# PREDICTION OF STABLE EXCAVATION SPANS
# FOR
# MINING AT DEPTHS BELOW
# 1,000 METERS IN HARD ROCK

MARCH 1980





Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

Copy 6 of 6
P.H.

**Golder Associates**
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

# PREDICTION OF STABLE EXCAVATION SPANS FOR MINING AT DEPTHS BELOW 1,000 METERS IN HARD ROCK

MARCH 1980

MNG-23943

Supported by the Canada Center for Mineral and Energy Technology (CANMET) Department of Energy Mines and Resources

DSS Serial No. OSQ80-00081
DSS File No. 17SQ.23440-0-9020

DISTRIBUTION:

5 copies - Canada Center for Mineral & Energy Technology
Elliott Lake, Ontario

2 copies - Golder Associates
Vancouver, British Columbia

April 1981



802-1571

134 pp.

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd)

## LIST OF APPENDICES



## LIST OF TABLES

## LIST OF FIGURES

# 1.0 INTRODUCTION

## 1.1 General

The open stopes referred to in this study are large openings produced by non-entry mining methods (Figure 1.1). They must remain open until all the ore is extracted, however some dilution resulting from wall or back failure can be tolerated. Open stoping has been practiced for many years, but the economics of the method have become increasingly attractive with improvements in large hole drilling, blasting and support practices. In-the-hole drills can now produce straight blast holes up to 60 m long, substantially reducing the amount of development and drilling required to prepare a stope for production.

Total ore recovery is usually achieved in two or three stages of mining. Primary stopes are mined first, leaving pillars for temporary ground support and subsequent recovery if economic. The percentage of ore recovered in the first stage of mining varies, depending on economics, pillar recovery strategy and geotechnical considerations. Primary stoping at depth cannot be carried out safely over the whole orebody without making pillar recovery difficult in the future. Therefore, the strategy for pillar recovery must be incorporated into the overall mining plans at an early stage. The placement of fill in completed open stopes, prior to the commencement of adjacent pillar recovery, is a key element in the sequence of activities.

At depths of 1,000 m, stress concentrations in pillars may be large enough to cause inelastic behaviour of the rock and yielding along joint surfaces. However, fill placed in completed stopes controls convergence and limits the regional zone of ground movement. If the resultant destressing of pillars and adjacent areas occurs without violence, then recovery conditions might well be improved. Monitoring of deformations in stope walls and pillars is necessary to help plan the sequence of mining activities in destressed ground.

Improvements in ground control have resulted from the use of slot and mass blasting techniques, cemented fill and long cable dowels. These are major factors contributing to the success of non-entry mining methods at depth. However, everything rests on being able to design a stable slot or primary open stope and the key question is "What are the geotechnical factors that must be considered in this regard"?

## 1.2 Scope

This report describes a study carried out to determine the information that is required to predict stable spans for open stopes at mining depths below 1,000 m. The study was commissioned by the Department of Energy Mines and Resources and was carried out under the direction of Dr. G. Herget of the Mining Research Laboratory at Elliott Lake. The project was completed during the period October 1980 to March 1981. Meetings to define the project and review progress were held on September 25th, 1980 in Calgary and February 3rd, 1981 in Vancouver.

Rock mass classification systems have been drawn up by various groups to provide objective comparisons between different rock types. The purpose of this study was to determine whether an empirical relationship existed between rock mass properties, mining depth and maximum stable open stope spans. The approach taken was to examine empirical relationships derived from rock mass classification systems described in the literature and assess their application to the design of open stopes. In addition, three mines were visited to obtain data on rock mass properties and open stope geometry and assess stability conditions. The data was obtained from:

- Heath Steele Mine, Newcastle, New Brunswick
- Geco Mine, Manitouwadge, Ontario
- CSA Mine, Cobar, Australia

Information on open stoping at other mines has been obtained from the literature.

Data collected during the site visits was used to modify an existing rock mass classification system to more closely reflect the requirements of open stope design. An empirical relationship between rock mass properties, mining depth and stope dimensions was determined, but this should be verified by the collection of additional data.

The report also includes information on the cost of collecting data on associated rock mass properties, and the cost of installing support in stopes, drill drifts and draw points.

## 1.3 Conclusions

(1) Rock mass classification systems are empirical but they offer the potential for useful application in the estimation of requirements for ground support.

(2) Most of the classification systems reviewed were oriented towards the prediction of support requirements for tunnels and permanent structures; as such, they reflect the experience of the people who developed them.

(3) Adjustments to the CSIR classification rating have been proposed by Laubscher and Taylor (1976) to assess (among other things) the feasibility of open stoping. The adjustments proposed are sound conceptually and merit further refinement through back analysis of data obtained from a variety of locations.

(4) The NGI classification system describes the rock mass well and has been tested in a wide range of geological conditions. However, it has shortcomings as a basis for the empirical design of open stopes. In particular, the stress reduction factor is too crudely defined to predict the stability of roof and wall exposures at depth.

(5) Adjustments to the NGI index considered necessary (to more accurately predict the stability of open stopes at depth) include:

   (a) modifications to the stress reduction factor to permit the assessment of the ratio of intact rock strength to induced stress acting parallel to exposed surfaces.

   (b) incorporation of additional factors to reflect the effects of persistent structure paralleling or intersecting exposed surfaces and unfavourable inclination of those surfaces.

(6) Adjustments to the NGI index to account for rock mass quality, the state of induced stress and the orientation of exposed surfaces should provide an empirical base for the estimation of a stability index or stability number.

(7) The analysis of roof and wall stability should incorporate a shape factor to reflect the two-way spanning characteristics of exposed surfaces in large excavations. A shape factor considered applicable has been defined by Laubscher and Taylor (1976) as the ratio of surface area to perimeter.

(8) The combination of a high stability number and a low value for the shape factor should reflect stable exposures, while the converse should reflect instability.

(9) A plot of stability number versus shape factor using data given in Table V-1, Appendix V, can be used to postulate zones defined as stable, unstable and caved (Figure 4.1).

(10) The stability zones defined in the text and depicted in Figure 4.2 are considered to be sound conceptually, but insufficient data was collected to confirm them. Additional site visits are needed to obtain confirmatory data (or otherwise).

(11) The relationship postulated deals with the analysis of the surfaces bounding single openings only. Numerical analysis techniques should be used to estimate the induced stresses in pillars which result from the interaction of multiple and adjacent openings.

(12) Virgin stress should be measured at deep sites because of the wide scatter about the mean of estimated values and the importance of the stress factor relating to the stability of excavations at depth.

(13) The additional cost of data collection for a mine contemplating open stoping at depth are considered to be trivial on a cost per tonne mined basis.

(14) The staff required to undertake rock mechanics activities at a mine varies according to the size, production rate, depth of workings, etc. However, it is postulated that a basic rock mechanics staff for a mine employing 150-200 men or more underground should comprise a rock mechanics engineer trained to oversee ground control practices and a Structural Geologist and/or Senior Technician trained to oversee structural and rock strength assessment.

## 2.0 ROCK MASS CLASSIFICATION SYSTEMS

Rock mass classification is a useful means of quantifying the properties of a rock mass for the purpose of predicting stable spans and support requirements for openings in different environments. A number of classification systems have been developed over the past 35 years and these have become progressively more refined with the passage of time (Hoek and Brown, 1980). Most of the systems described in the literature are applicable to the design of tunnels at shallow depths, i.e. less than about 500 m, and only one applies specifically to mining. The following is a brief review of existing classification systems and a discussion of the relative merits of each with respect to the design of open stopes at depths in excess of 1,000 m.

### 2.1 Terzaghi's Rock Load Classification

In 1946 Terzaghi proposed a simple rock classification system for use in estimating the loads to be supported by steel arches in tunnels. He described various types of ground and, based upon his experience in steel-supported railroad tunnels in the Alps, he assigned ranges of rock loads for various ground conditions. In estimating the rock loads, Terzaghi emphasized the importance of carrying out a geological survey, a major objective of which is to obtain information on the defects in the rock mass, e.g. joints and bedding planes. He stated that "the type and intensity of the defects may be much more important than the type of rock." He proposed several classes of rock mass which range from intact rock containing no fractures to swelling rock which advances into the tunnel on account of expansion.

Terzaghi's classification method cannot be used for open stope design because it relates to small, fully supported openings at shallow depth.

## 2.2 Stini and Lauffer's Classification

Stini (1950) proposed a rock mass classification which emphasizes the importance of structural defects in the rock mass. He also discussed many of the adverse conditions encountered in tunnelling.

Lauffer's (1958) system is concerned with the stand-up time and active span of tunnels. The stand-up time is defined as the length of time which an underground opening will stand unsupported after excavation and barring down, while the active span is defined as the largest unsupported span in the tunnel section between the face and the supports. The stand-up time is related to rock mass classes which generally correspond to those proposed by Terzaghi.

Both the active span and the stand-up time are important factors in open stope design and Lauffer's work shows that they can be related to rock mass classification. The major drawback to the direct use of Lauffer's chart for open stope design is that no account is taken of the effect of depth of working.

## 2.3 Deere's Rock Quality Designation

In 1964 Deere proposed a quantitative index of rock mass quality based upon core recovery from diamond drilling. This Rock Quality Designation (RQD) has come to be very widely used and is particularly useful in classifying rock masses for the selection of tunnel support. Deere proposed the following formula for calculating the value of RQD:

$$R.Q.D.\ (\%) = 100 \times \frac{\text{Length of core in pieces} > 100\text{ mm}}{\text{Length of borehole}}$$

An attempt was also made to relate the RQD values to Terzaghi's classification and it was found that a reasonable correlation existed between the two methods for steel-supported tunnels, but not for tunnels supported by rock bolts.

The major limitation of using RQD alone as a classification method is that it considers only one factor, the degree of fracturing in the rock mass. No account is taken of the effect of fracture orientation and stress conditions.

## 2.4 Influence of Fracture Infillings

Brekke and Howard (1972) point out that it is just as important, often more important, to classify fractures according to character as it is to note their scale parameters. They go on to discuss seven groups of discontinuity infillings which have a significant influence upon the engineering behaviour of the rock mass containing these discontinuities. These infillings range from joints healed with quartz or calcite to sand-like, cohesionless material.

Although this list does not constitute a rock mass classification, it does show the range of infilling that can occur and the stability problems that they can cause.

## 2.5 Patching and Coates' (1968) Classification

Patching and Coates (1968) used three parameters to describe the rock substance and two parameters to describe the rock mass as follows:

(a) geological name

(b) strength of the intact rock

(c) deformation characteristics, i.e., elastic or yielding

(d) gross homogeneity, i.e., massive or layered

(e) continuity, i.e., solid, blocky, slabby, broken or loose, tight.

This classification is very simple to use and is a useful means of comparing the quality of different rock masses. For design purposes it does not provide information on the strength and orientation of the fractures.

## 2.6 CSIR Classification

From discussion of the preceding classification systems, it is clear that no single, simple index is adequate as an indicator of the complex behaviour of the rock mass surrounding a stope. Consequently, some combination of factors such as RQD and the influence of infilling appears to be necessary. One such classification system developed for the South African Council for Scientific and Industrial Research (CSIR) uses five parameters (Bieniawski, 1974) as follows:

(a) Strength of intact rock, i.e., uniaxial compressive strength

(b) Rock Quality Designation

(c) Spacing of fractures

(d) Condition of fractures, i.e., frictional properties and continuity

(e) Ground water conditions.

Each of these parameters is given an importance rating for the particular situation. The total rating is an indicator of rock quality and ranges from less than 25 (worst rock conditions) to 100 (best rock conditions). The rating is then adjusted to account for the influence of the orientation of fractures on stability. This adjusted rating is used to classify the rock into one of five classes which are empirically related to the unsupported span and stand-up time of development openings.

The CSIR classification system was tested in this study and was found to be easy to use and gave consistent results. However, its drawbacks, for open stope design, are that it does not account for the effects of stress and of joints with continuous lengths of tens of meters. The adjustment for joint orientations gives an indication of the required reduction in rating for very continuous joints.

## 2.7 Laubscher and Taylor's (1976) Classification

This classification is based on the CSIR method but has been modified for use in mining and applied extensively at an asbestos mine in Zimbabwe. Modifications that account for the following factors have been made:

(a) The combined effect of a number of joint sets with different spacings.

(b) The shape, roughness and infilling of the fractures.

(c) Weathering, if it is likely to occur within the life of the excavation.

(d) Field sresses and stresses induced by the presence of adjacent openings.

(e) The orientation of fractures relative to the stope walls.

(f) Damage to the rock as a result of blasting.

The adjusted ratings have been used by Laubscher and Taylor (1976) to determine support requirements, cavability, angles of cave, pit slope angles and the feasibility of open stoping. The use of the classification system for this number of applications shows its value when employed by experienced personnel who have been able to test the system extensively in the field. However, the case studies presented deal with the assessment of support required for development openings. Only passing reference is made to applications related to the feasibility of open stoping. In this regard, some of the adjustments advocated rely heavily on extensive experience in the field, particularly those related to joint orientations, field and induced stresses and abutment stresses.

## 2.8 NGI Tunnelling Quality Index

On the basis of evaluation of a large number of case histories of underground excavation stability, Barton, Lien and Lunde (1974) of the Norwegian Geotechnical Institute (NGI) proposed an index for the determination of the tunnelling quality of a rock mass (Appendix I). The numerical value of the rock mass quality index Q is given by:

$$Q = \frac{RQD}{(Jn)} \times \frac{(Jr)}{(Ja)} \times \frac{(Jw)}{SRF}$$

where: RQD = Rock Quality Designation.

Jn = Joint set number (number of sets of fractures).

Jr = Joint roughness number (shape of fracture surfaces).

Ja = Joint alteration number (approximates the friction angle of joint surfaces).

Jw = Joint water reduction factor.

SRF = Stress reduction factor (stress conditions and the loosening of the rock mass).

Each of these factors is given an importance rating for the particular situation and the index Q ranges from 0.001 (exceptionally poor rock) to 1,000 (exceptionally good rock).

Knowing Q, the allowable span or excavation height, divided by the excavation support ratio (ESR) can then be selected for a given support requirement. The excavation support ratio "reflects construction practice in that the degree of safety and support required is determined by the purpose of the excavation, the presence of personnel, machinery, etc.". An ESR of 3-5 is suggested for temporary mine openings versus 1 for civil engineering type openings such as power stations, etc. The ESR is analogous to an inverse factor of safety to account for the use of the opening.

## 2.9 Application of Classification Systems to Open Stope Design

Laubscher and Taylor (1976) note that the objective of classifying rock masses is to assign a value and not a vague descriptive term to the rock mass. The classification should give an in situ rating of the rock mass which can be adjusted so that support requirements and the stability of underground excavations can be assessed. The key words are "stability of underground excavations" and the factors used in the various classification systems are summarized in Appendix II with this in mind.

The system advocated by Laubscher and Taylor (1976), based on the CSIR classification system, is the most comprehensive and "provides a useful guide as to whether an open stope and pillar recovery method can be employed". Barton (1977) has analyzed data obtained from the CSA Mine, Cobar in terms of both the NGI and CSIR indices. He concludes that although these classification systems are empirical and require further assessment, nevertheless, they offer the potential for useful application in the estimation of operational requirements for ground support. "Additional test cases of wide stope-type openings are urgently required".

Case history data for this study was gathered to conform to the requirements of both the CSIR and NGI systems of classification. Instructions for the gathering of data in the field are quite simple and with respect to the NGI system, they are unambiguous. However, it was decided at the outset that the NGI system would be used as a basis for development related to open stope design. This decision was based mainly on the background and experience of the authors and does not mean that the NGI system is considered to be superior to the CSIR system of rock mass classification. Both systems are empirical and reflect the experience of the people who designed them related to their specific objectives. No data or facts were found during the course of the study to warrant a review of the decision to use the NGI system of rock mass classification as a basis for the design of open stopes at depth. However, the index has quite a few shortcomings and these are discussed further.

The method of support/span selection is an attempt to quantify case example experience, but care must be exercised when circumstances differ. For example, the charts presented in Appendix I are valid for single openings only, as is most often the case for underground civil structures. For mine openings, particularly production openings, this is rarely the case. To illustrate the influence of ESR on allowable span, assume an excavation is to be developed in rock having an index Q of 40 with a range from 10-120. For a temporary mine opening where the ESR is 3.5, a span of 103 ft. with a range from 57.4 ft. to 160 ft. is indicated. Selecting an ESR of 1.6 for a permanent opening, the allowable span drops to 47 ft. and the range drops proportionately.

Consider the effect of RQD on span as interpreted by Barton et al (1974). When all other factors remain equal, the allowable span varies as (RQD): 0.39, that is, halving the RQD results in a 24 per cent reduction in allowable span. However, for the RQD to so drop, other factors will also change, thereby reducing the allowable span further.

The stress reduction factor, SRF, accounts for the influence of the pre-existing or virgin stresses on the selection of dimensions, and this parameter is high for both very low and very high stresses. Q varies inversely with SRF, indicating that small spans are required when the stresses are small or high. This reflects the possibility of block fall-out for low stresses and "rock bursting" for high stresses. Both cases can exist in deep mining but stresses will be relieved by adjacent mining. Hence the sequence of mining will influence the selection of safe spans.

No account is taken of joint orientation although this factor is discussed by Barton, et al (1974). For many of their case examples, the excavation was favourably orientated relative to the joint structure; hence their case examples are insensitive to this parameter. In many mining situations, the joint orientation most favourable for roof design is unfavourable for pillar stability; hence a compromise must be reached. Another omission in their classification system is the influence of blasting on support, although the influence can be included in the ESR factor. Hence for multiple openings in mining where blasting can occur nearby, a lower ESR should be used.

The degree of fracturing of the rock mass and the strength of these fractures is well described in the first four factors. Also, the range of values for each of these factors covers the rock properties and can be understood by engineers or geologists with a basic training in rock mechanics. As the NGI system has been tested in a wide range of geological conditions, it was selected for use with the following provisos:

(a) the stress reduction factor (SRF) should be modified to more accurately reflect the stresses acting parallel to the exposed surfaces of an open stope.

(b) additional factors should be incorporated to reflect the effects of persistent structure paralleling or intersecting exposed surfaces and unfavourable inclinations of those surfaces.

(c) a factor to account for the effect of poor blasting practice, although probably warranted, was not included in this initial analysis.

## 3.0 APPLICATION OF THE NGI CLASSIFICATION SYSTEM TO OPEN STOPING AT DEPTH

For open stoping to be viable, spans in excess of 20 m wide and walls up to 75 m high must be stable and extraction ratios exceeding 50 per cent attainable. In general, the number of variables to be considered is too high to permit other than empirical approaches to design. However, analytical methods are often used to identify excessive stress conditions or excessive deformations (Appendix III).

Three types of failure must be considered when designing open stopes:

(a) Structurally controlled failures caused by opening and movement along unfavourably oriented structure (usually zones of low stress).

(b) Stress controlled failure through intact rock.

(c) A combination of structural and stress controlled failure caused by movement along joints combined with failure of the bridge of intact rock between structure.

Laubscher and Taylor (1976) and others have defined rock mass strength and structure, state of stress and shape and size of the opening as being important variables related to the design of stable open stopes. Considering exposed surfaces, tunnels can be considered as one way spanning because the length is very long compared to the span. In the open stoping situation, exposed surfaces are two way spanning as the ratio of span to strike length can range from 2:1 to 1/2:1. One-way spanning changes to two-way spanning when the ratio of spans is less than 4:1 (slab design in civil structures).

For the purposes of this study, selected geotechnical factors have been combined as a stability number and plotted against a shape factor to assess empirically the stability of surfaces bounding an open stope. The stability number used accounts for rock mass quality, the state of stress and the orientation of exposed surfaces, while the shape factor accounts for the shape and size of the opening. On this basis, a high stability number and low shape factor should reflect stable exposures while the converse should reflect instability. It is necessary to assess the stability of each exposure in turn; roof, hangingwall, horizontal pillars and vertical pillars, etc. and adjust dimensions until all exposures are stable. Stresses applied should reflect anticipated values as accurately as possible considering the available data, e.g. field and induced stresses or abutment stresses resulting from the interaction of adjacent openings.

## 3.1 Calculation of the Stability Number (N)

Stability Number (N) = Q' x A x B x C

where: Q' = Modified NGI Rock Mass rating

A = Stress factor

B = Rock Defect orientation Factor

C = Design surface orientation factor

### 3.1.1 Modified NGI Rock Mass Rating (Q')

The rock mass quality index Q defined in Section 2.8 has been modified by setting the stress reduction factor (SRF) to 1. All the other factors are unchanged, hence the modified index Q' accounts for rock mass strength and structure only.

### 3.1.2 Rock Stress Factor (A)

The rock stress factor (A) replaces the stress reduction factor to more accurately reflect stresses acting on exposed surfaces of open stopes at depth. This factor is the ratio of intact rock strength to induced stress where:

(a) intact rock strength is defined as the unconfined uniaxial compressive strength, and

(b) induced stress is defined as the stress acting parallel to the exposed stope wall or roof under analysis.

Uniaxial compressive strength should be determined by unconfined compression testing of specimens of diamond drill core representative of the exposed surface under consideration. Compressive strength calculated from point load testing is also acceptable. Reference to tables which give a range of strength values for the rock type under consideration can be used as a last resort.

Bridging type failure between fractures should not occur when the ratio of intact rock strength to induced stress exceeds 10. Any failure under these conditions should be related to movement on defined structure only and for this case, the rock stress factor (A) is set to 1.0.

Failure will be stress controlled as the ratio of intact rock strength to induced stress approaches unity. For this case, the rock stress factor (A) approaches zero.

Combined stress and structurally controlled failure can occur between these extremes and a straightline relationship has been assumed to determine the factor A. From experience, the ratio of intact rock strength to induced stress should not be less than 2 as indicated in Figure 3.1 and this relationship can be used to flag potential problems.

Induced compressive stresses in walls and pillars are best estimated using analytical methods to model the actual situation under consideration. Measured virgin stresses should be used when the data is available but if not, the first step is to estimate the vertical stress and the ratio of average horizontal stress to vertical stress.

Herget (1980) has summarized the data available from virgin stress determinations carried out in the Canadian Shield in the Provinces of Manitoba and Ontario (Table 3-1). This data has been used by Herget (1980) to develop relationships for the estimation of vertical stress and average horizontal stress and these should be used as appropriate.

Hoek and Brown (1980) have presented an empirical relationship to estimate vertical stress based on data obtained from four continents. The relationship is graphed in Figure 3.2 for MPa and meter units as follows:

$$\text{Vertical Depth Stress } (\sigma_V) = 0.027 \text{ Depth (D)}$$

Figure 3.3 is a graph showing the variation of average horizontal stress ($\sigma_H$) with vertical stress ($\sigma_V$). The maximum value of this ratio (K) can be calculated from the empirical relationship:

$$K = \frac{1500}{D} + 0.5$$

## TABLE 3.1 RESULTS FROM GROUND STRESS DETERMINATION IN THE CANADIAN SHIELD (Herget, 1980)

| Depth (m) | $S_E$ (MPa) | $S_N$ (MPa) | $S_V$ (MPa) | $\sigma_1$ (MPa) (dir) | $\sigma_2$ (MPa) (dir) | $\sigma_3$ (MPa) (dir) | Rock Type | E (GPa) | $\nu$ | Method | Ref. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Nordic Mine, Elliot Lake** | | | | | | | | | | | |
| 335 | 20.7 | 17.2 | 10.3 | 20.7 (090/00) | 17.2 (000/00) | 10.3 (000/90) | Feldspathic quartz sandstone | 75.8 | 0.16 | 1) 2) | 1 |
| **Denison Mine, Elliot Lake** | | | | | | | | | | | |
| 305 | 36.5 | 20.0 | 11.0 | 36.5 (045/00) | 20.0 (135/00) | 11.0 (000/90) | " | 75.8 | 0.14 | 1) | 1 |
| 701 | 36.5 | 22.1 | 17.2 | 36.5 (090/00) | 22.1 (000/00) | 17.2 (000/90) | " | 75.8 | 0.14 | 1) | 1 |
| **G.W. McLeod Mine, Wawa** | | | | | | | | | | | |
| 366 | 21.0 | 20.3 | 16.3 | 21.4 (118/12) | 20.1 (027/12) | 16.1 (230/78) | Siderite | 114.5 | 0.32 | 1) | 2 |
| 366 | 36.0 | 32.3 | 23.5 | 42.5 (133/33) | 34.3 (229/09) | 15.1 (332/56) | Tuff | 74.5 | 0.25 | 2) | 2 |
| 479 | 29.2 | 27.9 | 19.4 | 30.0 (251/11) | 27.7 (343/08) | 18.7 (110/76) | Metadiorite | 77.2 | 0.31 | 1) | 2 |
| 570 | 38.8 | 40.6 | 28.6 | 47.2 (222/17) | 34.1 (315/09) | 26.7 (070/70) | Chert | 95.1 | 0.22 | 1) | 2 |
| 570 | 27.9 | 31.0 | 22.1 | 31.6 (162/11) | 27.9 (070/12) | 21.5 (295/74) | Tuff | 61.4 | 0.26 | 1) | 2 |
| 570 | 29.2 | 36.0 | 23.9 | 38.3 (356/22) | 29.5 (090/11) | 21.4 (206/66) | Tuff | 71.7 | 0.28 | 1) | 2 |
| 570 | 18.2 | 18.3 | 14.7 | 19.9 (224/04) | 16.6 (315/06) | 14.6 (100/83) | Tuff | 63.4 | 0.24 | 1) | 2 |
| **Kidd Creek Mine, Timmins** | | | | | | | | | | | |
| 488 | 31.9 | 25.4 | 13.4 | 33.1 (094/06) | 26.8 (186/23) | 10.7 (350/60) | Fine-grained andesite/diorite | 95.8 | 0.27 | 1) | 3 |
| 732 | 62.7 | 65.1 | 43.9 | 72.6 (258/19) | 64.7 (358/25) | 34.4 (135/58) | " | 79.9 | 0.27 | 1) | 3 |
| 853 | 51.0 | 52.5 | 20.9 | 53.3 (250/10) | 51.9 (342/09) | 19.1 (112/77) | " | 95.8 | 0.27 | 1) | 3 |
| 853 | 48.1 | 39.3 | 23.6 | 53.2 (077/12) | 39.9 (170/18) | 16.3 (318/70) | " | 95.8 | 0.27 | 3) | 4 |
| **Thompson Mine, Thompson** | | | | | | | | | | | |
| 610 | 44.2 | 46.3 | 26.8 | 58.5 (323/11) | 40.5 (061/32) | 18.3 (214/54) | Biotite gneiss | 57.3 | 0.23 | 1) | 5 |
| 1219 | 112.4 | 113.0 | 97.3 | 113.4 (207/03) | 112.7 (115/12) | 96.6 (305/77) | Granitic gneiss | 51.9 | 0.26 | 1) | 5 |
| **Birchtree Mine, Thompson** | | | | | | | | | | | |
| 457 | 24.8 | 40.8 | 17.8 | 42.5 (017/06) | 23.9 (108/17) | 16.9 (267/72) | Biotite schist | 69.4 | 0.20 | 1) | 5 |
| 838 | 30.5 | 26.9 | 16.1 | 31.7 (289/14) | 26.8 (022/6) | 15.0 (139/75) | " | 43.0 | 0.20 | 1) | 5 |
| **Madsen Mine, Red Lake** | | | | | | | | | | | |
| 1148 | 43.4 | 46.3 | 34.1 | 52.2 (049/26) | 44.1 (320/17) | 28.0 (267/59) | Talc schist | 84.1 | 0.06 | 2) | 6 |
| **Creighton Mine, Sudbury** | | | | | | | | | | | |
| 701 | | | | 32.5 (291/30) | 25.8 (034/22) | 16.6 (155/51) | Quartz-biotite gabbro | 69.6 | 0.19 | 3) | 7 |
| 701 | 28.4 | 27.3 | 22.0 | 37.4 (309/32) | 28.8 (205/21) | 14.1 (086/76) | " | 69.6 | 0.19 | 3) | 7 |
| 701 | 34.4 | 29.2 | 18.8 | 38.0 (300/25) | 27.0 (032/05) | 16.4 (128/66) | " | 76.5 | 0.19 | 3) | 7 |
| 1219 | 56.5 | 43.3 | 38.5 | 60.33 (250/13) | 45.7 (348/35) | 34.3 (144/52) | Meta-gabbro | 98.6 | 0.23 | 3) | 8 |
| 1219 | 33.4 | 16.6 | 18.2 | 34.5 (092/12) | 20.5 (348/47) | 13.3 (194/40) | " | 98.6 | 0.23 | 3) | 8 |
| 1219 | 71.3 | 45.9 | 40.1 | 80.7 (243/06) | 40.0 (358/76) | 36.6 (150/22) | " | 98.6 | 0.23 | 1) | 8 |
| 1707 | 118.3 | 85.8 | 87.6 | 128.7 (249/10) | 100.7 (350/52) | 62.3 (152/37) | " | 84.1 | 0.30 | 1) | 8 |
| 1707 | 123.6 | 86.7 | 102.2 | 131.7 (068/00) | 112.2 (339/61) | 68.9 (158/28) | Fine-grained gabbroic schist | 80.7 | 0.27 | 3) | 9 |
| 1707 | 77.6 | 47.5 | 53.6 | 84.1 (248/07) | 53.9 (133/73) | 40.5 (340/15) | " | 68.9 | 0.35 | 3) | 9 |
| 2073 | 132.6 | 124.2 | 124.7 | 133.6 (091/18) | 124.8 (194/34) | 123.1 (337/43) | Quartz-biotite schist | 66.9 | 0.34 | 1) | 9 |
| 2134 | 60.7 | 27.5 | 37.1 | 61.4 (265/08) | 37.1 (142/7) | 26.8 (358/13) | " | 55.1 | 0.28 | 3) | 9 |
| 2134 | 59.4 | 35.0 | 37.4 | 63.9 (266/22) | 39.2 (159/35) | 28.8 (022/46) | " | 49.6 | 0.28 | 3) | 9 |
| 2134 | 82.5 | 46.8 | 43.4 | 82.7 (267/02) | 48.2 (175/28) | 41.9 (000/61) | " | 51.7 | 0.27 | 3) | 9 |
| 2134 | 74.5 | 59.2 | 49.9 | 78.6 | 65.9 | 39.2 | " | 86.8 | 0.27 | 3) | 8 |
| **Onaping, Ontario** | | | | | | | | | | | |
| 1227 | 63.0 | 37.7 | 38.9 | 67.0 (268/20) | 39.8 (015/32) | 32.8 (151/51) | Norite | 48.3 | 0.20 | 4) | 10 |
| 1227 | 54.3 | 33.2 | 33.1 | 59.0 (264/22) | 33.4 (002/18) | 28.2 (127/61) | Norite | 56.8 | 0.17 | 4) | 10 |
| 1227 | 61.4 | 40.6 | 47.1 | 67.5 (260/26) | 44.6 (023/48) | 37.1 (154/30) | Norite | 52.8 | 0.19 | 4) | 10 |
| **Ottawa, Ontario** | | | | | | | | | | | |
| 0 | $(S_E + S_N)/2 = 2.76$ | | 0 | - | - | - | Limestone | 37.92 | - | 2) | 11 |
| **North Bay, Ontario** | | | | | | | | | | | |
| 0 | $(S_E + S_N)/2 = 7.59$ | | 0 | - | - | - | Granite | 39.53 | - | 2) | 11 |

**Footnotes to Table 1:**

Stress determination methods:

1) CSIR biaxial (doorstopper);
2) USBM biaxial;
3) CSIR triaxial;
4) CSIRO triaxial (Australia).

