

THE UNIVERSITY OF ZAMBIA
SCHOOL OF MINES
DEPARTMENT OF MINING ENGINEERING

AN ASSESSMENT OF ROOF SUPPORT DESIGN REQUIREMENTS
IN HARD ROCK MINING BASED ON THE MBR (MODIFIED BASIC
ROCK mass rating)

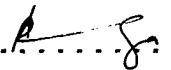
by
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A dissertation submitted in partial fulfillment of the degree
of Master of Mineral Sciences - Mining Engineering
(Rock Mechanics)

June 1988

DECLARATION

I, Robert Matembu Kamanga, declare that this thesis is my own work, and that it has not previously been submitted for a degree at this or another university.

Signed.....

APPROVAL

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of the degree of Master of Mineral Sciences in
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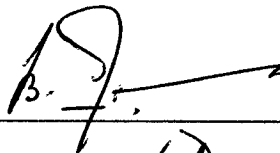
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ABSTRACT

This dissertation describes an investigation carried out at Baluba Mine to assess the quality of rock mass and support requirements in haulage drives developed in the footwall quartzite formations. The rock mass of the footwall formation is geotechnically quantified using the Modified Basic Rock mass rating system (MBR) and the Norwegian Geotechnical Institute index (NGI) or "Q"-system. Both classification systems are described and applied to three haulage drives, namely, the 245m, 300m and 480m levels. The purpose of this study was to determine whether the MBR was suitable and applicable to local conditions at Baluba Mine. A comparison of the results between the MBR and NGI showed that both systems use similar parameters except that they are expressed and used in different ways to quantify rock mass. The major difference is that MBR goes further to take into account production effects on haulage stability. The conclusions drawn from the field study data are that the four initial input geotechnical parameters used in the MBR are easily measurable and of the three first adjustment parameters used, blasting damage proved difficult to assess and to quantify. The main conclusion, however, is that despite limitations in quantifying blasting damage and determining Rock Quality Designation (RQD) on haulage sidewalls, the MBR is suitable and applicable to the mining

environment at Baluba Mine as long as the following recommendation is considered; blasting damage adjustment should be replaced by a Modified Fracture Frequency (MFF) which will take into account natural and artificial fractures. It is further recommended that in order to enhance the data base generated by the MBR, a complete hazard analysis of the drives be part of routine rock mass quality assessment.

In Appendix A, individual parameter rating tables and figures and support recommendation tables for both MBR and NGI are given. All field data are given in Appendix B.

CHAPTER ONE

1.0 INTRODUCTION

1.1 General Introduction

In recent years rock mass classification systems have gained immense recognition as a means of predicting ground conditions during exploration and actual mining. For a number of years these systems have also been used for estimating support requirements for tunnels and other civil engineering underground excavations. Most rock mass classification systems such as those by Bieniawski (1973, 1979), Laubscher (1977) and Barton et al. (1974) serve as means of correlating experiences of rock condition at different sites. Others are too general, such as Terzaghi (1946), to permit an objective evaluation of rock mass behaviour and provide no quantitative information while some, such as Bieniawski, (1973) and Barton et. al (1974), are too sophisticated.

The objective of a rock mass classification system is to quantitatively isolate categories of rocks that behave in a similar manner under certain controlling engineering and geological conditions. A classification system must enable an engineer with basic data to predict the support measures required under the

prevailing conditions and justify its use at all stages, from mine planning, development, production to post-production.

The principle of rock mass classification is that a numerical assessment is made of the physical and geological properties that control the behaviour of rock mass and a single rating is derived to represent the different properties. Based on experiences and study of numerous case histories, a final rating is related to support for various mining conditions.

1.2 Statement of the problem

One way of reducing costs in mining operations would be to improve methods of predicting ground behaviour. This is important because it can facilitate choice of better, cheaper and effective mining methods and support systems. A suitable choice of a rock mass classification system that would easily be part of normal routine geotechnical investigations would achieve such goals. The need, therefore for a straight forward classification technique that will be suitable and effective in local conditions is important.

The objective of this dissertation is to assess the applicability and efficiency of the Modified Basic Rock mass rating (MBR) system to well bedded and

moderately jointed rock masses at Baluba Mine of the Zambia Consolidated Copper Mines (ZCCM). The results obtained from the MBA tests are compared to the predictions by the Norwegian Geotechnical Institute (NGI) or Q-system. The MBA is chosen because it has never been tested at Baluba Mine. The classification is particularly interesting because it combines ideas of well known Bieniawski's (1973) 'Rock Mass Rating' system and Laubscher's (1977) 'Geomechanics' classification. As it will be shown in the next chapter, the method can easily be used to freshly developed haulage drives and adjusted to changing conditions after development.

CHAPTER TWO

2.0 REVIEW OF ROCK MASS CLASSIFICATION SYSTEMS USED IN TUNNELLING AND MINING.

The concept of classifying rock according to its qualities has been used for a number of years. During this time several classification techniques have been developed. In 1946 Terzaghi developed a classification for use in estimation of 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. However, Cecil (1970) found that Terzaghi's classification was too general to permit an objective evaluation of rock quality and that it provided no quantitative information on the properties of the rock mass.

The classification by Lauffer (1958) introduced a new parameter of performance, the 'stand-up time' of unsupported span. The classification has its foundation in the earlier work on tunnel geology by Stini (1950). Lauffer's original classification is no longer used. It has been modified a number of times by other engineers, notably by Pacher, Von Rabcewicz and Golser (1974). The

Lauffer and Pacher classification had a significant influence on the development of more recent rock mass classification system.

In 1966, Deere proposed a qualitative index based on a modified core recovery procedure which incorporates only sound pieces of core that are 100mm or greater in length. This rock quality designation (RQD) gained popularity rapidly and is now a standard type of core description method. Cording, Hendron, and Deere (1972) attempted to relate the RQD to Terzaghi's rock load factor. They found a reasonable correlation for steel-supported tunnels but not for openings supported by rock-bolts. Merritt (1972) found that the RQD could be of considerable value in estimating support requirements for rock tunnels, but pointed out a limitation of the RQD index in areas where joints contain the clay fillings or weathered material. On its own, RQD is not sufficient to provide an adequate description of a rock mass.

Bieniawski (1973) proposed Geomechanics classification which is also known as the Rock Mass Rating (RMA) system. The system utilizes six parameters; intact rock strength, RQD, joint spacing, condition and orientation and groundwater, each are assigned ratings similar to those proposed by Wickham et al. (1972) to give a rock mass rating(RMA) as a

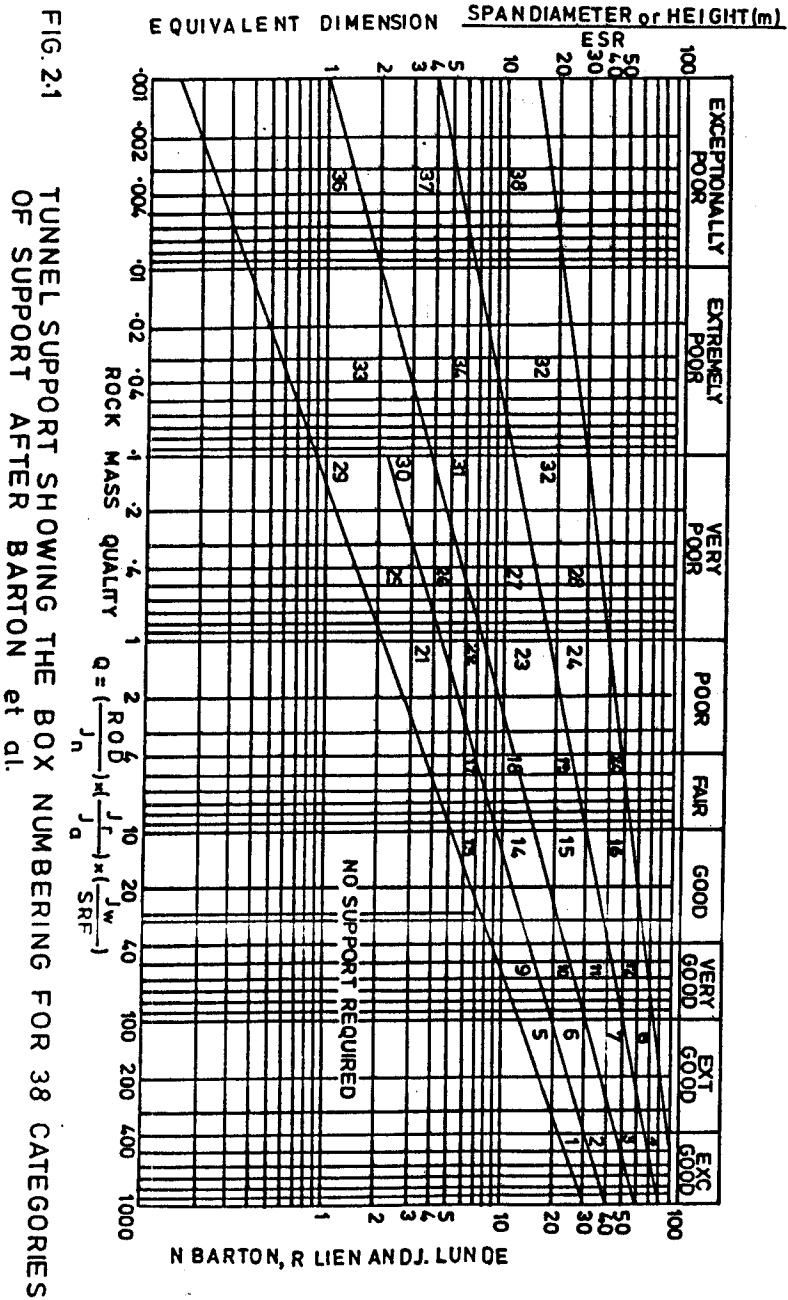
percentage. Five classes of rock mass were then defined, excavation support recommendations were made for each class and an estimate of the 'stand-up time', after Lauffer, was given for various sizes of excavations.

In 1974, Barton et al. basing their analysis on 200 case records of tunnel stability investigation produced a tunnelling quality index (Q) also known as Norwegian Geomechanics Institute (NGI) index. The index is related empirically to the amount and type of permanent support, with respect to excavation stability. The values of 'Q' ranges from 0.001 to 1,000 and 38 different support categories are accompanied by corresponding support recommendations as presented in Figure 2.1 and Tables A1, A2, A3, and A4 in appendix A. The Q-system is a function of six parameters which are rated according to their importance in the overall stability consideration. The six parameters include RQD, number of joint sets, joint roughness and alteration, water flow and stress reduction factor. The numerical value of the index 'Q' is defined by the following equation;

$$Q = RQD/J_w \times J_r/J_a \times J_w/SRF \dots\dots\dots 2.0$$

where RQD is defined below;

$$RQD\% = \frac{100 \times \text{length of core in pieces} > 100\text{mm}}{\text{length of borehole}} \dots\dots\dots 2.1$$



The other parameters in the equation are described as follows;

Jn is the joint set number

Jr is the joint roughness

Ja is the joint alteration number

Jw is the joint water reduction number

SAF is a stress reduction factor

The numerical value of each of the six parameters is determined from Table A5 in the appendix A. The reasoning behind this equation is that each of the quotients represents an important quantity of the rock mass. Thus RQD/J_n represents the structure of the rock mass and is a crude measure of a block. The quotient J_r/J_a represents interblock shear resistance or friction coefficient and J_w/SAF represents active stress. Although the Q-system appears to have been specifically developed for tunnels and chambers, Tinkler (1976) applied the system successfully to single mining drives such as those the author will consider in the next chapters.

2.1 Modified Basic Rock mass rating (MBR)

A particularly interesting rock mass classification system which was introduced in 1983 is the MBR system. The MBR classification system was developed by the Engineers International Incorporation (USA) (Cummings et al 1982 and Kendorski et al. 1983). The system encompasses a spectrum of geological and engineering factors. These factors are outlined in Figure 2.2. It uses the basic RMA approach by Bieniawski (1973, 1979) with some of the concepts of Laubscher (1977). The main difference lies in the arrangement of the initial rating terms and in the adjustment sequence. The MBR is a multiple-stage adjustment; the output at each stage can be related to support for various mining conditions. The flow chart in Figure 2.2 shows how the system works. The system is based on four initial input parameters namely intact rock strength, discontinuity density and condition and groundwater condition. Each of the four parameters is rated according to Figures A3, A4, A5, and Tables A8 and A9 in appendix A. The initial total MBR is obtained by summing up the individual parameter ratings. This MBR rating classifies general rock competence, without regard to the geometry of opening constructed in it. At this stage Figure A6 is consulted for support

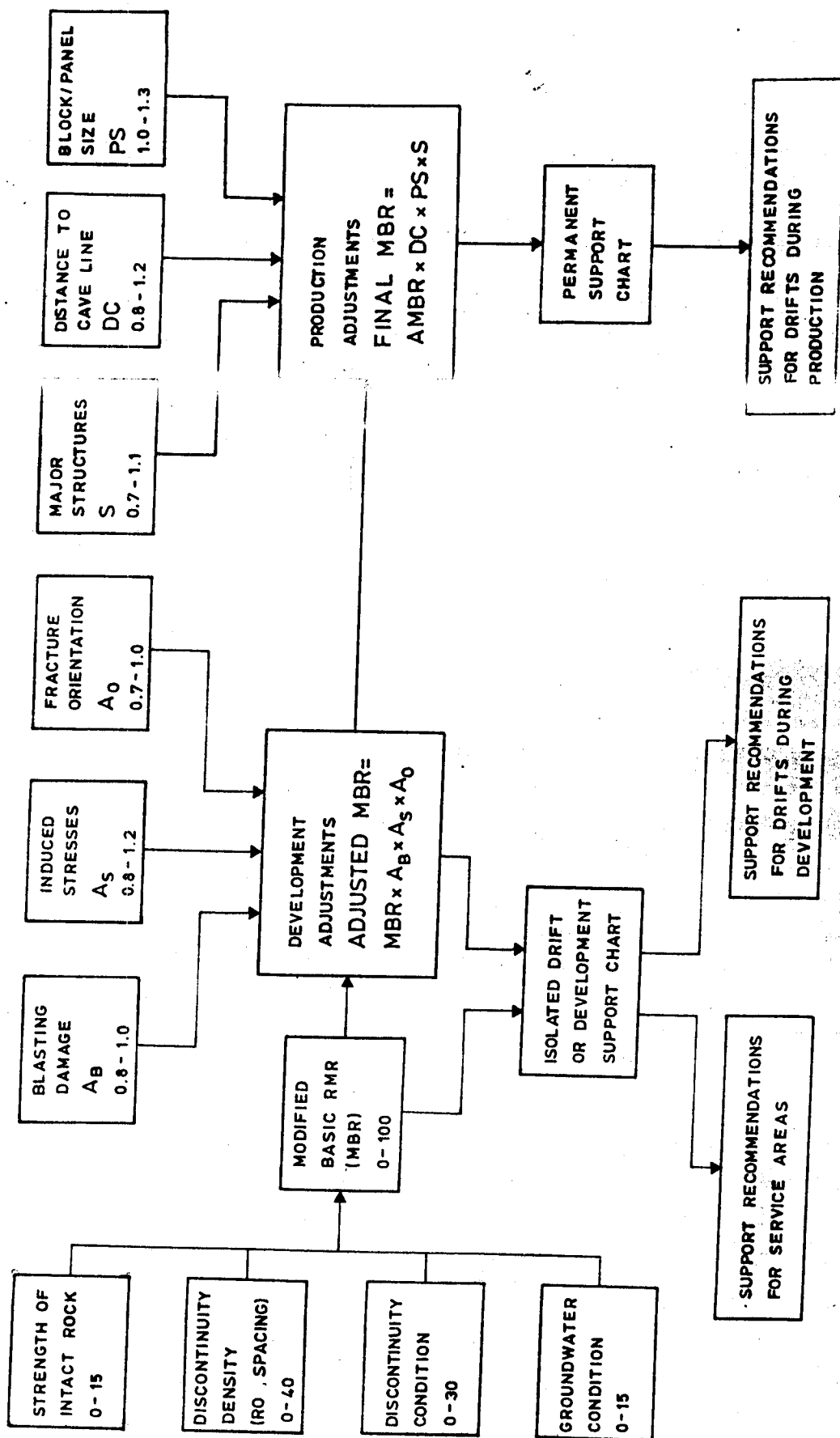


Figure 2-2 Organization of the MBR system (after Kendorski et al, 1983)

recommendations for isolated drives.

