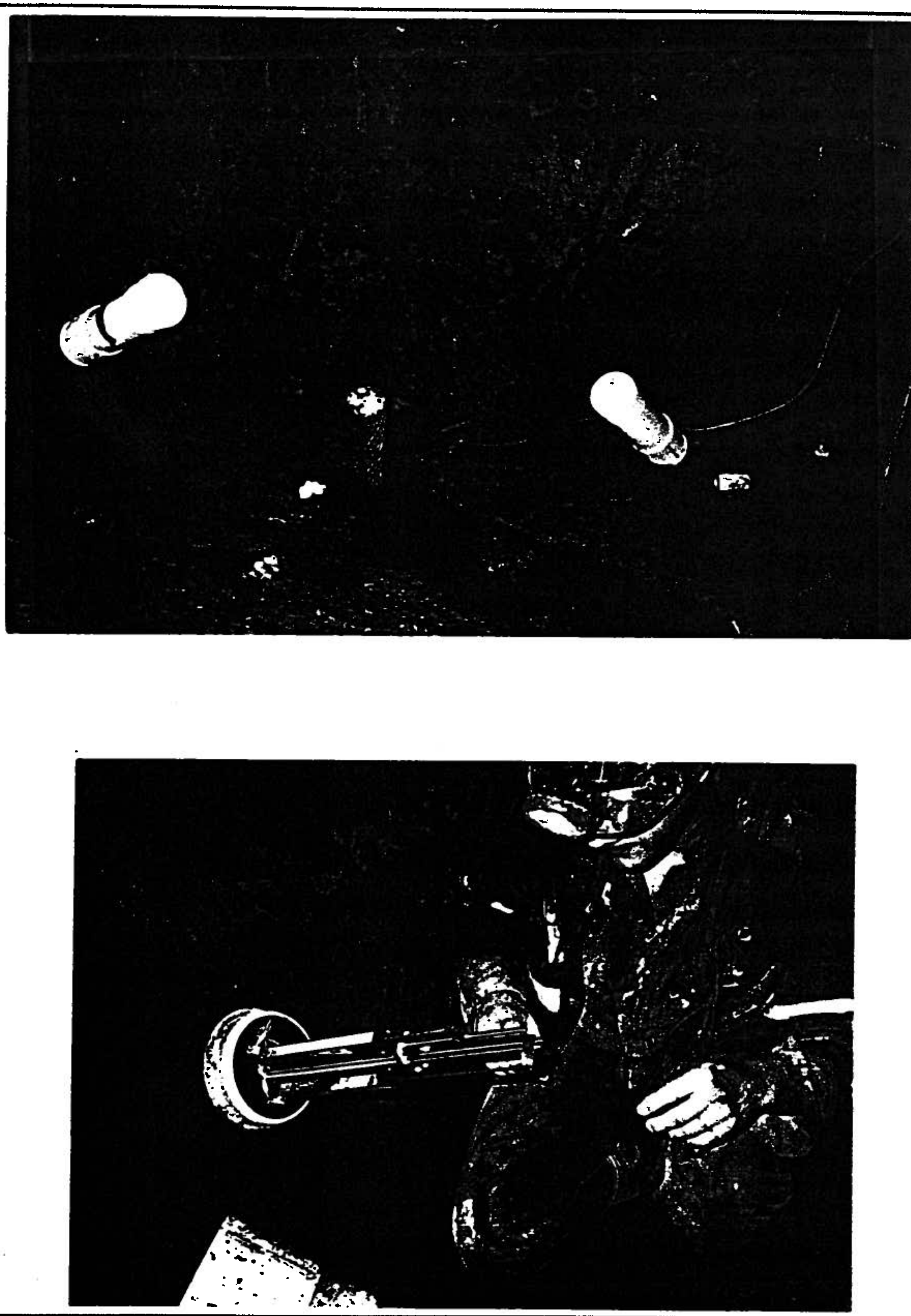
FIGURE 7.19



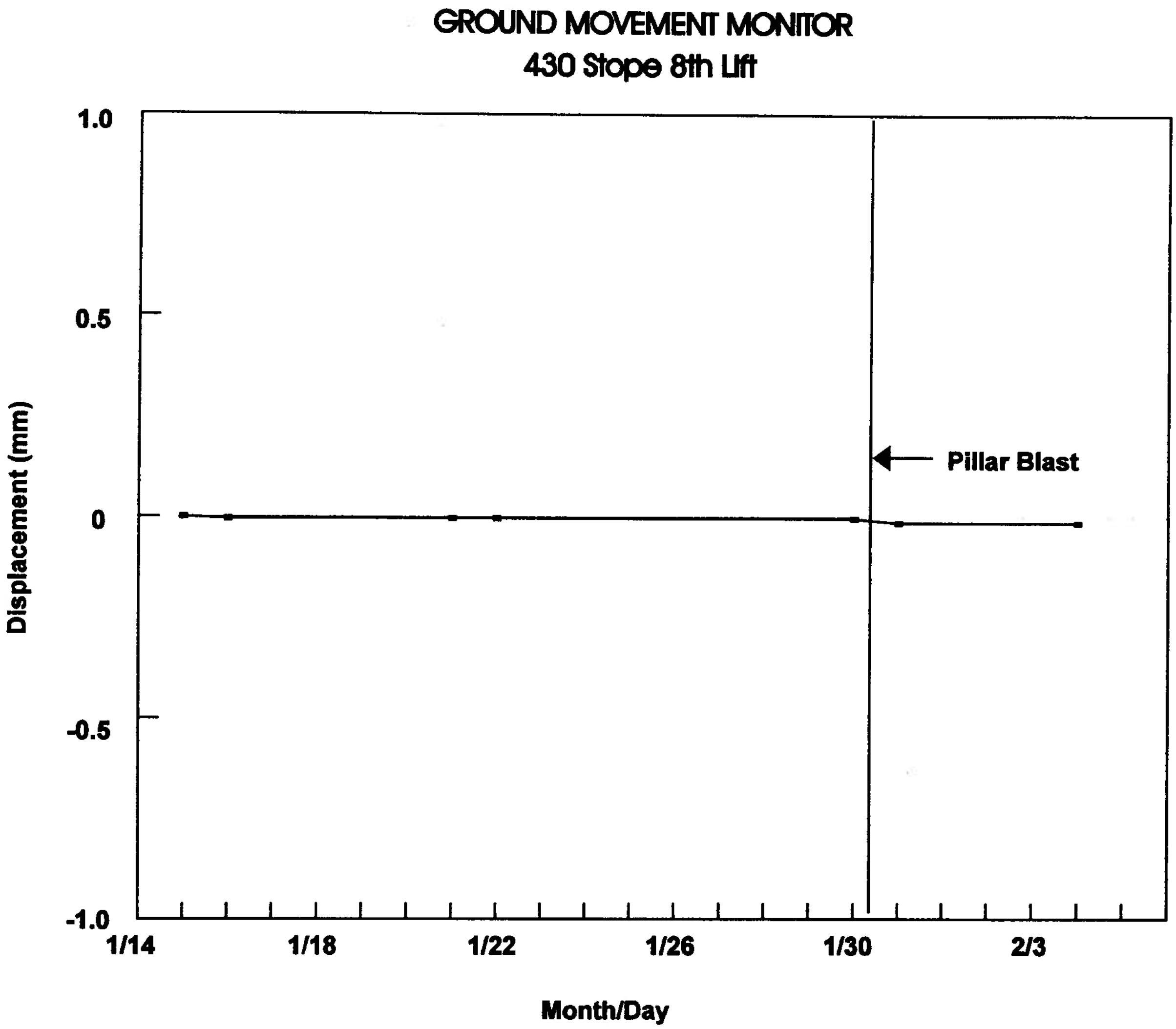
(a) Plated Double Steel Cable Bolts

(b) Post Pillar 941 (Looking South)