References:
1) Bielenstein, H.U., *et al.*, Proc. 6th Canadian Rock Mechanics Symposium, 1970.
2) Herget, G., Int. J. Rock Mechanics & Mining Science; vol 10; 1973.
3) Herget, G., *et al.*, CANMET, Energy, Mines & Resources Canada, Lab Report MRP/MRL 76-148; unpublished.
4) Herget, G., CANMET, Energy, Mines and Resources Canada, Lab Report MRP/MRL 77-2; unpublished.
5) Miles, P., *et al.*, CANMET, Energy, Mines and Resources Canada, Lab Report MRP/MRL 79-66; unpublished.
6) Sanden, B.H., BSc thesis, Mining Engineering Dept., Queen's Univ., 1971.
7) Herget, G., *et al.*, CANMET, Energy, Mines and Resouces Canada, Lab Report 78-53; unpublished.
8) Herget, G., *et al.*, Proc. 10th Canadian Rock Mechanics Symposium, 1975.
9) Miles, P., *et al.*, CANMET, Energy, Mines and Resources Canada, Lab Report 77-81, unpublished.
10) Falconbridge Nickel Mines; personal communication.
11) Grant, F., *et al.*, CANMET, Energy, Mines and Resources Canada, Lab Report FMP-64/125, unpublished.

$S_E/S_N/S_V$ normal stress components in east, north and vertical direction.

$\sigma_1/\sigma_2/\sigma_3$ principal compressive stress directions (maximum, intermediate, minimum (directions given in trend and plunge to true north).

E, $\nu$ Elastic modulus, Poisson's ratio, respectively.

To simplify the analysis ($\sigma_H$), is assumed to be constant for all orientations in the horizontal plane ($\sigma_H$ = average horizontal stress). For example, assume that the virgin stresses are to be estimated at a depth of 1,000 m. Then K = 2, $\sigma_V$ = 27 MPa and $\sigma_H$ = 54 MPa.

Adoption of the limiting formulae for (K) given above leads to high estimates of the horizontal stress ($\sigma_H$) for the range of depths under consideration (750 m to 1,500 m). Accordingly, it is recommended that a less conservative relationship $K = \frac{375}{D} + 1.0$ should be used (Figure 3.3). This relationship more closely represents a high average of the stresses actually measured in the field. For example, assume that the virgin stresses are to be estimated at a depth of 750 m. Then K = 1.5, $\sigma_V$ = 20.2 MPa and $\sigma_H$ = 30.4 MPa.

The above analysis shows the value of measuring virgin stress on site. Estimates based on empirical relationships are at best only rough approximations.

Calculated maximum concentrations of stress bounding single, rectangular openings are given in Figures 3-4 and 3-5 for increasing ratios of span. Induced stresses at the top of a rectangular opening can be estimated by referring to Figure 3-4 and in the wall by referring to Figure 3-5. For example, at a depth of 750 m, $\sigma_V$ is 20.2 MPa and K is 1.5. Referring to Figure 3-4, the ratio of induced stress ($\sigma_I$) to vertical stress ($\sigma_V$) is 3.3 for an opening with a height to span ratio of 4:1. Hence the induced stress $\sigma_I$ is 66.7 MPa.

Referring to Figure 3-5, it will be noted that tensile stresses can be induced in the walls of tall openings with increasing values of (K). When these conditions are encountered, $\sigma_I$ is set to zero, indicating that block fall out by gravity will be the condition of failure.

The levels of stress induced in pillars between multiple openings are related to the virgin stress, geometry of the openings and the width of pillars between openings (both horizontal and vertical). If adjacent orebodies are being mined, the spacing between openings in the orebodies must also be considered.

Induced stresses in pillars tend to increase as:

(a) the depth of workings increase,

(b) the height to span ratio of openings increases,

(c) the width of pillars decreases, and

(d) the spacing between orebodies decreases.

There are too many variables to consider unless a specific situation or layout is assessed. Even in this case, induced stresses are best estimated with numerical analysis techniques, using measured or empirically estimated data on virgin stress (Appendix III).

Referring to Figure 3-4, the ratio ($\sigma_I / \sigma_V$) is analogous to a stress concentration factor. With openings having a height to span ratio of 4 or more, stress concentrations of the order of 4 can be expected in the backs of single openings, resulting in "high" induced stresses at depths exceeding 1,000 m. Due to the superposition of stress, concentrations of 6-8 are possible in thin pillars between stopes.

The recommended design procedure is as follows:

(a) Assess the stable dimensions of single openings using the empirical procedures outlined in this report.

(b) Prepare an acceptable mining layout using the assessed dimensions of stable stopes.

(c) Estimate stresses in pillars using numerical analysis techniques.

(d) Redesign pillars if stresses in the pillars exceed an arbitrary level defined by $\sigma_c/\sigma_I < 2.0$.

### 3.1.3 Rock Defect Orientation Factor (B)

This factor is included to account for the presence of persistent structure paralleling or intersecting exposed surfaces (Figure 3-6). The value of the factor is 0.5 for structure paralleling the exposed surface and 1.0 for structure intersecting the exposed surface at right angles. The factor is selected as follows:

(a) Determine the orientation of the most persistent set of fractures based on relative spacing and continuity.

(b) Determine the angle of intersection with the exposed surface under consideration.

(c) Refer to Figure 3-6 and select the orientation factor (B).

Stereographic plots can be used to determine the relative angle between joint sets and design surfaces and it is recommended that they be used.

When selecting the orientation factor (B), it should be noted that the overall stability of the excavation is under assessment; some dilution in the form of block fallout or spalling can be accepted.

### 3.1.4 Orientation of Design Surface Factor (C)

Stope backs or roofs are inherently less stable that sidewalls because of the influence of gravity. Barton et al (1974) suggest that rock quality in a wall is hypothetically improved 5 times compared to a roof. They also recommend an ESR of 1.6 for permanent mine openings. However, some block fallout and spalling can be tolerated in a "non-entry" type of excavation. Therefore, it seems conservative to suggest that a vertical wall should be at least 5 x 1.6 = 8 times as stable as a horizontal roof. Referring to Figure 3.7, if the factor (C) for a horizontal surface is set to 1, then its value will increase as the angle of dip increases. The maximum value is 8 when the surface is vertical. The formula used to obtain the relationship between factor (C) and the angle of dip is given below and graphed in Figure 3-7.

$$\text{Factor } C = 8 - 7 \text{ cosine (angle of dip)}$$

The factor accounts for the effect of gravity and should be applied to roofs, hangingwalls and possibly steep footwalls when adverse structure has been identified.

## 3.2 Shape Factor (S)

As mentioned earlier, the exposed surfaces of open stopes can be regarded as two-way spans and a shape factor can be defined as the ratio of design surface area to design surface perimeter. This ratio is defined as the "hydraulic radius" by Laubscher and Taylor (1976). The relationship of shape factor (S) to span is given in Figure 3.8. It should be noted that as the ratio of spans increases beyond 4:1, the factor remains relatively constant and reflects one-way spanning situations.

## 3.3 Graph of Stability Number (N) vs. Shape Factor (S)

It is postulated that the stability of exposed surfaces in open stopes can be assessed empirically by plotting stability number (N) versus the shape factor (S).

Thus the stability of the exposed surface decreases as the shape factor (S) increases to the point of caving. Data obtained from the case studies and elsewhere has been tabulated and plotted in this form (Chapter 4).

## 4.0 APPLICATION OF THE MODEL TO CASE STUDIES

A graph of stability number (N) versus shape factor (S) has been plotted (Figure 4-1) using the data collected from the case study investigations (Appendix IV) and from various sources in the literature. This data is presented in table form in Appendix V, with notes on the source, reliability and estimates made to complete the table. The stability number (N) in Figure 4-1 has been plotted on a log scale in keeping with the presentation made by Barton et al (1974).

### 4.1 Discussion of Results

Divisions between stable, unstable and caving zones have been estimated from the scatter of data presented in Figure 4-1. The same divisions, without plotted data are presented in Figure 4-2 and are defined below.

(a) Stable

The excavation will stand unsupported with occasional localized ground support to control slabbing.

(b) Unstable

The excavation will experience some localized caving but will tend to form a stable arch. Open stoping is feasible if localized caving can be prevented by modifying extraction sequence, installing cable bolts, etc.

(c) Caving

The excavation will cave and will not stabilize until the void is full.

The divisions are considered to be sound conceptually but it is emphasized that at least two of the three sites should be revisited to obtain additional data. This data will be obtained when the opportunity to revisit arises and will be made available to CANMET. The basic problem is that site data was collected in the early stages of the study; sufficient data was collected to develop the concepts presented but not to confirm them. However, data obtained from CSA Mine at Cobar has been supplemented by published information presented by Barton (1977) and this has been used to present an example on the use of the graph plotting stability number (N) versus shape factor (S) in Figure 4-2.

## 4.2 Example on the Use of the Stability Graph

An open stope is to be developed at a depth of 1,000 m in ground having characteristics similar to the "average" ground occurring at the CSA Mine, Cobar. The orebody is assumed to be 25 m wide and dips at 80 degrees. It is desired that the stope length be a minimum of 30 m and the height a minimum of 75 m to permit the use of in-the-hole drilling equipment (Figure 4-3). The unconfined compressive strength of the intact rock averages 120 MPa and additional data on geotechnical parameters are given in the text. Determine whether the stope is stable according to the graph presented in Figure 4-2.

### 4.2.1 Modified NGI Rock Mass Rating (Q')

The rock mass quality data for the CSA Mine is summarized below from Barton (1977), Table 9 as follows:

| | Item | Description | Value |
|---|---|---|---|
| (1) | Rock Quality | Good | RQD = 85% |
| (2) | Joint sets | One joint set and random | = 3 |
| (3) | Joint roughness | Rough or irregular undulating | = 3 |
| (4) | Joint alteration | Unaltered with surface staining | = 1 |
| (5) | Joint water | Dry with minor surface staining | = 1 |
| (6) | Stress reduction | Single weakness zones containing clay | SRF = 2.5 |

$$\text{Hence } Q = \frac{85}{3} \times \frac{3}{1} \times \frac{1}{2.5} = 34$$

Determine the modified NGI Rock Mass Rating (Q') by setting SRF = 1, hence Q' = 85.

4.2.2 Rock Stress Factor (A)

Virgin stress has not been measured, hence the values must be estimated. Referring to Figure 3-2, the vertical stress ($\sigma_V$) is estimated at 27 MPa for a depth of 1,000 m. Hence (K), the ratio of average horizontal stress ($\sigma_H$) to the vertical stress ($\sigma_V$) from Figure 3-3 is 1.375 and $\sigma_H$ = 37.1 MPa.

Considering the values of virgin stress in the horizontal plane, assume that $\sigma_{H1} = \sigma_{H2} = \sigma_H$ = 37.1 MPa where $\sigma_{H1}$ is the virgin stress parallel to strike and $\sigma_{H2}$ is the virgin stress normal to strike (Figure 4-3).

Referring to Figure 3-4, the first step is to calculate the induced stresses in the back (top of vertical plane) and the strike end (end of horizontal plane) of the stope.

(a) Top of Mid Stope Vertical Plane

$\sigma_V$ = 27 MPa
$\sigma_{H2}$ = 37.1 MPa
K = $\sigma_{H2}/\sigma_V$ = 1.4

For a height to span ratio of 3 and K value of 1.4, then $\sigma_I : \sigma_V$ is estimated at 2.6. Hence $\sigma_I$ = 2.6 x 27 MPa = 70 MPa. Referring to Figure 3-1, the value of $\sigma_C : \sigma_I$ = 120:70 = 1.7. As this ratio is less than 2, the back is likely to be on the verge of instability. The height of the stope should be reduced but for the purposes of this example, assume a rock stress factor (A) = 0.1.

(b) Strike end of Mid Stope Horizontal Plane

$\sigma_{H1}$ = 37.1 MPa
$\sigma_{H}$ = 37.1 MPa
K = $\sigma_{H2}/\sigma_{H1}$ = 1

For a length to span ratio of 1.2 and K value of 1, then $\sigma_I : \sigma_{H1}$ is estimated at 1.0. Hence $\sigma_I$ = 1.0 x 37.1 MPa = 37.1 MPa. Referring to Figure 3-1, the value of $\sigma_c : \sigma_I$ = 120:37.1 = 3.2 and the rock stress factor (A) = 0.25.

Referring to Figure 3-5, the next step is to calculate the induced stresses in the hangingwall and footwall considering the vertical and horizontal mid-stope planes. The lowest value of the estimated rock stress factor (A) is used.

(c) Mid-Stope Vertical Plane (H/W and F/W)

K = $\sigma_{H2} : \sigma_V$ = 37.1:27 = 1.4

For a height to span ratio of 3 and K value of 1.4, then $\sigma_I : \sigma_V$ is estimated at -0.1. As the value is negative, it is set to zero and $\sigma_I$ is zero. Referring to Figure 3-1, the value of $\sigma_c : \sigma_I$ is greater than 10, hence the rock stress factor (A) = 1. It should be noted, however, that horizontal joints intersecting the hangingwall will open as the induced stress at the center of the hangingwall span is tensile.

(d) Mid-Stope Horizontal Plane (H/W and F/W)

K = $\sigma_{H2} : \sigma_{H1}$ = 37.1:37.1 = 1

For a length to span ratio of 1.2 and K value of 1, then $\sigma_I : \sigma_{H1}$ is estimated at 0.75. Hence $\sigma_I$ = 0.75 x 37.1 MPa = 27.8 MPa. Referring to Figure 3-1, the value of $\sigma_C : \sigma_I$ = 120:27.8 = 4.3 and the rock stress factor (A) is 0.35.

It should be noted that both the hangingwall and footwall are in compression in the direction of strike and tensile (near the mid span) in the direction of dip.

Summarizing, the rock stress factors (A) to be used are listed below:

| | | |
|---|---|---|
| Back | - A | = 0.1 |
| H/W | - A | = 0.35 |
| F/W | - A | = 0.35 |
| Vertical End | - A | = 0.25 |

4.2.3 Rock Defect Orientation Factor (B)

The principal joint set is flat dipping and joints are closely spaced in the range of 7 - 15 cm apart. Joint surfaces are unaltered with surface staining.

The orebody is 25 m wide and dips at 80 degrees. Referring to Figure 3-6, the orientation factor for the exposed surfaces are as follows:

| Exposed Surface | Orientation (degrees) | Value of (B) |
|---|---|---|
| Back | 0 | 0.5 |
| Hangingwall | 100 | 1.0 |
| Footwall | 80 | 1.0 |
| Vertical End | 90 | 1.0 |

4.2.4 Design Surface Orientation Factor (C)

The hangingwall and footwall dip at 80 degrees, hence the design surface orientation factors for the exposed surfaces are as follows (Figure 3-7):

| Exposed Surface | Inclination (degrees) | Value of (C) |
|---|---|---|
| Back | Horizontal | 1 |
| Hangingwall | 80 | 6.8 |
| Footwall | +90 | 8.0 |
| Vertical End | 90 | 8.0 |

4.2.5 Values of Stability Numbers (N)

Values of N = Q' x A x B x C for the exposed surfaces are summarized below:

| Exposed Surface | Value of (N) |
|---|---|
| Back | 4.3 |
| Hangingwall | 200.0 |
| Footwall | 240.0 |
| Vertical End | 170.0 |

4.2.6 Values of Shape Factor (S)

Values of (S) equal to the ratio of the area of the exposed surface to the perimeter of the exposed surface are given below.

| Exposed Surface | Value of (S) |
|---|---|
| Back | 6.8 |
| Hangingwall | 10.7 |
| Footwall | 10.7 |
| Vertical End | 9.4 |

4.2.7 Comments on Analysis

The values of stability number (N) versus shape factor (S) are plotted on Figure 4-4. All of the walls plotted in the stable area of the graph but the roof plots in the zone between "unstable" and "caving". If these results are accepted, then the height of the stope should be reduced to lessen roof stress.

Stresses are estimated for the central sections of the spans, but corner stresses will be much higher. Spalling can be anticipated in the upper corners along strike and cable bolting will not prevent this. However, cable bolting would be effective for general roof support and in areas that have spalled to a stable shape.

Localized sloughing can be expected from the central portion of the hangingwall (and footwall) but the vertical ends should be sound. However, secondary joint sets are randomly orientated hence some random spalling and sloughing can be anticipated.

The analysis was for a single opening and it can be appreciated that numerical analysis techniques are necessary to obtain a reasonable picture of stress distribution resulting from the interaction of multiple openings. The roof of the single stope analyzed may have spalled to a stable shape to permit support by cable bolts. However, if the roof was the bottom of a horizontal pillar separating a worked out stope above, then it probably would have failed completely. Failure would have resulted from the superposition or the interaction of stresses between the two openings to give much higher values. The effect of superposition of stresses increases as the thickness of the pillar decreases as illustrated by Hoek & Bray (1980), pp. 115. The horizontal pillar can only be stabilized by increasing pillar thickness or decreasing stope height and these measures may not be economic. The vertical end is quite stable, hence vertical pillars separating stopes on strike should also be stable if of reasonable length.

For rock of "good quality", experience has indicated that:

(a) pillar thickness should exceed the stope span by at least 25 per cent assuming "good" blasting practice, and/or

(b) the volume of rock in pillars should approach and/or exceed 50 per cent of the stope volume for depths below 750 m, even if fill is placed.

## 5.0 DATA COLLECTION AND ITS ESTIMATED COST

This chapter deals with the collection and analysis of the data required to use the model proposed. An estimation of the incremental cost of this data collection over normal mine exploration techniques employed for stope planning is also made. Most of the measurements or tests are those employed by geotechnical engineers on a well established and routine basis. Some engineering judgement will be necessary when classifying the rock mass. The procedures used at one mine for the measurement and classification of fractures are given by Mathews (1975). However, it is recommended that engineering staff unfamiliar with rock mechanics principles attend one of the short courses on this topic run by the various universities.

### 5.1 Determination of Rock Mass Quality

It is not unusual for the rock mass quality to vary from location to location within any given rock mass or rock type. Typically joint frequency may vary or the number of joint sets present may vary throughout the rock mass. However, in many instances, one or two factors can be recognized as dominating the variations in rock quality, e.g. variation in the persistence and spacing of bedding planes.

It is not recommended that a Q representing average conditions throughout the mine be used for design. Rather an estimate of Q should be made for the rock in the immediate vicinity of each surface being designed. A knowledge of the variation of rock mass quality would also be useful for designing pillars between stopes.

### 5.1.1 Rock Quality Designation (RQD)

Ideally RQD should be measured from NQ sized core (50 mm) using a double or triple tube core barrel. Other diameters of core will give different values of RQD in the same rock. Usually a larger diameter core such as HQ will give a higher RQD and a smaller diameter core such as BQ will give a lower RQD. The reason for this is that the mechanical effects of drilling on the defects in the rock are relatively greater on the smaller diameter core. A joint having a cohesion of 5 psi can theoretically support a slab of rock about 4 ft. thick under the force of gravity. However, this cohesion would probably be broken during drilling and handling of the core, even in an NQ sized hole.

When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, where the number of joints per meter for each joint set are added together. A simple relation can be used to convert this number to RQD for the case of clay free rock masses (Hoek and Brown, 1980, page 33):

$$RQD = 115 - 3.3\ JV \text{ (approximately)}$$

where JV = Total number of joints per $m^3$

$$(RQD) = 100 \text{ for } JV < 4.5)$$

It is not recommended that the above method be attempted before the measurer is familiar with measuring RQD from core. One should also beware of blast fractures (see Appendix VI).

One of the major discrepancies in the measurement of RQD lies in the way the core is extracted from the barrel by drillers and the method of transporting the core from drill site to the place where it is logged. Rough handling will cause extra breaks in core and lead to low RQD's. Driller education or how core should be extracted and transported is essential. When measuring core, only naturally occurring fractures should be logged and not obvious breaks caused by rough handling.

5.1.2 Joint Set Number (Jn)

Joints should be mapped along drifts or from oriented core. It is advisable to map joints in tunnels at right angles to each other to reduce bias. Only joints that can be traced for 3 m or more should be measured. Beware of blast fractures. Core orientation methods are well documented in the literature (Hoek and Brown, 1980, page 48). The minimum effort should be to piece core together, and then measure relative orientation of joint sets present. Sometimes core can be oriented from mapping joints exposed in surface outcrops or tunnels and identifying the same relative orientations.

Contoured stereographic plots of joint mapping should be prepared (see Appendix VI) and the number of joint sets (Jn) identified from these plots. These stereographic plots will also be used to determine the rock defect orientation factor (Figure 3-6).

5.1.3 Joint Roughness Number (Jr) and Joint Alteration Number (Ja)

These parameters should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. The value of

Jr/Ja should in fact, relate to the surface most likely to allow failure to initiate. This ratio should be determined from visual inspection of exposures underground. Judgement will be required as to which is the most significant joint set.

### 5.1.4 Joint Water Reduction Factor (Jw)

This parameter should be estimated from visual inspection underground. Most stoping areas are drained and dry with minor water inflow.

### 5.1.5 Uniaxial Compressive Strength

The uniaxial compressive strength may be estimated in the field by using the point load test on pieces of core. This is a simple and cheap test and is applicable to core obtained from rock which will form the design surface of the excavation. However, some uniaxial unconfined strength tests should be carried out in the laboratory to calibrate the curve used to assess the same value from the point load test (Hoek and Brown, 1980, page 52).

In rock having an anistropic strength such as schist, $\sigma_c$ should be measured both perpendicular and parallel to foliation. The appropriate $\sigma_c$ should be selected from inspection of the direction of the induced stress in relation to the orientation of the foliation.

### 5.1.6 Induced Stress

The method of assessing the induced stress parallel to the design surface has been discussed in Section 4. However, if it is planned to use numerical modelling to calculate induced stress, then values for Young's Modulus and Poisson's ratio must be determined from intact core specimens.

These can be obtained from strain gauging uniaxial unconfined compressive strength tests, flat jack tests or biaxial tests on over-cored rock from a stress measurement program.

## 5.2 Estimated Incremental Cost of Data Collection

The report deals with the design of openings below 1,000 m depth, hence it is assumed that a good deal of information is available on general mine geology and structure in the shallower mine workings. Also it is likely that development openings near the project area can be mapped in detail to supplement data obtained from the analysis of core.

Initial underground exploratory diamond drilling for open stoping layouts is usually done from exploration drifts located in the hangingwall. Holes (B size or greater) are drilled on sections spaced 150 m - 200 m apart through the orebody into the footwall. These holes define the orebody, confirm structure, rock type trends and permit preliminary mine planning such as stope and pillar and haulage layouts.

When block development is completed, short hole confirmatory drilling is then carried out from the additional development openings completed to provide ore limits for detailed stope planning. This definition drilling is usually A size or less and is done on sections spaced 20 m - 30 m apart, depending on the complexity of the situation.

Geotechnical data for design is usually obtained from:

(a) selected diamond drill holes from the exploratory phase, and

(b) the mapping of development openings as block development proceeds.

Occasionally, it is necessary to obtain additional geotechnical information from holes drilled at the confirmatory stage of exploration.

The additional work that should be undertaken for the collection of geotechnical data is summarized below.

(a) One or two structural diamond drill holes on sections spaced 150 m - 200 m apart.

(b) Structural logging and interpretation of the core.

(c) Structural mapping of available openings adjacent to and through the orebody and interpretation of data.

(d) Rock strength testing.

(e) Virgin stress measurement if this data is not available.

### 5.2.1 Structural Diamond Drilling

The standard of diamond drilling required for structural drilling is much higher than that required for normal exploratory drilling. Principle requirements include:

(a) hydraulic feed drilling machines,

(b) split tube, double or triple tube core barrels,

(c) as large a core size as practical,

(d) careful handling and boxing of the core, and

(e) positive motivation and skill of the drillers.