The second stage involves assigning numerical adjustments to the MBR so that ore block development process is taken into consideration. The adjustment relating to ore block development includes induced stress, fracture orientation and blasting effects. The three parameters are rated according to Figure A7 and Tables A10, A11, and A12 in appendix A. The total initial MBR is then multiplied by the three adjustment ratings, to give a range of preliminary support requirement for drift development. Figure A6 is again consulted for support recommendations.

The third stage involves applying additional adjustments that take into consideration geology and mining geometry. The three input parameter adjustments includes major structure adjustment (S), adjustment for proximity to the cave-line (DC), and adjustment for block/panel size (PS). The major structure adjustment is computed from Table A13, the adjustment for proximity to the cave-line is computed from Figure A8 and the adjustment PS is computed from Figure A9 in appendix A. The three adjustments are multiplied together and then multiplied by the adjusted MBR. The final MBR is correlated to permanent support recommendations given in Figure A10 in appendix A. From foregoing description of the MBR system it is clear that the system has potential

1

for application to the Zambian mines. This application is highlighted in chapter 5.

2.2 Discussion

From the preceding, and by no means complete review of rock mass classification systems it is apparent that rock mass classification systems are being continuously updated and modified to suit local conditions or particular projects.

The MBR is particularly attractive for mining applications. Its special features include the following:-

1. The MBR is a multiple adjustment system. The output at each stage during analysis can be related to support for various mining conditions. The system also facilitates the choice of support according to the performance period desired, since lighter support will be adequate in some rocks for short periods.
2. The classification system can be used with very preliminary geotechnical information from drill hole.
3. It combines the ideas of most favoured systems of Laubscher's (1977) Geomechanics and Bieniawski's (1973) RMR.

Consequently, the MBR classification system will be applied to the geological and mining environment described in the next chapter.

CHAPTER THREE

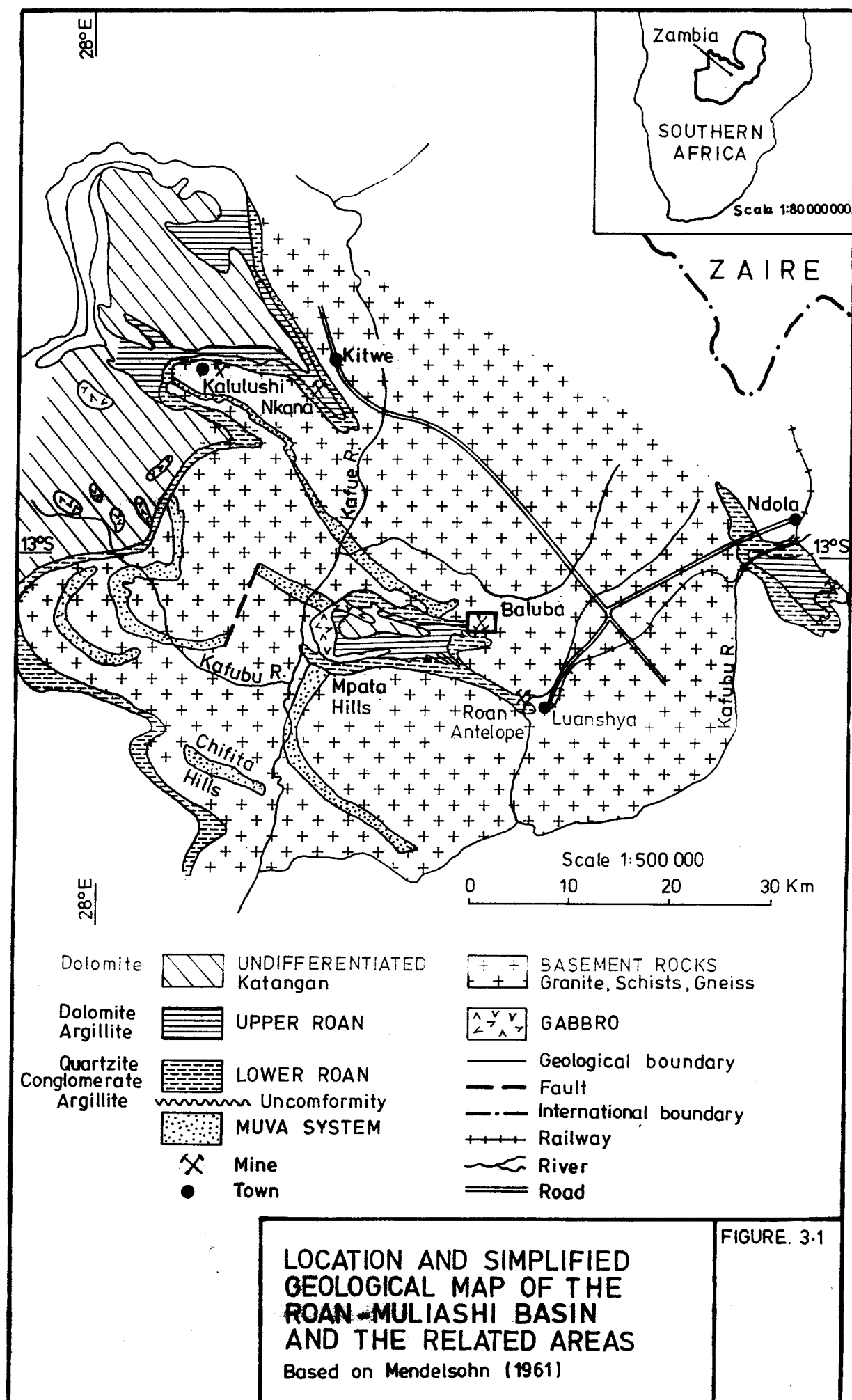
3.0 GEOLOGICAL AND MINING ENVIRONMENT AT BALUBA MINE

3.1 General Setting

Baluba mine is one of the operating copper and cobalt mines of the Zambia Consolidated Copper Mines (ZCCM). The mine is situated along the northern margin of the Roan-Muliashi basin as shown in Figure 3.1 and 3.2. The East to West trending Roan-Muliashi basin is about 2.7 Km long and upto 6.4 Km wide. The Baluba copper and cobalt deposit is situated on the northern limb of the basin and has a mineralised strike length of three kilometres. The economic copper and cobalt is mainly confined to a dolomite schist zone of an argillite-dolomite schist horizon (AL 6). The main rock types present in the Baluba area are shown in Figure 3.2 and 3.1.

3.2 Rock type and structural setting

The rock types within the subject region are comprised of sedimentary rocks of the Katanga System, which exists in a well bedded sequence of massive



LOCATION AND SIMPLIFIED GEOLOGICAL MAP OF THE ROAN-MULIASHI BASIN AND THE RELATED AREAS
Based on Mendelsohn (1961)

FIGURE. 3-1

GENERALIZED SURFACE PLAN OF BALUBA

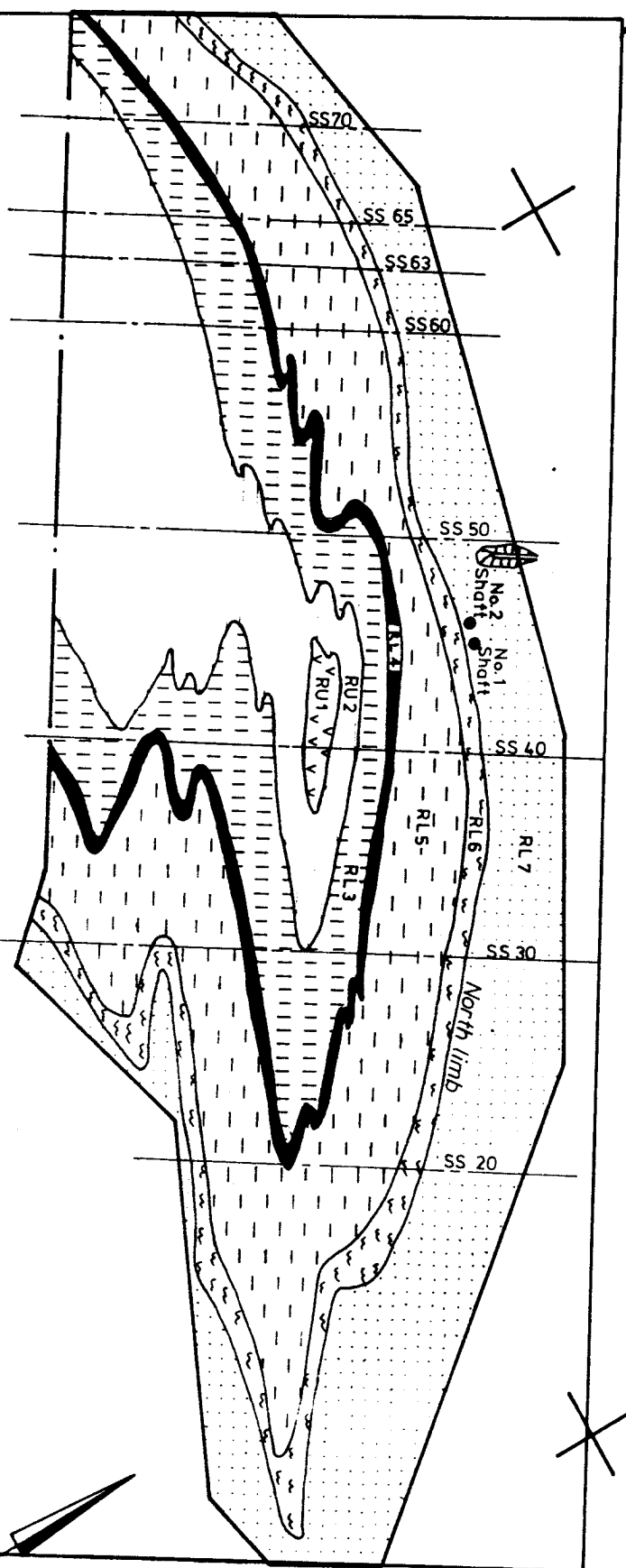
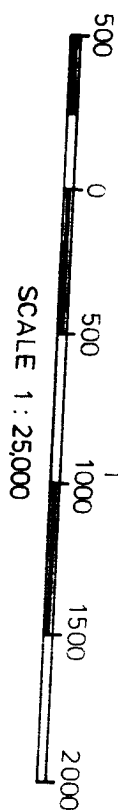


FIGURE 3.2

aeolian and argillaceous quartzites. Both these sequences are generally strong and considered suitable for the positioning of development. The Aeolian Quartzite is underlain by the Basal Conglomerate which lies unconformably on the Basement Complex. The Basement Complex- Conglomerate contact is generally weak. The Conglomerate varies in thickness, the maximum being 30 metres. Basement rocks, mainly quartz-biotite schists, are exposed in the development areas north of the shafts on lower levels (245m, 480m, and 580m). Basement Complex granite is also exposed on the 245m level workshops (Baluba East). The Argillaceous Quartzite is overlain by the RL6 formation. This formation, which consists of orebody dolomite and argillites, varies in thickness from 30 metres in the West to about 65 metres in the East and almost disappears in the extreme East towards the nose of the basin. The RL6 formation is overlain by RL5 formation which has a thickness of about 100 metres. The RL5 formation consists of dolomites, quartzites and argillites. The RL5 Dolomite may contain significant amounts of water and in some parts, the formation is also highly weathered. The RL4 which overlies the RL5 formation has a thickness of about 20 metres. It is composed mainly of dolomite, mica and chert in the lower part and greenish schistose argillite in the upper zone.

The RL4 is the main hangingwall aquifer, and must be drained prior to mining. Neither the RL4 nor any higher formations are exposed in underground workings at Baluba Mine. The stratigraphic sequence of the Baluba area is presented in Table 3.1.

Structurally the orebody has been divided into North, Centre, and South Limbs as shown in Figure 3.3. The syncline and anticline which form these limbs plunge gently towards the west until the North and Centre Limbs merge into the North Limb. There is a gradual increase in the dip of the rocks westwards, from as low as 10 degrees on section 33 (above 245m) to almost vertical on section 52. The orebody dip also varies with depth, generally steeper below 245m level.