PHOTOGRAPHS OF CABLE BOLT AND POST PILLAR 941 PRIOR TO PILLAR BLAST	FIGURE 7.20

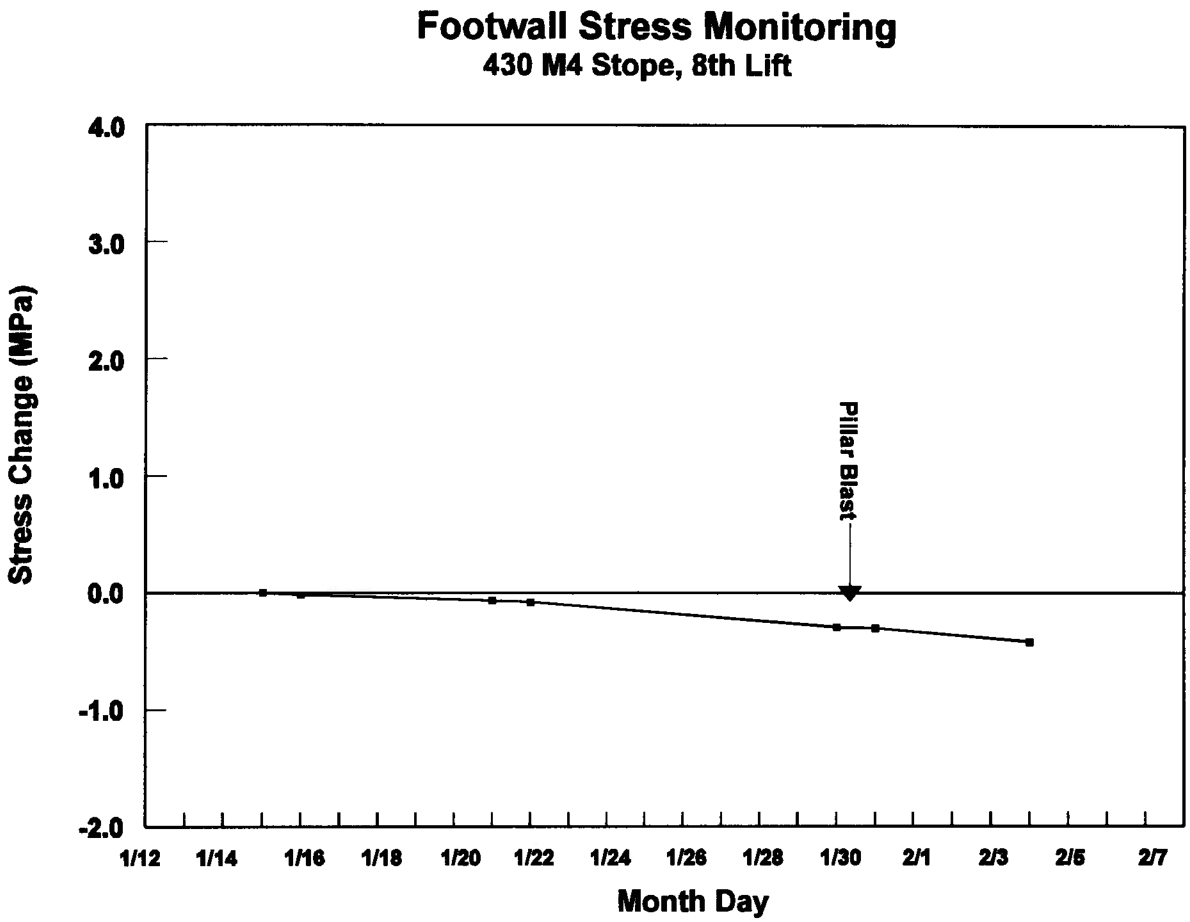


TRIPLE ANCHOR WIREFLEX EXTENSOMETER MONITORING HEADS (after Carter, 1990)	FIGURE 7.21

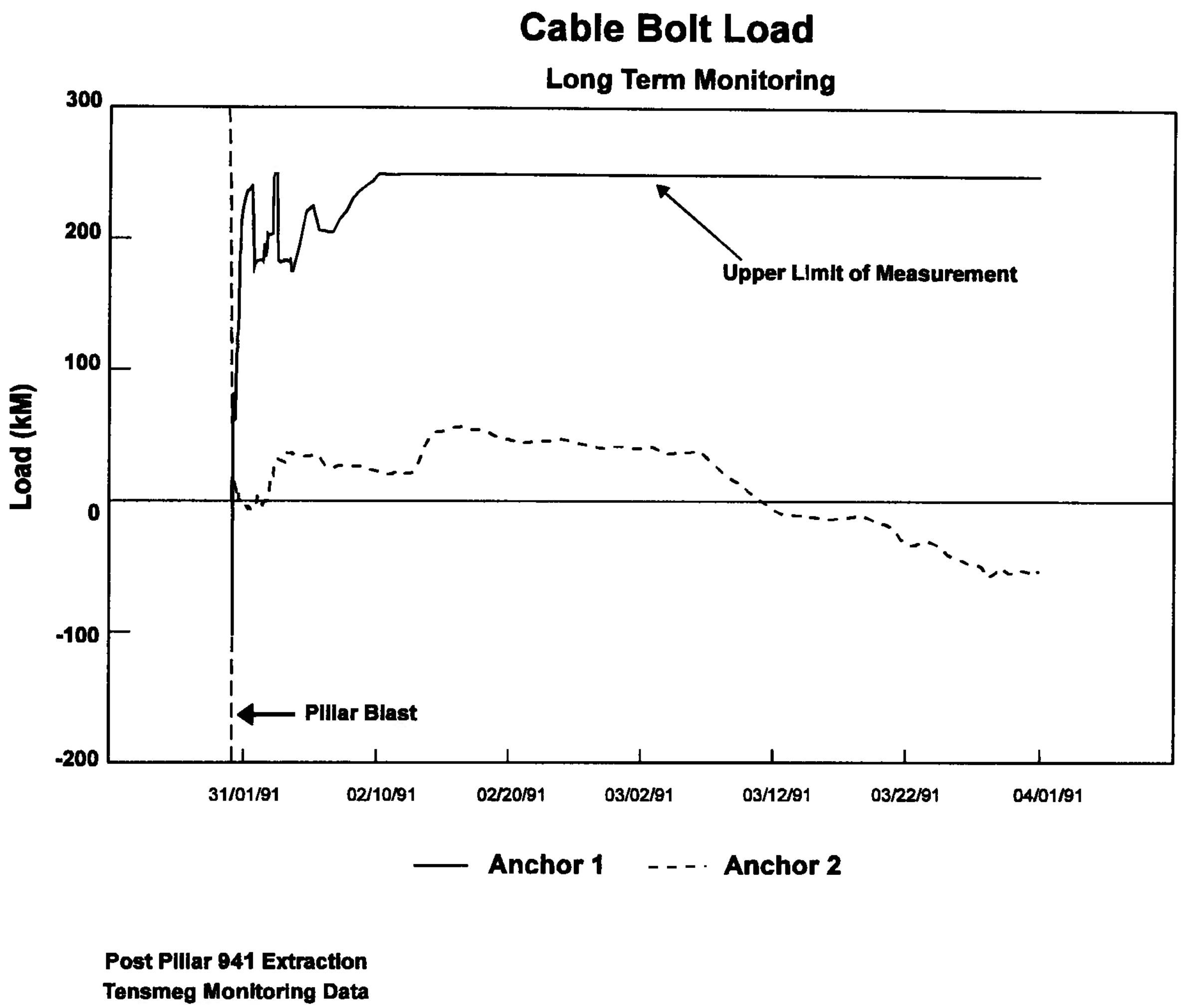


GROUND MOVEMENT MONITOR (GMM) MEASUREMENT OF BACK DISPLACEMENT

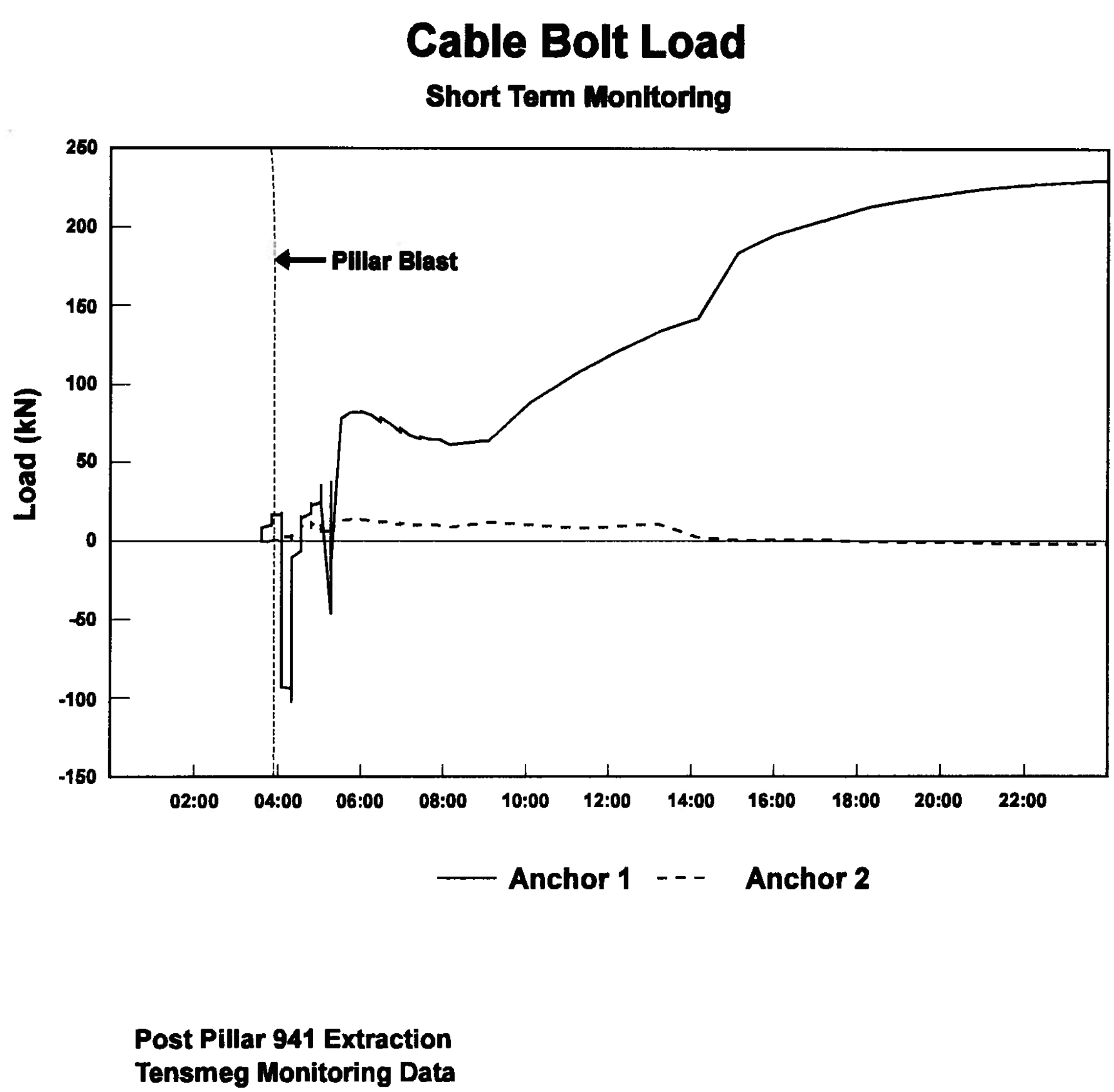
FIGURE 7.22



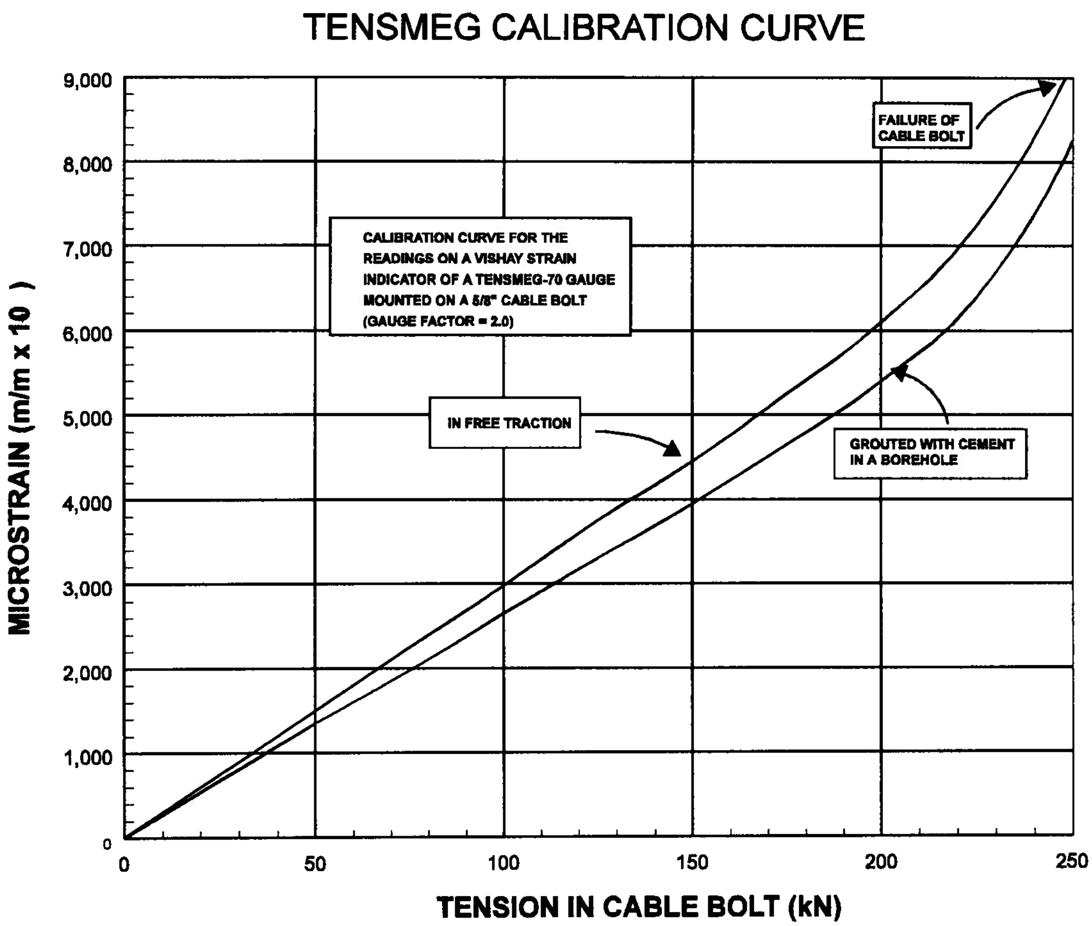
FOOTWALL STRESS CHANGE MONITORING	FIGURE 7.23



CABLE BOLT LOAD MONITORING USING TENSMEG STRAIN GAUGES - LONG TERM MONITORING	FIGURE 7.24

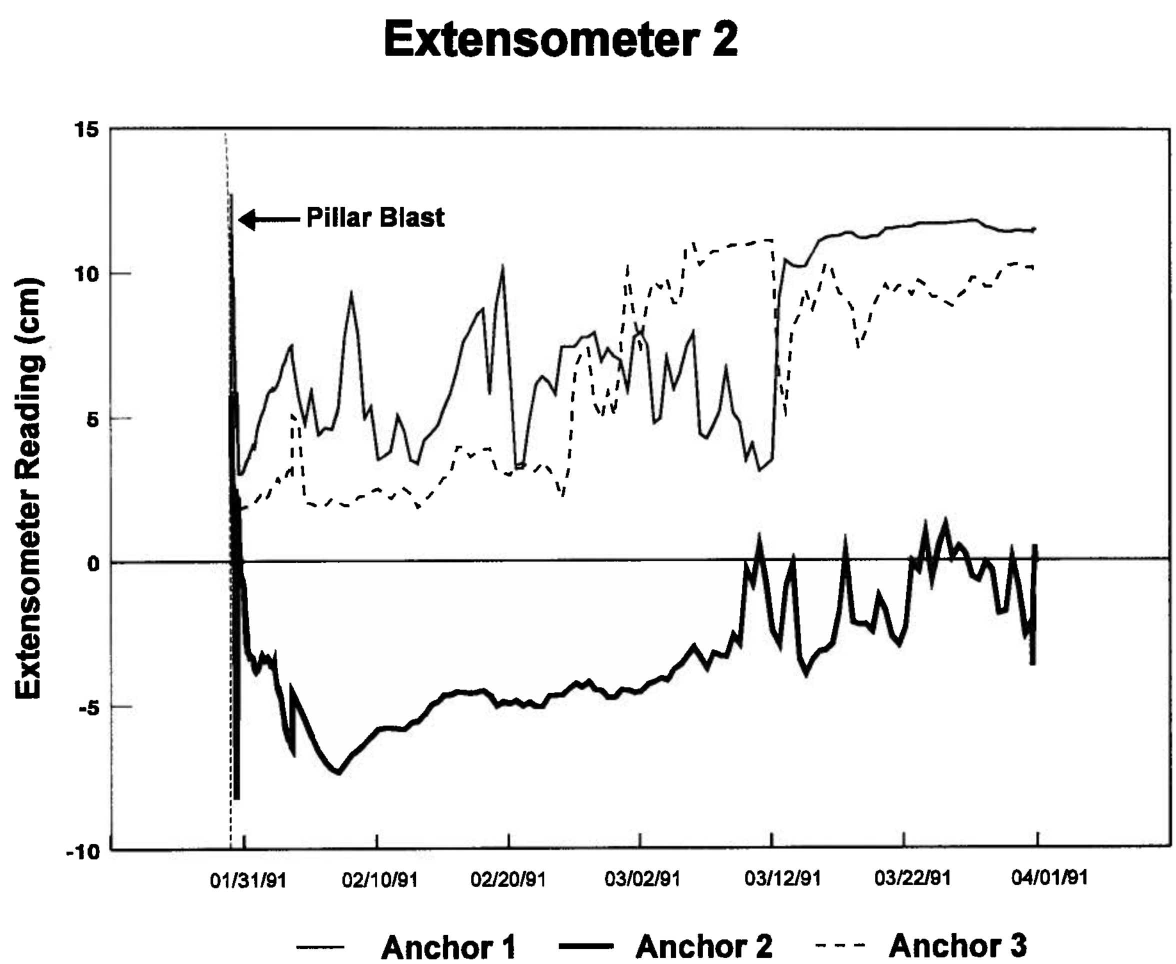


CABLE BOLT LOAD MONITORING USING TENSMEG STRAIN GAUGES - FIRST 24 HOURS	FIGURE 7.25



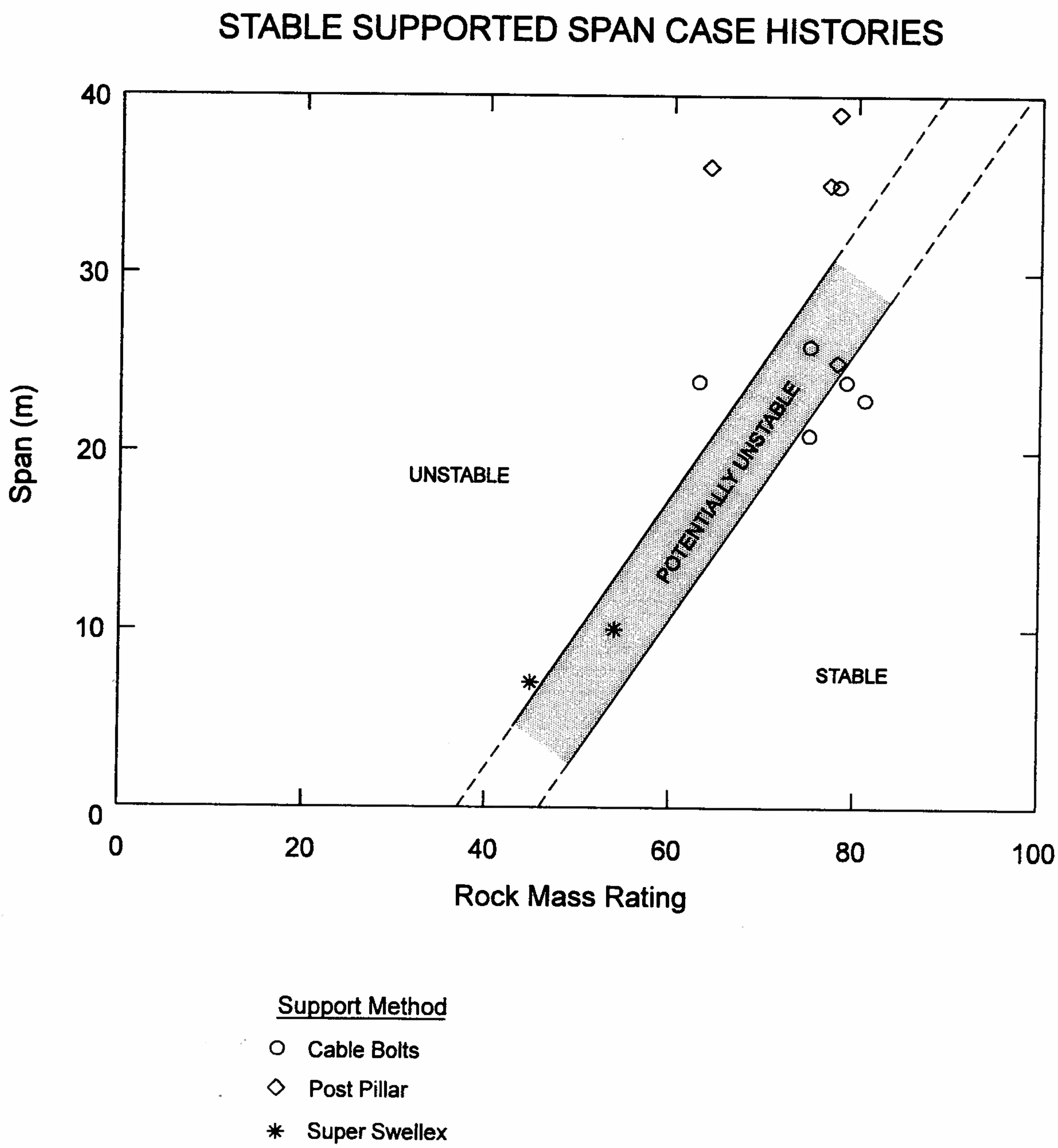
CALIBRATION CURVE FOR THE READINGS ON A VISHAYSTRAIN INDICATOR OF A TENSMEG-70 GAUGE MOUNTED ON A 5/8" CABLE BOLT
(GAUGE FACTOR - 2.0)

TENSMEG STRAIN GAUGE LOAD CALIBRATION CURVE	FIGURE 7.26



Post Pillar 941 Extraction
Extensometer Monitoring Data

EXTENSOMETER MONITORING OF BACK DISPLACEMENT	FIGURE 7.27



EFFECT OF CABLE BOLT AND POST PILLAR SUPPORT ON SPAN	FIGURE 7.28

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