Current costs for AX exploratory drilling to about 1,000 ft. is about $30 per meter and it is estimated that the cost per foot of an NX hole to the standards described above would be about $65 per meter. Hence the additional cost of drilling two structural holes (each 150 m long) per 150 m of strike would be about $5,000.

### 5.2.2 Rock Strength Assessment and Testing

Point load tests cost about $20 each and a uniaxial compressive strength test in a laboratory would cost about $80. Assuming that a uniaxial compression strength test is done on the core at 10 m intervals, and that a point load test is done at an average of 3 m intervals, then the additional cost of strength testing per foot of structural core obtained is about $17 per meter. This is equivalent to about $5,100 for 300 m of structural drilling per 150 m of strike.

### 5.2.3 Virgin Stress Measurement

The current cost for a virgin stress determination, including drilling costs, is about $70,000 for a 10 measurement compaign. In addition to the data on virgin stress, data is provided on:

(a) uniaxial compressive strength

(b) Young's Modulus

(c) Poisson's Ratio.

Individually, the cost per test for items (b) and (c) above is about $240.

### 5.2.4 Staff

The staff required to assess the data depends on the size of the mine and production rate. At the least, a Geologist, Structural Geologist or Senior Technician trained in structural assessment of core and development openings will be required. The Geologist should be assisted in the interpretation of the data and the application of the data to design by a Rock Mechanics Engineer. In a large mine, these will be full-time jobs, but in a small mine, outside assistance may be required.

## 6.0 EXCAVATION STABILITY AND SUPPORT COSTS

There are two aspects of the stability and support of open stopes which must be considered:

(a) the drawpoints and drill drifts

(b) the open stopes.

Because the support requirements of these two types of openings are very different, they are described separately below. In general, if open stoping is being used, the rock quality will be high and the most likely type of failure is one in which geological structure is a controlling factor. That is, blocks of rock, the size of which are defined by natural fractures, must be supported.

### 6.1 Drawpoints and Drill Drifts

The span of these openings varies from about 12 ft. to 20 ft. and they must be stable because men have to work in them throughout the life of the stope. During this time, the rock is subjected to blast vibrations and increasing stresses as the stope becomes larger, so it is essential that the support system remains effective under these changing conditions. The most appropriate types of support are rock bolts and cable bolts. Both of these are well proven systems that can be installed during development.

The primary function of both rigid bolts and cables is to confine the rock so that support is achieved by maintaining the interlock and normal stress between blocks. This is a more effective means of maintaining stability than relying on the tensile strength of the steel. Therefore, it is important that tension be maintained in the bolts.

The different applications of bolts and cables are as follows. Rock bolts are rigid and are usually limited in length to about 8 ft. because it becomes difficult to install longer bolts in standard size drifts. Because the bolt must be anchored in sound rock, the length of the bolt should penetrate about 2 ft. into sound rock for secure anchorage. The spacing between bolts and the bolt diameter depends upon the rock stress and the degree of fracturing.

Methods of designing rock bolt patterns are described by Hoek and Brown (1980) and Lang (1962).

The most usual type of anchor for rigid rock bolts is a mechanical wedge; the bolt is tensioned by tightening a nut on the face. This type of bolt will not maintain effective support if the anchor slips or rock on the face spalls away from under the plate. If this happens, the bolt should be retensioned by tightening the nut but this becomes difficult if the thread does not extend to the new position of the face.

The tension on bolts can be maintained more effectively by grouting the bolt over its full length using either cement or epoxy resin. In this way, the shear stress between the bolt and the rock is evenly distributed along the length of the bolt and failure of the rock at one point will not completely destroy the support provided by the bolt. If a fully grouted bolt is installed during development for the stope, it is not necessary to apply a tension to the bolt because any change in strain in the rock due to stoping will tension the bolt and thus prevent loosening of the rock.

If the zone of unstable rock has a thickness greater than about 4 - 6 ft., then it will be necessary to use cable bolts for support. Cable bolts are lengths of high tensile strength steel strand that can be over 100 ft. long in down holes and up to about 70 ft. long in up holes. They are cement grouted over their full length and are usually not tensioned. The tensile strength of cables is higher than that of most rigid rock bolts so it is likely that cables will become a more important means of support as open stoping is carried out at depths in excess of 1,000 m.

Cable bolts are used extensively in the drifts at the Geco Mine where the rock is not particularly competent. At the Heath Steele Mine, where the rock is more competent, rock bolts are used exclusively although there are plans to use cables as the depth of mining increases.

### 6.2 Open Stopes

The dimensions of open stopes are often an order of magnitude greater than drifts so there is a corresponding increase in the extent of instability. However, because men do not have to enter the stopes, minor rock falls are of no consequence and instability only becomes a problem if the dilution is excessive.

If the back of the stope is expected to cave upwards, it may be controlled by installing rock bolts or cable bolts from the drill drifts before the stope is mined. Caving of the walls can usually only be controlled with cable bolts if they are installed approximately at right angles to the wall. This requires that there be a drift in the stope walls from which the holes for the cables can be drilled. Caving of the hangingwall is the most usual problem, but unfortunately it is rare that there is access available in the hangingwall. In cases where cable bolts would be a suitable method of stabilizing the hangingwall, it may be worthwhile examining the economics of developing a drift in the hangingwall to install cables (to determine if this is less expensive than the cost of dilution).

The other means of controlling caving of the stope walls is to fill the stopes, using either tailings or rock fill. The fill need not be cemented unless it is planned to recover pillars between the stopes at a later date in which case, it is desirable that the fill can stand unsupported when the pillar is removed. There are two methods of using fill for support. Firstly, the stope can be kept full with broken ore and waste rock at all times by tipping fill into the stope as the ore is drawn out at the bottom. When the stope is completed, the fill can be cemented by pouring cement in at the top. This is the method which is used with much success at the Geco Mine where the stope walls cave soon after they are exposed. The second method is to fill the stope when all the ore has been drawn. This method is used when there is no serious dilution problem, but the fill is required to prevent caving from occurring when adjacent stopes are mined and the induced stresses in the rock increase.

As open stoping is practiced at greater depths, the use of fill is likely to become more common. The Geco method could be used in competent rock where the high stresses cause stability problems as soon as the ore is drawn.

## 6.3 Support Costs

Six mines were requested to provide data on support costs and five responded. Average costs are given for the categories listed below.

(a) Rock Bolts (1.8 m tensioned)

| | |
|---|---|
| Materials | \$ 4.30 each |
| Installation | \$ 7.40 each |
| Total Cost | \$11.70 each |

(b) Grouted Cable Bolts

Tensioned - \$18 per meter installed
Untensioned - \$16.50 per meter installed

(c) Fill Costs

| | |
|---|---|
| Quarried rock fill | - \$4.5 per $m^3$ placed (swell factor 1.33) |
| Hydraulic fill | - \$2 per $m^3$ placed |
| Cemented hydraulic fill (30:1) | - \$7 per $m^3$ placed |

## 7.0 FUTURE WORK

Most of the classification systems reviewed for this study were concerned primarily with the prediction of support for civil engineering projects. The adjustments to the CSIR classification rating described by Laubscher and Taylor (1976) are the first published record of a systematic effort to use rock mass classification as a basis for the empirical design of mine excavations. Barton (1977) when comparing the CSIR and NGI indices for support prediction in stopes at the CSA Mine, Cobar concluded that:

"The design systems which have been described are empirical and require further assessment, but, nevertheless, offer the potential for useful application in the estimation of operational requirements for ground support. Additional test cases of wide stope-type openings are urgently required".

In this study, the NGI index was modified specifically for the purpose of obtaining relationships to predict empirically the stability of open stopes at depth. Results obtained from the limited data available were considered sufficient to develop the concepts presented, but insufficient to confirm them. At least two of the sites should be revisited to obtain additional data to permit back analysis and further refinement.

The success of the application of rock mass classification systems to support prediction for civil engineering structures is well documented in the literature. There is no reason why similar success cannot be achieved by developing adjustments applicable to the major mining methods used in Canada. Research effort is warranted to gather sufficient data for the development of models.

The collection and analysis of geotechnical data must be supervised by trained personnel and universities should be encouraged to run courses from time to time on these topics. Management should also be encouraged to send staff to these training courses. The publication of successful applications of rock mechanics principles, particularly those affecting safety and economics, would be of assistance in this regard.

Mention was made in the introduction that at depths of 1,000 m, stress concentrations in pillars may be high enough to cause inelastic behaviour of the rock and yielding along joint surfaces. There is an excellent opportunity here to develop concepts that take advantage of this behaviour and permit non-entry mining methods to be practiced in partly destressed ground. These concepts will be more readily developed based on a thorough understanding of the mechanisms involved. Empirical relationships based on back analysis are powerful predictive tools, particularly if combined with numerical modelling and analysis techniques.

## 8.0 ACKNOWLEDGEMENTS

Golder Associates wishes to acknowledge the valuable assistance and co-operation provided by the following organizations during site visits to gather data for this study.

- Heath Steele Mines Ltd., New Brunswick
- Geco Mines Ltd., Manitouwadge, Ontario
- CSA Mine, Cobar, N.S. Wales, Australia

The various mining companies that provided cost information on ground support, diamond drilling and fill (and wish to remain anonymous) are also thanked.

Particular recognition is given to Dr. G. Hergert, who was also the Scientific Authority on the project, for the help and constructive criticism given during the course of the investigations.

Yours very truly,

GOLDER ASSOCIATES

D. C. Wyllie

for K.E. Mathews, P. Eng.

E. Hoek

E. Hoek, P. Eng.

D. C. Wyllie

D.C. Wyllie, P. Eng.

SBV Stewart

S.B.V. Stewart, P. Eng.

KEM/EH/DCW/SBVS/bn

802-1571

## LIST OF REFERENCES


Barton, N., Lien, R., and Lunde, J. (1974). Engineering classification of rock masses for the design of tunnel support. Rock Mechanics, Vol. 6, No. 4, pp. 189-236.

Barton, C.M. (1977). A geotechnical analysis of rock structure and fabric in the CSA Mine, Cobar, N.S.W. Division of Applied Geomechanics Technical Paper No. 24, CSIRO, Australia.

Bieniawski, Z.T. (1974). Geomechanics classification of rock masses and its application in tunnelling. Proc. Third International Congress on Rock Mechanics, ISRM, Denver, Volume 11A, pp. 27-32.

Brekke, T.L., and Howard, T. (1972). Stability problems caused by seams and faults. Proc. First North American Rapid Excavation and Tunnelling Conference, AIME, New York, pp. 25-41.

Deere, D.U. (1964). Technical description of rock cores for engineering purposes. Rock Mechanics and Engineering Geology, Vol. 1, No. 1, pp. 17-22.

Herget, G (1980). Regional stress measurements in the Canadian Shield. Underground Rock Engineering, C.I.M. Special Volume 22, pp. 9-16.

Hoek, E., and Brown, E.T. (1980). Underground excavations in rock. IMM London, U.K. Printed by Stephen Austin & Sons.

Lang, T.A. (1962). Theory and practice of rock bolting. AIME Transactions (Mining), Vol. 223.

Laubscher, D.H., and Taylor, H.W. (1976). The importance of geomechanics classification of jointed rock masses in mining operations. Proc. of Symposium on Exploration for Rock Engineering, Johannesburg.

Lauffer, H. (1958). Gebirgsklassifizierung fur den stollenbau. Geologie und Bauwesen, Vol. 24, No. 1, pp. 46-51.

Mathews, K.E. (1975). Fracture measurements and analysis of data for excavation design. Proc. 10th Canadian Rock Mechanics Symposium, Queens University, Kingston, Ontario.

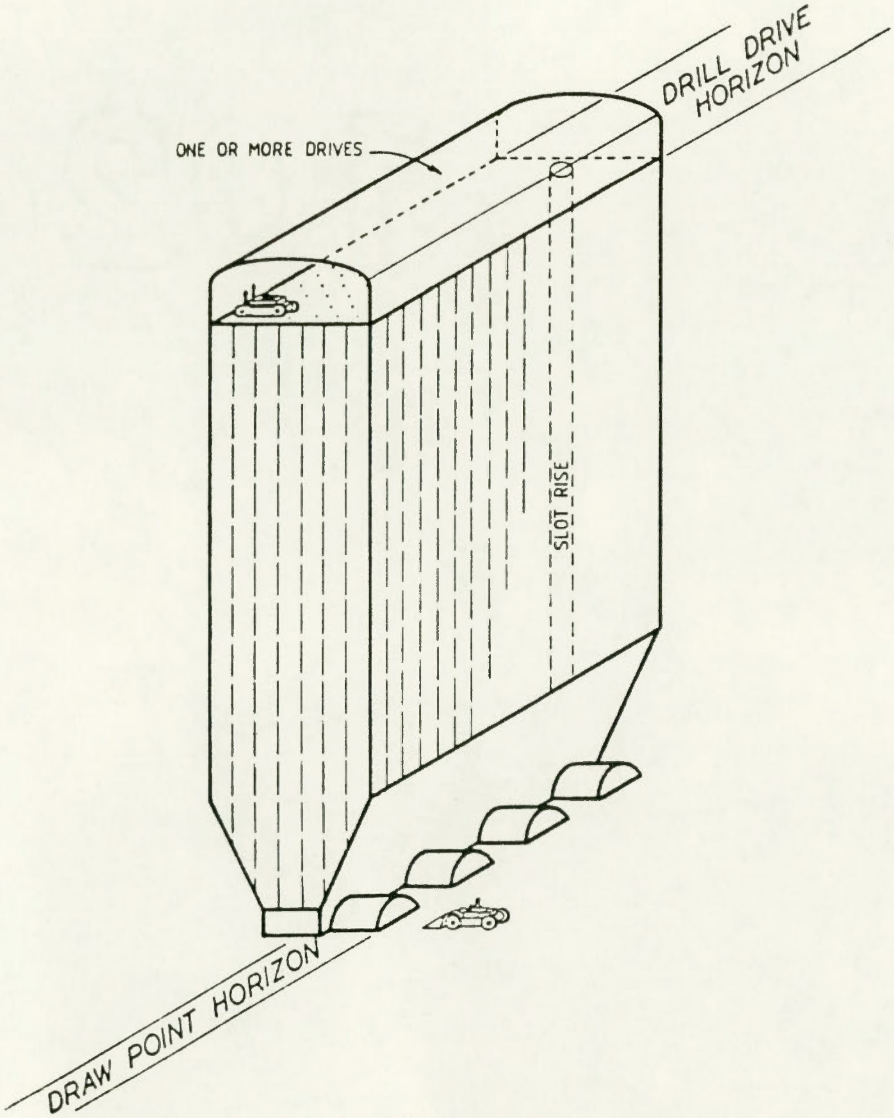
Patching, T.H., and Coates, D.F. (1968). A recommended rock classification for rock mechanics purposes. CIM Bulletin, October.

Stini, I. (1950). Tunnelbaugeologie. Springer-Verlag, Vienna.

Terzaghi, K. (1946). Rock defects and loads on tunnel supports. Rock Tunnelling with Steel Supports. Published by Commercial Shearing and Stamping Co., Youngstown. pp. 15-99.

SCHEMATIC OF AN OPEN STOPE

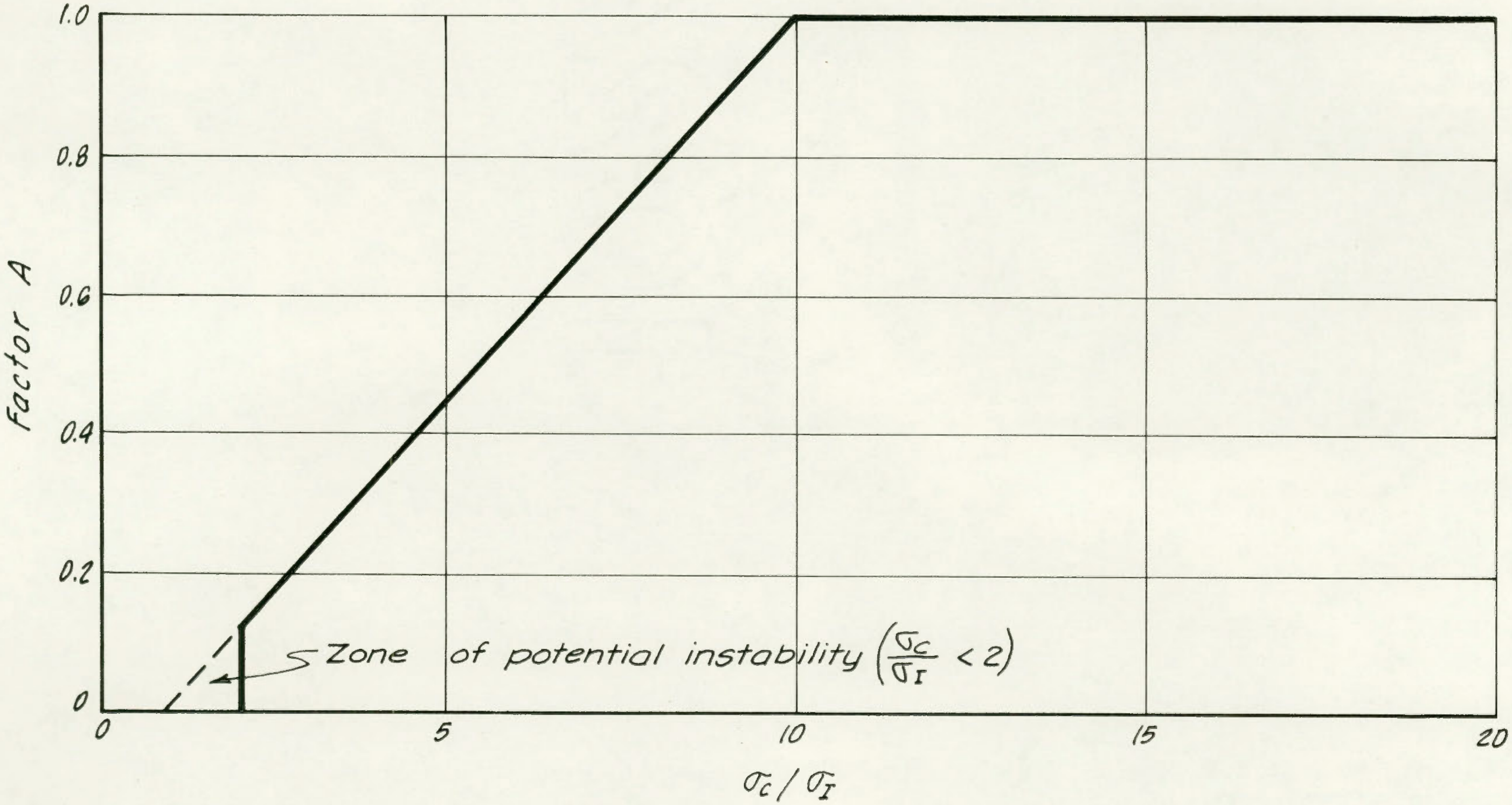
**Figure 1-1**




Project No. 802-1571 Drawn R.D. Reviewed Date Mar. '81

**Golder Associates**

Project No. 802-1571 Drawn [illegible] Reviewed [illegible] Date Feb.'81




$\sigma_c$ = Uniaxial compressive strength of intact rock.
$\sigma_I$ = Induced compressive stress.

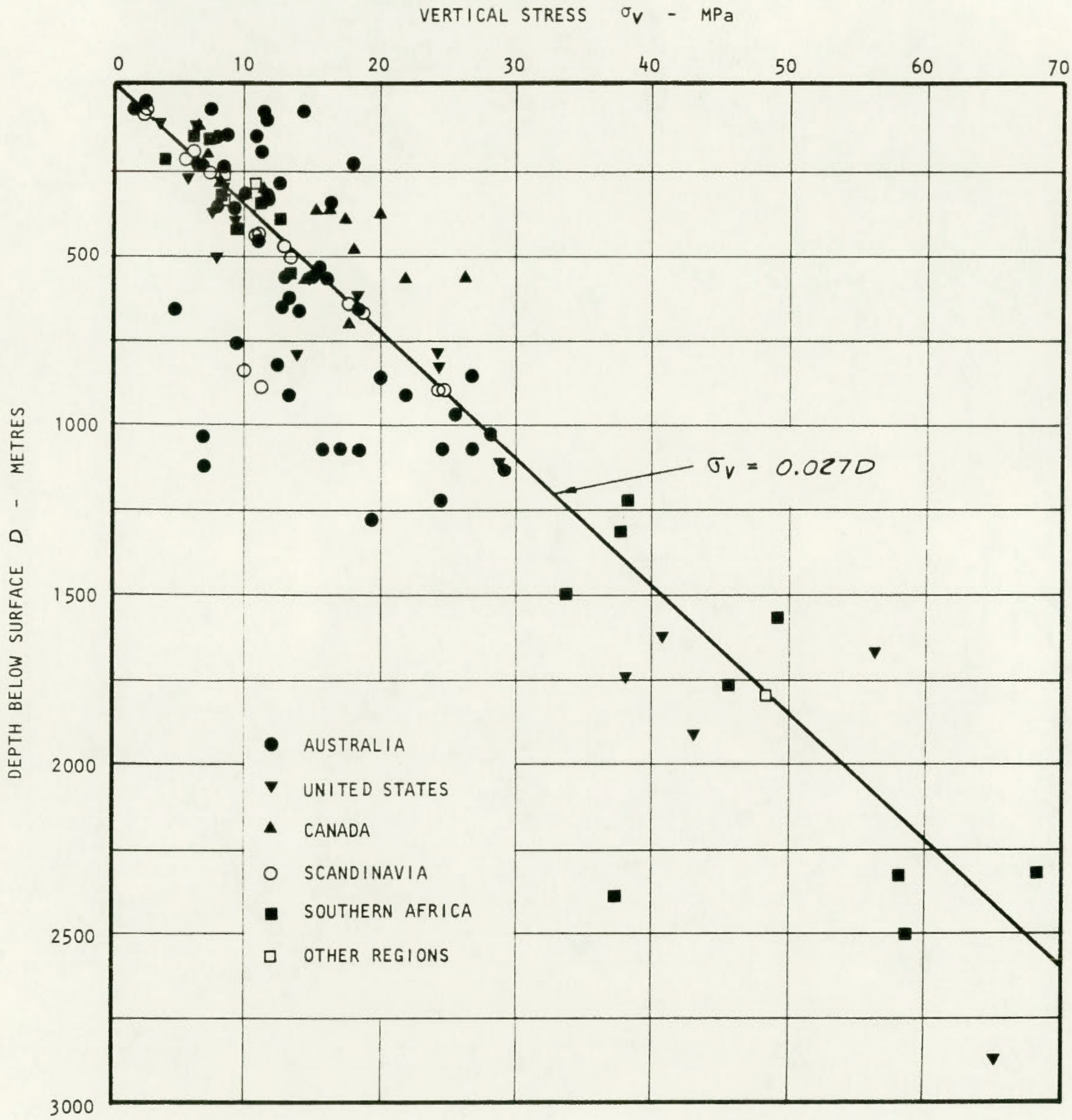
**Golder Associates**

GRAPH TO DETERMINE THE ROCK STRESS FACTOR (A)

**Figure 3-1**

# PLOT OF VERTICAL STRESSES AGAINST DEPTH BELOW SURFACE

**Figure** 3-2

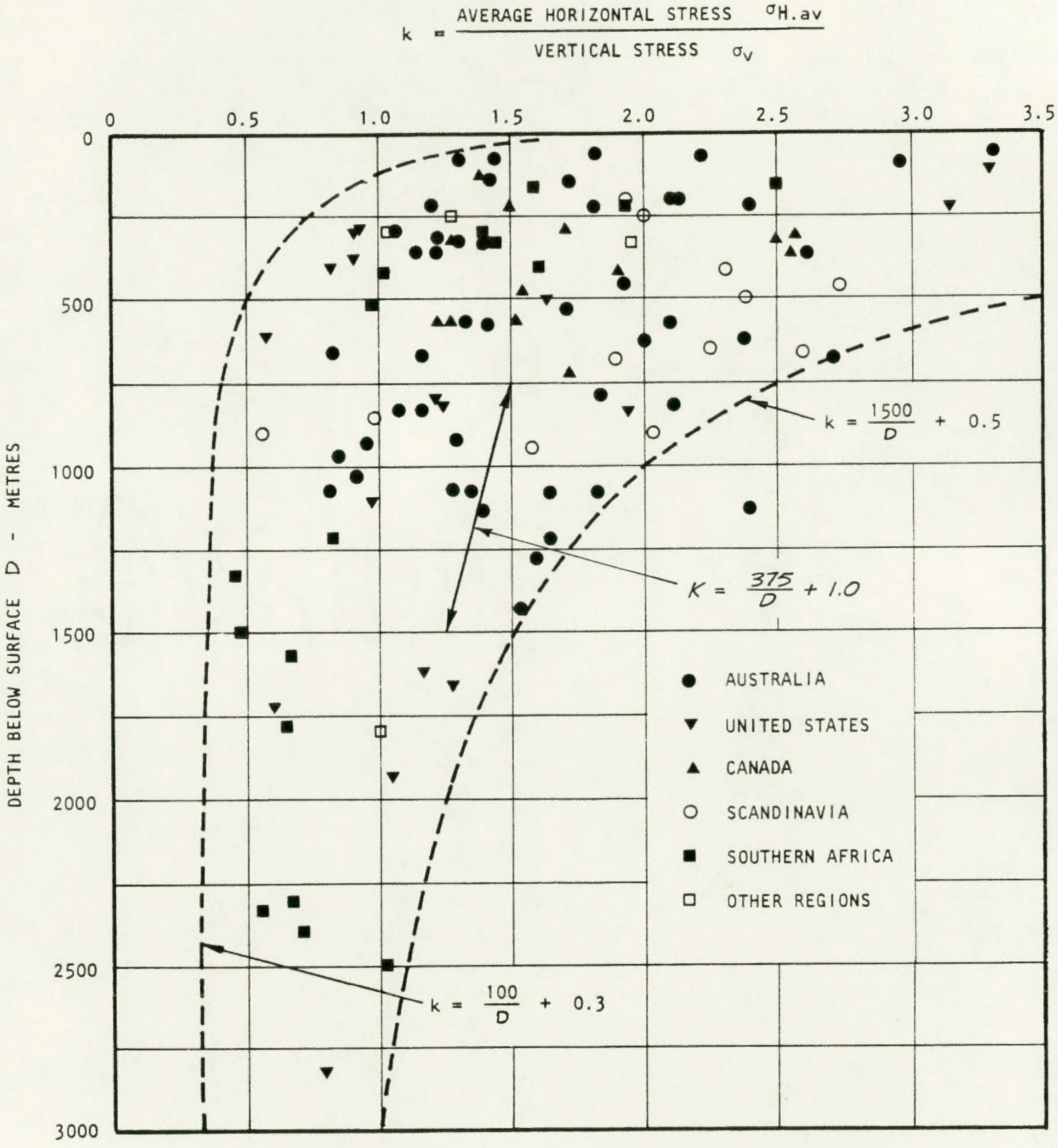



(After Hoek and Brown, 1980)