3.3 Mining

3.3.1 Mining method

Baluba Mine, operating since 1973, currently produces about 2 million tonnes of ore per year. The mining method used is the standard sublevel open stope, with variations in the methods of extraction. The extraction methods employed depends on the orebody configuration including its dip. For steeply dipping sections (dips greater than 60 degrees), grizzly ore

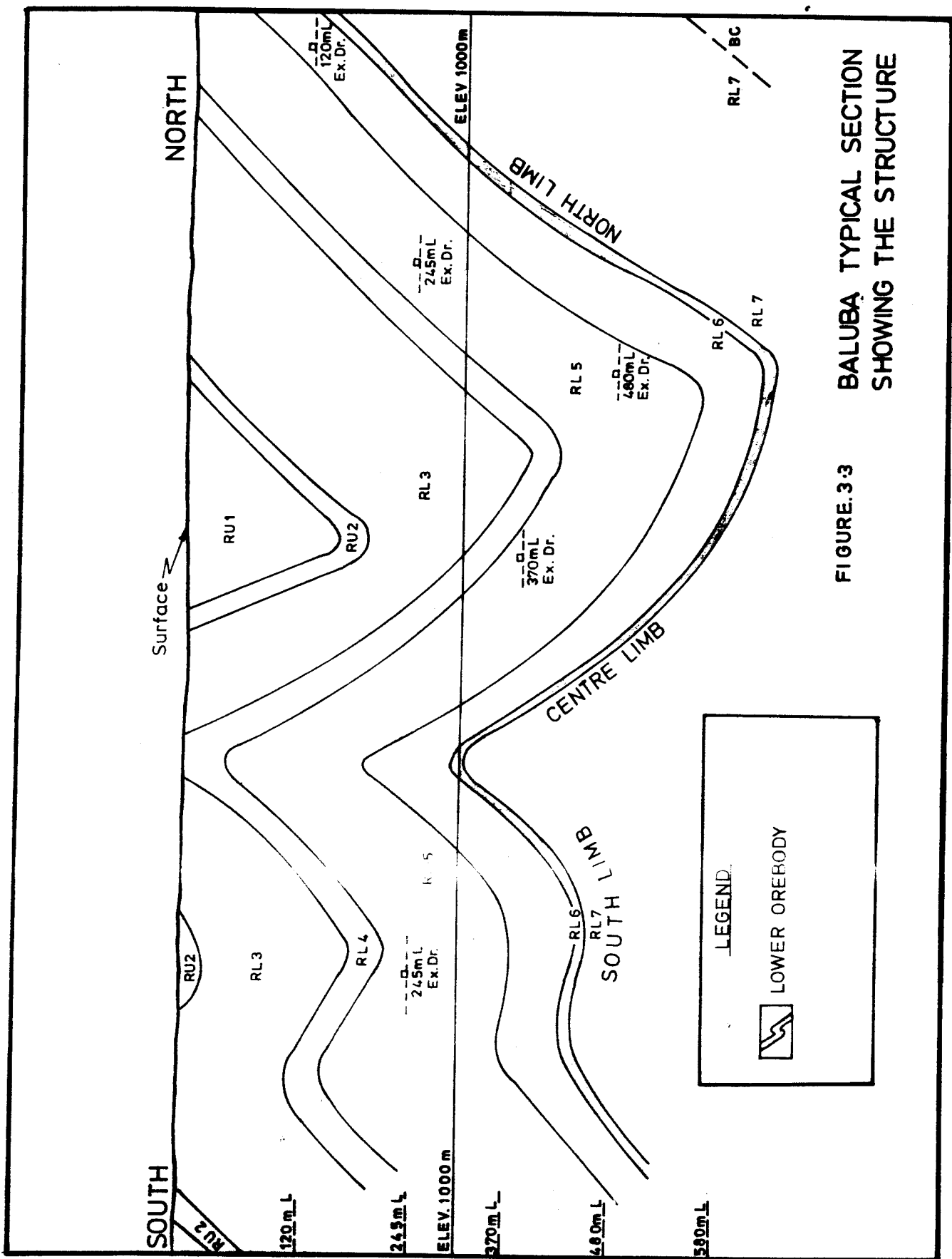


FIGURE 3.3 BALUBA TYPICAL SECTION
SHOWING THE STRUCTURE

Table 3.1 The stratigraphic sequence in the Baluba Area

SYSTEM	SERIES	GROUP	FORMATION	THICKNESS	DESCRIPTION
Katanga	Kundelungu			?	Tillite-weak clay matrix. Not present at Baluba.
			Mwashia	100m	Finely banded carbonaceous shale Not present at Baluba.
		Upper Roan			
			RU1	500m	Dolomite and dolomitic agillite U.C.S* 10-40 MPa
			RU2	30-50m	Argillite with some dolomite and quartzite. U.C.S 50-80 MPa
	Mine				
			RL3	100-130m	Pebbly arkose and feldspathic quartzite. U.C.S 110-140 MPa
			RL4	40-30m	Dolomite undelying green shale. Weak and cavernous. U.C.S 5-20 MPa
		Lower Roan			
			RL5	60-120m	Interbedded quartzite and quartzitic argillite. U.C.S 70-100 MPa
----- (unconformity) -----					
LUFUBU	Basement Complex				Quartzite-biotite schist. Intrusive granite in Places.

* U.C.S = Average unconfined compressive strength.

extraction is often used and for orebodies with dips less than 60 degrees, tracked or trackless drawpoint extraction is employed. The mine is serviced by two shafts and a portal to a depth of 675m level and 480m level respectively. Service and extraction drives are developed parallel to the orebody strike using access crosscuts from the shafts and the portal.

The footwall drives are developed in 'competent' argillaceous and aeolian quartzites (unconfined compressive strength 150-180 MPa). The tramming and extraction haulage drives are about 3.6m wide and 3.6m high. The rock mass in which the drives are developed varies from highly to poorly jointed. Highly jointed sections are usually wet to very wet, with water flows upto 90m³/hr in some areas. The footwall access drives which are also established from the portal are about 2.4 metres wide and 2.4 metres high. Vertical development spacing between footwall drilling drives and the main extraction drives varies from 10 metres for shallow orebody to about 18 metres for steeply dipping orebody. The minimum horizontal spacing between tramming and extraction drives is about 8 metres.

3.3.2 Support methods currently used

The main types of support used are, grouted wire dowels, steel arch sets, timber sets and props, split sets, untensioned rebars, bamboo dowels (temporary) and shotcrete. Table 3.2 shows the support elements commonly used in various underground excavations at Baluba Mine. There is, however, an increasing use of wire dowels from discarded winding rope.

The general setting and mining conditions having been discussed, detailed rock mass characteristics of three study sites are presented in chapter 4.

Table 3.2 Support elements presently used

Support commonly used	Type of excavation
Wire dowels-grouted but untensioned Ring spacing=1.5m Bolt spacing=0.75-1	Main haulage drives, workshops, footwall access drives, drawpoint crosscuts, ramps and drainage drives.
Bamboo dowels-grouted Ring spacing=1.0m Dowel spacing=0.75-1m	Stopes, footwall and hangingwall drives.

Steel arch sets	Main haulage drives, ramps,
Spacing varies with	pump chambers and workshops.
ground condition	

Timber sets and props	Main haulage drives, footwall access drives, drilling drives and drawpoint crosscuts.
-----------------------	---

Shotcrete	Main haulage drives, workshops,
Thickness varies with	main access crosscuts, and pump
ground condition	chambers.

Untensioned grouted	Main haulage drives, footwall
rebars.	access and drilling drives and
Ring spacing=1m	workshops.
Rebar spacing=0.5-1m	

Split sets with or	Main haulage drives and pump
without wire mesh	chambers.
Spacing varies with	
ground condition.	

CHAPTER FOUR

4.0 SITE INVESTIGATION

4.1 Introduction

To adequately assess the requirements for the design of roof supports, the process of collecting geotechnical information is important in underground excavations. The purpose of collecting this information is to characterize the rock mass, to predict likely future geological conditions, identify possible unstable or hazardous areas and to determine support requirements.

In most underground excavations, it is the presence of the discontinuities and the degree of cohesiveness of the rock material which control the rock mass behaviour and its interaction with support.

In this chapter geotechnical parameters that control the overall behaviour of rock mass are discussed. Three major production levels, namely the 245m, 300m, and 480m levels are considered. The geotechnical parameters obtained from the three levels are used in the next chapter as input data into the MBR and 'Q' classification systems for subsequent assessment of ground conditions and support

recommendations. The haulage drives were particularly chosen because of their importance as accesses to the remote eastern part of Baluba as well as their importance as tramming and extraction drives. They were also chosen after taking into consideration the fact that in sublevel stoping, drives are developed years ahead of production and hence the need to support them adequately before commencement of mining the orebody.

4.2 Location of study areas

The three drives from which geotechnical data were obtained for this investigation are: the 245 metre level extraction and tramming haulage West, located in the Centre Limb; the 300 metre level extraction haulage East and the 480 metre level tramming haulage East, both are situated in the North Limb.

4.2.1 The 245ml haulage drive West

At the time of this investigation the 245m level haulage drive was still being developed westwards. On completion, it will serve as a tramming and extraction haulage drive. The drive is developed in well bedded, moderately jointed and dry RL 7 Argillaceous Quartzite. The section of the haulage

under study is spot bolted and is shown in Figure B11 in appendix B.

4.2.2 The 300ml haulage drive East

The 300 metre level extraction haulage East is developed in well bedded (about 60 degrees dip) moderately jointed and dry RL 7 Argillaceous Quartzite. When production commences the drive will be used for extracting the orebody which is located about 18 metres to the south of it. The drive follows approximately, an East-West Route, along the strike. The section of the drive under study is spot bolted and is shown in Figure B12 in appendix B.

4.2.3 The 480ml tramming haulage drive East

The 480 metre level tramming haulage drive East is developed along the contact between RL 7 Argillaceous Quartzite to the South and RL 7 Aeolian Quartzite to the North. Although the contact appeared strong at the time of mapping, the possibility of deterioration when production commences cannot be ruled out. The drive follows approximately, an East-West route, along the strike. The Aeolian Quartzite is massive and moderately jointed. The Argillaceous Quartzite is well bedded

(dipping at 70 degrees south) and moderately jointed. The section of the haulage under study is shown in Figure B13 in appendix B.

4.3 Methods of Investigation

Underground structural mapping was conducted in the three haulage drives already discussed in section 4.2. The geometry and spatial position of the joints and bedding were recorded and the scanline technique by Sen et al. (1984), Karzulovic et al. (1984) and Priest (1981, 1985) was applied as a means of obtaining statistical samples of joint spacing and AQD. Priest (1981) recommended use of the stereographic method for the statistical identification of joint sets and the determination of the possible mode of failure of rock wedges was carried out using schmidt equal area net (lower hemisphere). An attempt to quantify the roof and side wall conditions by using non-destructive insitu relative strength tests were conducted on some prominent joint and bedding planes. The instrument which was used for these tests was the N2-4 type Schmidt Hammer. The recommended procedure for point load testing by the International Society for Rock Mechanics, Commission on Testing Methods (ISRM, 1985) was carried out on a total of 38 cores of 42mm and 47mm diameters (refer to Tables

B14, B15, and B16 in appendix B). The size correlation factor suggested by Brook (1985) was applied.

4.4 245m1 haulage - Geotechnical parameters

4.4.1 Fracture survey

Engineering geological mapping was carried out between structural sections (SS) 28 and 33, in dark-grey moderately bedded RL 7 Argillaceous Quartzite (figure B11 in appendix B). In MBR and 'Q' systems, RQD values occupy prominent position. The procedure for obtaining RQD is based on recording of fractures intersected by underground drift walls. On the basis of the number of discontinuities per metre along the scanline on the northern wall, mean discontinuity and mean frequency were calculated. This information was in turn used to calculate RQD values. The RQD was calculated according to Priest and Hudson, (1976) :-

$$RQD = 100^{-(0.1\lambda)} (0.1\lambda + 1) \quad \text{-----} \quad 4.1$$

where λ = mean discontinuity frequency.

A list of observation and statistical values is given in Table 4.1 to characterize strata in which the haulage is developed. Detailed structural mapping data is also presented in Table B17, appendix B.

Table 4.1 Strata characteristics

Fracture data	Location : 245ml northern wall
	Rock type: RL 7 argillaceous
	Quartzite
Number of discontinuities	48
Length of scanline	150m
Mean discontinuity	0.32/m
Mean population frequency	3.1
RQD%	96

4.4.2 Discontinuity concentration

A total of 73 discontinuities (mainly bedding and joints) were measured in the 245ml haulage drive West. Individual plots on Schmidt equal area (lower hemisphere) diagram in Figure 4.1 revealed four joint sets and one bedding plane (Figure 4.2). The prominent of the four joint sets was set 2 which plotted joint density greater than 12 percent. A few scattered random joints were also observed as shown in Table B17 in appendix B. A summary of discontinuity set data is also shown in Table 4.2. Further analysis involving the

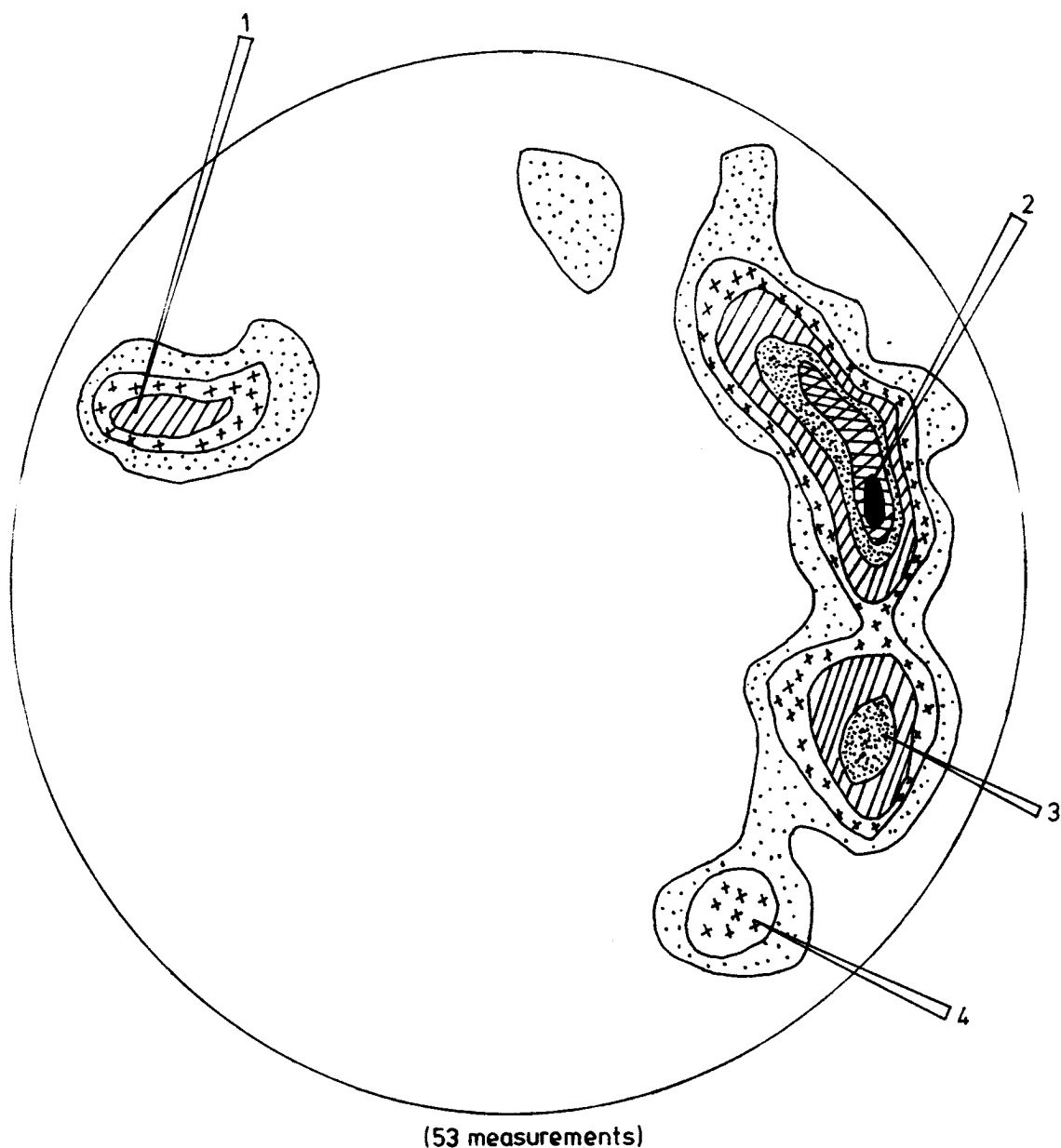


FIGURE. 4.1

KEY		
DENSITY CONTOUR LEVELS(%)		
CONTOUR PLOT OF JOINTS	 <2%	 >10≤12%
LOWER HEMISPHERE	 >2≤4%	 >12%
EQUAL AREA PROJECTION	 >4≤6%	
245m LEVEL TRAMMING HAULAGE WEST (CENTRE LIMB)	 >6≤8%	
	 >8≤10%	