Based on the results of a survey of cut and fill operations in Canada, it has been shown that there is not a consistent or well established method of designing spans for underground entry-type excavations. Most operators are relying on past experience with ground conditions at their operations as a guide to designing future stopes; however, this experience is not being systematically documented. Existing methods of span design, including beam theory, Voussoir block theory, structural failure analysis, empirical design methods, and numerical modeling methods have been reviewed in Chapter 3. Beam theory cannot be applied because it assumes an unjointed rock mass, which does not normally occur in underground metal mines. Voussoir block theory assumes that the back contains regularly spaced vertical joints which would be uncommon in underground metal mines but may be applicable in stratified deposits. General purpose empirical design methods have been proposed; however they have been developed from largely civil engineering case histories. Other empirical span design methods such as the Modified Stability Graph Method, have been developed from open-stope case histories and should not be used for the design of entry-type excavations.

Any design procedure for entry-type stopes must attempt to reconcile two conflicting goals. First, a high enough safety factors is required that recognizes the higher risks which accompany mining entry-type stopes. Secondly, the design must be balanced with the requirement for a relatively low factor of safety as compared to civil engineering excavations, recognizing the short term nature of the stopes and the costs associated with support. In this study, a database was established to develop an empirical span design technique specifically for entry-type excavations. 172 case histories were collected in which stability, Rock Mass Rating, span, and major structure was recorded. The Detour Lake Mine, a large underground gold mine in northern Ontario, where the observations were collected, is well suited as a data collection site because there is a good range of stope spans and rock qualities from which to collect the data.

An empirical span design chart, called the Stability Graph for Entry-Type Excavations was developed by plotting the span against Rock Mass Rating for the observed case histories. A statistical analysis of the data was carried out to define stable, potentially unstable, and unstable groups. A form of discriminant analysis which employs the generalized Mahalanobis Distance was used to create boundaries between the three groups.