Project No. 802-1571 Drawn Date Feb. '81

# VARIATION OF RATIO OF AVERAGE HORIZONTAL STRESS TO VERTICAL STRESS WITH DEPTH BELOW SURFACE

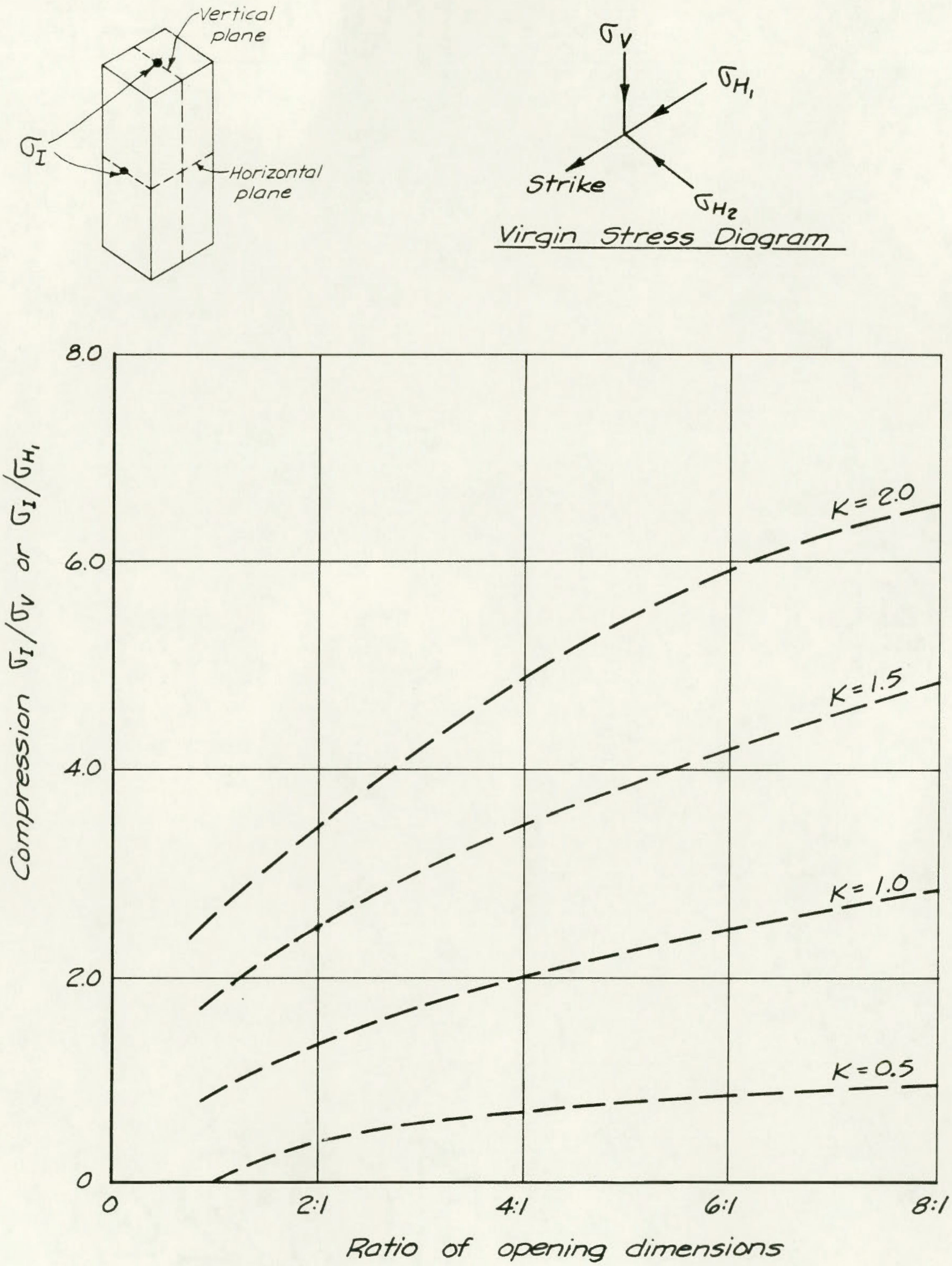
**Figure** 3 - 3




(After Hoek and Brown, 1980)

Project No. 802-1571 Drawn - Reviewed SS Date Feb. '81

GRAPH TO DETERMINE INDUCED STRESSES IN BACKS AND ENDS OF SINGLE OPENINGS **Figure 3-4**




LEGEND

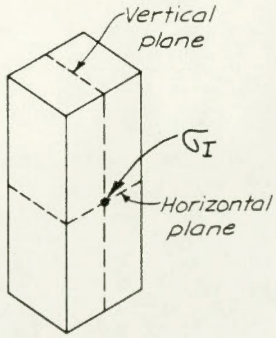
$\sigma_I$ = Induced stress
$\sigma_V$ = Vertical virgin stress
$\sigma_{H_1}$ = Horizontal virgin stress on strike
$\sigma_{H_2}$ = Horizontal virgin stress normal to strike
Horizontal plane, $K = \sigma_{H_2}/\sigma_{H_1}$
Vertical plane, $K = \sigma_{H_2}/\sigma_V$

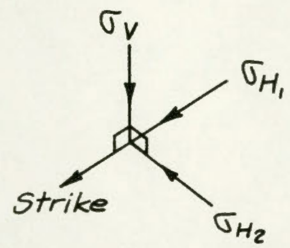
Project No. 802-1571 Drawn R.D. Reviewed Date Mar. '81

**Golder Associates**

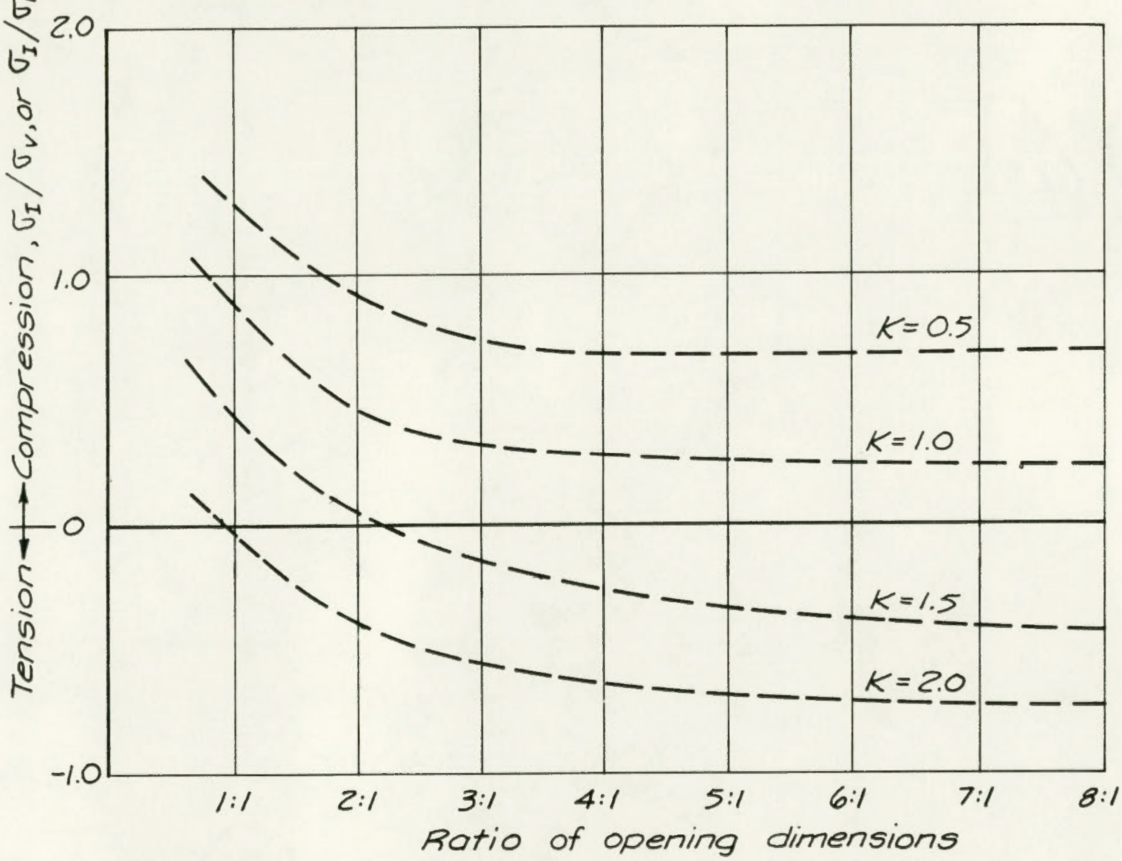
## GRAPH TO DETERMINE INDUCED STRESSES IN HANGINGWALLS OF SINGLE OPENINGS

**Figure 3-5**




Virgin Stress Diagram



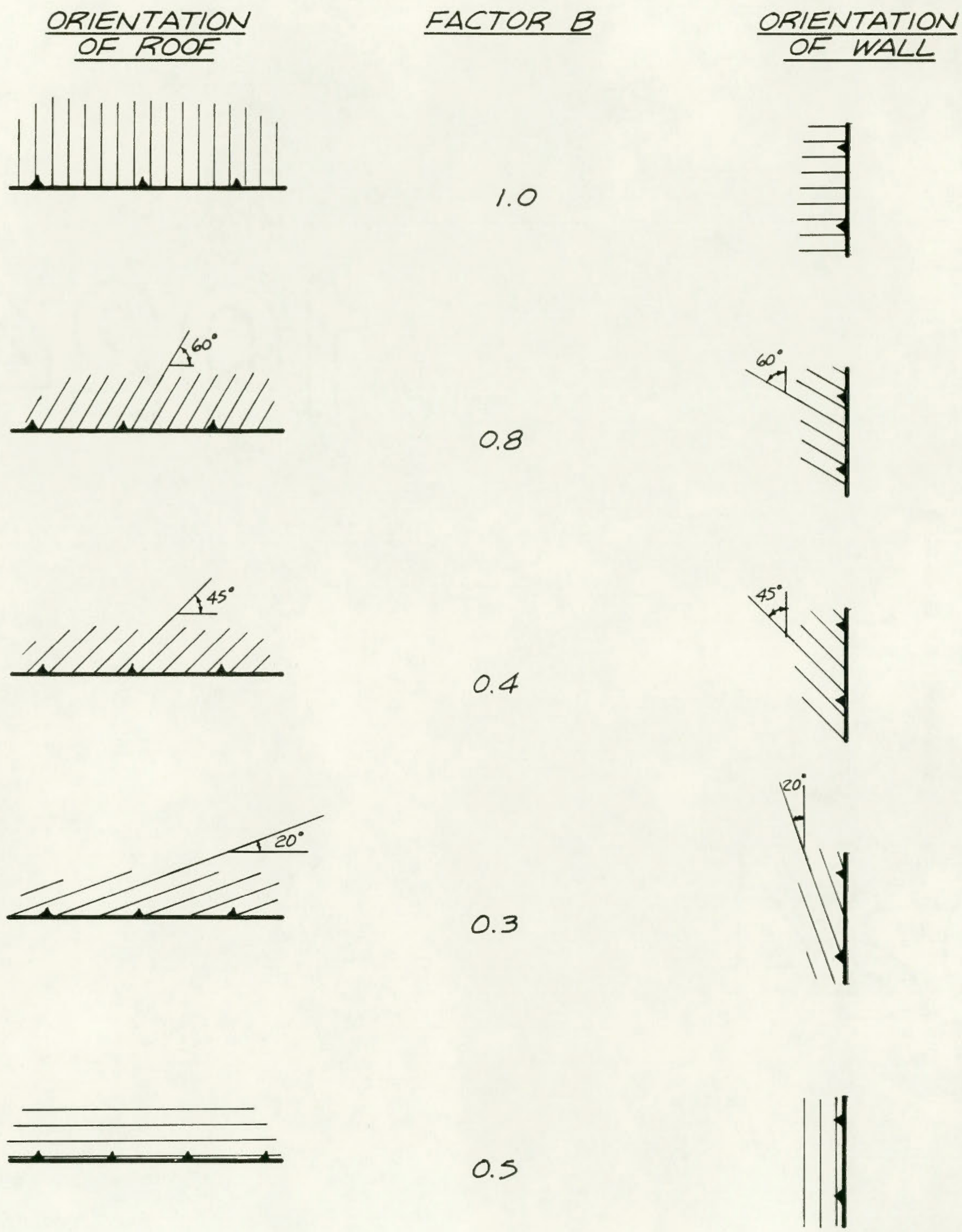

LEGEND

$\sigma_I$ = Induced stress
$\sigma_V$ = Vertical virgin stress
$\sigma_{H_1}$ = Horizontal virgin stress on strike
$\sigma_{H_2}$ = Horizontal virgin stress normal to strike
Horizontal plane, $K = \sigma_{H_2}/\sigma_{H_1}$
Vertical plane, $K = \sigma_{H_2}/\sigma_V$

Project No. 802-1571 Drawn R.D Reviewed Date Mar. '81

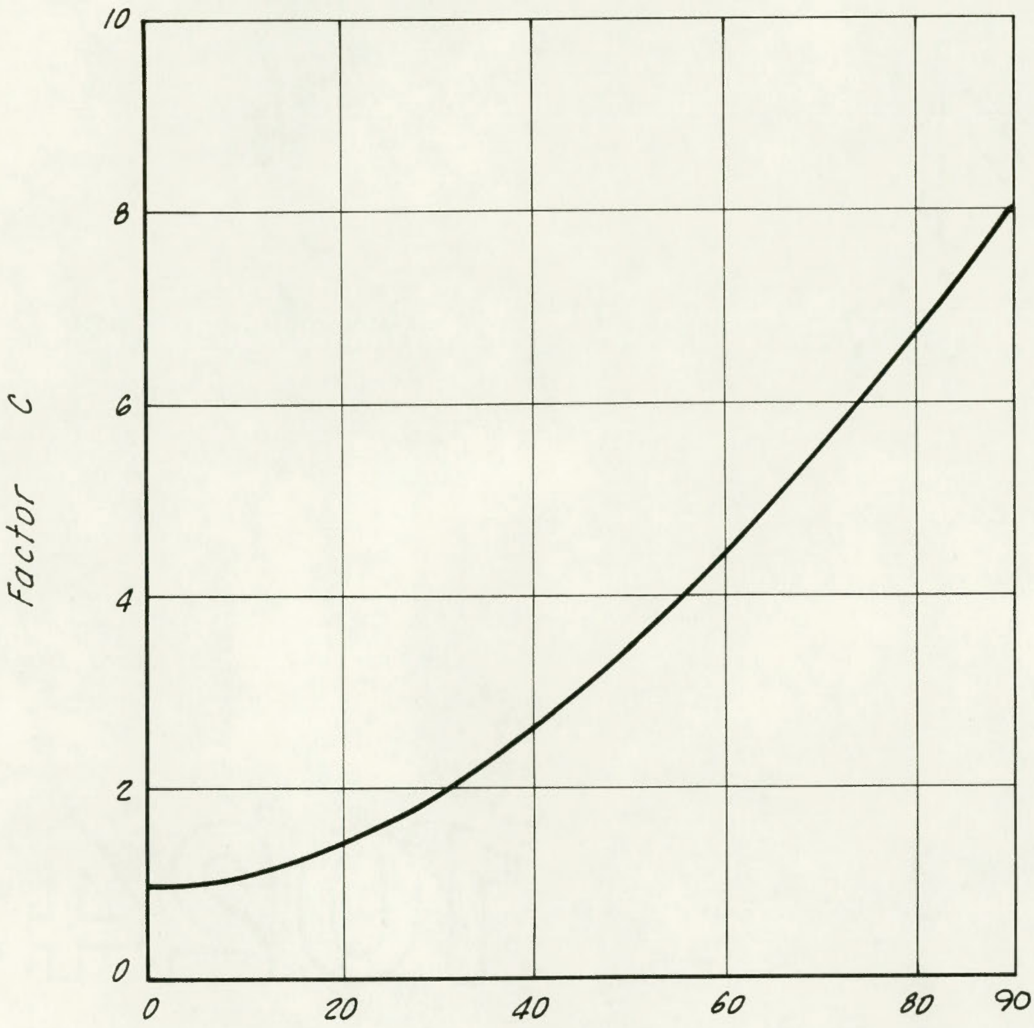
Golder Associates

# ROCK DEFECT ORIENTATION FACTOR (B) **Figure 3-6**




Project No. 802-1571 Drawn R.D. Reviewed [illegible] Date Mar. '81

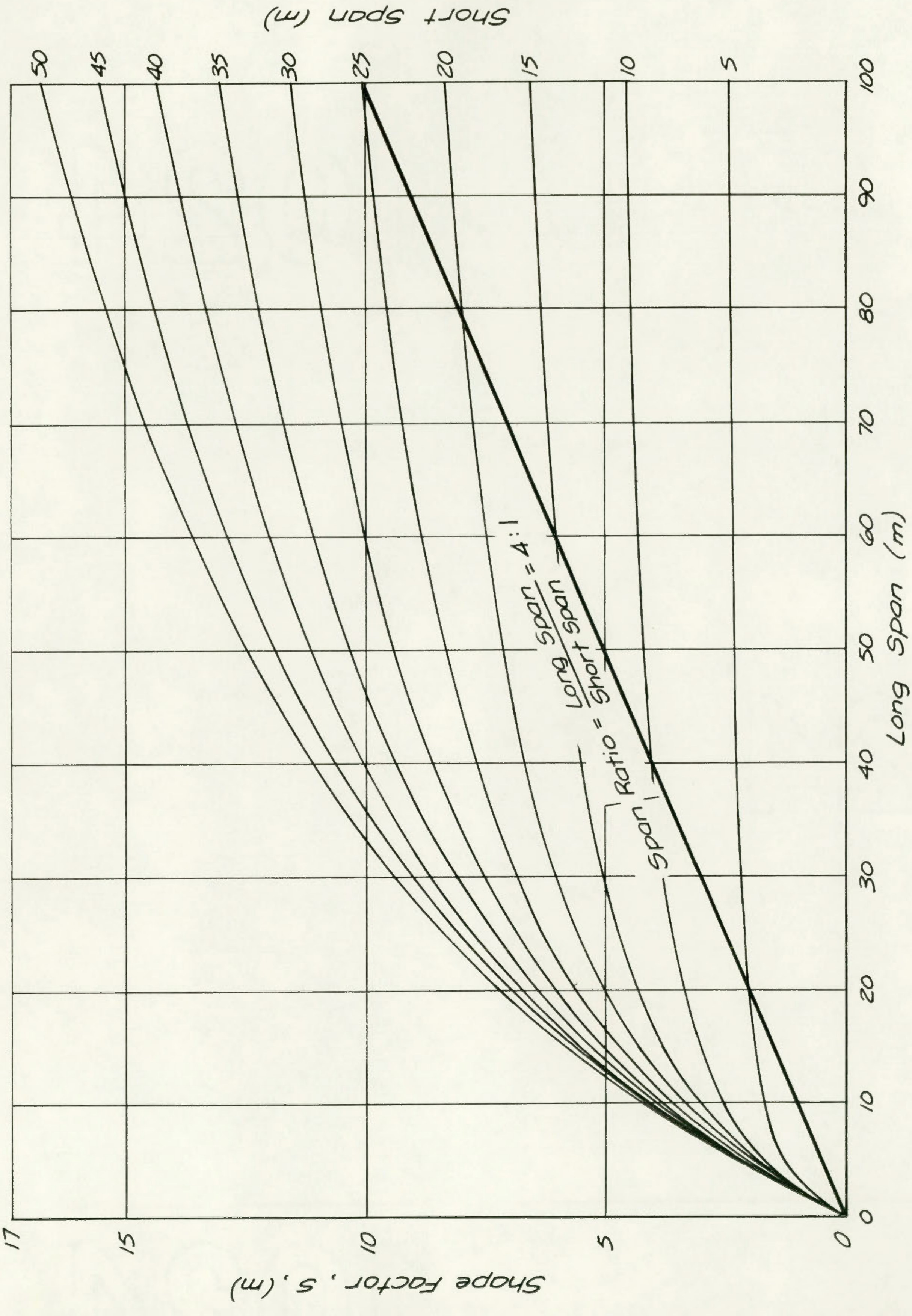
DESIGN SURFACE ORIENTATION FACTOR (C) **Figure** 3-7




Factor C = 8 - 7 Cosine (angle of dip)

Project No. 802-1571 Drawn R.D. Reviewed Date Feb. '81

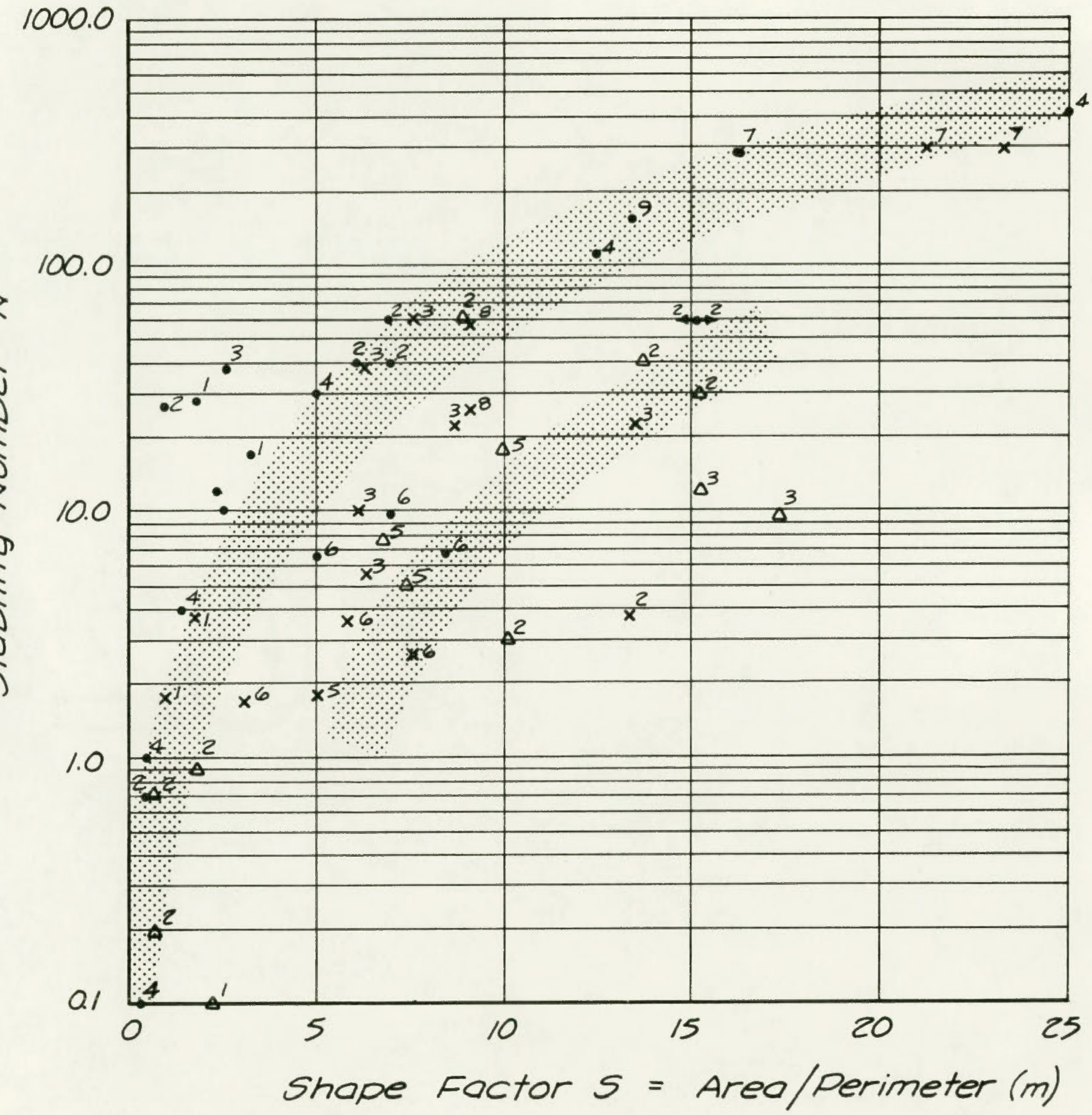
## RELATIONSHIP OF SHAPE FACTOR VS SPANS **Figure 3-8**




Project No. 802-1571 Drawn R.D Reviewed Date Mar. '81

## PLOT OF STABILITY NUMBER (N) VERSUS SHAPE FACTOR (S)

**Figure 4-1**



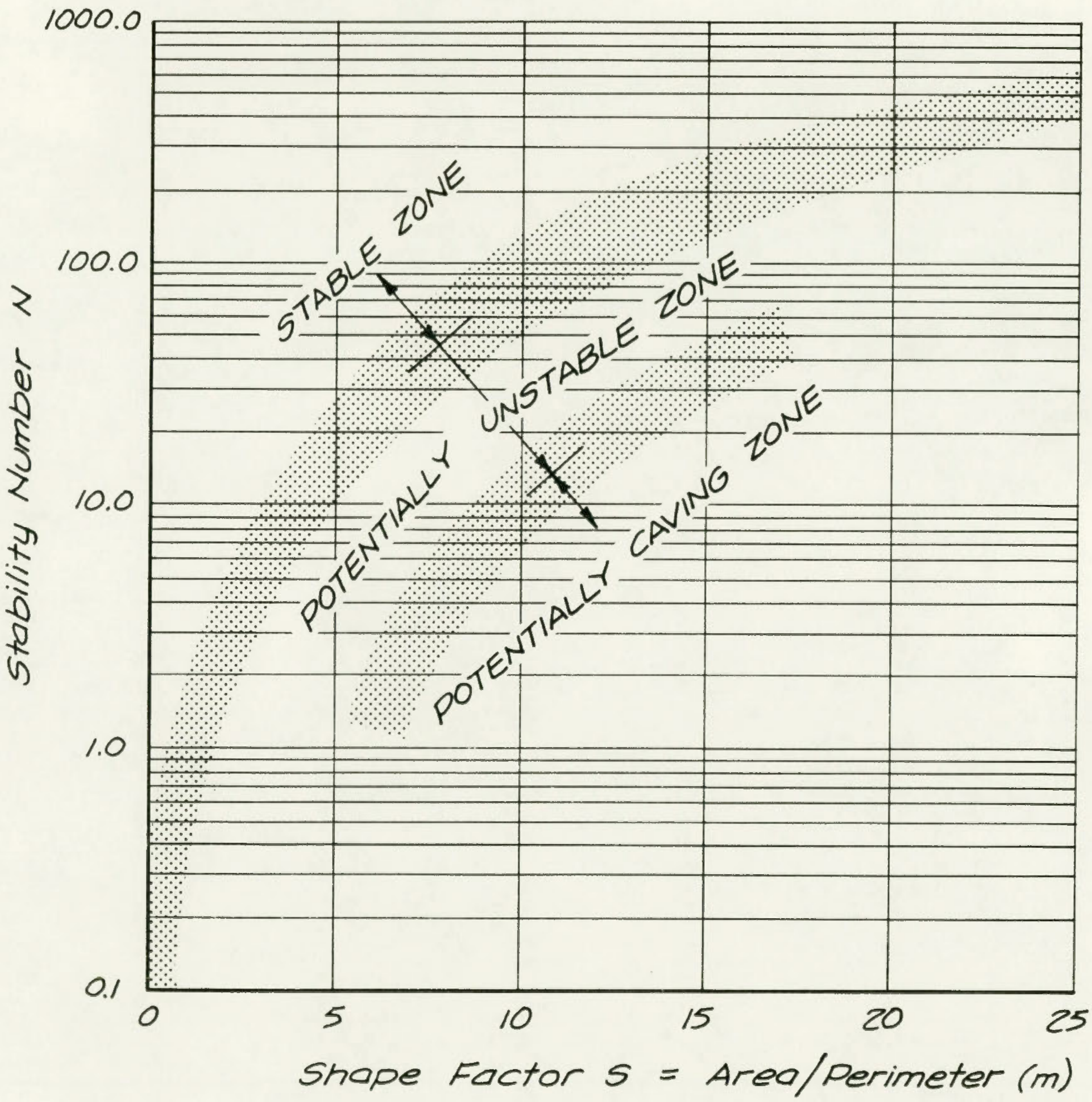

LEGEND

- • Stable
- × Unstable
- △ Cave

Numeral refers to set number in table V-1 of Appendix V.