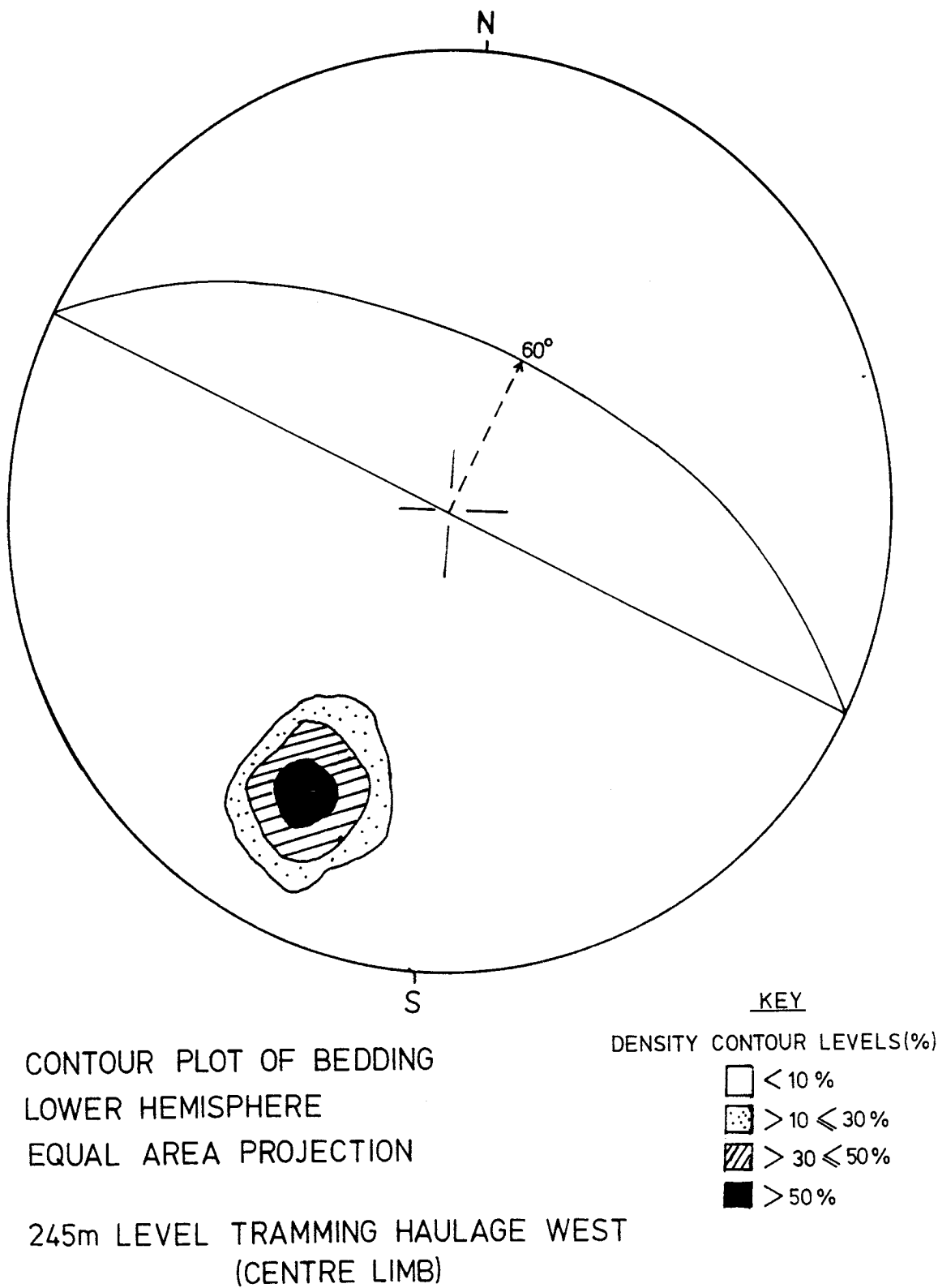


FIGURE 4.2

planes representing the poles of most prominent joints was conducted to determine potentially unstable wedges formed by intersecting planes. Figure 4.3 shows a tetrahedral wedge bounded by four joint planes and a bedding plane. The points of intersection in the figure surround the centre of the net, an indication that the wedge can fall freely from the roof provided there is no cohesion on joint planes or stress that will tend to force it out of its position. Detailed structural data is presented in Table B17, appendix B.

Table 4.2 Discontinuity set data

Discontinuity set	Number of joints/ bedding	Dip direction (degrees)	Discontinuity separation (average)	Gouge
1	8	113/64	1-2mm	clay
2	17	255/50	1-3mm	infill
3	14	295/64	1-3mm	"
4	5	325/66	1-2mm	"
Random	9	-	-	"
Bedding	20	-	-	"

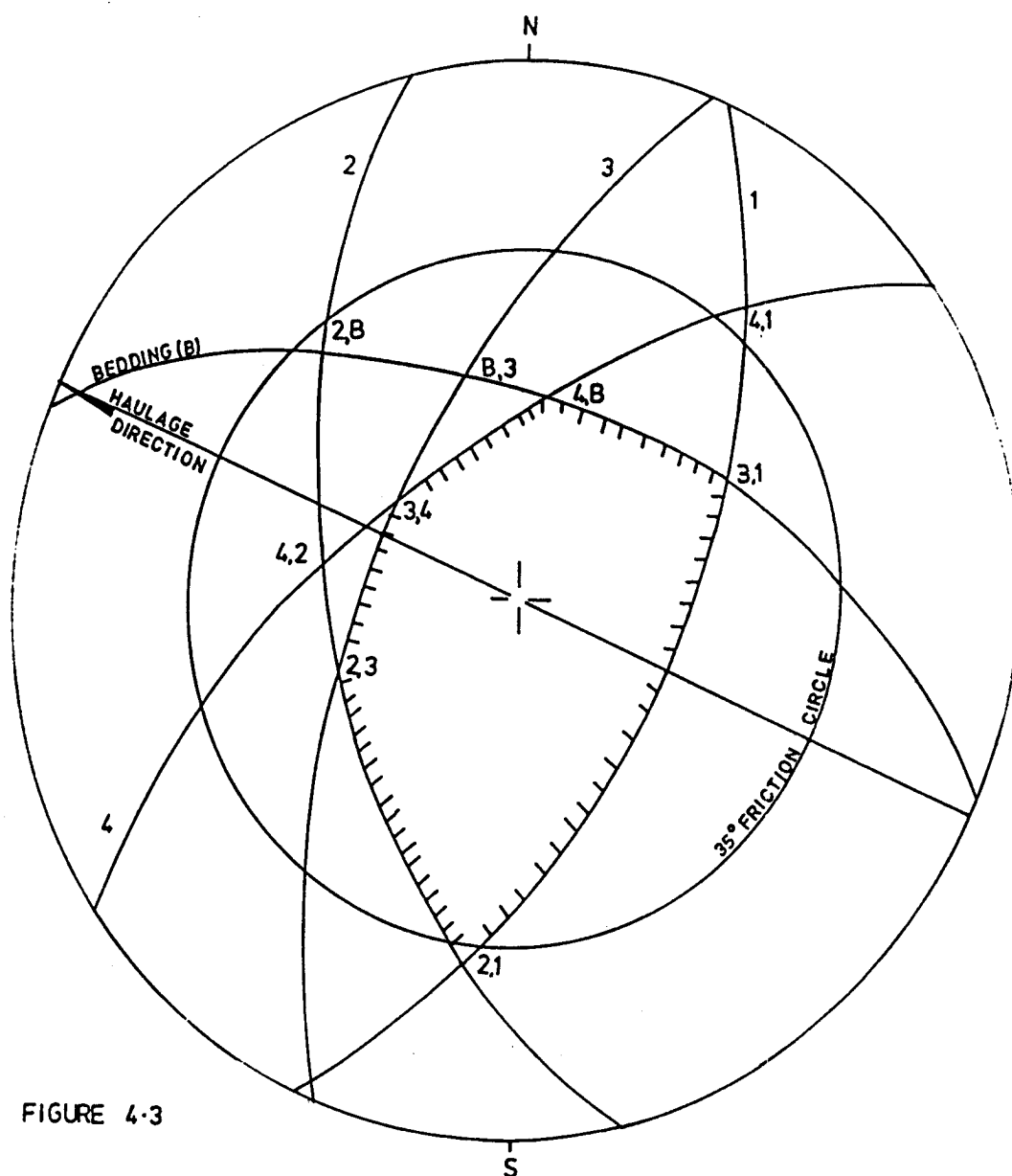


FIGURE 4-3

PLOTS OF PROMINENT JOINT
PLANES SHOWING LARGEST
TETRAHEDRAL BLOCK
245m LEVEL ARGILLACEOUS QUARTZITE

KEY

1, 2, 3, 4 Joint planes

B Bedding

 Largest
Tetrahedral
block

4.4.3 Discontinuity condition

Generally the majority of discontinuities observed in the 245m level haulage drive West had infilling material. The most common were red clay and calcareous cement, mainly dolomite and calcite. Red clay, mostly occurs as thin film on joint planes. The clay filled joints appeared weaker than the dolomite or calcite filled ones. Some surfaces had slickensides and others were generally rough. The joints observed in borehole, BX 1740, confirmed site observations.

Schmidt Hammer tests on five joints, one from each joint set indicated joint wall strength greater than 64MPa for the RL 7 Argillaceous Quartzite. Table 4.3 shows the results obtained on the 245m haulage West.

Table 4.3 Schmidt Hammer (S.H) test results in RL7
Argillaceous Quartzite - 245ml

Station	Joint set	Joint direction/ dip	Description of joint surface	Mean S.H. No.	S.H. strength MPa
9	1	131/55	smooth with <1mm thick clay	46	60
21	2	252/65	smooth, slickensided <1mm clay	48	65
22A	3	293/54	smooth with <1mm thick clay	49	65
22B	4	317/70	smooth to moderately rough, no gouge	50	69
22C	random joints	varies	smooth with <1mm clay	45	60

Average joint compressive strength = 64 MPa

4.4.4 Intact rock strength

In the MBR classification system intact rock strength (IRS) is used directly as an input parameter and indirectly used in Q-system. On the 245m level, IRS was determined from 11 core samples taken at every 0.5m interval from borehole number BX 1740. A Farnell point load testing apparatus was used for this purpose. Due to non-availability of the 50mm recommended core size for diametral point load tests, Brook's (1985) method for size correlation was used. The following formular was used:-

$$Is(50) = F.P/D^2 \dots\dots\dots 4.2$$

$$\text{where } F = (De/50mm)^{0.45} \dots\dots\dots 4.3$$

$Is(50)$ = point load index for 50mm
diameter core

D = diameter of core sample

De = equivalent diameter

P = failure load

Brook's conversion factor of 22 was also used to convert the point load index ($Is(50)$) to uniaxial compressive strength (UCS). This factor was justified by the results from the plot of $Is(50)$ against

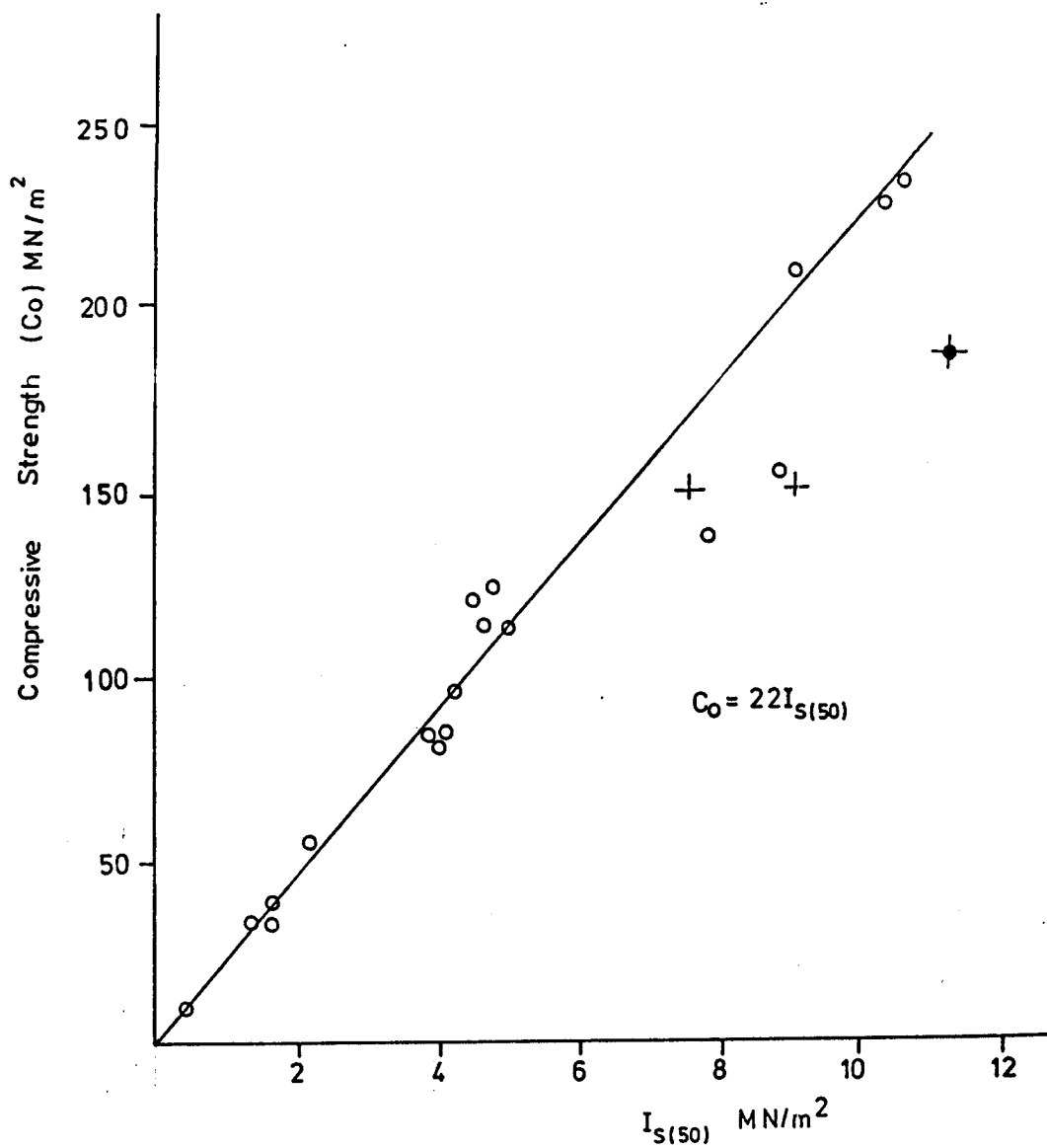


FIGURE. 4.4

A graph showing the relationship between compressive strength, C_o in MN/m^2 and $I_{s(50)}$ in MN/m^2 for a wide variety of rock types (after Brook 1985)