The Stability Graph for Entry-Type Excavations is an easy to use method for designing spans in entry-type stopes but it is one which must be used with a reasonable degree of engineering judgment. To use the stability graph, the engineer must first establish the expected Rock Mass Rating for the stope being designed. Ideally, stope design should be carried out on a lift-by-lift basis so the RMR is measured on previous lifts of a stope. Alternatively, the RMR can be measured in nearby workings or estimated from geotechnical core logging. It is recommended that the design span should fall below the lower boundary of the potentially unstable zone. Sound engineering judgment must be applied to determine the degree to which the design span approaches the lower boundary of the potentially unstable zone. At Detour Lake Mine, it is possible to design spans close to this lower bound because geologists, front-line supervisor's, and stope leaders are given special training in recognizing and responding to hazardous ground conditions. Where a potentially hazardous condition arises, instrumentation is installed to detect and monitor instability. An effective reporting system is also important so that changes in rock quality can be quickly communicated to the engineering department and design changes can be made. The more confidence a mine has in its ability to quickly recognize and respond to changes in rock mass quality, the closer the design span can approach the potentially unstable zone boundary.

The Stability Graph for Entry-Type Excavations that has been proposed is a significant improvement over existing methods of predicting stable spans in cut and fill stopes. The potentially unstable zone on the graph recognizes the real-world uncertainty which exists between stable and unstable excavations. It is important to emphasize the limitations of the database which control its applicability. These are:

- The term span refers to spans with key block support only;
- The term stable refers to short term stability (approximately 3 months);
- The graph is considered applicable over the RMR range 40 to 85;
- High horizontal stresses are not assumed to be a factor controlling stability; and
- The graph applies to horizontal design surfaces.

Three-dimensional boundary element modeling and vibrating wire stress meters were used to assess the state of stress in the back of the cut and fill stopes. The modeling indicated the confining stress in the pillar to be near zero in the immediate back. Average σ_1 stresses were well below the unconfined compressive strength of the rock, indicating that high stresses were not a contributing factor to cases of instability recorded at Detour Lake Mine. Rather, the lack of confining stress in the immediate back results in key-block failure, wedge failure, or rock mass failure.

The role of support in increasing the allowable span before instability occurs has been briefly examined in this study. Yielding post pillars have been successful at Detour Lake Mine and elsewhere for increasing the span which can be mined using cut and fill methods. A trial support program was carried out at Detour Lake Mine to determine whether a concentrated pattern of cable bolts installed at the centre of the span would provide the same benefit as a post pillar. Based on the results of instrumentation readings and visual observations, the cable bolts were successful in maintaining stability in the stope after the post pillar was removed and the span increased to 35 metres. Previous case histories without support indicated that instability occurs when the span exceeds approximately 22 metres in the rock where the trial was located. Two additional cable bolt pillars have since been installed at Detour Lake Mine, and both have been successful in increasing the stable span beyond 30 metres.

It is suggested that the Stability Graph for Entry-Type Excavations be used as part of an integrated design approach which also uses analytical and numerical modeling techniques as described in Section 3.3 and illustrated on the flow chart in Figure 3.30. After obtaining intact rock strength parameters from laboratory testing, and rock mass characteristics from geotechnical mapping, the design engineer should determine whether there are any major geological structures which will control stability. If so, a wedge stability analysis should be done as described in Section 3.1.3, and if necessary, the span should be designed to limit the formation of a wedge. Alternatively, a means of supporting the wedge can be specified. If there is foliation parallel to the dip of the deposit, a quick assessment of chimney failure potential should be made using Equation 3.29. Numerical modeling would be required at this stage to determine the horizontal stress in the sill pillar for Equation 3.29, and it would also indicate if pillar crushing was becoming a potential failure mechanism. After structural failure along recognized structures has been accounted for in the design, the most likely mode of failure is rock mass failure which can be assessed using the Stability Graph for Entry-Type Excavations as described above.

The design of a stope should be an on-going activity. During and after excavation of the stope, the rock mass rating should be monitored on an on-going basis so the design can be adjusted accordingly. Visual monitoring, supplemented by instrumentation readings, should be used to assess stability of individual stopes on a regular basis.

The above span design procedure has been incorporated into the mine design approach at Detour Lake Mine, enabling the mine to maximize extraction of the orebody and improve profitability, while maintaining safe working conditions. In 1992, this approach contributed to the Detour Lake Mine being awarded the "Award of Excellence" by the Mines Accident Prevention Association of Ontario as the safest mine in Ontario. During this time, the mine was also successful in reducing the cost of gold production from $382/ounce to $340/ounce (US).

8.2 RECOMMENDATIONS

The ideal database for the empirical study described above would contain a uniform density of observations on the Stability Graph for Entry-Type Excavations; however, this was not possible under actual mining conditions. The concentration of data in some areas and the lack of it in others is a source of error in the statistical analysis presented in Chapter 6. Additional unstable and potentially unstable case histories would be desirable particularly at higher rock mass ratings. Potential sources for this data are the provincial mines inspection authorities who may keep records of major groundfall incidents.

The effect of post pillar support and cable bolt support has been examined briefly in this thesis. It has been shown how cable bolts have been used to increase the stable span at DLM from 22 to 35 metres. Additional case histories will be required before empirical guidelines can be established for confidently predicting the span which can be achieved for a given support. The author envisages that the Stability Graph for Entry-Type Excavations can eventually be combined with supported entry-type excavation case histories which would contain several design bands reflecting various support systems and intensities of support. This would be a worthwhile and interesting subject for future researchers.

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APPENDIX A

SURVEY OF CANADIAN CUT AND FILL MINES

QUESTIONNAIRE

PLACER DOME INC. - PAKALNIS AND ASSOCIATES - CANMET

NAME OF OPERATION: *FALCONBRIDGE LTD. (Overview of C&F operations)*
CONTACT/POSITION: *DOUG HANSON, SENIOR GEOMECHANICS ENGINEER*
PRODUCTION RATE FROM UNDERGROUND:
DEPTH OF MINING: *From 2000 to 4000 feet*
DIP OF OREBODY: *20° to 60°*
TYPE OF ORE: *Massive Sulphide and stringer ore*

1.

Mining Method	Percent of Total Production	Backfill (Y/N)
Overhand Cut and Fill	*20*	*Y*
Post Pillar Cut and Fill	*50*	*Y*
VCR Bench	*15*	*Y*
VCR Crater	*15*	*Y*

2. **How do you determine the spans (FW-HW) within your cut and fill system?**
Currently evaluating back spans utilizing Modified Mathews stability graph method and recommending support spacing and length. Will be incorporating the 3DEC-GC package which functions from the rock mass Q database.

3. **Have there been instances where full extraction of the ore was not possible by C/F because the span would be too wide? If so, what was the span? The rock mass rating? Percentage of ore left behind?**
Early 1980's Strathcona Mine had groundfalls with dimensions of m x m between 5 m x 5 m post pillars. Q'≈ 20-40
4-11% of ore left behind.

4. **Do you use post pillars? If so, how were they designed?**
Original concept based on 1973 design by K. Singh which essentially uses D. Hedley's formulas. Currently a post pillar project is underway involving a new empirical approach combined with instrumentation and FLAC, UDEC, and 3DEC sensitivity parameter analysis.

5. **Do you use cable bolts? If so, what is the pattern and type? How was the pattern designed?**
Currently some mines use the MRA cable bolt design manual but we are working with Noranda to improve on the design. General patterns are 1.5m x 1.5m single strand 6.0 m long (length varies). Again, 3DEC-GC program now being introduced will analyze necessity for support, and reanalysis will determine if support is adequate.

6. **What is your spacing between sill pillars? How thick are your sill pillars? How were they designed?**
Spacing: 60-70 m, Thickness: 15 m
Truthfully, a certain number of horizons are identified to achieve a certain production rate. However this results in high stressed burst prone sills as the mine matures.

7. **Have you experienced any cases of pillar failure?**
Horizontal sill pillars commonly burst at thicknesses ≥ 15 m. Vertical rib pillars experience stress fractures and vertical slabbing and hourglassing more from the secondary panel extraction side.