GRAPH OF STABILITY NUMBER (N) VERSUS SHAPE FACTOR (S)

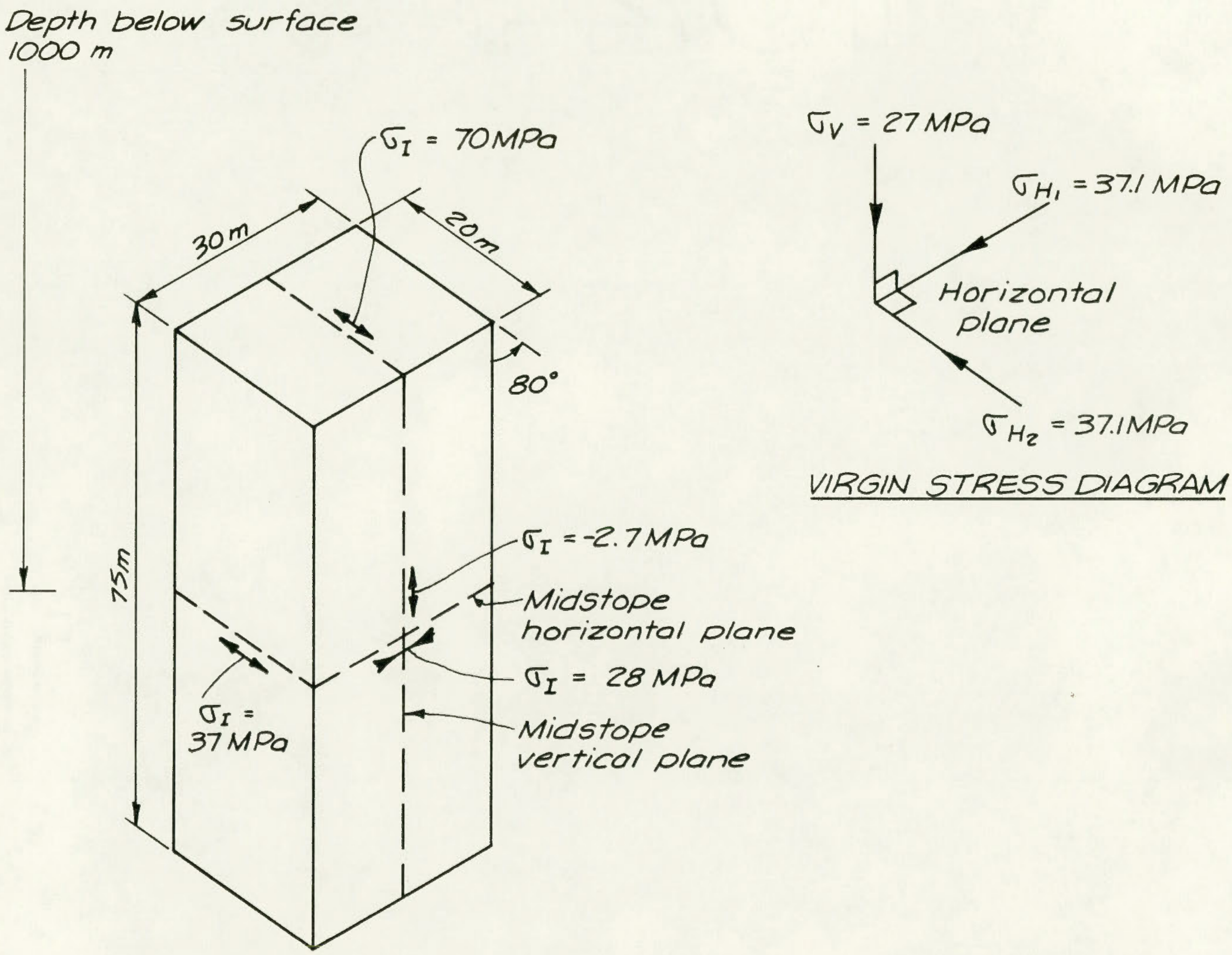
**Figure 4 - 2**




Project No. 802-1571 Drawn R.D. Reviewed Date Mar. '81

DIAGRAM FOR OPEN STOPE DESIGN EXERCISE

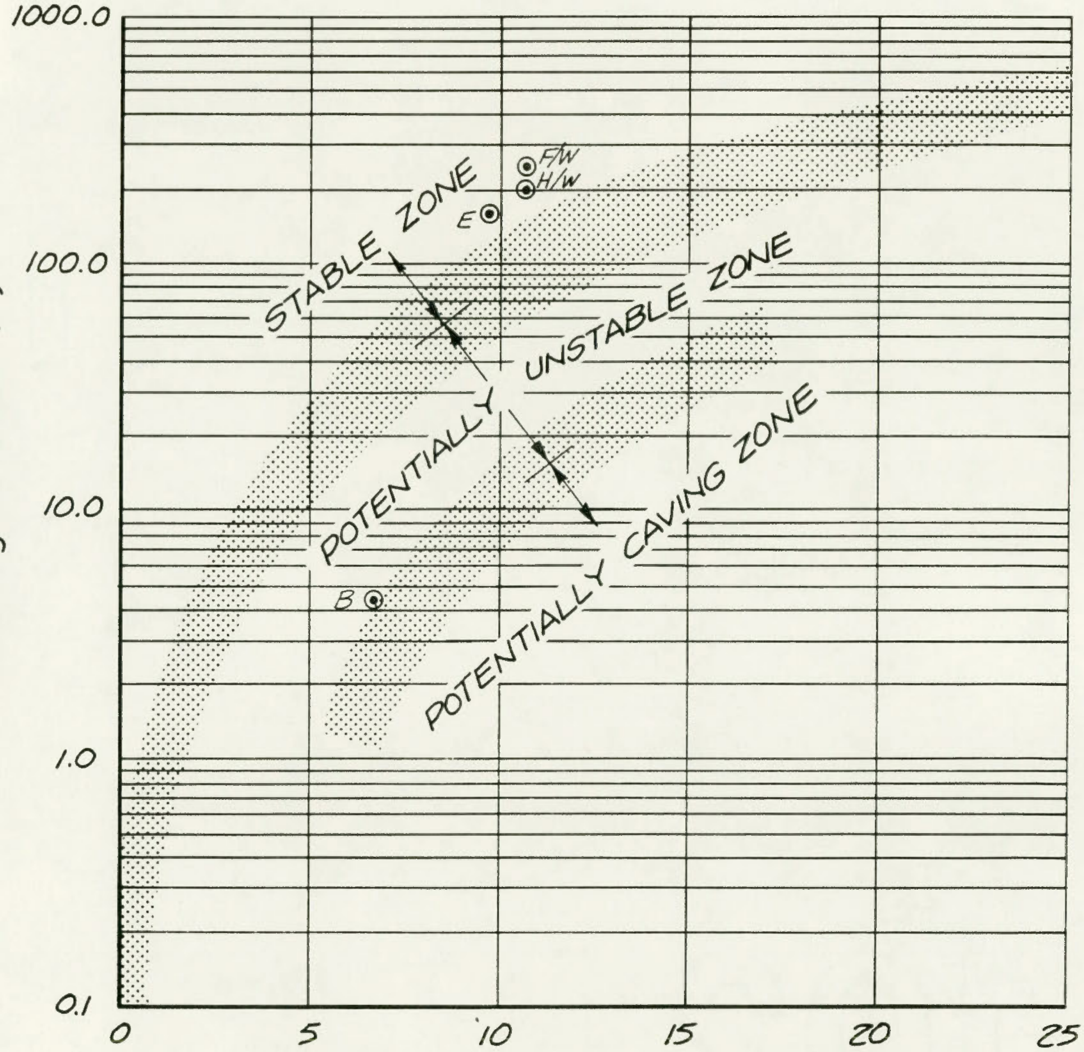
**Figure 4-3**




Project No. 802-1571 Drawn R.D. Reviewed Date Mar. '81

PLOT OF STABILITY RESULTS - OPEN STOPE DESIGN EXERCISE

**Figure 4-4**




⊙ B = Back (roof)
H = Hangingwall
F = Footwall
E = Vertical end

Project No. 802-1571 Drawn R.D. Reviewed Date Mar. '81

# APPENDIX I

## SUMMARY DESCRIPTION OF NGI QUALITY INDEX

(Extract From "Underground Excavations in Rock", Hoek & Brown 1980)

NGI Tunnelling Quality Index

On the basis of an evaluation of a large number of case histories of underground excavation stability, Barton, Lien and Lunde[1] of the Norwegian Geotechnical Institute (NGI) proposed an index for the determination of the tunnelling quality of a rock mass. The numerical value of this index Q is defined by :

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

where

RQD is Deere's Rock Quality Designation as defined on page 18,

$J_n$ is the joint set number,

$J_r$ is the joint roughness number,

$J_a$ is the joint alteration number,

$J_w$ is the joint water reduction factor, and

SRF is a stress reduction factor.

The definition of these terms is largely self-explanatory, particularly when the numerical value of each is determined from Table 7.

In explaining how they arrived at the equation used to determine the index Q, Barton, Lien and Lunde offer the following comments :

" The first quotient ($RQD/J_n$), representing the structure of the rock mass, is a crude measure of the block or particle size, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400. If the quotient is interpreted in units of centimetres, the extreme "particle sizes" of 200 to 0.5 cms are seen to be crude but fairly realistic approximations. Probably the largest blocks should be several times this size and the smallest fragments less than half the size. ( Clay particles are of course excluded ).

The second quotient ($J_r/J_a$) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favour of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will tend to dilate strongly when sheared, and that they will therefore be especially favourable to tunnel stability.
When rock joints have thin clay mineral coatings and fillings, the strength is reduced significantly. Nevertheless, rock wall contact after small shear displacements have occurred may be a very important factor for preserving the excavation from ultimate failure.
Where no rock wall contact exists, the conditions are extremely unfavourable to tunnel stability. The "friction angles" given in Table 7 are a little below the residual strength values for most clays, and are possibly downgraded by the fact that these clay bands or fillings may tend to consolidate during shear, at least if normally consolidated or if softening and swelling has occurred. The swelling pressure of montmorillonite may also be a factor here.
The third quotient ($J_w/SRF$) consists of two stress parameters. SRF is a measure of: 1. loosening load in the case of an excavation through shear zones and clay bearing rock, 2. rock stress in competent rock and 3. squeezing loads in plastic incompetent rocks. It can be regarded as a total stress parameter. The parameter $J_w$ is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. Water may, in addition,

cause softening and possible outwash in the case of clay-filled joints. It has proved impossible to combine these two parameters in terms of inter-block effective normal stress, because paradoxically a high value of effective normal stress may sometimes signify less stable conditions than a low value, despite the higher shear strength. The quotient ($J_w$/SRF) is a complicated empirical factor describing the "active stresses".

It appears that the rock tunnelling quality Q can now be considered as a function of only three parameters which are crude measures of :

1. block size ($RQD/J_n$)
2. inter-block shear strength ($J_r/J_a$)
3. active stress ($J_w$/SRF)

Undoubtedly, there are several other parameters which could be added to improve the accuracy of the classification system. One of these would be joint orientation. Although many case records include the necessary information on structural orientation in relation to excavation axis, it was not found to be the important general parameter that might be expected. Part of the reason for this may be that the orientations of many types of excavation can be, and normally are, adjusted to avoid the maximum effect of unfavourably oriented major joints. However, this choice is not available in the case of tunnels, and more than half the case records were in this category. The parameters $J_n$, $J_r$ and $J_a$ appear to play a more important general role than orientation, because the number of joint sets determines the degree of freedom for block movement (if any), and the frictional and dilational characteristics can vary more than the down-dip gravitational component of unfavourably orientated joints. If joint orientation had been included the classification would have been less general, and its essential simplicity lost."

The large amount of information contained in Table 7 may lead the reader to suspect that the NGI Tunnelling Quality Index is unnecessarily complex and that it would be difficult to use in the analysis of practical problems. This is far from the case and an attempt to determine the value of Q for a typical rock mass will soon convince the reluctant user that the instructions are simple and unambiguous and that, with familiarity, Table 7 becomes very easy to use. Even before the value of Q is calculated, the process of determining the various factors required for its computation concentrates the attention of the user onto a number of important practical questions which can easily be ignored during a site investigation. The qualitative "feel" for the rock mass which is acquired during this process may be almost as important as the numerical value of Q which is subsequently calculated.

In order to relate their Tunnelling Quality Index Q to the behaviour and support requirements of an underground excavation, Barton, Lien and Lunde defined an additional quantity which they call the *equivalent dimension* $D_e$ of the excavation. This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the *excavation support ratio* ESR.

Hence :

$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio}}$$

The excavation support ratio is related to the use for which the excavation is intended and the extent to which some degree of instability is acceptable. Barton[29] gives the following suggested values for ESR :

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

The ESR is roughly analogous to the inverse of the *factor of safety* used in the design of rock slopes[2].

The relationship between the Tunnelling Quality Index Q and the Equivalent Dimension $D_e$ of an excavation which will stand unsupported is illustrated in figure 7. Much more elaborate graphs from which support requirements can be estimated were presented by Barton, Lien and Lunde[1] and Barton[29]. A discussion of these graphs will be deferred to a later chapter in which excavation support will be discussed more fully.

*Practical example using the NGI Tunnelling Quality Index.*

An underground crusher station is to be excavated in the limestone footwall of a lead-zinc ore body and it is required to find the span which can be left unsupported. The analysis is carried out as follows :

| | *Item* | *Description* | *Value* |
|---|---|---|---|
| 1. | Rock Quality | Good | RQD = 80% |
| 2. | Joint sets | Two sets | $J_n = 4$ |
| 3. | Joint roughness | Rough | $J_r = 3$ |
| 4. | Joint alteration | Clay gouge | $J_a = 4$ |
| 5. | Joint water | Large inflow | $J_w = 0.33$ |
| 6. | Stress reduction | Medium stress | SRF = 1.0 |

Hence

$$Q = \frac{80}{4} \times \frac{3}{4} \times \frac{0.33}{1} = 5$$

TABLE 7 - CLASSIFICATION OF INDIVIDUAL PARAMETERS USED IN THE NGI TUNNELLING QUALITY INDEX

| *Description* | *Value* | *Notes* |
|---|---|---|
| 1. ROCK QUALITY DESIGNATION | RQD | |
| A. Very poor | 0 - 25 | 1. Where RQD is reported or measured as $\leq 10$ ( including 0 ), a nominal value of 10 is used to evaluate Q. |
| B. Poor | 25 - 50 | |
| C. Fair | 50 - 75 | |
| D. Good | 75 - 90 | 2. RQD intervals of 5, i.e. 100, 95, 90 etc are sufficiently accurate. |
| E. Excellent | 90 - 100 | |
| 2. JOINT SET NUMBER | $J_n$ | |
| A. Massive, no or few joints | 0.5 - 1.0 | |
| B. One joint set | 2 | |
| C. One joint set plus random | 3 | |
| D. Two joint sets | 4 | |
| E. Two joint sets plus random | 6 | |
| F. Three joint sets | 9 | 1. For intersections use $(3.0 \times J_n)$ |
| G. Three joint sets plus random | 12 | 2. For portals use $(2.0 \times J_n)$ |
| H. Four or more joint sets, random, heavily jointed 'sugar cube', etc | 15 | |
| J. Crushed rock, earthlike | 20 | |
| 3. JOINT ROUGHNESS NUMBER | $J_r$ | |
| *a. Rock wall contact and* | | |
| *b. Rock wall contact before 10 cms shear.* | | |
| A. Discontinuous joints | 4 | |
| B. Rough or irregular, undulating | 3 | |
| C. Smooth, undulating | 2 | |
| D. Slickensided, undulating | 1.5 | |
| E. Rough or irregular, planar | 1.5 | 1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3m. |
| F. Smooth, planar | 1.0 | |
| G. Slickensided, planar | 0.5 | 2. $J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided the lineations are orientated for minimum strength. |
| *c. No rock wall contact when sheared.* | | |
| H. Zone containing clay minerals thick enough to prevent rock wall contact. | 1.0 | |
| J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact. | 1.0 | |
| 4. JOINT ALTERATION NUMBER | $J_a$ | $\phi_r$(approx.) |
| *a. Rock wall contact.* | | |
| A. Tightly healed, hard, non-softening, impermeable filling | 0.75 | — |

| | $J_a$ | $\phi_r$ (approx.) | |
|---|---|---|---|
| B. Unaltered joint walls, surface staining only | 1.0 | $(25^\circ - 35^\circ)$ | |
| C. Slightly altered joint walls non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc | 2.0 | $(25^\circ - 30^\circ)$ | 1. Values of $\phi_r$, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present. |
| D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening) | 3.0 | $(20^\circ - 25^\circ)$ | |
| E. Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2mm or less in thickness) | 4.0 | $(8^\circ - 16^\circ)$ | |
| *b. Rock wall contact before 10 cms shear.* | | | |
| F. Sandy particles, clay-free disintegrated rock etc | 4.0 | $(25^\circ - 30^\circ)$ | |
| G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5mm thick) | 6.0 | $(16^\circ - 24^\circ)$ | |
| H. Medium or low over-consolidation, softening, clay mineral fillings, (continuous, < 5mm thick) | 8.0 | $(12^\circ - 16^\circ)$ | |
| J. Swelling clay fillings, i.e. montmorillonite (continuous, < 5 mm thick ). Values of $J_a$ depend on percent of swelling clay-size particles, and access to water | 8.0 - 12.0 | $(6^\circ - 12^\circ)$ | |
| *c. No rock wall contact when sheared.* | | | |
| K. L. M. Zones or bands of disintegrated or crushed rock and clay (see G,H and J for clay conditions) | 6.0<br>8.0<br>8.0 - 12.0 | $(6^\circ - 24^\circ)$ | |
| N. Zones or bands of silty- or sandy clay, small clay fraction, (non-softening) | 5.0 | | |
| Q. P. R. Thick, continuous zones or bands of clay ( see G, H and J for clay conditions) | 10.0 - 13.0<br>13.0 - 20.0 | $(6^\circ - 24^\circ)$ | |

| 5. JOINT WATER REDUCTION FACTOR | $J_w$ | approx. water pressure ($Kgf/cm^2$) | |
|---|---|---|---|
| A. Dry excavations or minor inflow, i.e. < 5 lit/min. locally | 1.0 | < 1.0 | |
| B. Medium inflow or pressure, occasional outwash of joint fillings | 0.66 | 1.0 - 2.5 | |
| C. Large inflow or high pressure in competent rock with unfilled joints | 0.5 | 2.5 - 10.0 | 1. Factors C to F are crude estimates. Increase $J_w$ if drainage measures are installed. |
| D. Large inflow or high pressure , considerable outwash of fillings | 0.33 | 2.5 - 10.0 | |
| E. Exceptionally high inflow or pressure at blasting, decaying with time | 0.2 - 0.1 | > 10 | 2. Special problems caused by ice formation are not considered. |
| F. Exceptionally high inflow or pressure continuing without decay | 0.1 - 0.05 | > 10 | |

### 6. STRESS REDUCTION FACTOR

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

| | SRF |
|---|---|
| A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth) | 10.0 |
| B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50m) | 5.0 |
| C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50m) | 2.5 |
| D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth ) | 7.5 |
| E. Single shear zones in competent rock (clay free), (depth of excavation < 50m) | 5.0 |
| F. Single shear zones in competent rock (clay free), (depth of excavation > 50m) | 2.5 |
| G. Loose open joints, heavily jointed or 'sugar cube' (any depth) | 5.0 |

*b. Competent rock, rock stress problems*

| | $\sigma_c/\sigma_1$ | $\sigma_t/\sigma_1$ | SRF |
|---|---|---|---|
| H. Low stress, near surface | >200 | >13 | 2.5 |
| J. Medium stress | 200-10 | 13-0.66 | 1.0 |
| K. High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability) | 10-5 | 0.66-0.33 | 0.5-2 |
| L. Mild rock burst (massive rock) | 5-2.5 | 0.33-0.16 | 5-10 |
| M. Heavy rock burst (massive rock) | <2.5 | <0.16 | 10-20 |

*c. Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure*

| | SRF |
|---|---|
| N. Mild squeezing rock pressure | 5-10 |
| O. Heavy squeezing rock pressure | 10-20 |

*d. Swelling rock, chemical swelling activity depending upon presence of water*

| | SRF |
|---|---|
| P. Mild swelling rock pressure | 5-10 |
| R. Heavy swelling rock pressure | 10-20 |

1. Reduce these values of SRF by 25 - 50% if the relevent shear zones only influence but do not intersect the excavation.
2. For strongly anisotropic virgin stress field (if measured) : when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce $\sigma_c$ to $0.8\sigma_c$ and $\sigma_t$ to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce $\sigma_c$ and $\sigma_t$ to $0.6\sigma_c$ and $0.6\sigma_t$, where $\sigma_c$ = unconfined compressive strength, and $\sigma_t$ = tensile strength (point load) and $\sigma_1$ and $\sigma_3$ are the major and minor principal stresses.
3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

### ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in the tables:

1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay free rock masses :
   $RQD = 115 - 3.3J_v$ (approx.) where $J_v$ = total number of joints per $m^3$
   ($RQD = 100$ for $J_v < 4.5$)
2. The parameter $J_n$ representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible, or only occasional breaks in the core due to these features, then it will be more appropriate to count them as "random joints" when evaluating $J_n$.
3. The parameters $J_r$ and $J_a$ (representing shear strength) should be relevant to the *weakest significant joint set or clay filled discontinuity* in the given zone. However, if the joint set or discontinuity with the minimum value of $(J_r/J_a)$ is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of $J_r/J_a$ should be used when evaluating Q . *The value of $J_r/J_a$ should in fact relate to the surface most likely to allow failure to initiate.*
4. When a rock mass contains clay, the factor SRF appropriate to *loosening loads* should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may

TABLE 7 CONTINUED

| |
|---|
| become the weakest link, and the stability will then depend on the ratio rock-stress/ rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.<br>5. The compressive and tensile strengths ($\sigma_c$ and $\sigma_t$) of the intact rock should be evaluated in the saturated condition if this is appropriate to present or future in situ conditions. A very conservative estimate of strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions. |

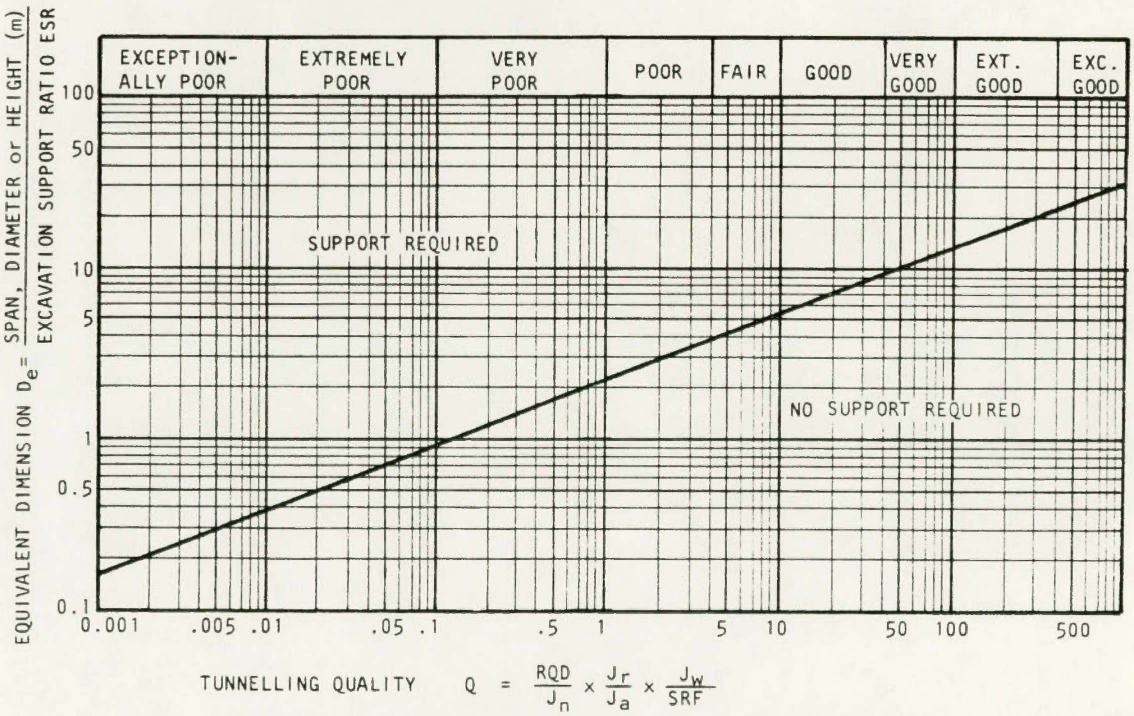



Figure 7 . Relationship between the maximum equivalent dimension $D_e$ of an unsupported underground excavation and the NGI tunnelling quality index Q. (After Barton, Lien and Lunde[1])

From figure 7, the maximum equivalent dimension $D_e$ for an unsupported excavation in this rock mass is 4 metres. A permanent underground mine opening has an excavation support ratio ESR of 1.6 and, hence the maximum unsupported span which can be considered for this crusher station is $ESR \times D_e = 1.6 \times 4 = 6.4$ metres.

## Discussion on rock mass classification systems

Of the several rock mass classification systems described in this chapter, the CSIR system proposed by Bieniawski[25,26] and the NGI system proposed by Barton, Lien and Lunde[1] are of particular interest because they include sufficient information to provide a realistic assessment of the factors which influence the stability of an underground excavation. Bieniawski's classification appears to lay slightly greater emphasis on the orientation and inclination of the structural features in the rock mass while taking no account of the rock stress. The NGI classification does not include a joint orientation term but the properties of the most unfavourable joint sets are considered in the assessment of the joint roughness and the joint alteration numbers, both of which represent the shear strength of the rock mass.

Both classification systems suggest that the influence of structural orientation and inclination is less significant than one would normally tend to assume and that a differentiation between *favourable* and *unfavourable* is adequate for most practical purposes. While this may be acceptable for the majority of situations likely to be encountered in the field, there are a few cases in materials such as slate where the structural features are so strongly developed that they will tend to dominate the behaviour of the rock mass. In other situations, large blocks may be isolated by a small number of individual discontinuities and become unstable when the excavation is created. In such cases, the classification systems discussed in this chapter may not be adequate and special consideration may have to be given to the relationship between the geometry of the rock mass and that of the excavation. This subject will be dealt with in chapter 7 of this book.

The authors have used both the CSIR and the NGI systems in the field and have found both to be simple to use and of considerable assistance in making difficult practical decisions. In most cases, both classifications are used and both the Rock Mass Rating (RMR) and the Tunnelling Quality (Q) are used in deciding upon the solution to the problem. It has been found that the equation $RMR = 9 \, Log_e Q + 44$ proposed by Bieniawski[26] adequately describes the relationship between the two systems.

When dealing with problems involving extremely weak ground which result in squeezing, swelling or flowing conditions (see Terzaghi's classification in Table 1 on page 17), it has been found that the CSIR classification is difficult to apply. This is hardly surprising given that the system was originally developed for shallow tunnels in hard jointed rock. Hence, when working in extremely weak ground, the authors recommend the use of the NGI system.

In discussing the CSIR and NGI classification systems, the authors have concentrated upon the basic rock mass classification and on the indication given by this classification of whether support is required or not. Bieniawski[25,26] and Barton, Lein and Lunde[1] went on to apply these classifications to the choice of specific support systems. The detailed design of support for underground excavations, including the use or rock mass classifications to assist in the choice of support systems, will be discussed in chapter 8 of this book.

# APPENDIX II

## SUMMARY COMPARISON OF ROCK MASS CLASSIFICATION SYSTEMS DISCUSSED

## APPENDIX II

## COMPARISON OF ROCK MASS CLASSIFICATION SYSTEMS DISCUSSED

Table II-1 shows the data that is required for each of the classification systems described in Chapter 2 of the report.