○ after Brook 1985

+ plot of Baluba Argillaceous Quartzite(1987)

◆ plot of Baluba Aedian Quartzite (1987)

laboratory determined UCS as shown in Figure 4.4. The points as shown in the figure fell within the general trend of Brook's curve. Table 4.4 shows a summary of point load results for the RL7 Argillaceous Quartzite. Detailed point load indices and converted results are also presented in Table B14, in appendix B.

4.5 300ml haulage - Geotechnical parameters

4.5.1 Fracture survey

On the 300ml, geological mapping was carried out between Mining Sections (MS) 7399 and 7385 in the HL7 Argillaceous Quartzite. The drive is developed parallel to the strike and intersects a weak shear plane between MS 7399 and 7391 as shown in Figure B12. The RQD value in Table 4.5 was calculated using the formula number 4.1. By Deere's (1964) classification the RQD which was obtained indicated competent ground. This result compared well with the RQD obtained in the 245ml drive. Other fracture data is presented in Table 4.5. Detailed mapping results are also shown in Table B17.

Table 4.4 Summary of point load results for the RL7 Argillaceous Quartzite. Results from borehole, BX 1740.

	No. of samples	Range of point load strength Is(50)	Mean point load strength Is(50)	Is(50) x22 (range) MPa	Is(50) x22 (mean) MPa	U.C.S* (range) MPa	U.C.S. # (mean) MPa	Standard deviation %
Point load Strength MPa	9	4.8-10.0	7.7	136-220	170.4	-	-	21
U.C.S* MPa	-	-	-	-	-	125-155	140	-

* U.C.S = Unconfined Compressive Strength obtained using compression machine.
Note: Values of Is(50) resulting from failure along a plane of weakness such as a joint plane are not included.

Table 4.5 Strata characteristics

Fracture data	Location: 300ml-North well
	Rock type: RL7 Argil. Quartzite

Number of discontinuities	12
Length of scanline	37m
Mean discontinuity	0.3/m
Mean population frequency	3.1
RQD%	96

4.5.2 Discontinuity concentration

A total of 29 joints including a shear plane were identified in the 300 metre level extraction haulage drive. Due to consistence in bedding orientation within the area of study, only five readings were taken. Plotted poles to the joint planes in Figure 4.5 revealed three major joint sets. Generally, the discontinuity distribution on 300ml is fairly uniform with most joints cutting across the entire width of the haulage drive. Results of the plots of the planes representing prominent joints in Figure 4.6 shows a tetrahedral wedge which is bounded by two joints (1 and 2) and the bedding plane. The fact that the wedge plots inside the friction circle indicates that the angle of the sliding surface of the wedge dips at an angle greater than the angle of friction. Hence, the likelihood of the wedge to slide from the tunnel roof. Table 4.6 shows joint sets and bedding from structural mapping on 300ml. Detailed structural data is also presented in table B1B in appendix B.

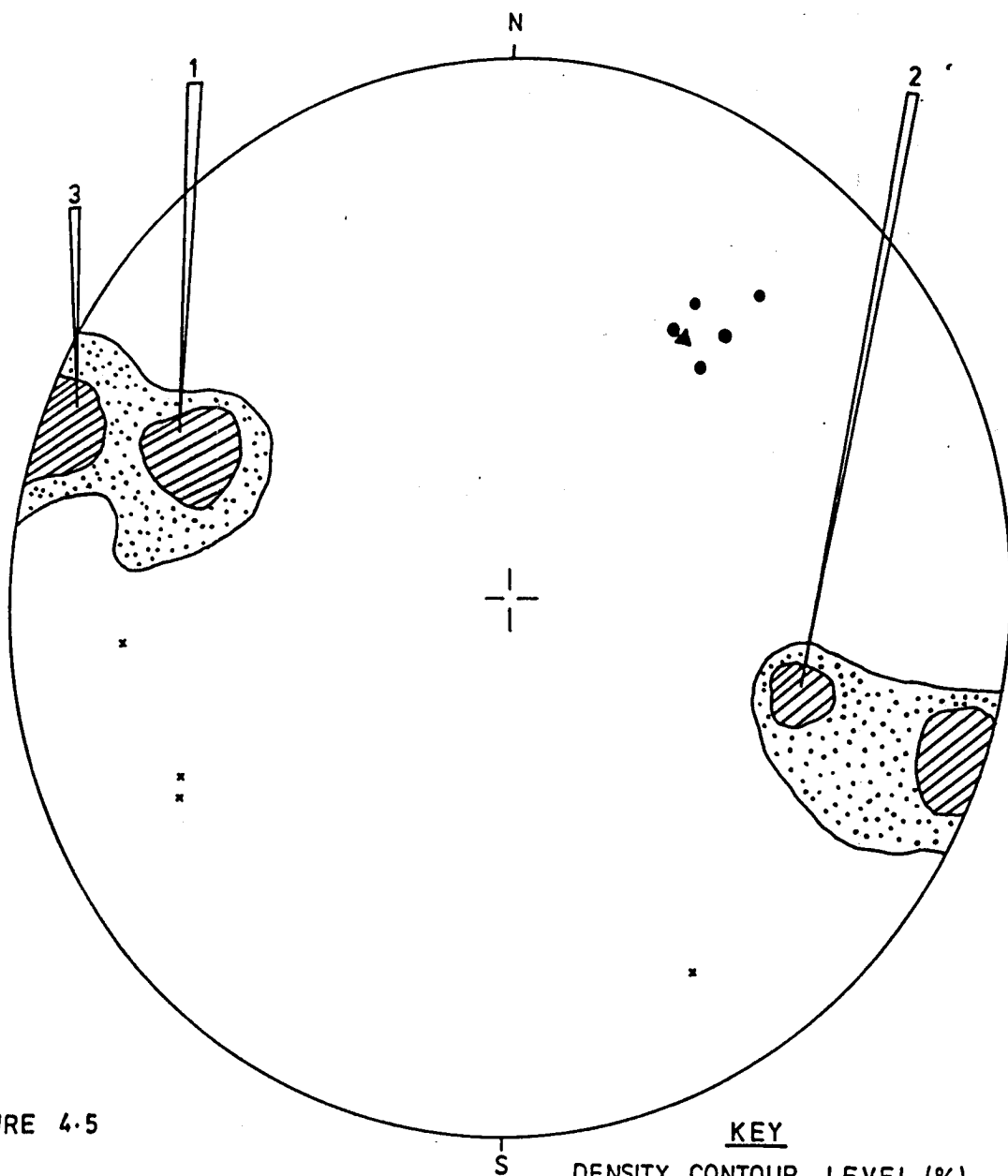


FIGURE 4.5

CONTOUR PLOT OF BEDDING
AND JOINTS

LOWER HEMISPHERE

EQUAL AREA PROJECTION

300m Level- Extraction haulage.

(Argillaceous quartzite)

Joints (25 readings)

Bedding

KEY

DENSITY CONTOUR LEVEL (%)

 > 8 < 12 %

 > 12%

▲ Shear plane

● Poles to bedding

x Random joints (poles)

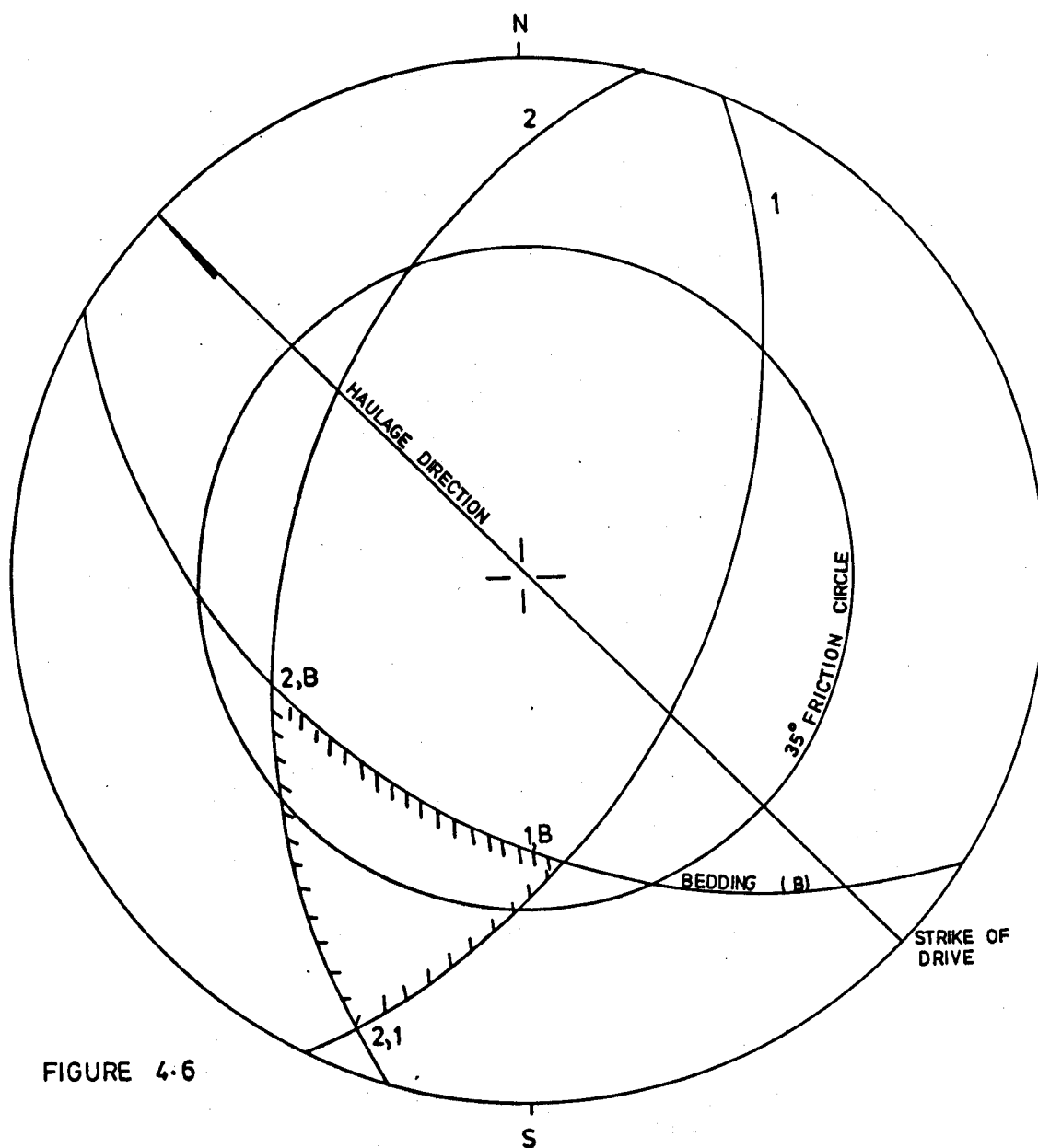


FIGURE 4-6

PLOTS OF PROMINENT JOINT
PLANES SHOWING LARGEST WEDGE
300m LEVEL-ARGILLACEOUS QUARTZITE

- | | | | |
|---|---|---|---------------|
| 1 | 2 | 3 | Joint planes |
| B | | | Bedding |
| | | △ | Largest wedge |

Table 4.6 Discontinuity set data

Joint set	Number of joint/bedding observed	Dip direction/ dip	Joint separation (average)	gouge
1	15	115/60	1-2mm	clay
2	5	285/50	1-2mm	clay
3	5	110/90	1-2mm	clay
shear plane	1	217/50	0.2m	clay
random joints	4	-	1-2mm	clay
bedding	5	215/50	-	-

4.5.3 Discontinuity condition

The discontinuity conditions described in section 4.4.3 are similar to those observed on the 300ml. Nearly all discontinuities observed in the drive contain clay with minor talcose material. Most of the joints are rough with slickensides on few. The 0.2m thick shear zone between MS 7399 and 7385 is generally weak and contains clay and talcose material infilling. The Schmidt Hammer test results obtained on some joint sets indicate compressive strength of 60MPa. Table 4.7 shows the results obtained on the 300ml.

Table 4.7 Schmidt hammer test results in RL7
Argillaceous Quartzite - 300ml

Station No.	Joint set	dip direction/ dip	Description of joint surface	Mean S.H rebound No.	S.H strength MPa
8	2	287/50	rough with 1-2mm joint separation clay infilled	61	83
11	3	110/90	rough, 1-2mm joint separation	49	65
12	1	115/60	rough, 1-2mm joint separation	55	73
18	random	83/70	rough, 1-2mm joint separation	57	83
23	random	332/70	rough, 1-3mm joint separation	46	60

4.6 480 haulage - Geotechnical parameters

4.6.1 Fracture survey

The main feature on 480m1 between Structural Sections (SS) 33 and 31 is that the drive is developed along a lithological contact between the RL7 Argillaceous and Aeolian Quartzites. The drive is 3.5m wide and 3.5m high. The Argillaceous and Aeolian Quartzites have two and four joint sets respectively. The RQD was calculated using Deere's formula given in section 4.4.1.