8. **Have you experienced any cases of back failure?**
Some back failure in overcuts(gravitational, key block, low stress) where preliminary rock support placed. Generally no failure where cable bolt support placed.

9. **What kind of monitoring instruments do you use in the back?**
Ground movement monitors (GMM's) standard and variable lengths. Sometimes use Geokon stress meters to examine stress arching across back.
In the pillars?
Horizontal GMM's, DISTOFOR (Telemax Extensometer) Geokon Stress meters, Rock Spys

10. **Have you experienced bursting of ground?**
Yes

11. **Would you describe failures to date as being structural or stress controlled?**
Bursting activity usually occurs first followed by debonding of key block wedges and gravitational failure.

ROCK MECHANICS DATABASE

Indicate with a ✓ which of the following parameters have been estimated at your mine.

Rock Strength Parameters	Unit Weight, γ	✓
	Elastic Modulus, ε	✓
	Poisson's Ratio, ν	✓
Stress Investigations	In-Situ Measurement	✓
	Photo-Elastic Moduli	
	Computer Modeling	✓
Laboratory Testing	Compressive Strength, σ_c	✓
	Tensile Strength, σ_t	✓
	Triaxial Strength, σ_c / σ_t	
	Shear Strength, τ	
	Failure Criterion	✓
Rock Mass Classification	RQD	✓
	NGI Rating	✓
	CSIR Rating	✓
	Laubscher MRMR Rating	
	Structural Mapping	✓
Monitoring	Multi-wire Extensometer	✓
	Boroscope Observation	
	Compression Pad	
	Closure Station	✓
	Leveling Survey Station	
	Piezometer	

Mining Method **	Stope Dip	Depth (feet)	Footwall			Hangingwall			Ore			Opening		Lift Height	Span FW-HW	Post Pillar Size	Drilling	Sill Pillar Size	Rock Mass Classification	
			Strength	Fracture Spacing	Joint Condition	Strength	Fracture Spacing	Joint Condition	Strength	Fracture Spacing	Joint Condition	Width	Length				Uppers=U Breast=B		NGI- Q	CSIR-RMR
Strathcona Cut and Fill	*20°-60°*	*2000-4000*	*S*	*W*		*S*	*W*		*M*	*W*	*M*	*40 feet*	*300 feet*	*10 feet*	*40-80 feet*	*16 x16 feet*	*B*	*60 feet*	*40*	
Fraser Cut and Fill	*20°-60°*	*2000-4500*	*S*	*W-C*	*M*	*W*	*W-C*	*M*	*S*	*C*	*S*	*100 feet*	*300 feet*	*5 metres*	*30 metres*	*5 x 5 metres*	*B*	*?*	*40*	
Lockerby VCR and Cut and Fill	*50°-90°*	*2000-4500*	*S*	*W*	*S*	*W*	*W-C*	*W*	*M*	*C*	*M*	*50 feet*	*400 feet*	*5 metres*	*20 metres*	*	*B*	*60-80 feet*	*Q'=20* *Q'=6*	*(granite)* *(dorite)*
Craig Cut and Fill	*50°-60°*	*2500-4300*	*S*	*W*	*S*	*S*	*W*	*M*	*M*	*W*	*M*	*30 metres*	*12 m*	*5 metres*	*30 metres*	*5 x 5 metres*	*B*	*?70 feet*	*Q'=20* *Q'=40* *Q'=10*	*(ore)* *(FW)* *(HW)*
Onaping Cut and Fill	*45°-50°*	*3000-4600*	*S*	*W*	*M*	*S*	*W*	*M*	*M*	*W*	*M*	*15 metres*	*60 metres*	*5 metres*	*15 metres*	*5 x 5 metres*	*B*	*?15 metres*	*Q'=20* *Q'=40* *Q'=10*	*(ore)* *(FW)* *(HW)*
East VCR	*70°*	*3000-6400*	*S*	*VW*	*S*	*W*	*W*	*W*	*S*	*W*	*S*	*15 metres*	*60 metres*	*N/A*	*15 metres*	*N/A*	*N/A*	*N/A*	*Q=30* *Q=4*	*(FW)* *(HW)*

* Cable Replacement in future
** List individual stopes

ROCK STRENGTH

Indicated by uniaxial compressive strength or as follows:

Weak	<6,000 psi
Moderate	6,000-15,000 psi
Strong	>15,000 psi

FRACTURE SPACING

	Fractures/metre	Fractures/foot	RQD
Very Close	>16	>5	0-20
Close	10-16	3-5	20-40
Wide	3-10	1-3	40-70
Very Wide	<3	<1	70-100

JOINT CONDITION

Weak:	Clean joint with smooth surface or filled with material whose strength is less than rock mass strength
Moderate:	Clean joint with rough surface
Strong:	Joint is filled with a material that is equal to or stronger than the rock mass strength

QUESTIONNAIRE

PLACER DOME INC. - PAKALNIS AND ASSOCIATES - CANMET

NAME OF OPERATION: *DOME MINE*
CONTACT/POSITION: *S. SELDON, ROCK MECHANICS ENGINEER*
PRODUCTION RATE FROM UNDERGROUND : *3300 Tons milled per day*
DEPTH OF MINING: *From Surface to*
DIP OF OREBODY: *Varies 0-90°*
TYPE OF ORE: *Gold associated with different host rocks and structural features*

1.

Mining Method	Percent of Total Production	Backfill (Y/N)
Narrow Vein	*2.5*	
Panel Cut & Fill	*28*	
Longhole	*26*	
Other	*20.5*	
Open Pit	*23*	

2. How do you determine the spans (FW-HW) within your cut and fill system?
Experience / Orebody Type

3. Have there been instances where full extraction of the ore was not possible by C/F because the span would be too wide? If so, what was the span? The rock mass rating? Percentage of ore left behind?
Yes - 30 feet , Q=8-15, Recovery=80%, Panel Stopes

4. Do you use post pillars? If so, how were they designed?
Yes, Study by McGill University (July, 1988) gives design criteria - not generally used to date.

5. Do you use cable bolts? If so, what is the pattern and type? How was the pattern designed?
Some cases - Normally 4' to 8' pattern
20' to 40' long, design from experience and analysis of strength/stress expectations

6. What is your spacing between sill pillars? How thick are your sill pillars? How were they designed?
130 feet between sill pillars, 20 feet thick.

7. Have you experienced any cases of pillar failure?
Yes, 5 tons to 500,000 tons. Large failures occur over long time periods in cave stopes.

8. Have you experienced any cases of back failure?
Yes, massive in cave stopes.

9. What kind of monitoring instruments do you use in the back?
Ground Movement Monitors, stress meters, slough meters

In the pillars?
Stress meters, extensometers

10. Have you experienced bursting of ground?
Yes, rare

11. Would you describe failures to date as being structural or stress controlled?
Stress failure is rare (overstress). Controlled by tensile failure and structure. Schistosity-major structural feature.

ROCK MECHANICS DATABASE

Indicate with a ✓ which of the following parameters have been estimated at your mine.

Rock Strength Parameters	Unit Weight, γ	✓
	Elastic Modulus, ε	✓
	Poisson's Ratio, ν	✓
Stress Investigations	In-Situ Measurement	✓
	Photo-Elastic Moduli	
	Computer Modeling	✓
Laboratory Testing	Compressive Strength, σ_c	✓
	Tensile Strength, σ_t	✓
	Triaxial Strength, σ_c / σ_t	✓
	Shear Strength, τ	
	Failure Criterion	✓
Rock Mass Classification	RQD	✓
	NGI Rating	✓
	CSIR Rating	✓
	Laubscher MRMR Rating	
	Structural Mapping	✓
Monitoring	Multi-wire Extensometer	✓
	Boroscope Observation	
	Compression Pad	✓
	Closure Station	✓
	Leveling Survey Station	
	Piezometer	