The purpose of this table is to show the number of rock mass properties that various geologists and engineers consider to be relevant to the design of underground openings. The table also shows the number of different purposes for which classification systems have been used.

The successful use of rock mass classification for engineering purposes depends upon the user keeping in mind the purpose of the work, and also using a system that is suited to the rock in which the opening will be excavated. Therefore, in some cases, it may be necessary to modify an existing system to suit particular conditions. Table II-1 shows the number of parameters which may be considered for the modified classification system presented in this study.

TABLE II - 1

ROCK MASS CLASSIFICATION SYSTEMS

| Parameter | Intact Strength | Deformability | Fracture Intensity | RQD | Infilling Type | Joint Spacing | Joint Friction | Joint roughness | Joint Orientation | Water Pressure | Stress Conditions | Weathering | Rock Damage | Purpose of Classification |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Terzaghi | ● | ● | ● | | | | | | | | ● | | | Estimate rock load on steel supports in tunnels. |
| Stini and Lauffer | ● | ● | ● | | | | | | ● | | ● | | | Estimate active span and stand-up time in tunnels. |
| Deere | | | | ● | | | | | | | | | | Estimate support requirements, i.e. no support, rock bolts, steel ribs in tunnels. |
| Brekke and Howard | | | | | ● | | | | | | | | | Classification of infilling types and their influence on tunnel stability. |
| Patching and Coates | ● | ● | ● | | | | | | | | | | | General engineering classification of rock mass. |
| CSIR | ● | | ● | ● | | | ● | ● | ● | ● | | | | Classification relates stand-up time to unsupported tunnel span. |
| Laubscher and Taylor | ● | | ● | ● | ● | ● | ● | ● | ● | ● | ● | ● | ● | Used in underground mining to determine support requirements, cavability, angles of cave, open stoping feasibility. Also pit slope angles. |
| NGI | ● | | ● | ● | | | ● | ● | ● | ● | ● | | | Classification relates span to support requirements and intended use of excavation. Primarily designed for tunnels |
| Golder Associates | ● | | ● | ● | | | ● | ● | ● | ● | ● | | | Design of open stopes |

# APPENDIX III

## BRIEF SUMMARY OF THE APPLICATION OF NUMERICAL MODELLING TECHNIQUES

# APPENDIX III - LIST OF FIGURES

# APPENDIX III

## BRIEF SUMMARY OF THE APPLICATION OF NUMERICAL MODELLING TECHNIQUES

The last fifteen years have seen significant advances in the development of computer programs to model stress and displacements around excavations underground. Photo-elastic and electric analogue models which were important before the mid sixties, are now little used because computer based models offer greater flexibility in modelling.

The finite element method is a well accepted tool in rock mechanics. It is used for elastic and yielding models and can include slip or separation effects on special joint elements. Complex excavation shapes can be modelled in two and three dimensions. The only limitations are the time taken to set up the meshes to model a suitable volume of rock around the excavation and the computing costs for large models.

Although the finite element method is still important, several alternatives based on the face element or boundary element method offer similar results with less data preparation and reduced computing costs. In these methods, the influence of the surrounding rock mass is represented using elastic theory and the model definition requires only the excavation boundaries and sometimes special joint elements. The first mining application of this type was developed by Salamon and others to model the extensive tabular excavations of the South African gold mines. This led to the MINSIM computer program and later extensions of it to MSIM3D and NFOLD. Related developments are the two dimensional displacement discontinuity method and various boundary integral methods in two and three dimensions.

Numerical modelling techniques are developed to assist in design decisions. Often, their function is to provide predictions where no previous experience exists or to extrapolate from previous experience, preferably at the same site or in a similar geological setting. For either function, an appropriate numerical model must be selected if it is to adequately represent the prototype which is to be simulated.

A large number and variety of numerical modelling techniques have been described in the literature. Golder Associates have a library of programs for rock stress analysis in underground mining and for the purposes of this brief summary, they are listed below.

- FES2D - Finite elements in two dimensions with joints
- DDJ2D - Displacement discontinuity method in two dimensions
- BEM2D - Boundary element in two dimensions
- BITEF - Boundary integral in two dimensions with inhomogeneous elastic material
- MSIM3D - Tabular excavations, single plane
- NFOLD - Tabular excavations with folds and multiple planes having parallel strike
- RSAP3D - Finite elements in three dimensions.

All these modelling programs require base data on stresses and rock properties. Applications are given below.

- BEM2D
- BITEF

For two dimensional problems with wide excavations the boundary element method provides a cheap method of stress analysis producing stress and displacement results at selected grids or points.

Data preparation is simple as only the excavation outlines and result points need to be defined.

BEM2D is suitable for typical cases with holes in a uniform elastic material. Program BITEF is similar but can deal with zones having a different thickness or elastic modulus.

FES2D

This is a well tested finite element program which has extensive data checking and output plotting capabilities. It provides for elastic elements, ubiquitous joint elements, specific joint elements, mine fill and sequential mining steps.

DDJ2D (Figure III-1)

Two dimensional analysis by displacement discontinuity method. Ideal applications of this method are illustrated in diagrams (a) and (b) of Figure III-1. In case (a), DDJ2D provides, very cheaply, the pattern of stress and displacement induced by mining multiple thin orebodies. In case (b) the pattern of slip (frictional sliding) and separation on bedding planes is also obtained.
Case (c) is more complex with blocky rock, involving slip or separation on bedding planes and joints. Large displacements cannot be modelled but an incipient caving situation can be studied.

MSIM3D (Figure III-2)

For complex mining layouts in a tabular, single plane orebody, the program determines stresses and displacements. It has been used on a number of mining projects to compare alternative stoping sequences and pillar layouts.

The program determines stresses in pillars and other intact ore, and the pattern of hangingwall to footwall convergence and ride over the whole problem area. An elastic rock mass is assumed in the hangingwall and footwall. On the mining plane, special material properties can be included to model brittle behaviour in pillars, enabling potential rock burst areas to be predicted. Other characteristics which can be modelled include:

- closure of hangingwall to footwall;
- compressible fill;
- variations in ore thickness.

The ability to "scale" a model from an initial broad study to successive more detailed studies is important to obtain accurate results without ignoring the effects of adjacent large excavations. Where possible, previous studies have commenced with modelling of actual mine layouts to "calibrate" the model, particularly to determine the pillar strength and yield characteristics. The model is then used to compare alternative future layouts.

NFOLD (Figure III-3)

This is the three dimensional equivalent of DDJ2D. It is similar to MSIM3D but has the following additional features:

- can model folded multiple orebodies with any dip, but uniform strike;
- faults capable of slip or separation can be included, provided they have the same strike as the orebodies.

Figure III-3 shows an NFOLD model to study stress in a major rib pillar of orebody 1, included by mining in orebodies 1 and 2.

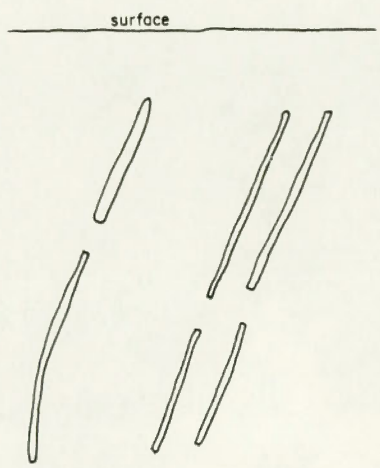
RSAP3D

A three dimensional finite element program, suited to detailed studies of mining layouts. It incorporates the following features:

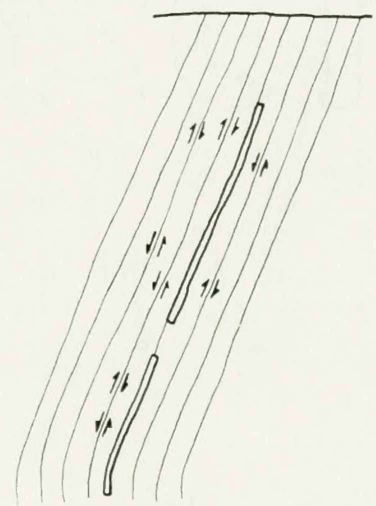
- multistage excavation modelling, simulating mining, backfilling, reinforcement, etc.;
- detailed grid checking and mesh generation facilities;
- interactive graphical display of results through nominated planes of the model;
- incorporates isoparametric elements for modelling major geological discontinuities cutting through the region under investigation;
- inelastic behaviour simulated by piece-wise non-linear analysis;
- a variety of element types to model struts, supports, etc.

# DIAGRAM ILLUSTRATING A TYPICAL APPLICATION FOR PROGRAM DDJ2D

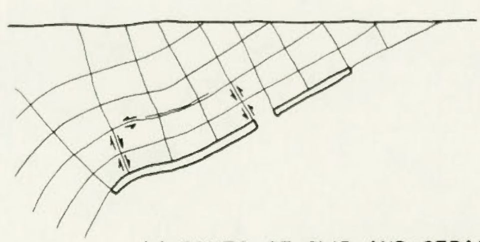
**Figure III - 1**




(a) INTERACTION BETWEEN MULTIPLE OREBODIES.



(b) PREDICTING ZONES OF SLIP AND SEPARATION ON BEDDING PLANES.

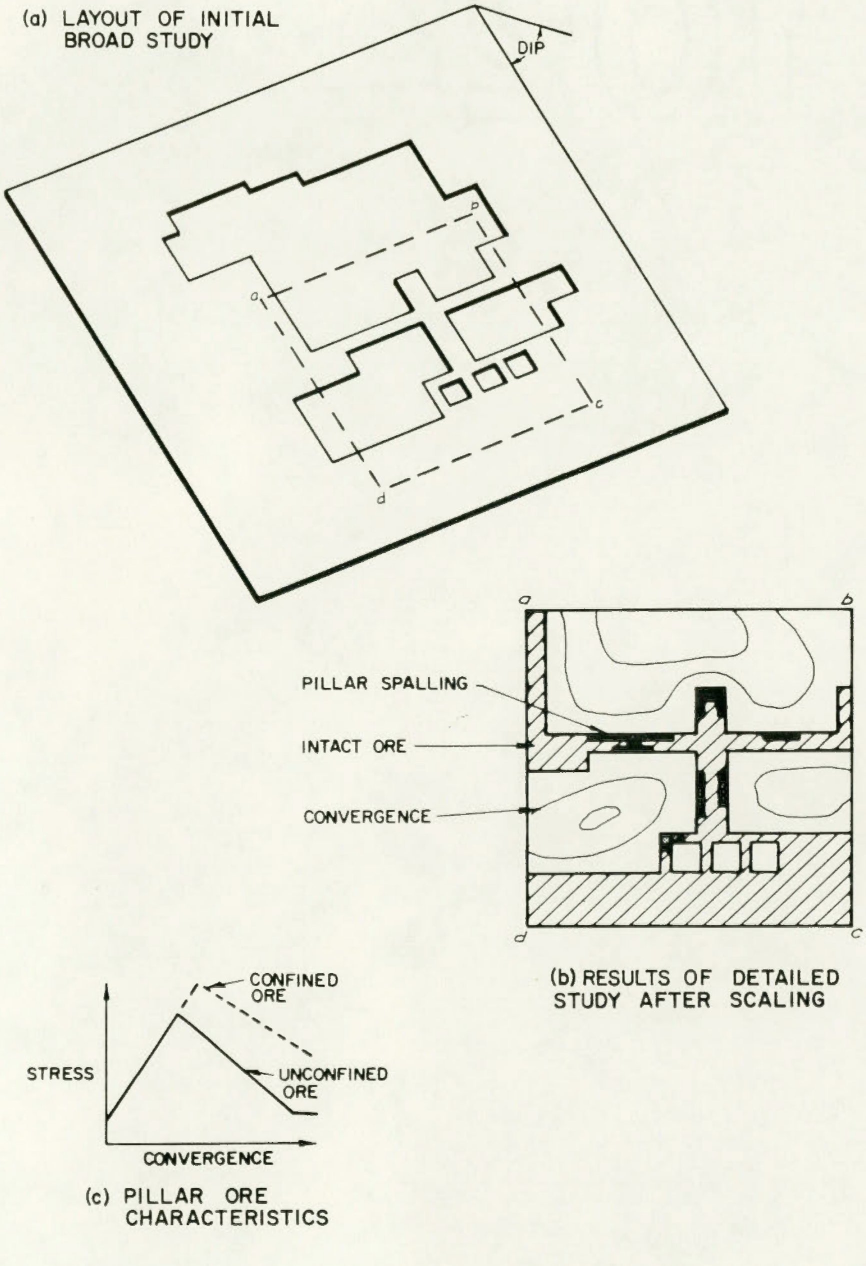


(c) ZONES OF SLIP AND SEPARATION ON BEDDING PLANES AND JOINTS

Project No 802-1571 Drawn - Reviewed Date Mar '81

## DIAGRAM ILLUSTRATING A TYPICAL APPLICATION FOR PROGRAM MSIM3D

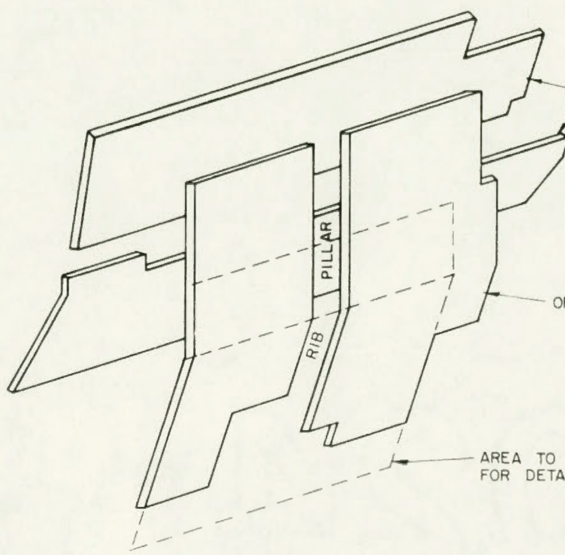
**Figure III - 2**




Project No. 802-1571 Drawn — Date Mar. '81

DIAGRAM ILLUSTRATING A TYPICAL APPLICATION FOR PROGRAM NFOLD

**Figure III - 3**




Project No. 802-1571 Drawn - Reviewed SRS Date Mar. '81

# APPENDIX IV

## NOTES ON MINE VISITS

# APPENDIX IV - LIST OF FIGURES



## LIST OF TABLES

# APPENDIX IV-1

## NOTES ON VISIT TO GECO MINE, MANITOUWADGE, ONTARIO

### INTRODUCTION

The Geco Mine is located at Manitouwadge in northwestern Ontario. Manitouwadge is situated 34 miles north of the Trans Canada Highway halfway between Sault Ste Marie and Thunder Bay, Ontario. Production started in October 1957 and present production is around 4,500 tons per day. It is a copper/zinc mine with some lead, silver and gold.

### GEOLOGY

Rock formations in the Manitouwadge area are listed below:

- Diabase Dykes
- Granite Pegmatite, and Gneissic granite
- Migmatite, highly granitized gneisses
- Sedimentary Gneisses
- Garnetiferous - Amphibole Gneiss and Biotite

Grey Gneiss group - a series of quartz-feldspar biotite gneisses. Sericite schist, or quartz muscovite schist is the host rock of the Geco orebody and occurs at the top of the grey gneiss series.

The Geco orebody, in common with other known orebodies of the Manitouwadge area, is associated with a dragfold on the south limb of the Manitouwadge syncline. This major geological structure, the nose of which is about five miles west of the Geco plant, has an easterly plunge, and all of the orebodies have a similar plunge.

A cross-section from south to north across the Geco orebody shows the following sequence of formations: grey gneiss, including biotitic quartzite; sericite schist, containing the orebody; and biotite-amphibole-garnet gneiss. Intrusive into these formations are basic dykes, granite, pegmatite dykes and diabase dykes.

The orebody forms a tabular mass lying more or less vertical, and raking eastward at from 20 degrees to 30 degrees. In cross-section, the orebody has the shape of an onion, with the bulbous bottom portions conforming to the curvature of the dragfold.

The grade of the ore averages better than 2 per cent copper, 4 per cent zinc and 2 oz/ton of silver. There is a rough zoning of the ore at right angles to the line of the rake, with copper concentrated at the deeper horizons and zinc at the shallower.

Multiple folding in the ore bearing schist, transverse to the main dragfolding, aggravates the ground weaknesses induced by faulting and fracturing, and in some places increases the tendency to slough.

As a rule, the pegmatite dykes are not mineralized, except where in contact with the massive sulphide core. They are, therefore, rarely included in a stope, but may form a stope wall. Since the large dykes (over 3 ft.) are extensively fractured, they tend to slab and break off when exposed over a wide surface. By contrast, the massive sulphide core of the orebody is relatively free from joints and fractures and has been observed standing solidly over horizontal lengths of 70 ft. and vertical heights of over 300 ft. Thus the structural weaknesses of the ore-bearing formation consists of:

(a) foliation and some faulting in an east-west direction.

(b) jointing and minor faulting in a north-south direction.

(c) weak contacts along diabase dykes and along quartz diorite/quartz muscovite schist contacts.

(d) regional, drag and cross folding.

(e) irregular fractures and joints in broad pegmatites.

Jointing

Two main joint sets were identified from the mapping carried out during the visit to the mine. Both are steeply dipping, one set had an east-west strike and the other strikes roughly north-south. This is in agreement to the findings of the government study carried out by Raven and Gale (1977). An interesting note was the absence of a persistent near horizontal joint set underground at Geco.

MINING METHOD AND STABILITY CONDITIONS OF STOPES

The mining methods used at Geco are well documented in the literature (Schwartz 1971). The rocks at Geco are not the best suited to open stoping as they will slough readily when exposed in the large areas of a stope wall. Geco have overcome the problem by modifying the mining method by introducing fill as the ore is drawn; thus keeping the stopes full at all times. To prevent instability in the backs, they are cable bolted using tensioned 9.1 m long cable bolts.

Over the years a system of mining has evolved that is quite successful. A sublevel stoping block would be about 150 to 180 m high consisting of three 21 m wide primary stopes separated by two 37 m pillars and flanked by two boundary pillars 46 m wide. The primary stopes are mined first and drawn under rock fill and then consolidated with the introduction of cemented hydraulic fill. The two 37 m pillars are then removed between the filled stopes. These pillars are usually mined in 60 to 90 m lifts to minimize dilution from the fill walls.

To date most of the primary stoping is completed and pillar mining contributes a large share of the production.

ROCK MASS CLASSIFICATIONS

The footwall hangingwall and orebody rocks were classified in various locations visited (see Figure IV-1).

Values obtained are listed below:

2850 level 28-54.5 Cross-cut

Sericite Schist (Two Ratings)

| | NGI | NGI |
|---|---|---|
| RQD | 60 | 50 |
| Jn | 4 | 6 |
| Jr | 2 | 2 |
| Ja | 0.75 | 1.0 |
| Jw | 1.0 | 1.0 |
| SRF | 2 | 1.0 |
| Q = | 20 | 16.7 |

| | CSIR |
|---|---|
| Intact Strength | 7 |
| RQD | 13 |
| Spacing of Joints | 10 |
| Condition of Joints | 12 |
| Ground Water | 10 |
| | 52 |

Hangingwall Ramp Below 2850 L

Biotite Garnet Gneiss (Two Ratings)

| | NGI | NGI |
|---|---|---|
| RQD | 60 | 90 |
| Jn | 6 | 4 |
| Jr | 3 | 2 |
| Ja | 1 | 0.75 |
| Jw | 1 | 1 |
| SRF | 2.5 | 1 |
| Q = | 12 | 60 |

| | CSIR |
|---|---|
| Intact Strength | 7 |
| RQD | 13 |
| Spacing of Joints | 20 |
| Condition of Joints | 6 |
| Ground Water | 10 |
| | 56 |

1850 Level Hangingwall Drift off 18-36 Cross-Cut

Hangingwall Schist (Two Ratings)

| | NGI | NGI |
|---|---|---|
| RQD | 90 | 60 |
| Jn | 2 | 3 |
| Jr | 1 | 1.5 |
| Ja | 1 | 2 |
| Jw | 1 | 1 |
| SRF | 2.5 | 2.5 |
| Q = | 18 | 6 |

| | CSIR |
|---|---|
| Intact Strength | 7 |
| RQD | 20 |
| Spacing of Joints | 20 |
| Condition of Joints | 6 |
| Ground Water | 10 |
| | 63 |

1850 Level in Footwall of 19-40 Pillar Stope

Footwall Sericite Schist (Two Ratings)

| | NGI | NGI |
|---|---|---|
| RQD | 75 | 60 |
| Jn | 4 | 3 |
| Jr | 1 | 2 |
| Ja | 4 | 2 |
| Jw | 1 | 1 |
| SRF | 2.5 | 2.5 |
| Q = | 1.9 | 8 |

| | CSIR |
|---|---|
| Intact Strength | 7 |
| RQD | 13 |
| Spacing of Joints | 20 |
| Condition of Joints | 6 |
| Ground Water | 10 |
| | 56 |

1850 Level 44 Pillar Sill

Massive Sulphides (Two Ratings)

| | NGI | NGI |
|---|---|---|
| RQD | 60 | 80 |
| Jn | 9 | 9 |
| Jr | 1 | 1.5 |
| Ja | 2.0 | 0.75 |
| Jw | 1.0 | 1 |
| SRF | 4.0 | 1 |
| Q | 0.8 | 17.8 |

| | CSIR |
|---|---|
| Intact Strength | 7 |
| RQD | 13 |
| Spacing of Joints | 20 |
| Condition of Joints | 6 |
| Ground water | 10 |
| | 56 |

2250 level 27-61 Stope

Footwall Schist (Two Ratings)

| | NGI | NGI |
|---|---|---|
| RQD | 50 | 70 |
| Jn | 4 | 3 |
| Jr | 1 | 2 |
| Ja | 4 | 3 |
| SRF | 2.5 | 2.5 |
| Q = | 1.3 | 6.2 |

| | CSIR |
|---|---|
| Intack Strength | 7 |
| RQD | 13 |
| Spacing of Joints | 20 |
| Condition of Joints | 6 |
| Ground Water | 10 |
| | 56 |

Values obtained for the footwall schist varied from 1.3 to 20 (poor to good rock) 0.8 to 17 for the sulphides (poor to good rock) and from 6 to 60 for the hangingwall schist (poor to very good rock).

A very important factor in the footwall rocks was the presence of mica on the foliation. Where mica was present, the rock tended to slough quite readily when exposed in stope walls.

Rock Mechanics Data

Apart from the massive sulphides, not very much strength testing of the rocks has been carried out at Geco. Therefore, values obtained for the footwall and hangingwall rocks should be used with caution.

| Rock Type | E x $10^{-6}$ (psi) | Poisson's Ratio | UCS (psi) |
|---|---|---|---|
| Quartz Biotite Schist (hornfels) | - | - | 6,000 |
| Quartzite | - | - | 23,000 |
| Sericite Schist | - | - | 4,000 |
| Quartz Biotite Gneiss (sillimanite) | - | - | 7,500 |
| Quartz Biotite Muscovite Schist | - | - | 15,500 |
| Hornblende Biotite Quartz Schist | - | - | 7,000 |
| Granite (Gneiss) | - | - | 9,000 |
| Massive Sulphides | 15.0 | 0.31 | 14,625 |
| Granite Biotite | - | - | 10,500 |
| Quartz Biotite | - | - | 27,000 |
| Quartz Muscovite Schists (Sericite) | - | - | 13,500 |

Wall Closure Measurements

Wall closure measurements are made at various locations in the mine using a micrometer between opposing wall stations grouted 3 ft. into the drift sidewalls. These measurements have limited value in that they only measure relative wall displacements but do appear to be useful in predicting imminent collapse of the drift into stope voids.

Virgin Stress Measurement

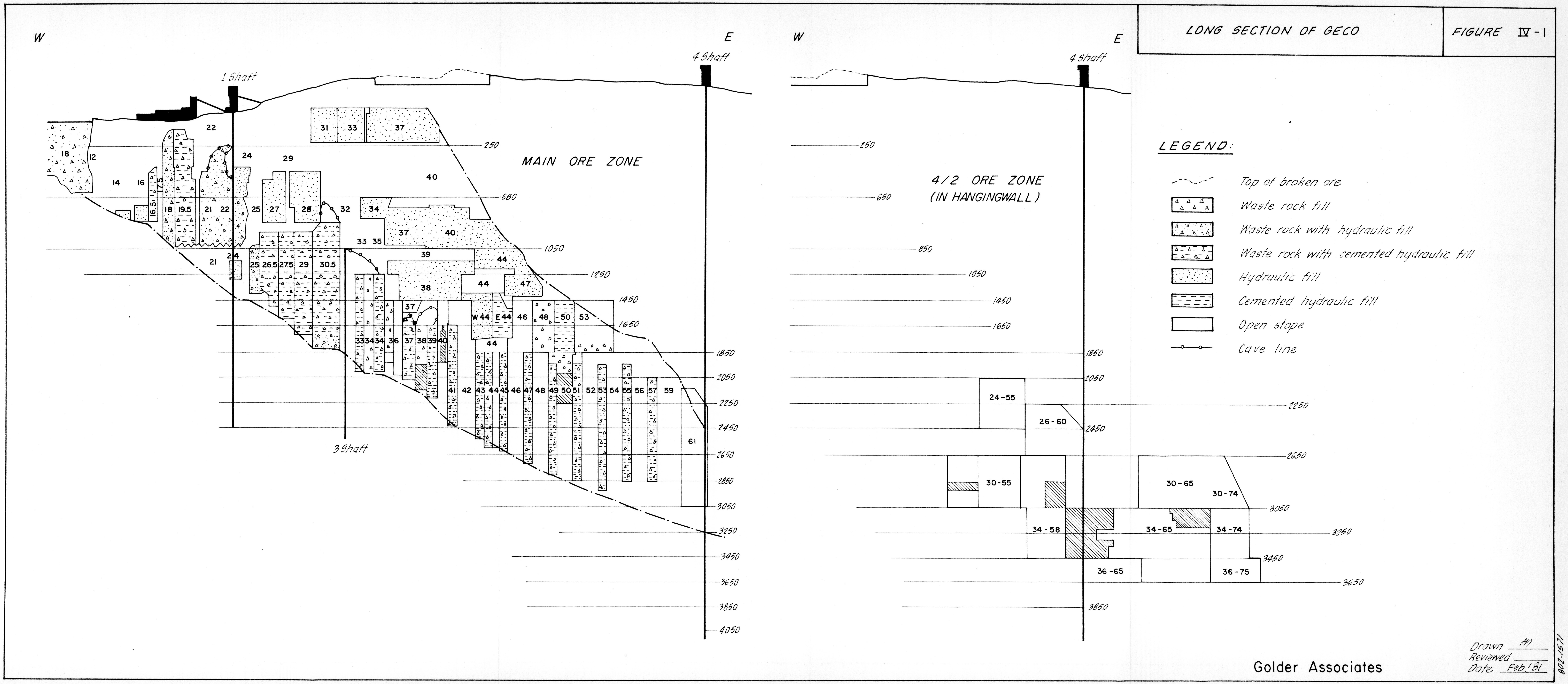
No measurements have been made.