Table 4.8 Strata characteristics

		Location and lithology	

Fracture data	North wall:		South wall:
	Argillaceous		Aeolian
	Quartzite		Quartzite

Number of discontinuities	50		22
Length of scanline		125	125
Mean discontinuity		0.40/m	0.18/m
Mean population frequency	2.5		5.7
RQD %		97	99

In both Argillaceous and Aeolian Quartzites, the RQD is very high. This is an indication of competent ground ('excellent ground' by Deere's (1964) classification). A diamond drillhole, BX1751, drilled horizontally to the South of the tramming haulage (Figure B13) indicate an average RQD of 85 percent an indication of a slight deterioration of ground condition to the south.

4.6.2 Discontinuity concentration

A total of 72 discontinuities were measured between SS33 and SS31 section in the 480ml haulage drive. Of the 72, 50 were measured in the Aeolian Quartzite and the rest in the Argillaceous Quartzite. Due to consistence in bedding orientation only ten bedding planes were measured. The plotted poles to the joint planes in Figure 4.7 and 4.8 revealed two major sets in the Argillaceous Quartzite and four in the Aeolian Quartzite respectively. The Tables 4.9 and 4.10 show summaries of the values obtained during mapping. Figures 4.9 and 4.10 show tetrahedral wedges bounded by two joint sets and bedding in Argillaceous Quartzite and four joint sets in the Aeolian Quartzite. Detailed structural data is also presented in Table B19 and B20 in appendix B.

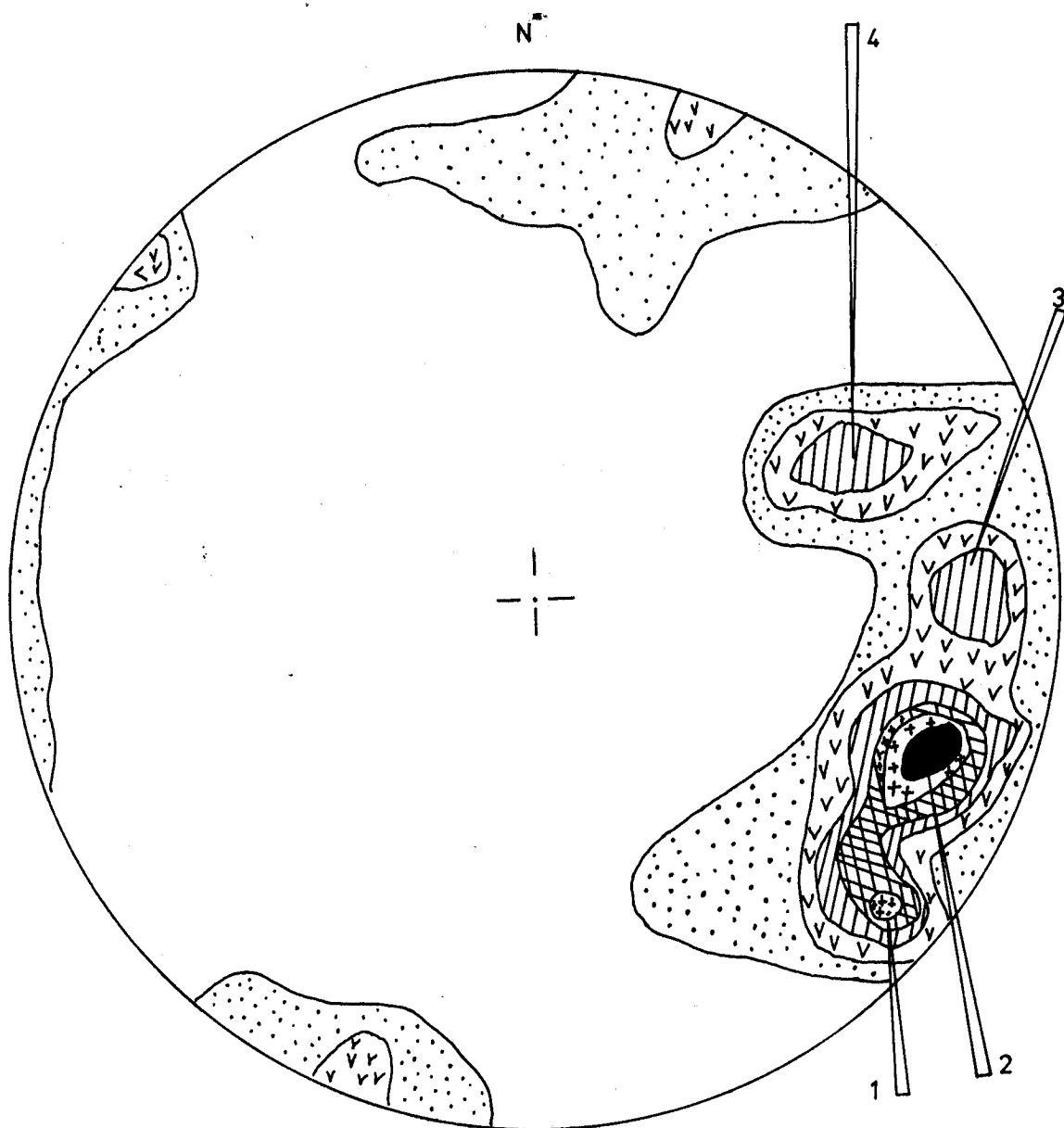


FIGURE 4-7

	KEY
CONTOUR PLOT OF JOINTS	DENSITY CONTOUR LEVELS(%)
LOWER HEMISPHERE	<2%
EQUAL AREA PROJECTION	>2 ≤ 4%
480m LEVEL TRAMMING	>4 ≤ 6%
HAULAGE EAST-	>6 ≤ 8%
AEOLIAN QUARTZITE	>8 ≤ 10%
(50 measurements)	>10 ≤ 12%
	>12%

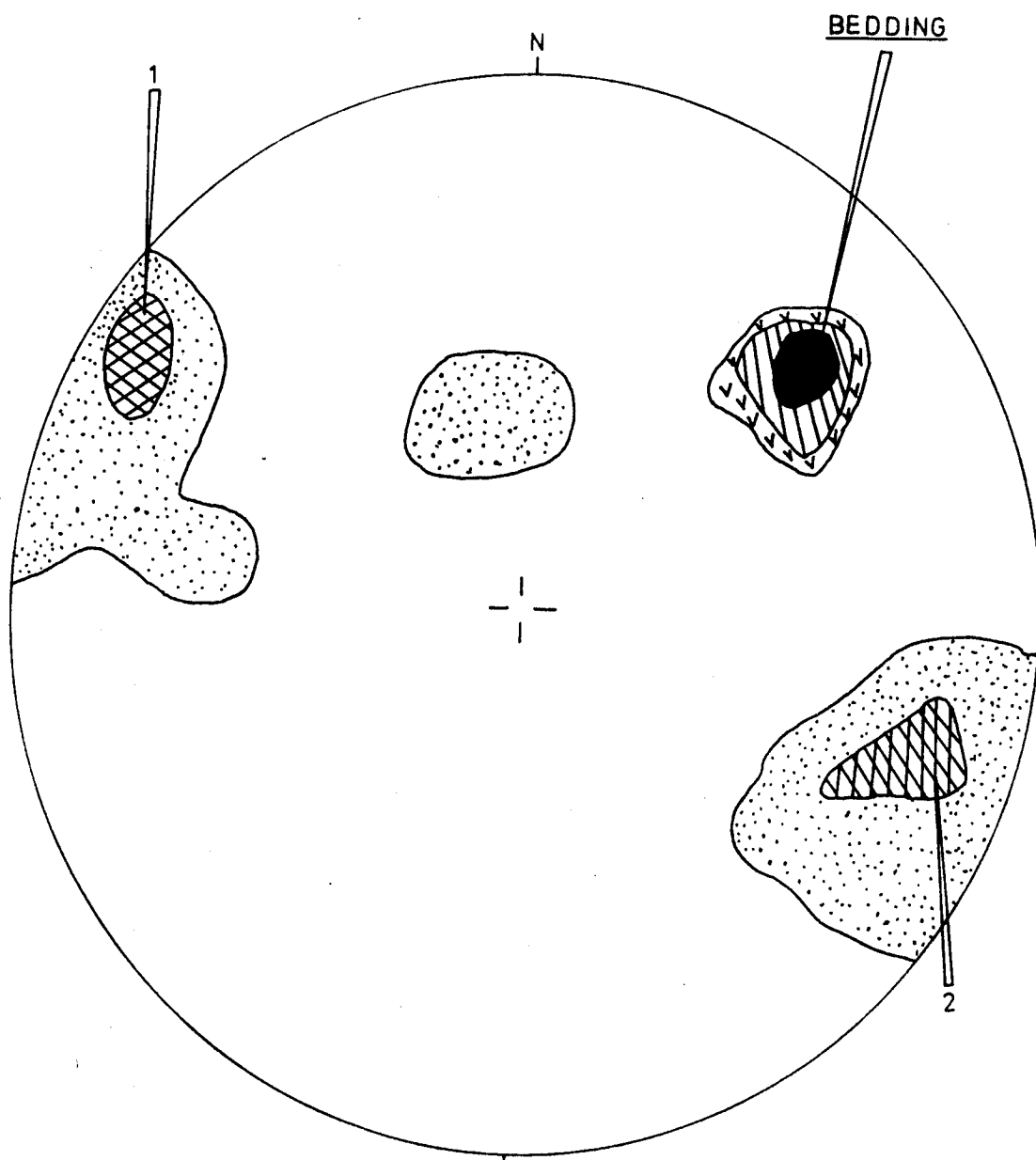
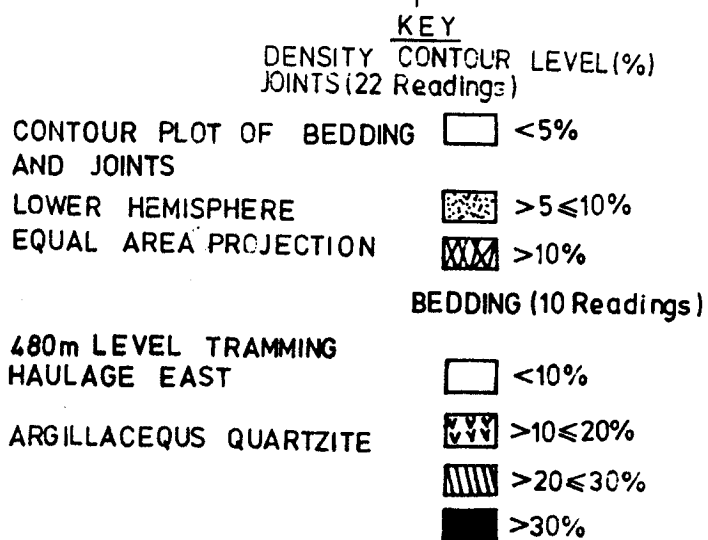