Mining Method **	Stope Dip	Depth (ft)	Footwall			Hangingwall			Ore			Opening		Lift Height	Span FW-HW	Post Pillar Size	Drilling	Sill Pillar Size	Rock Mass Classification	
			Strength	Fracture Spacing	Joint Condition	Strength	Fracture Spacing	Joint Condition	Strength	Fracture Spacing	Joint Condition	Width	Length				Uppers=U Breast=B		NGI- Q	CSIR-RMR
Panel	*60°*	*3000*	*M*	*C*	*M*	*M*	*C*	*M*	*M*	*C*	*M*	*30 feet*	*100 feet*	*25 feet*	*30 feet*	*10x10 feet*	*U+B*	*20 feet*	*±10*	*±50*
Narrow Vein	*70°*	*2500*	*M*	*C*	*M*	*M*	*C*	*M*	*M*	*C*	*M*	*10 feet*	*100 feet*	*16 feet*	*10 feet*		*B*	*20 feet*	*±15*	*±60*

ROCK STRENGTH

Indicated by uniaxial compressive strength or as follows:

Weak	<6,000 psi
Moderate	6,000-15,000 psi
Strong	>15,000 psi

** List individual stopes

FRACTURE SPACING

	Fractures/metre	Fractures/foot	RQD
Very Close	>16	>5	0-20
Close	10-16	3-5	20-40
Wide	3-10	1-3	40-70
Very Wide	<3	<1	70-100

JOINT CONDITION

Weak:	Clean joint with smooth surface or filled with material whose strength is less than rock mass strength
Moderate:	Clean joint with rough surface
Strong:	Joint is filled with a material that is equal to or stronger than the rock mass strength

QUESTIONNAIRE

PLACER DOME INC. - PAKALNIS AND ASSOCIATES - CANMET

NAME OF OPERATION: *TROUT LAKE MINE*
CONTACT/POSITION: *J. ROMANOWSKI, MINE ENGINEER*
PRODUCTION RATE FROM UNDERGROUND: *2500 Tons per day*
DEPTH OF MINING: *From 50 m to 400 m , potentially to 800 m*
DIP OF OREBODY: *50°-70°*
TYPE OF ORE: *Solid to disseminated sulphides within the altered zone*

1.

Mining Method	Percent of Total Production	Backfill (Y/N)
Cut and Fill	*80-100*	*Y*
Longhole Open Stoping	*0-20*	*N*

2. How do you determine the spans (FW-HW) within your cut and fill system?
It is determined by mineralization - full extraction

3. Have there been instances where full extraction of the ore was not possible by C/F because the span would be too wide? If so, what was the span? The rock mass rating? Percentage of ore left behind?
It has not happened so far.

4. Do you use post pillars? If so, how were they designed?
No.

5. Do you use cable bolts? If so, what is the pattern and type? How was the pattern designed?
Yes, 7 strand, 270K steel cables, installed in vertical holes drilled into the back (65 feet) and inclined (60°-80°) holes drilled into the HW (30'-55'). Basic pattern: 1.8m x 1.8m, length and dip of inclined HW cables depends on local conditions. Basic pattern was proposed by Golder Assoc. and was based on case histories rather than strictly designed. It is sometimes modified.

6. What is your spacing between sill pillars? How thick are your sill pillars? How were they designed?
Spacing: 60-65 metres
Thickness: approximately 10 metres
They were designed to contain the load of backfill over the maximum stope span.

7. Have you experienced any cases of pillar failure?
No

8. Have you experienced any cases of back failure?
Yes, Failure started always close to HW contact and frequently was controlled by natural jointing. It occurred within fractured, destressed zone in the back below the "pressure arch".

9. What kind of monitoring instruments do you use in the back?
None

In the pillars?
None

10. Have you experienced bursting of ground?
Not Yet

11. Would you describe failures to date as being structural or stress controlled?
Both. Most of them start in spots where we can expect stress concentration - so they are stress related. Structure of the back and particularly HW controls extent and volume of failure.

ROCK MECHANICS DATABASE

Indicate with a ✓ which of the following parameters have been estimated at your mine.

Rock Strength Parameters	Unit Weight, γ	✓
	Elastic Modulus, ε	✓
	Poisson's Ratio, ν	✓
Stress Investigations	In-Situ Measurement	
	Photo-Elastic Moduli	
	Computer Modeling	✓
Laboratory Testing	Compressive Strength, σ_c	✓
	Tensile Strength, σ_t	✓
	Triaxial Strength, σ_c / σ_t	
	Shear Strength, τ	
	Failure Criterion	
Rock Mass Classification	RQD	
	NGI Rating	✓
	CSIR Rating	
	Laubscher MRMR Rating	✓
	Structural Mapping	✓
Monitoring	Multi-wire Extensometer	
	Boroscope Observation	
	Compression Pad	
	Closure Station	
	Leveling Survey Station	
	Piezometer	

Mining Method **	Stope Dip	Depth (m)	Footwall			Hangingwall			Ore			Opening		Lift Height	Span FW-HW	Post Pillar Size	Drilling	Sill Pillar Size	Rock Mass Classification	
			Strength	Fracture Spacing	Joint Condition	Strength	Fracture Spacing	Joint Condition	Strength	Fracture Spacing	Joint Condition	Width	Length				Uppers=U Breast=B		NGI- Q	CSIR-RMR
Cut and Fill	*50°-70°*	*180 to 390*	*50-60 MPa*	*W*	*M-S*	*40-50 MPa*	*C-W*	*W-M*	*100-150 MPa*	*VW*	*M-S*	*up to 25 m*	*up to 250 m*	*5 m*	*up to 25 m*		*B*	*≈10m*	*35-40 (back) 3 HW/FW*	*87 (back) 40 HW/FW*

ROCK STRENGTH

Indicated by uniaxial compressive strength or as follows:

Weak	<6,000 psi
Moderate	6,000-15,000 psi
Strong	>15,000 psi

** List individual stopes

FRACTURE SPACING

	Fractures/metre	Fractures/foot	RQD
Very Close	>16	>5	0-20
Close	10-16	3-5	20-40
Wide	3-10	1-3	40-70
Very Wide	<3	<1	70-100

JOINT CONDITION

Weak:	Clean joint with smooth surface or filled with material whose strength is less than rock mass strength
Moderate:	Clean joint with rough surface
Strong:	Joint is filled with a material that is equal to or stronger than the rock mass strength

QUESTIONNAIRE

PLACER DOME INC. - PAKALNIS AND ASSOCIATES - CANMET

NAME OF OPERATION: *INCO LIMITED - ONTARIO OPERATIONS*
CONTACT/POSITION: *P.H. OLIVER/SENIOR SPECIALIST - ROCK MECHANICS*
PRODUCTION RATE FROM UNDERGROUND : *11,450,000 TONS - 1989*
DEPTH OF MINING: *From Surface to 7200 feet*
DIP OF OREBODY: *Flat to vertical - Normal case is for dips >55°*
TYPE OF ORE: *Massive to Disseminated sulphides*

1.

Mining Method	Percent of Total Production	Backfill (Y/N)
VRM	*46.3 (1989)*	*Y*
Blasthole	*21.0 (1989)*	*N*
SLC	*29.0 (1989)*	*N*
Cut & Fill	*3.7 (1989)*	*Y*

2. How do you determine the spans (FW-HW) within your cut and fill system?
For true ore widths less than 35 feet - mine full width longitudinal. For widths>35 feet mine with transverse stopes and ribs or with post pillars. Stoping spans range from25 to 40 feet depending on rock quality, mining depth, fill quality and stress state.

3. Have there been instances where full extraction of the ore was not possible by C/F because the span would be too wide? If so, what was the span? The rock mass rating? Percentage of ore left behind?
No. The effective ore widths have never exceeded the critical span which for our stress state and yielding pillar approach to cut and fill mining is believed to be in excess of 600 feet at depths in excess of 600 feet.

4. Do you use post pillars? If so, how were they designed?
Yes, Empirically. Yielding pillars yield and therefore have safety factors less than 1.0. The sizing of pillars and stope spans is a function of the rock quality, the horizontal stress state, and the fill quality. Overly stiff pillars can lead to high back s stresses. Overly soft pillars can permit tensile conditions to develop in the stope backs. Their stiffness also influences the timing and the nature of the failure of the sill pillar in wide orebodies.

5. Do you use cable bolts? If so, what is the pattern and type? How was the pattern designed?
Yes, for specific situations. For fall of ground or wedge situations, cable strength x No. of cables > weight, and cable spacing not to exceed point where the grout bond strength to cables exceeds 500 psi. (Grout w/c ratio <0.45) For pre-pinning, spacing governed by 500 psi bond strength =7 ft x 7 ft for single cables, 10 ft x 10 ft for double cables

6. What is your spacing between sill pillars? How thick are your sill pillars? How were they designed?
200 foot spacing. Nil thickness, they are mined out. They were mined through failure between 30 feet and 40 feet thick. They were the recovered by VC&F.