TABLE IV-1

GECO STOPE DETAILS FOR PLOTTING GRAPH OF FIGURE 4-1

| Stope | Depth (m) | Width (m) | Height (m) | Strike Length (m) | Support | Remarks |
|---|---|---|---|---|---|---|
| 33/34/34.5 | 366 | 18 | 134 | 70 | Yes* | Cave possibly associated with fill pass above |
| 27/61 | 655 | 12 | 150 | 61 | Yes* | Unstable |
| 21/22.5 | 137 | 24 | 150 | 107 | No | Caved |
| 30.5 | 244 | 17 | 244 | 70 | Yes* | Caved possibly associated with fill pass above |
| 24/41 | 505 | 88 | 125 | 21 | Yes* | Unstable |

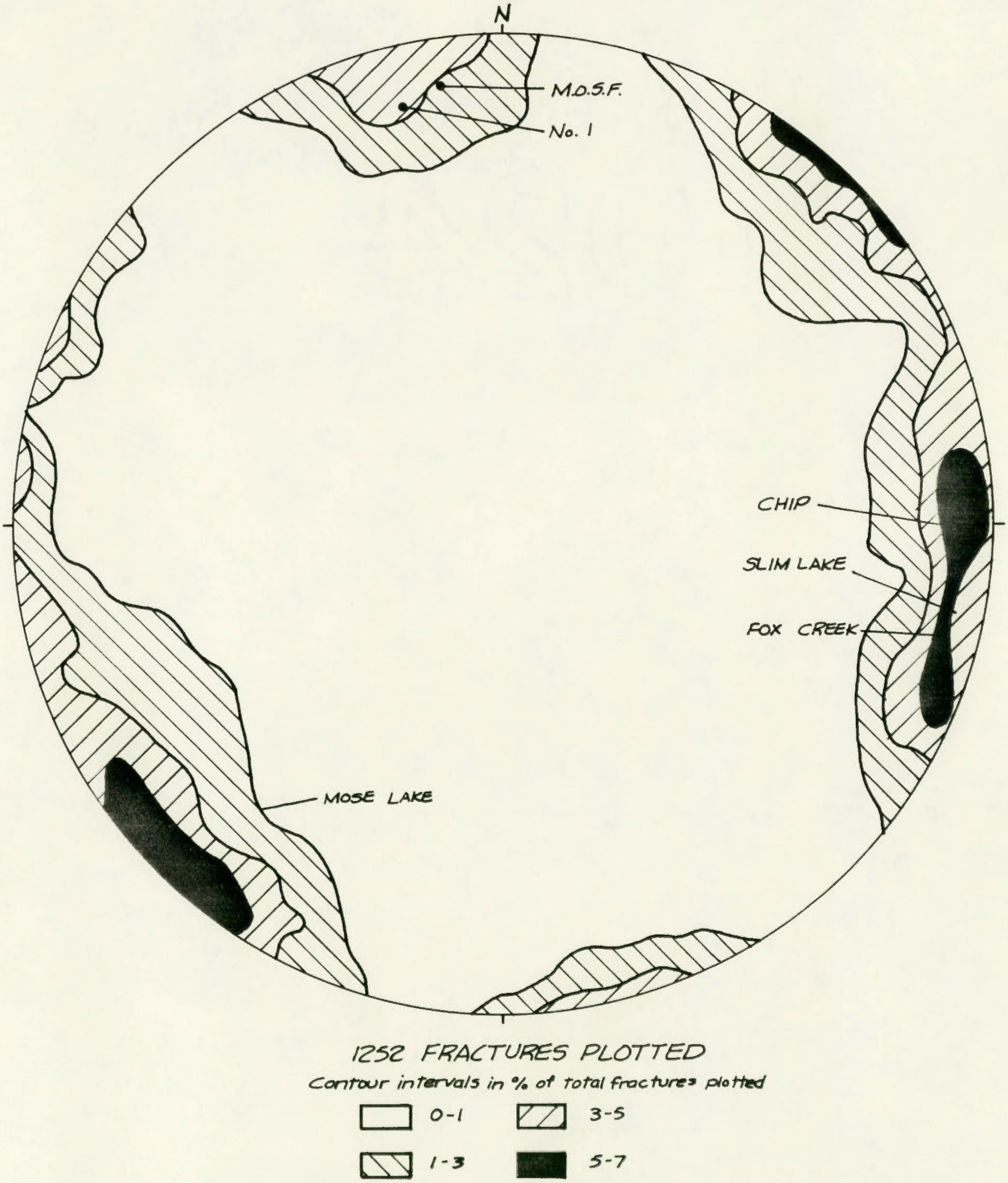
*Back typically supported by fans of 10 m long tensioned cable bolts spaced at about 1.2 m



LONG SECTION OF GECO
FIGURE IV-1
W
E
1 Shaft
4 Shaft
3 Shaft
MAIN ORE ZONE
4/2 ORE ZONE
(IN HANGINGWALL)
LEGEND:
Top of broken ore
Waste rock fill
Waste rock with hydraulic fill
Waste rock with cemented hydraulic fill
Hydraulic fill
Cemented hydraulic fill
Open stope
Cave line
250
680
650
850
1050
1250
1450
1650
1850
2050
2250
2450
2650
2850
3050
3250
3450
3650
3850
4050
24-55
26-60
30-55
30-65
30-74
34-58
34-65
34-74
36-65
36-75
Golder Associates
Drawn
Reviewed
Date Feb.'81
802-1571

GECOMINE SURFACE FRACTURES
(RAVEN AND GALE, 1977)

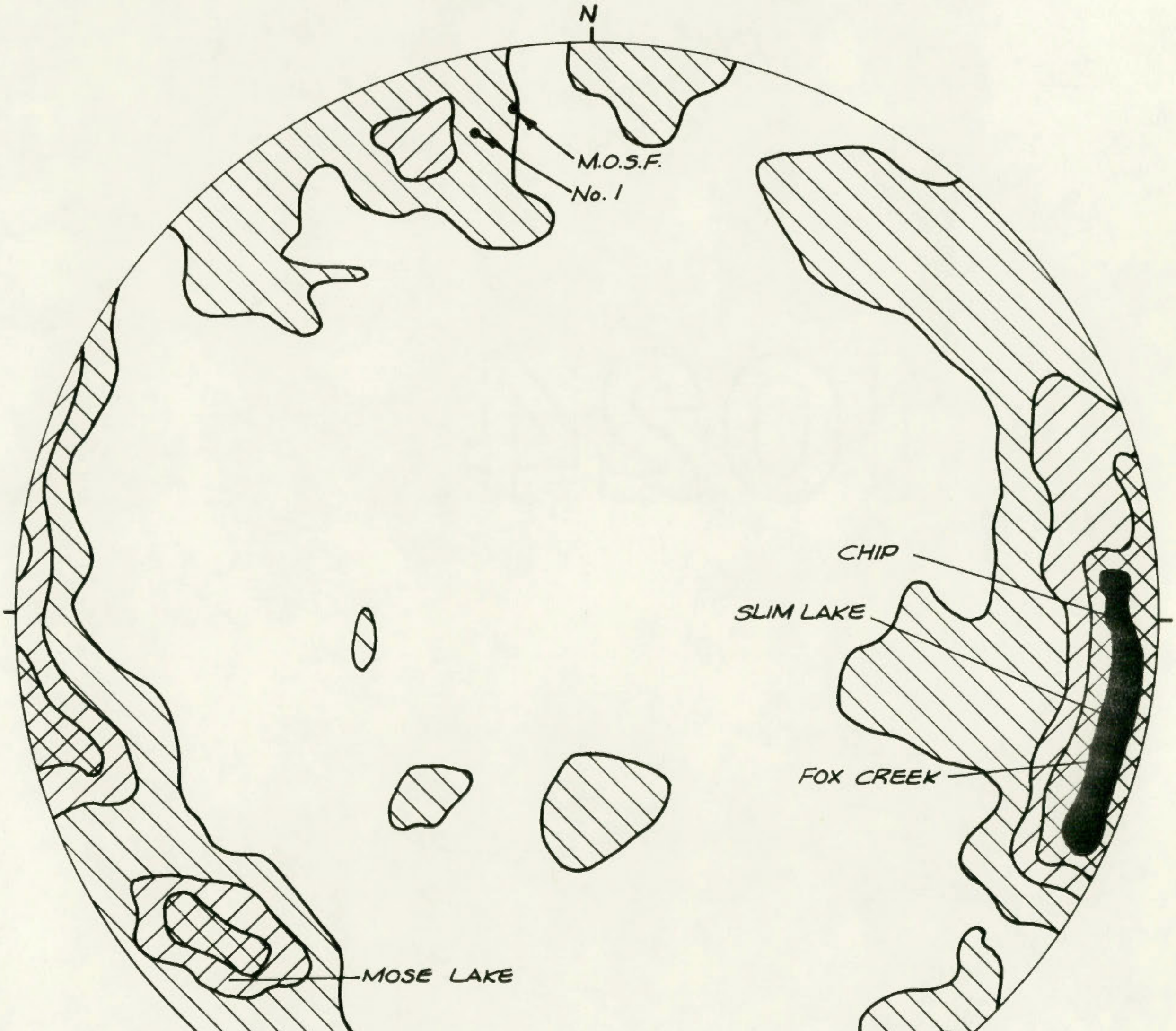
**Figure IV - 2**




Project No. 802-1571 Drawn R.Q. Reviewed ______ Date APR. '81

GECOMINE SUBSURFACE FRACTURES
75 m to 1112 m BELOW GROUND LEVEL
(RAVEN AND GALE, 1977)

**Figure IV - 3**




Project No. 802-1571 Drawn R.D. Reviewed Date Apr. '81

## APPENDIX IV-2

## NOTES ON VISIT TO HEATH STEELE MINE, NEWCASTLE, NEW BRUNSWICK

### LOCATION

Heath Steele Mines Limited is a wholly owned subsidiary of Noranda Mines Limited and operates the mine in a joint venture with the American Smelting and Refining Company. The mine is managed and operated by Heath Steele Mines Ltd., which has a 75 per cent interest in the joint venture.

Heath Steele is situated in the Bathurst-Newcastle area of northern New Brunswick, Canada. The property is accessible by route 430 and is 35 miles from Newcastle and 40 miles from Bathurst.

### GEOLOGY

The massive sulphide stratiform deposits of northern New Brunswick are hosted by the Tetagouche rock group. This rock group is highly folded, middle Ordovician in age and covers a circular area approximately 35 miles in diameter.

The massive sulphide deposits lie within the rhyolite unit in close proximity to the quartz feldspar crystal tuff, which is also known as augen schist and porphyry.

The stratigraphic rock units in the area of the ore zone top towards the north which is indicated by the metal zoning in the sulphides and the graded bedding in the sediments. These units listed from youngest to oldest are as follows:

(1) Banded Quartz Feldspar Crystal Tuff

(2) Banded Quartz Crystal Tuff

(3) Banded Iron Formation

(4) Massive Sulphides

(5) Acid Tuff

(6) Clastic Sedimentary Rocks.

STRUCTURE

The B zone is a tabular shaped vertical or steep northerly dipping massive sulphide body, which strikes at N73°E. The massive sulphides have a strike length of approximately 3,800 ft., vary in thickness from a few inches to 250 ft. and has been traced to a depth of 3,600 ft.

Folding is the primary structural control and although there is minor faulting, faults have had no major influence on the shape of displacement of the ore zone. The ore body has undergone five periods of folding (Allcott and Archibald 1980) which are numbered one to five in time sequence as they occurred.

JOINTING

There are two major joint sets evident throughout the mine. Both are steeply dipping, with one set approximately parallel to the strike of the ore body and one approximately transverse to the strike. A third set of near horizontal joints were observed which appeared to be more prominent in the sulphides.

The joint set parallel to the strike indicated a spread in strike direction and is probably a combination of two joint sets. As the mapping carried out was only superficial in nature, it was not possible to determine conclusively whether the jointing varies significantly from locale to locale whithin the mine. Joints were usually planar or slightly undulating and spaced at about 1 m to 3 m. At times difficulty was experienced in distinguishing between blast fractures and naturally occurring joints.

Mining Method and Stability Conditions of Stopes and Pillars

The rock at Heath Steele is quite competent and lends itself well to the open stoping method of mining.

No problems were encountered with primary stoping at the mine in the upper levels. Stope dimensions were typically 46 m strike length, 137 m high, with 15 m rib pillars. Problems have, however, been encountered when recovering rib pillars between primary stopes. In the first instance, a rib pillar that was instantaneously blasted caused the adjacent rib pillar to burst and initiated a cave in the back extending over a 150 m strike length. After this, stope lengths were limited to 43 m; height to 85 m; and rib pillar length to 18 m.

Ground trouble occurred with increasing frequency as the depth from surface increased. The second instance of caving was also associated with rib pillar recovery. In this case, however, the pillar between two primary stopes had failed causing a cave to initiate over a strike length of about 90 m.

The typical chain of events of pillar failure would arise at first as spalling backs and sides in rib pillar cross-cuts, accompanied by some light sound, followed by sloughing from the rib pillar face. Later with much more sound the backs of adjacent stopes would begin to slab off.

Dilution from hangingwall and footwall waste rocks usually commenced at the same time or immediately after the backs of stopes failed. On one occasion the acid tuff forming the footwall began sloughing before the rib pillar was visibly stressed.

At the present time critical areas are backfilled, and for the future it is planned to remove rib pillars only between filled stopes. Dimensions of stopes planned on the 7430 level are 30 m along strike, 60 m high with 30 m rib pillars separating them.

ROCK MASS CLASSIFICATIONS

The footwall, hangingwall and orebody rocks were classified in various locations of the mine using the NGI system. Core from exploratory diamond drill holes was examined.

NGI Rock Mass Classification

Values obtained varied from 13 (good rock) to 124 (extremely good rock) for the sulphides; from 32 to 42 (good to very good) for the hangingwall porphyry; and 40 to 84 (good to very good) for the footwall rock.

Results of rock mass classifications are given below as well as an estimated CSIR rating for comparison purposes. Two classifications were independently made at each location.

77-92 Cross-Cut Footwall - Chlorite Tuff

| | NGI | |
|---|---|---|
| RQD | 95 | 90 |
| Jn | 3 | 6 |
| Jr | 2 | 2 |
| Ja | 0.75 | 0.75 |
| Jw | 1.0 | 1.0 |
| SRF | 1.0 | 1.0 |
| Q | 84 | 40 |

| | CSIR |
|---|---|
| Intact Strength | 7 |
| RQD | 20 |
| Spacing of Joints | 25 |
| Condition of Joints | 6 |
| Ground Water | 10 |
| | 68 |

## 77-92 Cross-Cut Sulphides

| | NGI | |
|---|---|---|
| RQD | 85 | 95 |
| Jn | 6 | 6 |
| Jr | 1 | 1 |
| Ja | 0.75 | 0.75 |
| Jw | 1.0 | 1.0 |
| SRF | 1.0 | 1.0 |
| Q | 18.9 | 21 |

| | CSIR |
|---|---|
| Intact Strength | 12 |
| RQD | 17 |
| Spacing Joints | 25 |
| Condition of Joints | 6 |
| Ground Water | 7 |
| | 67 |

## 77-92 Cross-Cut Hangingwall Porphyry

| | NGI | |
|---|---|---|
| RQD | 95 | 95 |
| Jn | 6 | 3.0 |
| Jr | 2.0 | 1.0 |
| Ja | 0.75 | 0.75 |
| Jw | 1.0 | 1.0 |
| SRF | 1.0 | 1.0 |
| | 42 | 42 |

| | CSIR |
|---|---|
| Intact Strength | 7 |
| RQD | 20 |
| Spacing of Joints | 25 |
| Condition of Joints | 6 |
| Ground Water | 10 |
| | 68 |

## 75 - 85 Cross-Cut Massive Sulphides

| | NGI | |
|---|---|---|
| RQD | 95 | 95 |
| Jn | 2 | 3 |
| Jr | 2 | 2 |
| Ja | 0.75 | 0.75 |
| Jw | 1.0 | 1.0 |
| SRF | 1.0 | 1.0 |
| | 127 | 84 |

CSIR

| | |
|---|---|
| Intact Strength | 12 |
| RQD | 20 |
| Spacing of Joints | 25 |
| Condition of Joints | 6 |
| Ground Water | 10 |
| | 73 |

72-84 Hangingwall Drift Porphyry

NGI

| | | |
|---|---|---|
| RQD | 90 | 95 |
| Jn | 3 | 4 |
| Jr | 1.0 | 1 |
| Ja | 0.75 | 0.75 |
| Jw | 1.0 | 1.0 |
| SRF | 1.0 | 1.0 |
| | 40 | 32 |

CSIR

| | |
|---|---|
| Intact Strength | 12 |
| RQD | 20 |
| Spacing of Joints | 25 |
| Condition of Joints | 6 |
| Ground Water | 4 |
| | 67 |

Difficulty arose in estimating RQD as blast fractures were very misleading. Generally, it was felt that the user of the classification system needs to be experienced in its use to get good reproductibility.

The large variation in the sulphides was determined by the joint set number Jn. It is interesting to note that the areas where horizontal jointing was strongly in evidence were areas where caving had taken place.

## Rock Mechanics Data

A program of rock strength testing has been carried out at Heath Steele. Average values for unconfined compressive strength, Young's modulus and Poisson's ratio are tabled below. Results were obtained by taking a logarithmic average of test results available.

| Rock Type | E x $10^{-6}$ (psi) | Poisson's Ratio | UCS (psi) |
|---|---|---|---|
| F/W Chlorite Tuft | 9.94 | 0.25 | 12,200 |
| H/W Quartz Porphyry | 9.97 | 0.19 | 13,250 |
| Massive Sulphides | 17.3 | 0.24 | 25,600 |

Most of the results are from samples taken from diamond drill holes normal to the orebody strike and dip. As the rocks are foliated, they may be expected to have anistropic strengths. The figures given will apply in the main to strengths perpendicular to the foliation.

Extensometers

In 1969 a program of extensometer installation was initiated. The extensometers were designed to measure pillar closure and as such the data obtained to date is very useful. Predictions can be made on pillar strength using available data. No extensometers, however, have been installed to monitor stope wall or back strains.

The extensometer data has been augmented by regular patrols of areas to perform crack surveys and record occurrences of seismic noise and sloughing of stopes.

Virgin Stress Measurements

No virgin stress measurements have been made at the mine. Measurements have been made at a mine in the same rock formation about 50 km away. The results of these measurements indicate that at a depth of about 700 m, the major horizontal stress is perpendicular to the orebody and about twice the magnitude of the vertical stress. The vertical stress was found to be proportional to the depth times the average density of the overlying rocks.

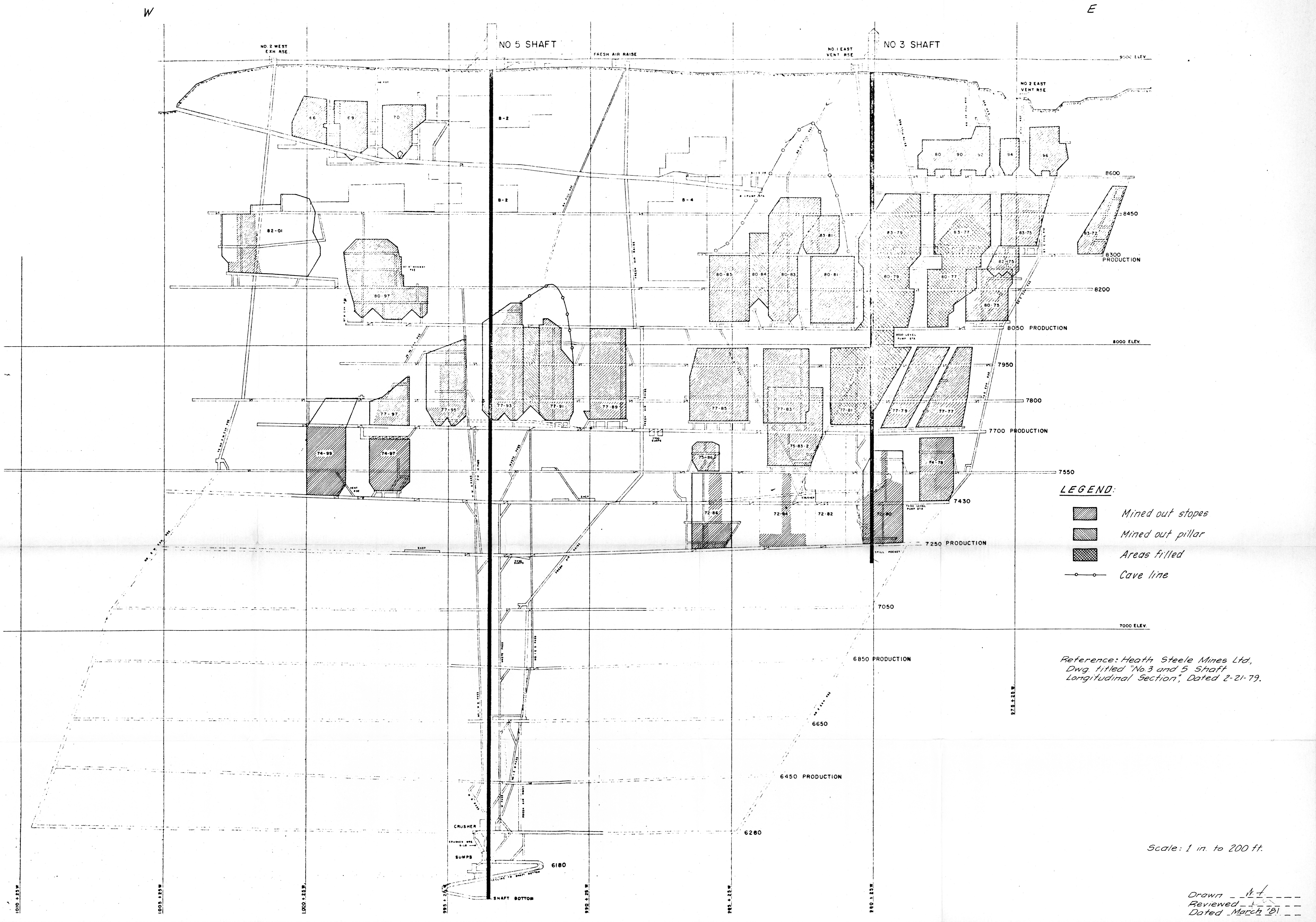
TABLE IV-2

HEATH STEELE STOPE DETAILS FOR PLOTTING GRAPH OF FIGURE 4-1

| Stope | Depth (m) | Width (m) | Height (m) | Strike Length (m) | Support | Remarks |
|---|---|---|---|---|---|---|
| 72-84 | 435 | 43 | 70 | 6 | Rock bolts and some mesh | Back stable |
| 72-86 | 435 | 27 | 81 | 23 | None | Back failed to stable arch |
| 75-86 | 400 | 30 | 30 | 30 | None | Back failed to stable arch |
| 74-97 | 395 | 43 | 55 | 30 | None | Back failed to stable arch |
| 74-99 | 385 | 70 | 72 | 43 | None | Back failed to stable arch |
| 77-97 | 340 | 17 | 38 | 43 | None | Back failed to stable arch |
| 77-91/93 | 270 | 46 | 116 | 92 | None | Caved upwards after central pillar blasted |
| 77-89 | 280 | 18 | 96 | 40 | None | Back failed to stable arch |
| 80-81/85 | 185 | 46 | 122 | 168 | None | Caved upwards after central pillar blasted |

LONG SECTION OF MINE SHOWING OUTLINE OF STOPES AT HEATH STEELE

FIGURE IV - 4



# APPENDIX IV-3

## NOTES ON VISIT TO CSA MINE, COBAR, NEW SOUTH WALES

### INTRODUCTION

The CSA mine produces copper, zinc and lead from a series of relatively narrow lenticular orebodies. The mine has an annual output of about 600,000 tons per annum from approximately four to six open stopes. When the mine first commenced production in 1969, cut and fill methods were used. The transition to open stoping was introduced gradually from the introduction of a trial open stope during a period of depressed metal prices.

The mine has been the site of a number of rock mechanics research programs involving, stress measurement, microseismic noise monitoring, strain measurement, rock strength and elastic modules testing, and rock structure studes.

### GEOLOGY

A geological description and comprehensive analysis of rock structure is given by Barton (1977). The main features affecting open stoping are summarized below:

Orebodies are located within quartz-rich shear zones which strike approximately north-south, dip at 75 - 85 degrees east and pitch steeply to the north. The separate orebodies or shoots are generally lenticular and en echelon and occur within a 300 - 400 m wide tabular mineralized envelope. Ore shoots average between 60 and 120 m in length but may extend, through weakly mineralized areas, over total distances of over 300 m.

## STRUCTURE OF THE MINE AREA

The structure of the mine area has been established by a succession of geologists. In general aspect, but certainly not in detail, the structure is very simple. Bedding generally strikes at 340°N and dips steeply at approximately 80 degrees to the west. Occasional steepening of dip and overturning is evident.

The main defects in the rock structure are cleavage and shears (faults) parallel to the orebodies. Some of the shear zones contain talc and black chlorite which create extensive weak planes. The predominant joint set is flat dipping with the joints approximately perpendicular to cleavage. Diamond drill core shows bedding breaks, cleavage and joints. Bedding is rarely a significant break plane in underground excavations. Occasionally one or two extensive joints have isolated a massive block which has fallen out of the back of a cut and fill stope. Barton reports that joint spacing tends to be more intense in the vicinity of the orebodies. On the other hand, siliceous ore is often a strong massive cherty material (termed elvan).

Main "shears" and many mesoscopic faults extend across several mine levels and can be traced between isolated intersections in boreholes and underground openings. Progressive mapping of these features is effected by the mine geologists as development proceeds.

## MINING METHOD AND STABILITY CONDITIONS OF STOPES AND PILLARS

Before 1977, most of the production came from cut and fill stopes. Now all production is from open stopes. The general mining layout is shown in Figure IV-3. Stope lengths have generally been limited by barren zones which have in effect provided rib pillars. Where orebody wall lengths were considered too large for stability, pendant rib pillars were introduced. Stope heights up to 90 m have frequently been used.

Blastholes in the early open stopes were 57 m diameter. In-the-hole (100 mm diameter) drills were introduced in 63W1 and 150 mm diameter blastholes were introduced in 45W3 stope. A blast hole length of 45 m is considered an optimum from the point of view of drilling performance and accuracy. Final clean up of a stope before filling is done with remote control loaders.

Usually, three stopes are available for production and current scheduling allows blast hole drilling to be kept six months ahead of mining.

The lift height is limited to between 35 and 45 m for optimum drilling results. Uncemented mine tailings and development waste are used as fill.

Wall exposures were gradually increased in stope 11CE which had extensometers in the footwall. Footwall overbreak did occur on the 2nd and 3rd lifts. Apart from this stope (and recently 63W2) monitoring of wall rock movement has not been carried out. Overbreak has occurred which has no doubt had some effect on production but dilution and loss of ore has not been excessive. The extent of overbreak is generally only roughly known from visual observation, grades and tonnages drawn.

Crown pillars have overbroken, notably in two places:

(a) 18CB-CE:

This failure occurred in a narrow pillar where strong cleavage and chiloritic shears sub-parallel to the pillar were subjected to stope blast vibrations.

(b) 42E4, 42E2:

An isolated remnant in the eastern orebody system, probably subjected to moderately high stress.

Failures in both cases were probably induced by a combination of rock stress, rock defects and the interaction with neighbouring stopes. The 63W2 crown pillar is being subjected to fairly high stress but has shown only slight overbreak. The rock is apparently strong (> 100 MPa) and massive.

It appears that wall stability conditions can be summarized as follows:

Around a long, narrow excavation a destressed zone will form along each of the long walls ($\sigma_3 < 0$, $\sigma_2 < 0$). In a slightly larger zone partial destressing occurs ($\sigma_3 < 0$, $\sigma_2 > 0$). Within the fully destressed zone, overbreak may occur in the form of block sliding on one or two major joints or by ravelling if jointing is intense, depending on the rock mass quality.

The other important case is where closed spaced bedding or cleavage occurs parallel to the wall. This can permit slabbing to develop within the destressed zone as described above. There can be extension of overbreak into the partially destressed zone. This depends on rock stress, strength, friction between layers and thickness of layers.