FIGURE. 4-8



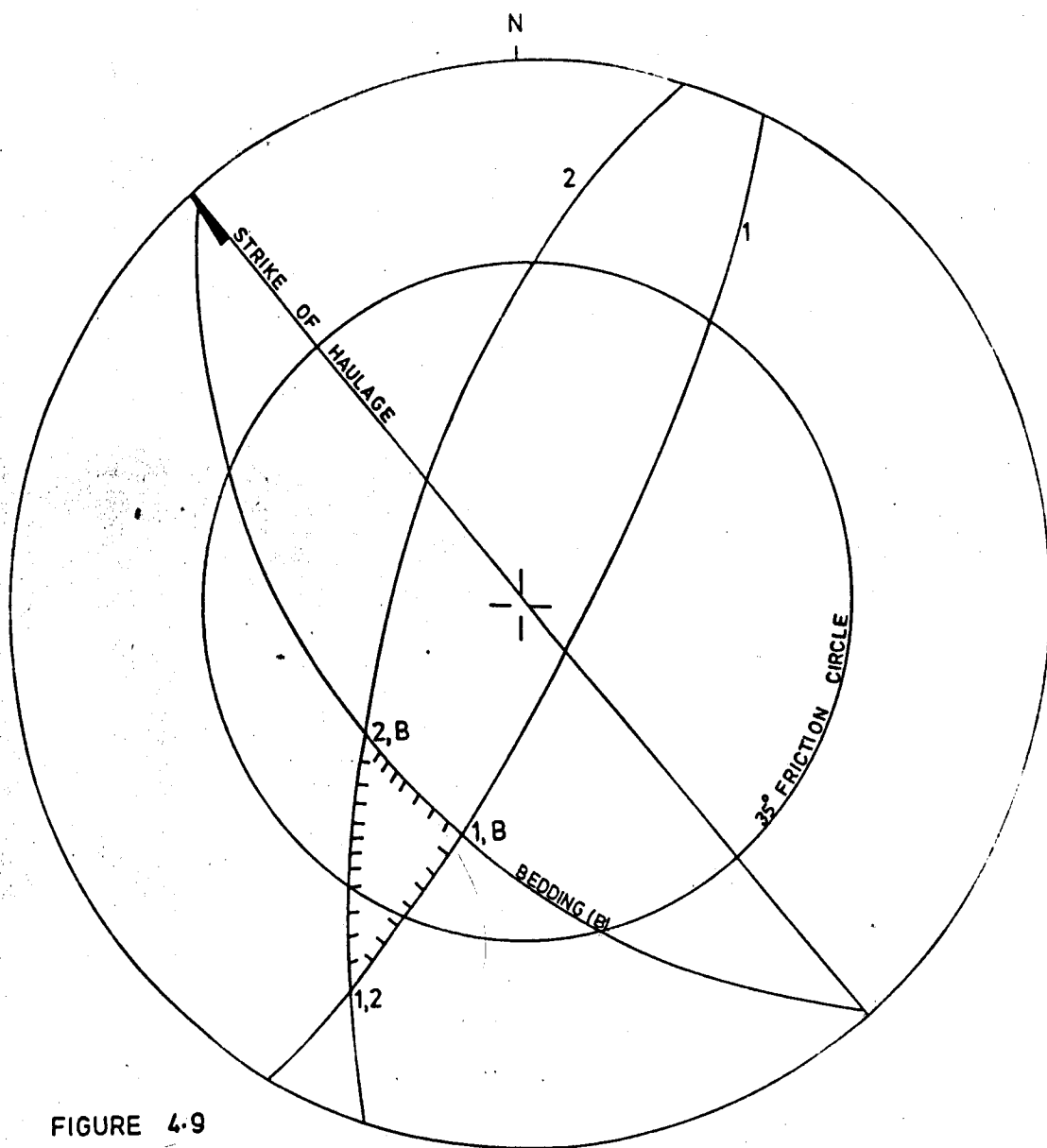


FIGURE 4.9

PLOTS OF PROMINENT JOINT
PLANES SHOWING LARGEST WEDGE
480m LEVEL-ARGILLACEOUS QUARTZITE

KEY

- 1, 2, 3 Joint plane
- B Bedding
-  Largest wedge

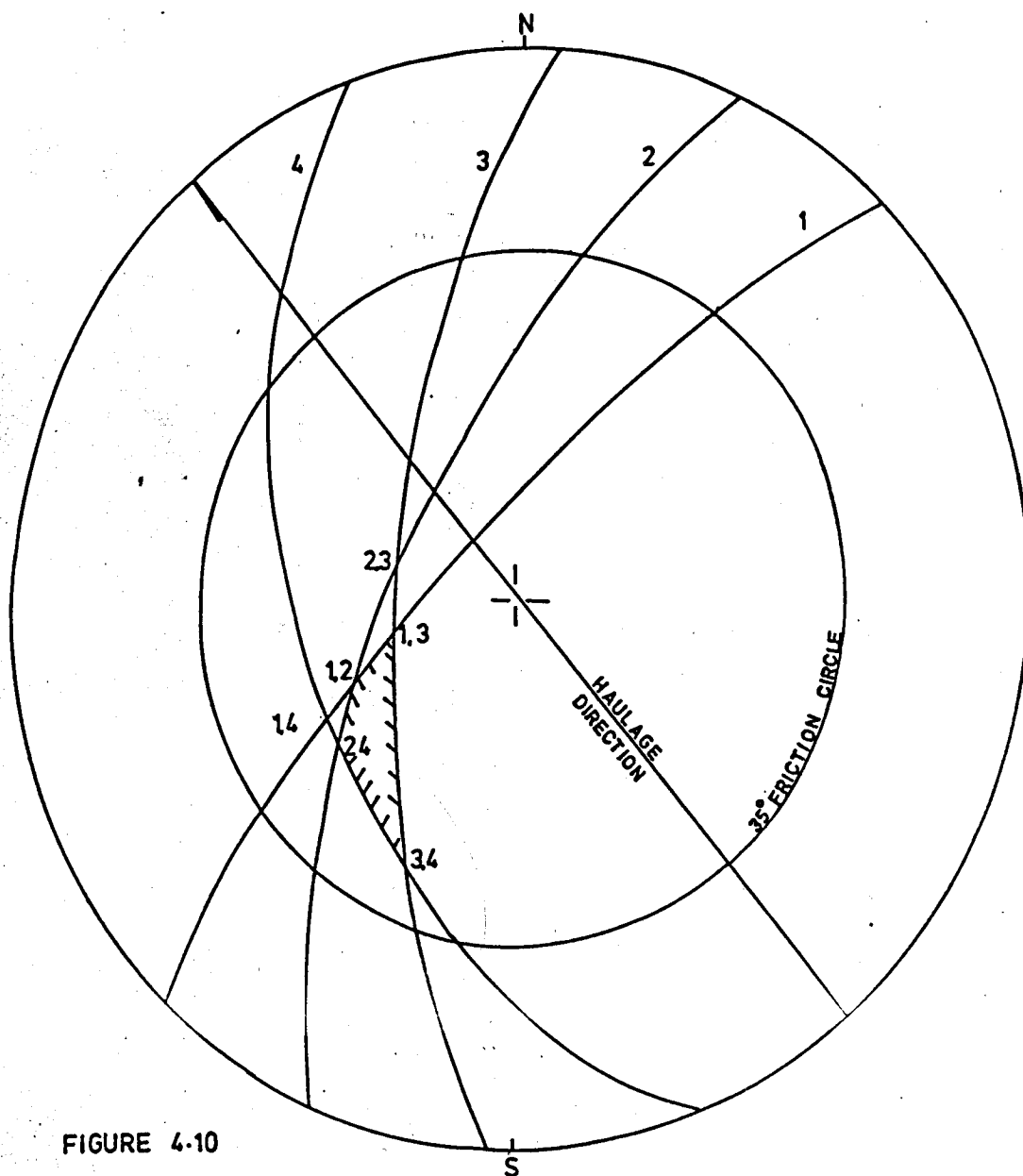


FIGURE 4-10

PLOTS OF PROMINENT JOINT
PLANE SHOWING LARGEST WEDGE
480m LEVEL-AEOLIAN QUARTZITE

KEY

1, 2, 3 Joint planes



Largest wedge

Table 4.9 Discontinuity set data in Argillaceous
Quartzite - 480ml

Joint set	No. of joints observed	Dip direction/ dip(degrees)	discontinuity separation	Gouge
1	8	120/80	1-2mm	clay+calc material
2	10	288/70	1-2mm	clay+calc material
Random joints	4	-	1-2mm	clay+calc material
Bedding	10	230/60	-	-

calc = calcareous.

Table 4.10 Discontinuity set data in Aeolian Quartzite

Joint set	No of joints observed	Dip direction/ dip	Discontinuity separation	Gouge
1	8	314/78	<1mm	none
2	11	294/70	<1mm	thin film of calc. material
3	11	273/70	<1mm	thin film of calc. material.
4	9	249/54	<1mm	thin film of calc. material
Random joints	11	-	<1mm	thin film of calc. material
Bedding	poorly bedded (none observed)			

4.6.3 Discontinuity condition

The majority of discontinuities in the RL7 Aeolian Quartzite are clean and smooth. Only in few cases are joints filled with very thin (less than 1mm) clay and calcareous material. None of those observed had slickensides. In the RL7 Argillaceous Quartzite, on the other hand, joints, in some cases, were slightly open (1-2mm) and filled with clay and calcareous material. Schmidt Hammer test results on a few joint sets in the Aeolian and Argillaceous Quartzites indicated joint compressive strengths greater than 83 MPa and 72 MPa respectively. Table 4.11 shows Schmidt Hammer test results obtained during mapping.

4.6.4 Intact rock strength

The point load intact strength of the RL7 Argillaceous and Aeolian Quartzites were determined from 11 core samples from each of the above mentioned lithologies. The borehole, BX1751 (47mm diameter core) provided samples for the Argillaceous Quartzite while re-drilled samples from the tramming haulage were used to obtain the average strength for the Aeolian Quartzite. Both lithologies yielded relatively high intact strengths compared to results obtained using unconfined compressive machine. The difference was

Table 4.11 Schmidt hammer test results from 480m1

Rock Type	Station	Joint set	Joint direction/ Dip (degrees)	Description of joint surface	Mean S.H. rebound No.	S.H. Strength MPa
Argillaceous Quartzite	No.S2	2	280/70	Rough with 2mm thick clay filling	59	83
	No.S5	2	308/55	Slickensided surface with less than 1mm clay filling	53	73
	No.S7	Random	180/30	Smooth to moderately rough	62	83
	No.S13	2	290/80	Rough with less than 1mm clay filling	56	83
	No.S14	1	120/75	Rough with less than 1mm clay filling	52	72
	No.S15	Random	250/90	Rough	56	83
Aeolian Quartzite	No.S6	1	313/60	Clean surface	68	83
	No.S10	2	290/80	Clean surface	66	83
	No.S16	4	245/65	Clean surface	68	83
	No.S17	3	270/80	Smooth surface with less than 1mm thick calcareous filling material	66	83
	No.S1		270/65	Clean and smooth surface	48	65
	No.S3	3	264/70	Clean and smooth surface	66	83
	No.S4	3	275/80	Clean and smooth surface	68	83
	No.S19	Random	336/65	Clean and smooth surface	60	83
	No.S20	Random	180/60	Clean and smooth surface	68	83

S.H = Schmidt Hammer

attributed to varied rock core condition at the time of tests, varied anisotropy conditions of the rocks and the limited number of samples tested. A summary of results is presented in Table 4.12. From Table 4.12 both rock types would be classified as very to extremely strong (classification based on Geological Engineering Society Working Party, 1970. Detailed point load results are given in Table B14, B15, and B16 in appendix B.

4.7 Comments on geotechnical parameters

From the foregoing outline of the geotechnical parameters obtained from mapping of the three levels, it is evident that no single parameter can describe fully and quantitatively rock mass behaviour. A combination of the parameters will, however, contribute to achievement of a complete quantitative assessment of rock mass condition. The results of the mapping on 245m, 300m and 480m levels also shows that roof behaviour depends upon structural anisotropy, and the system of discontinuities rather than on the rock material. The discontinuity orientation and concentration determines the size of wedges that are likely to form.

The AQD although forms part of the input parameter for the MBR, it was not easy to appreciate it when development walls and diamond drill core were

Table 4.12 Summary of intact rock strengths of Argillaceous and Aeolian Quartzites.

Rock Type	No. of samples	Point load strength		Point load strength		Is(50) x22		Is(50) x22		U.C.S.*		U.C.S.*		Standard deviation
		(range)	(mean)	(range)	(mean)	(range)	(mean)	(range)	(mean)	(range)	(mean)	(range)	(mean)	
		Is(50)	Is(50)			MPa	MPa	MPa	MPa	MPa	MPa	MPa	MPa	%
Point load strength MPa	Arg.	11	5.0-11.3	6.9	101.2-	151.4	-	-	-	-	-	-	-	25
	Quartzite				228.8									
U.C.S.* MPa	Aeolian	11	10.0-14.5	11.5	213.4-	253.9	-	-	-	-	-	-	-	16
	Quartzite				310.2									
U.C.S.* MPa	Arg.	-	-	-	-	-	-	-	-	125-155	140	-	-	-
	Quartzite													
	Aeolian	-	-	-	-	-	-	-	-	170-205	188	-	-	-
	Quartzite													

* U.C.S. = Uniaxial Compressive Strength obtained from compression machine.
 Arg. = Argillaceous

logged. The mean discontinuity concentration, on the other hand, appeared to have an advantage in this respect because it provided a more quantitative description of the rock fragment size. The fracture per metre was not only easy to visualize, but also easy to measure in the three drives.

The rock hardness values obtained using Schmidt Hammer type N2-4 and point load apparatus were very high in both Argillaceous and Aeolian Quartzites. Without considering the influence of discontinuities on roof stability, the strength parameter, on its own, is misleading. In the next chapter, the rock properties outlined in this section are combined into one overall classification rating for MBR rock mass system.

CHAPTER FIVE

5.0 APPLICATION OF MBR AND NGI SYSTEMS TO BALUBA HAULAGE DRIVES

5.1 Introduction

In this chapter the MBR system is applied to the 245m, 300m, and 480m level drives. The recommended support obtained is compared to the actual support installed in the three drives. The 'Q' system is also applied for comparison with the results procured from the MBR classification system. The geotechnical parameters that are used have already been discussed in chapter four. The description of how the two systems work has also been presented in chapter two. The MBR is applied to Baluba Mine for the first time. Its adequacy or inadequacy in terms of assessment of ground condition and support recommendation are discussed.

5.2 MBR classification system

Rock masses in the 245m, 300m and 480m haulage drives were classified according to the following parameters.

5.2.1 245m level drive

Table 5.1 MBR parameter rating - Argillaceous
Quartzite

Parameter	Description	Rating	Reference Figure/ Table No.
Intact rock strength	170 MPa - Point Load. (64 MPa - Schmidt Hammer)	13	Fig. A3
Discontinuity density:	RQD = 96%	19	Fig. A4
-spacing	joint spacing=3.1m	20	Fig. A4
Discontinuity condition:	rough (R)		Table A8
-wall separation	clay-slightly weathered	25	
-joint filling			
Groundwater	dry	15	Table A9
Total MBR		92	Fig. A6

Table S.2 Development adjustments

Parameter	Description	Rating	No. of joints	Ref. Table
Blasting damage (A_b)	Fair conventional blasting-moderate damage.	0.9	-	A12
Fracture orientation (A_o)	Joint set 1 113/64	0.9	8	A10
	Joint set 2 255/50	1.0	17	A10
	Joint set 3 294/64	1.0	14	A10
	Joint set 4 325/66	1.0	5	A10

The weighted fracture orientation (A_o) is calculated as follows:

$$A_o = \frac{(\text{No. of joints} \times \text{rating}) + (\text{No. of joints} \times \text{rating})}{\text{Total No. of joints measured}}$$

$$A_o = \frac{(8 \times 0.9) + (1 \times 17) + (1 \times 14) + (1 \times 5)}{8 + 17 + 14 + 5}$$

$$= 0.98$$

therefore the adjusted MBR computation is as follows:

$$A_o \times A_b = 0.88$$

$$MBR = 0.88 \times 0.92 = 81.1$$

Next, the production adjustments to take into account orebody extraction between SS28 and SS30 are computed;

Table S.3 Production adjustments

parameter	Description	Rating	Reference Figure/
Distance to Cave Line (DC)	Horizontal distance to the orebody. (orebody not yet caved)	0.99	Fig. A8
Block Panel size (PS)	The length of the haulage mapped within ore reserve block i.e l=150m	1.0	Fig. A9

$$\text{The final MBR rating} = 81.1 \times 0.99$$

$$= 80.3$$

According to the chart in Figure A10, the 245m haulage

drive West requires SPOT BOLTING to restrain local spalling or stop individual rock wedges from falling. Plate 1 shows typical ground conditions on 245m level.

S.2.2 300 metre level drive

Table S.4 MBR parameter rating - Argillaceous
Quartzite

Parameter	Description	Rating	Reference Table/Fig.
Intact rock strength	140 MPa	11	Fig. A3
Discontinuity density	RQD = 96%	19	Fig. A4
-spacing	Joint spacing = 3.1m	20	Fig. A4
Discontinuity condition	Rough (R)	25	Table A8
Wall condition	1-2mm		
Joint filling	Clay, slightly weathered		
Groundwater	Dry	15	Table A9
Total MBR		90	Fig. A6



PLATE 1

The weighted fracture orientation (A_0) is calculated as
The 245m level extraction haulage West showing sidewall
spalling caused by northern dipping beds. (3)

$$8 + 3 + 3$$

MBA development rating = 80.3

Support recommendation: Spot bolting

$$A_0 \times A_b = 0.87$$

NGI rating = 12.1

Support recommendation: Systematic untensioned rock

bolts spaced at 1.5m - 2m centers.