7. Have you experienced any cases of pillar failure?
Yes, All ribs, post, and sill pillars fail. Preferably by yield. Rib and post pillars in the 30 ft to 50 ft mining height region. Sill pillars anywhere from 110 feet to 40 feet thick.

8. Have you experienced any cases of back failure?
Who has not? There have been localized structurally bound groundfalls and material displaced by rockbursts. There have been no massive collapses such as would occur if the critical span was exceeded.

9. What kind of monitoring instruments do you use in the back?
No routine instrumentation other than visual observations. The adequacy of the design becomes apparent once the rib or post pillars yield, through observation of the back conditions. There is normally little that can be done if the pillars are too soft
In the pillars?
for the stope spans and the back goes tensile

10. Have you experienced bursting of ground?
Yes

11. Would you describe failures to date as being structural or stress controlled?
Both
Comment: There seems to be too little emphasis placed on the role of horizontal stress state as it influences stable spans.

ROCK MECHANICS DATABASE

Indicate with a ✓ which of the following parameters have been estimated at your mine.
**Not necessarily routinely for all applications*

Rock Strength Parameters	Unit Weight, γ	✓
	Elastic Modulus, ε	✓
	Poisson's Ratio, ν	✓
Stress Investigations	In-Situ Measurement	✓
	Photo-Elastic Moduli	
	Computer Modeling	✓
Laboratory Testing	Compressive Strength, σ_c	✓
	Tensile Strength, σ_t	✓
	Triaxial Strength, $\sigma_c/\ \sigma_t$	✓
	Shear Strength, τ	
	Failure Criterion	
Rock Mass Classification	RQD	✓
	NGI Rating	✓
	CSIR Rating	✓
	Laubscher MRMR Rating	
	Structural Mapping	✓
Monitoring	Multi-wire Extensometer	✓
	Boroscope Observation	
	Compression Pad	
	Closure Station	✓
	Leveling Survey Station	
	Piezometer	

Mining Method **	Stope Dip	Depth (m)	Footwall			Hangingwall			Ore			Opening		Lift Height	Span FW-HW	Post Pillar Size	Drilling	Sill Pillar Size	Rock Mass Classification	
			Strength	Fracture Spacing	Joint Condition	Strength	Fracture Spacing	Joint Condition	Strength	Fracture Spacing	Joint Condition	Width	Length				Uppers=U Breast=B		NGI- Q	CSIR-RMR
VRM and C&F	*>55°*	*to 7200 feet*	*S*	*W*	*M*	*S*	*W*	*M*	*S*	*W*	*M*	*25-60 feet*	*40-550 feet*	*12 feet C&F 50-300 ft VRM*	*20 to 550 ft*	*20 feet when used*	*U & B for C&F*	*mined*		*65-75*
BLASTHOLE	*>35°*		*S*	*W*	*M*	*S*	*W*	*M*	*S*	*W*	*M*	*40-70 feet*	*70-200 feet*	*140-340 ft*	*20 to 550 ft*			*40-80 feet*		*65-75*
S.L.C.	*>55°*		*S*	*W*	*M*	*S*	*W*	*M*	*S*	*W*	*M*	*+30 feet*	*Ore Strike*	*35 ft sub -levels*	*20 to 550 ft*		*FAN*			*65-75*

ROCK STRENGTH

Indicated by uniaxial compressive strength or as follows:

Weak	<6,000 psi
Moderate	6,000-15,000 psi
Strong	>15,000 psi

** List individual stopes

FRACTURE SPACING

	Fractures/metre	Fractures/foot	RQD
Very Close	>16	>5	0-20
Close	10-16	3-5	20-40
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JOINT CONDITION

Weak:	Clean joint with smooth surface or filled with material whose strength is less than rock mass strength
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QUESTIONNAIRE

PLACER DOME INC. - PAKALNIS AND ASSOCIATES - CANMET

NAME OF OPERATION: *WESTMIN MINE*
CONTACT/POSITION: *MICHAEL CULLEN, GEOTECHNICAL ENGINEER*
PRODUCTION RATE FROM UNDERGROUND : *3650 TONS PER DAY*
DEPTH OF MINING: *FROM 400 m TO 600 m*
DIP OF OREBODY: *60° to Subhorizontal*
TYPE OF ORE: *Massive Sulphides*

1.

Mining Method	Percent of Total Production	Backfill (Y/N)
Mechanized Cut and Fill	*75*	*Y*
Longhole	*25*	*Y*

2. **How do you determine the spans (FW-HW) within your cut and fill system?**
Determined by structural stability, i.e. beneath the faulted hangingwall instability occurs at approximately 6-8 metres, In massive sulfide instability occurs at approximately 10 -15m.

3. **Have there been instances where full extraction of the ore was not possible by C/F because the span would be too wide? If so, what was the span? The rock mass rating? Percentage of ore left behind?**
Stopes greater than 10 m span (most) mined by post pillar. In the past, 10 m stopes running strike length beneath the faulted hangingwall utilized a two pass system. 86% extraction in post pillar stopes. 100% extraction in double pass
Q<1 for HW fault, Q>10 for massive sulfides

4. **Do you use post pillars? If so, how were they designed?**
Yes, Tributary area/ Hedley pillar strength / Exposed Span

5. **Do you use cable bolts? If so, what is the pattern and type? How was the pattern designed?**
Cable Type: 0.6" diameter, 7 strand, 2 cables
Pattern: 2m x 2 m in waste, 2m x1.5m in ore
Design: Gravity Load, Experience, and practice at other mines

6. **What is your spacing between sill pillars? How thick are your sill pillars? How were they designed?**
N/A

7. **Have you experienced any cases of pillar failure?**
N/A

8. **Have you experienced any cases of back failure?**
Back failures common in areas of faulting and geological contacts.

9. **What kind of monitoring instruments do you use in the back?**
Multi-point extensometers

In the pillars?
Vibrating wire strain gauges

10. **Have you experienced bursting of ground?**
No

11. **Would you describe failures to date as being structural or stress controlled?**
Predominantly structurally controlled, however, stress related failures are increasing as extraction ratio increases.

ROCK MECHANICS DATABASE

Indicate with a ✓ which of the following parameters have been estimated at your mine.

Rock Strength Parameters	Unit Weight, γ	✓
	Elastic Modulus, ε	✓
	Poisson's Ratio, ν	✓
Stress Investigations	In-Situ Measurement	✓
	Photo-Elastic Moduli	
	Computer Modeling	
Laboratory Testing	Compressive Strength, σ_c	✓
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	Triaxial Strength, $\sigma_c/\ \sigma_t$	
	Shear Strength, τ	✓
	Failure Criterion	
Rock Mass Classification	RQD	✓
	NGI Rating	✓
	CSIR Rating	✓
	Laubscher MRMR Rating	✓
	Structural Mapping	✓
Monitoring	Multi-wire Extensometer	✓
	Boroscope Observation	
	Compression Pad	
	Closure Station	
	Leveling Survey Station	
	Piezometer	

Mining Method **	Stope Dip	Depth (m)	Footwall			Hangingwall			Ore			Opening		Lift Height	Span FW-HW	Post Pillar Size	Drilling	Sill Pillar Size	Rock Mass Classification	
			Strength	Fracture Spacing	Joint Condition	Strength	Fracture Spacing	Joint Condition	Strength	Fracture Spacing	Joint Condition	Width	Height				Uppers=U Breast=B		NGI- Q	CSIR-RMR
Post Pillar Cut and Fill	60°	550	M	C-W	W	W	VC	W	S	VW	M-S	6-8 (m)	6-8 (m)	4 m	35 m	5.5X5.5 m	B		10	
Post Pillar Cut and Fill	20°	510	M	C-W	M	M	C	M	S	VW	S	8 m	8 m	4 m	65 m	5.5X5.5 m	B		20	
Post Pillar Cut and Fill	60°	550	M	C	M	W	VC	W	S	W	M-S	6-8 (m)	6-8 (m)	4 m	30 m	5.5X5.5 m	B		10	

** List individual stopes

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