The typical overbreak at footwall and hangingwall appears to be predominantly withn the destressed zone, rock mass quality is probably the main control and rock stress only a minor factor.

## Rock Mass Classification

C.M. Barton (1977) has carried out a rigorous analysis of the rock structure at Cobar. The rocks were classified using the CSIR and NGI systems. Results of rock mass classifications are listed below:

Interstope Rock

| NGI | | CSIR | |
|---|---|---|---|
| RQD | 95 | Strength | 10 |
| Jn | 3 | RQD | 20 |
| Jr | 3 | Spacing of Joints | 20 |
| Ja | 1 | Condition of Joints | 16 |
| Jw | 1 | Ground Water | 10 |
| SRF | 2.5 | Rating | 76 |
| Q = | 38 | | |

Stope - Average Ground

| NGI | | CSIR | |
|---|---|---|---|
| RQD | 85 | Strength | 10 |
| Jn | 3 | RQD | 17 |
| Jr | 3 | Spacing of Joints | 10 |
| Ja | 1 | Condition of Joints | 16 |
| Jw | 1 | Ground Water | 10 |
| SRF | 2.5 | Rating | 63 |
| Q = | 34 | | |

Stope (very bad ground)

| NGI | |
|---|---|
| RQD | 3 |
| Jn | 6 |
| Jr | 1 |
| Ja | 12 |
| Jn | 1 |
| SRF | 10 |
| Q = | 0.004 |

General Estimate of NGI Q for Wall Rocks by

Golder Associates

| | |
|---|---|
| RQD | 75 |
| Jn | 3.0 |
| Jr | 2.5 |
| Ja | 2.0 |
| Jw | 1.0 |
| SRF | 2.5 |
| Q = | 25 |

It is noted that Barton (1977) estimated some better values (RQD = 85, Ja = 1) which would lead to Q = 34 for stopes. The disagreement is small. It is suggested that a value Q = 30 be used at this stage.

The Q calculated is applicable to estimating roof support. To apply the rock mass quality to estimation of wall support requirements, Barton et al suggest using an effective wall quality of 5Q, i.e. Q = 150. These authors suggest that no support will be required provided:

where span is the width of wall (or roof) exposed and ESR is an "Excavation Support Ratio" appropriate to the type of excavation and its degree of permanence. For a typical Cobar open stope with a span of 70 m and an effective rock mass quality of 150, it appears that ESR shuld be set at:

$$ESR = 70/(2 \times 150^{0.4})$$

$$ESR = 4.7$$

It is tentatively concluded that for the walls of Cobar stopes, using Qw equal to 5 times the assessed Q applicable to a tunnel roof, that a no support condition is obtained in equation (1) by using an ESR value of approximately 5.

## ROCK MECHANICS DATA

### Rock Strength

Laboratory testing of the rock properties is summarized below. Results showed large variation on samples from various locations in the mine.

Average Unaxial Compressive Strength = 116 MPa

Average Modulus of Electricity from = 67 GPa
uniaxial and flat jack testing

## Virgin Stresses

Several measurements of virgin stress have been made at the CSA mine and results are summarised in Reference 3. The results are reasonably consistent and indicate that all three principal stresses incrase linearly with depth below surface, as follows:

| | | | | |
|---|---|---|---|---|
| (a) | minor | - | vertical | 30 kPa per meter depth |
| (b) | intermediate | - | hor. north-south | 33 kPa per meter depth |
| (c) | major | - | hor. east-west | 63 kPa per meter depth |

The vertical stress is equal to the overburden pressure, which gives some confidence in the results. The east-west horizontal stress is tabulated below for three levels.

| | | | |
|---|---|---|---|
| 4 level | : | 23 MPa | (3300 psi) |
| 6 level | : | 34 MPa | (4900 psi) |
| 7 level | : | 40 MPa | (5800 psi) |

## Open Stope Wall Deformation

Measurements of footwall deformations during mining of Stages 2 and 3 of 11CE stope were made by CSIRO using arrays of borehole extensometers. Results show zones of rock loosening occurred to depths of about 10 m and 20 m into the footwall during mining of Stages 2 and 3, respectively. These represent zones of potential overbreak, depending on geological conditions. The areas of rock fall are considerably smaller and occur mainly in the convex area of the footwall.

## Stress Concentrations in Crown Pillars

A stress monitoring station has been established in 18CC crown pillar at 32 section (Figure IV-3). This consists of the following instrumentation installed in several boreholes extending from Z1 access cross-cut to the 18CC drill drives.

(a) rigid inclusion cell

(b) mechanical extensometers

(c) resistance wire extensometers

(d) acoustic emission detectors

(e) borehole deformation cells

(f) strain gauged grouted dowel

An in situ stress measurement (USBM and CSIRO cells) has been carried out at this site and there are additional acoustic emission stations located along the length of the pillar.

## TABLE IV-3

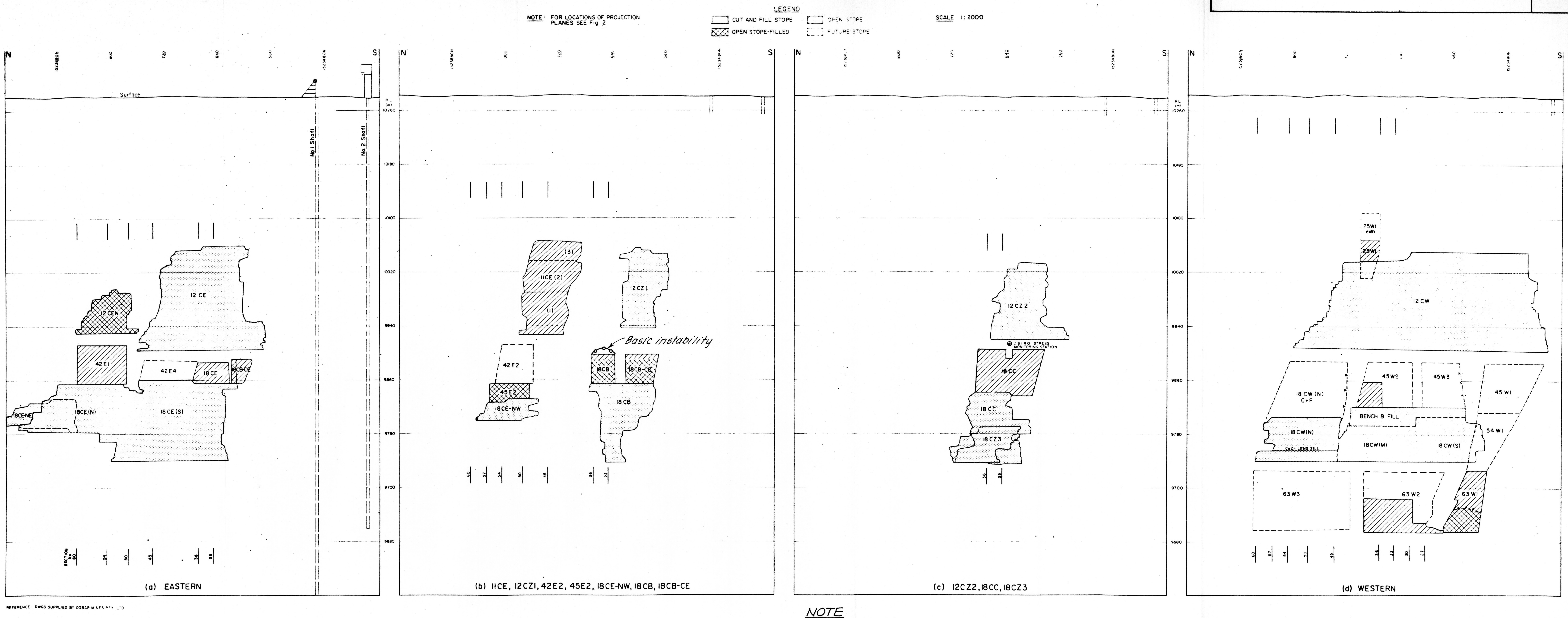
## COBAR STOPE DETAILS FOR PLOTTING GRAPH OF FIGURE 4-1

| Stope | Depth (m) | Width (m) | Height (m) | Strike Length (m) | Support | Remarks |
|---|---|---|---|---|---|---|
| 12 CEN | 320 | 12 | 60 | 60 | No | Stable |
| 11 CE 1 | 290 | 18 | 65 | 65 | No | Stable |
| 2 | | 22 | 110 | 70 | No | Footwall overbreak on |
| 3 | | 14 | 140 | 70 | Some Bolts | talc chlorite shears |
| 63 W1 | 600 | 11 | 100 | 40 | No | Footwall overbreak on talc chlorite shear, possibly overdrilling |
| 18 CC | 410 | 20 | 60 | 90 | Yes* | Used pendant rib pillar |
| 42 E1 | 400 | 20 | 60 | 70 | Rock Bolts | Stable |
| 42 E2 | 400 | 7 | 60 | 60 | Yes* | Footwall overbreak at re-entrant bulge |
| 45 W2 | 430 | 9 | 70 | 80 | No | Large hangingwall slab formed |
| 45 W3 | 430 | 10 | 70 | 60 | Yes* | Little overbreak |
| 63 W2 | 600 | 16 | 50 | 100 | Yes* | Little overbreak. Pendant pillar |

*Support by grouted 16 mm Ø Dywidag dowels placed in fans from drill drives under crown. Typically 4 dowels per fan, fans 4 m apart.

## LONG SECTION OF COBAR MINE

FIGURE IV-5




NOTE

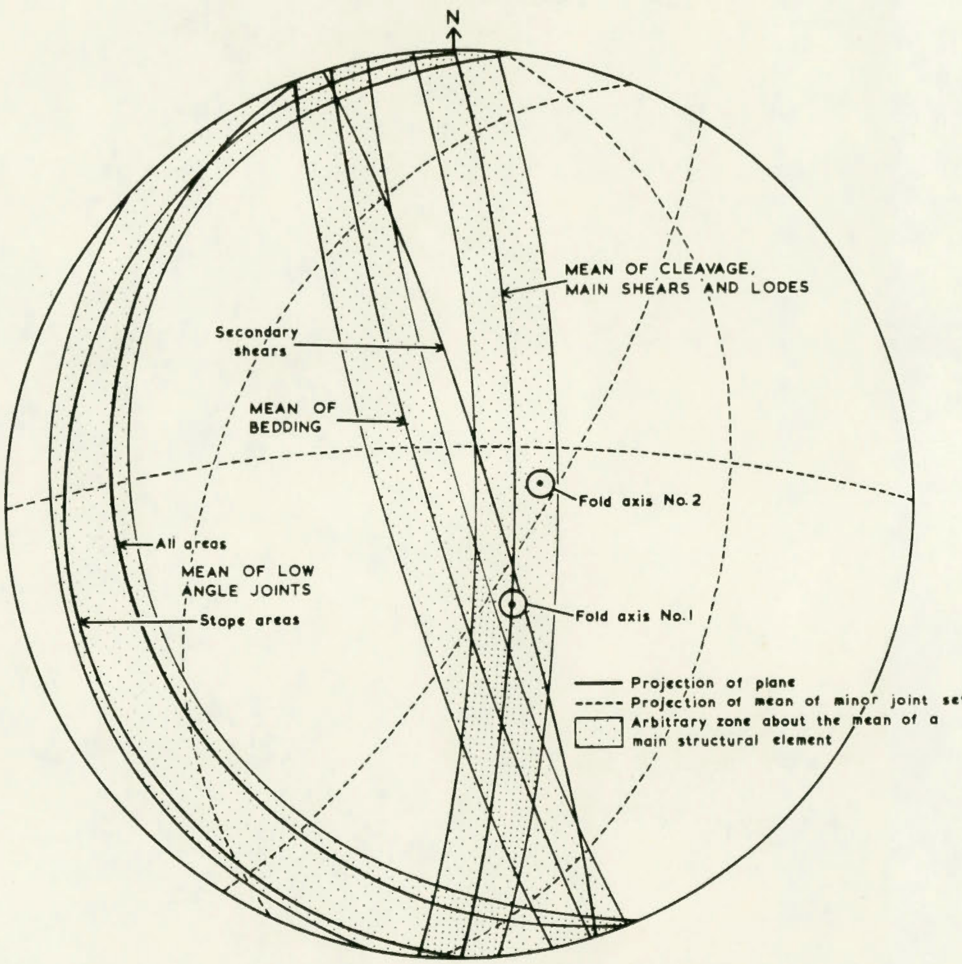
*Figures of orebodies from left to right are superimposed and run from footwall to hangingwall.*

Golder Associates

Drawn
Reviewed
Date Jan. '81

# EQUAL AREA PROJECTION OF MAIN STRUCTURAL ELEMENTS CSA MINE COBAR (BARTON, 1977)

**Figure IV-6**




Project No. 802-1571 Drawn R.D. Reviewed ____ Date Apr. '81

# APPENDIX IV - LIST OF REFERENCES


Allcott, G.A., and Archibald, D.E. (198). Pillar design at Heath Steele Mines. Internal Report.

Barton, C.M. (1977). A geotechnical analysis of rock structure and fabric in the CSA Mine, Cobar, N.S.W. Technical Paper No. 24, Division of Applied Geomechanics, CSIRO, Australia.

Bray, R.C.E. (1967). Control of ground movement at the Geco Mine. Internal Report.

Brown, W.L., R.C.E. Bray and Mine Staff (1960). The geology of the Geco Mine. Trans. CIM. Vol. LXIII, pp. 1-9.

GA Report to Cobar (1977). Review of Open Stoping CSA Mine. February.

GA Report to Cobar (1979). Rock Mechanics Review of Open Stoping CSA Mine. January.

GA Report to Cobar (1980). CSA Mine Rock Mechanics Review. January.

Raven, K.G. and Gale, J.G. (1977). Subsurface containment of solid radioactive waste. A study of the surface and subsurface structural and ground water conditions at selected underground mines and excavations. Energy Mines and Resources/Geological Survey of Canada, Internal Report No. 1/77.

Schwartz, A. (1971). Trends in sublevel stoping practice a the Geco Mine. CIMM Convention, April.

Udd, J.E. and Students. Strength and grade relationships for a copper-zinc sulphide ore. Geco Internal Report.

# APPENDIX V

## SUMMARY TABLE OF DATA USED TO COMPILE STABILITY NUMBERS VS. SHAPE FACTOR

# APPENDIX V

## SUMMARY TABLE OF DATA USED TO COMPARE STABILITY NUMBER (N) VS. SHAPE FACTOR (S)

Data used to plot Figure 4.1 in Chapter 4 was obtained from the mines visited and published case histories and is tabulated in Table V-1 below. When data was unavailable, it was estimated; such data is noted with an asterisk in Table V-1. It should be noted that the empirical relationships presented in Figure 4.1 are based on this data and additional site visits are required to verify the hypothesis.

Notes on the sets of data used are given below.

Set 1 was taken from the paper by Laubscher and Taylor (1976) entitled "The importance of geomechanics classification of jointed rock masses in mining operations". The data used is presented in Table VIII of the paper and the formula RMR = 9 log Q + 44 was used to convert Bieniawski's rating to Barton's Q rating.

Set 2 was taken from Tables 7-2 and 7-3 of Chapter 7 of the SME Mining Engineering handbook.

Set 3 was taken from data collected during the visit to Heath Steele.

Set 4 was taken from Figure 137 of "Underground Excavations in Rock" by Hoek and Brown (1980). It had to be assumed that the data used were from excavations developed near the surface. The points represents the line described by span = 2 x $Q^{0.66}$. The shape factor was conservatively calculated using equal spans.

Set 5 was taken from data collected during the visit to the Geco Mine.

Set 6 was taken from data collected during the visit to the CSA Mine at Cobar and from the paper entitled "A Geotechnical Analysis of Rock Structure and Fabric in the CSA Mine, Cobar, NSW", by Barton (1977).

All the above data deals with back stability. Sets 7, 8 and 9 were plotted for walls from data obtained from the mines visited.

## TABLE V-1

## SUMMARY OF DATA COLLECTED FROM MINE VISITS AND THE LITERATURE

| Location | Q' | (MPa) | (MPa) | | Factors | | | Stability Number | Shape Factor (m) | Condition | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | A | B | C | | | | |
| SET 1 (BACKS) | | | | | | | | | | | |
| 570 L. S. Haulage | 11.5* | 138 | 6.1* | 22.6 | 1.0 | 1.0* | 1.0 | 11.5 | 2.4 | Stable | 1) Orientation of structure |
| 480 L. Expl. Drive | 3.8* | 131 | 5.1* | 25.5 | 1.0 | 1.0* | 1.0 | 3.8 | 1.8 | Unstable | with respect to back not |
| 365 L. F/W Haulage | 0.06* | 7 | 3.9* | 1.8 | 0.17 | 1.0* | 1.0 | 0.01 | 2.4 | Cave | known. Factor B = 1.0. |
| 570 L. Access | 2.7* | 41 | 6.1* | 6.8 | 0.63 | 1.0* | 1.0 | 1.7 | 1.0 | Unstable | 2) Q' calculated from RMR = |
| 480 L. X/C South | 16.1* | 117 | 5.1* | 22.8 | 1.0 | 1.0* | 1.0 | 16.1 | 3.2 | Stable | 9 $\log_e$ Q + 44 |
| 480 L. Expl. Drive | 28.0* | 103 | 5.1* | 20.1 | 1.0 | 1.0* | 1.0 | 28.0 | 1.8 | Stable | |
| SET 2 (BACKS) | | | | | | | | | | | |
| White Pine | 40* | >88 | 4.3* | 20.5 | 1 | 1.0* | 1.0 | 40 | 6.1 | Stable | 1) Orientation of structure |
| White Pine | 40* | >88 | 4.3* | 20.5 | 1 | 1.0* | 1.0 | 40 | 13.7 | Cave | with respect to back not |
| San Manuel | 0.15* | >119 | 11.8* | 10.1 | 1 | 1.0* | 1.0 | 0.2 | 0.8 | Cave | known. Factor B = 1.0. |
| Hockley | 200* | 40* | 15.0* | 2.7 | 0.2 | 1.0* | 1.0 | 40 | 7.0 | Stable | |
| Jenifer | 6.3* | 8 to 46 | 3.2* | 2.4 14.2 | 0.58* | 1.0* | 1.0 | 3.7 | 13.4 | Unstable | |
| Grace | 1.1* | 103* | 16.1* | 6.4 | 0.64 | 1.0* | 1.0 | 0.7 | 0.4 | Stable | |
| Grace | 1.1* | 103* | 16.1* | 6.4 | 0.64 | 1.0* | 1.0 | 0.7 | 0.5 | Cave | |
| Climax Area A | 67* | 138* | 15.6* | 8.8 | 0.87 | 1.0* | 1.0 | 58.3 | <15.2 | Stable | |
| Climax Area B | 34* | 122 | 15.6* | 7.8 | 0.75 | 1.0* | 1.0 | 25.5 | 1.0 | Stable | |
| Climax Area B | 67* | 138* | 15.6* | 8.8 | 0.87 | 1.0* | 1.0 | 58.3 | >15.2 | Unstable | |
| Climax Area B | 34* | 138* | 15.6* | 8.8 | 0.87 | 1.0* | 1.0 | 29.6 | 15.2 | Cave | |
| Climax Area C | 4* | 122 | 15.6* | 7.8 | 0.75 | 1.0* | 1.0 | 3.0 | 10.1 | Cave | |
| Climax Area D | 1.0* | 138* | 15.6* | 8.8 | 0.87 | 1.0* | 1.0 | 0.9 | 1.9 | Cave | |
| Crestmore | 93* | 30 to 137 | 8.6* | 3.5 16.0 | 0.64* | 1.0* | 1.0 | 59.5 | 7.0 | Stable | |
| Crestmore | 93* | 30 to 137 | 8.6* | 3.5 16.0 | 0.64* | 1.0* | 1.0 | 59.5 | 8.7 | Cave | |
| SET 3 (BACKS) | | | | | | | | | | | |
| 72-84 | 100 | 177 | 41.7 | 4.24 | 0.36 | 1.0 | 1.0 | 36 | 2.7 | Stable | |
| 72-86 | 100 | 177 | 39.4 | 4.48 | 0.38 | 1.0 | 1.0 | 38 | 6.1 | Unstable | |
| 75-86 | 100 | 177 | 27.0 | 6.5 | 0.6 | 1.0 | 1.0 | 60 | 7.6 | Unstable | |
| 74-97 | 40 | 177 | 30.7 | 5.8 | 0.54 | 1.0 | 1.0 | 21.6 | 8.8 | Unstable | |
| 74-79 | 40 | 177 | 28.9 | 6.1 | 0.56 | 1.0 | 1.0 | 21.6 | 13.4 | Unstable | |
| 77-99 | 20 | 177 | 32.6 | 5.4 | 0.49 | 0.5 | 1.0 | 9.8 | 6.1 | Unstable | |
| 77-91/93 | 20 | 177 | 31.7 | 5.6 | 0.51 | 0.5 | 1.0 | 10.2 | 15.2 | Cave | |
| 77-89 | 20 | 177 | 50.8 | 3.5 | 0.27 | 0.5 | 1.0 | 5.4 | 6.4 | Unstable | |
| 80-81/85 | 13 | 177 | 22.8 | 7.7 | 0.73 | 0.5 | 1.0 | 9.5 | 17.4 | Cave | |
| SET 4 (BACKS) | | | | | | | | | | | |
| | 1,000 | | | >10* | 1.0 | 1.0* | 1.0 | 1,000 | 50.0* | Stable | 1) Assumed low stress envi- |
| | 400 | | | >10* | 1.0 | 1.0* | 1.0 | 400 | 25.0* | Stable | ronments. Factor A = 1.0 |
| | 110 | | | >10* | 1.0 | 1.0* | 1.0 | 110 | 12.5* | Stable | 2) Orientation of structure |
| | 30 | | | >10* | 1.0 | 1.0* | 1.0 | 30 | 5.0* | Stable | with respect to backs not |
| | 10 | | | >10* | 1.0 | 1.0* | 1.0 | 10 | 2.5* | Stable | known. Factor B = 1.0. |
| | 4 | | | >10* | 1.0 | 1.0* | 1.0 | 4 | 1.3* | Stable | 3) Shape factor calculated |
| | 1 | | | >10* | 1.0 | 1.0* | 1.0 | 1 | 6.5* | Stable | assuming equal span in |
| | 0.1 | | | >10* | 1.0 | 1.0* | 1.0 | 0.1 | 0.3* | Stable | both directions. |
| SET 5 (BACKS) | | | | | | | | | | | |
| 33/34/34.5 | 18 | 103 | 28.6 | 3.6 | 0.28 | 1.0 | 1.0 | 5.0 | 7.3 | Cave | |
| 27-61 | 18 | 103 | 51.2 | 2.0 | 0.1 | 1.0 | 1.0 | 1.8 | 5.0 | Unstable | |
| 21/22.5 | 18 | 103 | 8.6 | 12.0 | 1.0 | 1.0 | 1.0 | 18 | 9.9 | Cave | |
| 30.5 | 18 | 103 | 20.2 | 5.1 | 0.44 | 1.0 | 1.0 | 7.9 | 6.8 | Cave | |
| SET 6 (BACKS) | | | | | | | | | | | |
| 11CE Stage 1 | 85 | 100 | 34.0 | 3.0 | 0.22 | 0.5 | 1.0 | 9.4 | 7.0 | Stable | 1) Q' used is the average |
| 11CE Stage 2 | 85 | 100 | 40.0 | 2.5 | 0.16 | 0.5 | 1.0 | 6.8 | 8.4 | Stable | determined for stoping |
| 11CE Stage 3 | 85 | 100 | 56.6 | 1.77 | 0.08 | 0.5 | 1.0 | 3.4 | 5.8 | Slightly Unstable | areas by Barton (1977). |
| 42E2 | 85 | 100 | 70.8 | 1.41 | 0.04 | 0.5 | 1.0 | 1.7 | 3.1 | Unstable | |
| 63W2 | 85 | 100 | 62.4 | 1.6 | 0.06 | 0.5 | 1.0 | 2.6 | 7.6 | Unstable | |
| 12CEN | 85 | 100 | 44.2 | 2.3 | 0.15 | 0.5 | 1.0 | 6.4 | 5.0 | Stable | |
| SET 7 (STOPE WALLS) | | | | | | | | | | | |
| 45W3 | 85 | 100 | -8.9 | N/A | 1.0 | 0.5 | 6.8 | 289 | 16.1 | Stable | 1) For tensile stress A = |
| 11CE Stage 1 | 85 | 100 | -5.3 | N/A | 1.0 | 0.5 | 6.8 | 289 | 16.1 | Stable | 1.0 |
| 11CE Stage 2 | 85 | 100 | -5.3 | N/A | 1.0 | 0.5 | 6.8 | 289 | 21.3 | Some F/W Instability | |
| 11CE Stage 3 | 85 | 100 | -7.0 | N/A | 1.0 | 0.5 | 6.8 | 289 | 23.3 | Unstable | |
| SET 8 (STOPE WALLS) | | | | | | | | | | | |
| 24-41 H/W | 30 | 48.3* | 9.2* | 5.27 | 0.47 | 0.5 | 8.0 | 56.4 | 9.1 | Unstable | |
| 24-41 F/W | 12.5 | 48.3* | 9.2* | 5.27 | 0.47 | 0.5 | 8.0 | 23.5 | 9.1 | Unstable | |
| SET 9 (STOPE WALLS) | | | | | | | | | | | |
| 77-97 | 40 | 91.3 | -4.4 | N/A | 1.0 | 0.5 | 8.0 | 160.0 | 13.1 | Stable | 1) For tensile stress A = 1.0 |

*Denotes that the value is estimated.

# APPENDIX VI

## GUIDELINES ON THE USE OF THE NGI SYSTEM OF ROCK MASS CLASSIFICATION

# APPENDIX VI

## GUIDELINES ON THE USE OF THE NGI SYSTEM OF ROCK MASS CLASSIFICATION

The six parameters that are quantified in the NGI system are described on pages 194 through 196 of the original paper (Barton, Lien and Lunde (1974)) and are summarized in Appendix I. This is adequate information for use in the field during mapping, although the following comments may help the user to obtain consistent and reliable results.

(a) It is useful if both diamond drill core logging and underground mapping can be carried out. The drill core provides good information on the frequency of fractures, particularly if there has been substantial blast damage to the rock underground and the rock surface is covered with mud or diesel exhaust deposits.

(b) Underground openings provide information on the continuity of continuous fractures which cannot be detected in the core. Continuous fractures often have a significant influence on the stability of open stopes.

(c) In underground mapping, RQD measurements can be made by the method described by Barton, et al (1974) pp. 196-197. The procedure consists of measuring the number of joints per meter for each joint set. Care should be taken to distinguish between natural and blast fractures. Blast fractures are usually planar and have no infilling. In the walls and the back they are usually aligned parallel to the blast hole direction, while in the face they form a rosette pattern.

(d) In determining the number of joint sets, it is useful to map drifts, cross-cuts and possibly raises because fractures which are parallel to the direction of the opening will be rarely visible compared to fractures at right angles to the opening. Similarly, horizontal fractures can best be mapped in raises.

(e) In order to distinguish between sets of fractures and random fractures, it is necessary to measure the orientation of the fractures with a geological compass. A stereoplot of the data will show how many sets are present. This is particularly useful when mapping a ramp for example, where it is difficult to maintain ones orientation.

(f) One should keep in mind the purpose of the mapping and pay particular attention to those fractures which will have the greatest influence on stability. For example, if high, narrow stopes are to be mined, then the stability of the back will be critical and good information on the character and occurrence of horizontal fractures should be obtained.

(g) If using the design procedure described in this report, the SRF factor should be set to 1.0 and appropriate modifications made to the Q value described in Chapter 3.