Table S.5 Development adjustments

Parameter	Description	Rating	No. of Joints	Ref. Table.
Blasting damage (A_b)	Fair conventional blasting-moderate damage	0.9	-	A12
Fracture orientation (A_o)	Joint set 1 115/60 Joint set 2 285/50 Joint set 3 110/90	1.0 0.9 1.0	5 3 3	A10 A10 A10

The weighted fracture orientation (A_o) is calculated as follows;

$$A_o = \frac{(1 \times 5) + (0.9 \times 3) + (1 \times 3)}{5 + 3 + 3}$$

$$= 0.97$$

Therefore the adjusted MBR computation is as follows;

$$A_o \times A_b = 0.87$$

$$MBR = 0.87 \times 0.90$$

$$= 78.6$$

from figure A6 development support recommendation is given as follows;

Wide pattern bolting with spacing of 1.5 to 1.8 metres

Production adjustments are computed as follows;

Table 5.6 Production adjustments

Parameter	Description	Rating	Reference Figure
Distance to Cave-line (DC)	Orebody is approx. 13m from the centre of the haulage	0.88	Fig. A8
Block Panel size	Maximum void to be opened when production commences is 37m x 6m.	1.05	Fig. A9

The final MBR rating is therefore:-

$$= 78.6 \times 0.88 \times 1.05$$

$$= 72.6$$

According to the chart in Figure A10, the production support recommendation is as follows;

Spot bolting to restrain individual blocks of loose rock from falling. Plate 2 shows typical ground condition in the 300m level haulage drive.



PLATE 2

The 300m level extraction haulage East (South wall haulage/crosscut intersection) showing side wall wedging.

MBA development adjusted rating = 78.6

Support recommendation: Wide pattern bolting spaced at 1.5m ~ 1.8m centers.

5.2.3 480 metre level drive

Table 5.7 MBR parameter rating - Argillaceous
Quartzite

parameter	Description	Rating	Reference Table/Fig.
Intact rock strength	151 MPa (point-load)	12	Fig. A3
Discontinuity density	RQD = 99.0%	19	Fig. A4
Spacing	Joint spacing=5.7m	20	Fig. A4
Discontinuity condition	Rough (A)	25	Table A8
-wall separation	1-2mm		
-joint filling	Clay filling		
Groundwater	Dry	15	Table A9
Total MBR		91	Fig. A6

Table 5.8 Development adjustments

Parameter	Description	Adjust- ment	No. of joints	Ref. Table
Blasting damage (A_b)	Fair conventional blasting- moderate damage	0.90	-	A12
Fracture spacing (A_o)	Joint set 1 120/80 Joint set 2 288/70	1.00 0.90	8 10	A10 A10

The weighted fracture orientation (A_o) in Argillaceous Quartzite formation is calculated as follows;

$$A_o = \frac{(0.9 \times 10) + (1.0 \times 8)}{10 + 8}$$

$$= 0.94$$

The final adjusted MBR is therefore computed as follows;

$$A_o \times A_b = 0.85$$

$$\text{MBR} = 0.85 \times 91$$

$$= 77.4$$

From the support chart in Figure A10, it is readily seen that the southern side of the 480ml haulage drive which is in Argillaceous Quartzite would require wide pattern

bolts spaced at 1.5m to 1.8m centers.

Table 5.9 MBR determination for the 480m level-Aeolian
Quartzite

Parameter	Description	Rating	Reference Fig./Table
Intact rock strength	253 MPa (point-load)	15	Fig. A3
Discontinuity density	RQD = 97.4%	20	Fig. A4
Spacing rating	joint spacing=2.5m	20	Fig. A4
Discontinuity condition	Smooth to slightly rough	20	Table A8
-wall separation	Less than 1mm		
-joint filling	None		
Groundwater	Dry	15	Table A9
Adjusted MBR		90	Fig. A6

Table S.10 Development adjustments

Parameter	Description	Rating	No.of joints	Reference Table
Blasting damage (A_b)	Fair conventional blasting-moderate damage	0.90	-	A12
Fracture orientation (A_o)	joint set 1 314/78 joint set 2 294/70 joint set 3 273/70 joint set 4 249/54	0.90 0.90 0.90 0.90	8 11 11 9	A10 A10 A10 A10

The weighted fracture orientation (A_o) in RL7 Aeolian Quartzite formation is calculated as follows:

$$A_o = \frac{(0.90 \times 8) + (0.90 \times 11) + (0.90 \times 9) + (0.90 \times 11)}{8 + 11 + 9 + 11}$$

Therefore the final adjusted MBR is computed as follows:

$$A_o \times A_b = 0.81 \quad \text{where } A_o = 0.90, A_b = 0.90$$

$$\text{MBR} = 0.81 \times 90$$

$$= 72.9$$

It must be noted that the extraction ratio adjustment rating did not apply to this case although there are two parallel drives which are being developed. The reason

is that the conversion adopted, according to Kendorski, (1983), is that only openings within 1.5 drift diameters of each rib are considered in computing the extraction ratio. The distance between the parallel haulages on 480m level is 14 metres measured sidewall to sidewall. The above consideration means that the stability of one haulage does not influence the other for a large drive separation.

The development support recommendation for the northern side of the 480m level haulage as given in Figure A10 is wide pattern bolting with bolt spacing of 1.5m to 1.8m. Plate 3 shows typical condition of the AL7 Aeolian Quartzite.

5.3 NGI CLASSIFICATION SYSTEM

5.3.1 245 metre level drive

The information presented in this section was collected for the purpose of satisfying the NGI input requirements. The indices obtained provided relative ratings of the rocks in which the 245, 300, and 480 metre levels are developed.



PLATE 3

The 480m level tramming haulage east (northern wall- Aeolian Quartzite) showing smooth joint planes.

MBA development rating = 72.9 joint walls 1.0 Table A5

Support recommendation: Wide pattern bolting with bolt number (Jo) spacing at 1.5m - 2m centers.

NGI rating = 16.2

Support recommendation: Systematic untensioned rock bolting with bolt spacing at 1.5m -2m centers.

Table 5.11 NGI parameter rating for 245m level drive

Parameter	Description	Value	Reference Table/Fig.
RQD	100% core recovery generally intact with few natural breaks	97%	Table A5
Joint set number (Jn)	Three main joint sets plus minor and random joints.	12	Table A5
Joint roughness number (Jr)	Generally rough and irregular	1.5	Table A5
Joint alteration number (Ja)	Unaltered joint walls with surface red clay stains.	1.0	Table A5

Table S.11 continued.....from page 75

Joint water	dry	1.0	Table A5
factor (Jw)			

Stress	compressive strength/	1.0	
reduction	tensile strength		Table A5
factor (SRF)	$6c/6t = 140/170=0.82$		

Hence, the tunnel quality index is as follows:

$$\begin{aligned}
 Q &= RQD/J_n \times J_r/J_a \times J_w/SRF \\
 &= 97/12 \times 1.5/1 \times 1/1 \\
 &= 12.1 \text{ (good ground)}
 \end{aligned}$$

Support estimate computation is as follows:

245m level drive span = 3.5m

Excavation support ratio (ESR) = 1.6 (refer to Table A5)

$$\text{span/ESR} = 2.2 \text{ m}$$

Therefore, for $Q = 12.2$ the support category is 13

According to Table A1, category 13 recommends the following PERMANENT SUPPORT:

Systematic untensioned rock bolts, spaced at
1.5m to 2m centers.

TEMPORARY SUPPORT: none is required.

Maximum unsupported span is calculated as follows:

$$\begin{aligned}\text{Span} &= 2 \times \text{ESR} \times Q^{0.4} \\ &= 8.7\text{m}\end{aligned}$$

The Q rating of 12.1 classifies the Argillaceous Quartzite in the 245m drive as "Good rock". Plate 1. shows typical rock condition in the drive.

5.3.2 300 metre level drive

Table 5.12 NGI parameter rating for 300m level drive

Parameter	Description	Value	Reference Table/Fig.
RQD	100% core recovery good intact rock.	97%	Table A5
Joint set number (Jn)	three joint sets plus random ones.	12	Table A5
Joint roughness (Jr)	Generally rough and irregular	1.5	Table A5
Joint alteration number (Ja)	Unaltered joint walls. Only red clay staining was observed on joint surface	1.0	Table A5
Joint water factor (Jw)	dry	1.0	Table A5
Stress reduction	compressive strength/ tensile strength	1.0	Table A5

$$\text{factor (SRF)} \quad 6c/6t = 140/170 = 0.82$$

Hence, the tunnelling quality index is:

$$\begin{aligned} Q &= 96/12 \times 1.5/1 \times 1/1 \\ &= 12 \quad (\text{Good ground}) \end{aligned}$$

Support estimate computation is as follows:

$$300 \text{ metre level drive span} = 3.5\text{m}$$

$$\text{ESR} = 1.6$$

$$\text{span/ESR} = 2.2\text{m}$$

For $Q = 12$, support category is 13 and according to

Table A1, the recommended PERMANENT SUPPORT is

SYSTEMATIC BOLTING - untensioned with bolt spacing 1.5m

to 2.0m. The maximum unsupported span is calculated as

follows:

$$\begin{aligned} \text{span} &= 2 \times \text{ESR} \times Q^{0.4} \\ &= 8.6\text{m} \end{aligned}$$

5.3.3 480 metre level drive

Table 5.13 NGI parameter rating for the 480 metre
level drive - Argillaceous Quartzite

Parameter	Description	Value	Reference Table/Fig.
RQD	Excellent ground (Deere, 1964)	99%	Table A5
Joint set number (Jn)	2 joint sets plus random joints	6	Table A5
Joint roughness number (Jr)	Generally rough	1.5	Table A5
Joint alteration number (Ja)	Unaltered joint walls Surface red clay staining only	1.0	Table A5
Joint water factor (Jw)	Dry	1.0	Table A5

Table S.13 Continued from page 80

Stress	Compressive strength/	1.0	Table A5
reduction	Tensile strength		
factor	$6c/6t = 140/151$		
(SRF)	$= 1.13$		

Hence, the Tunnelling Quality index is:-

$$Q = 99/6 \times 1.5/1 \times 1/1$$

$$= 24.8 \quad (\text{good ground})$$

Support estimate is computed as follows:-

480 metre level drive span = 3.5m

$$ESR = 1.6$$

$$\text{span}/ESR = 2.2\text{m}$$

For $Q = 24.8$ the support category is given as 13 and according to Table A1, the recommended PERMANENT support is SPOT BOLTING.

Maximum unsupported span is calculated as follows:

$$2 \cdot ESR \cdot Q^{0.4} = 11.5\text{m}$$

Table 5.14 NGI parameter rating for 480 metre level-
Aeolian Quartzite

Parameter	Description	Value	Reference Table/Fig.
RQD	Excellent ground (deere, 1964)	97.4%	Table A5
Joint set number (Jn)	4 joints (2 major and 2 minor plus random ones)	6	Table A5
Joint roughness number (Jr)	Generally smooth joints	1.0	Table A5
Joint alteration number (Ja)	Unaltered joint walls. Only surface staining.	1.0	Table A5
Joint water factor (Jw)	dry	1.0	Table A5
Stress reduction factor (SRF)	Compressive strength/ Tensile strength $6c/6t = 187.5/254 = 0.74$	1.0	Table A5

Hence, the value of $Q = 97.4/6 \times 1/1 \times 1/1$

$$= 16.2 \quad (\text{good ground})$$

For $Q = 16.2$, the support category according to Table A1 is 13. The PERMANENT support for this value of "Q" is as follows; systematic untensioned bolts, 1.5 to 2m bolt spacing.

The maximum unsupported span is calculated as follows;

$$2 \times ESR \times Q^{0.4} = 9.7m$$

Plate 3 shows typical Aeolian Quartzite condition in the 480m tramming drive East.

5.4 COMMENTS ON THE RESULTS

From the summary presented in Table 5.15, the following comments are advanced regarding the applicability of the MBA and NGI rock mass systems in the local conditions prevailing at Baluba mine.

In the 245 metre level drive the MBA recommends spot bolting during development and further spot bolting when the block commences production. The NGI system, on the other hand, prescribes untensioned bolts with ring spacing of 1.5 to 1.8 metres. The maximum unsupported span is given as 8.7 metres. The prediction based on the NGI system, compared favourably with the support actually installed although systematic roof bolting seems to be the standard practice at Baluba mine irrespective of the ground conditions prevailing.

The NGI system does not recommend support when the block comes into production. The requirements during development is final. In contrast, the MBA system is initially conservative, but it compensates this conservativeness by further recommending spot bolting when the block comes into production. Production support recommendation by MBA is meant to take into account stress re-distribution around excavations during and after mining.

In the 300 metre level drive the support

Table S.15 Summary of support recommendations for all levels.

Level	Rock Type	Rock mass classification		Actual Support Installed	
		NGI-Recommendations	MBA Recommendations		
		Development Support		Production Support	
		Spot bolting	Spot bolting	Spot bolting	Systematic bolting
245ml	Argillaceous Quartzite	Systematic rock bolting-unensioned with bolt spacing 1.5m to 2.0m			(grouted wire dowels -Ring spacing=1.5m (max) Bolt length: 2.4m (min)
300ml	Argillaceous Quartzite	Systematic rock bolting-unensioned with bolt spacing 1.5m to 2.0m	Wide pattern bolting with bolt spacing of 1.5m to 1.8m	Spot bolting	Systematic bolting Ring spacing:1.5m (max) Bolt length: 2.4m (min)
480ml	Argillaceous Quartzites	Spot bolting	Wide pattern bolting with bolt spacing of 1.5m to 1.9m	Not required	Systematic bolting Ring spacing:1.5m (max) Bolt length: 2.4(min)
480ml	Aeolian Quartzite	Systematic unensioned rock bolting with bolt spacing if 1.5m to 2.0m	Wide pattern bolting with bolt spacing of 1.5m to 1.8m	Not required	Systematic bolting Ring spacing:1.5m (max) Bolt length: 2.4m(min)

Note: Bolts in spot bolting through close pattern bolting as given in figure, are considered to be 19mm in diameter, fully grouted or resin-anchored standard rockbolts; mechanical anchors are acceptable in material of MBA greater than 60. Split-set installation is at the discretion of the operator.

recommendation by the MBR and the NGI tie in favourably with the support pattern that has been installed. The grouted wire dowels installed in the drive have ring spacing of 1.5 metres and each dowel is spaced at 1 metre centers. The MBR, however, goes further to recommend spot bolting when production on this level commences.

In the southern part of the 480 metre level drive (in Argillaceous Quartzite), the correlation with the presently installed bolt pattern was better for the MBR than the NGI. The support recommendation for the northern part of the drive (in Aeolian Quartzite) also compared favourably with the support installed. The NGI, on the other hand, recommends two different support patterns for the northern and southern parts of the drive (refer to Table S.15). This difference perhaps highlights the sensitivity of the NGI with regards to isolated drives.

CHAPTER SIX

6.0 CONCLUSIONS AND RECOMMENDATIONS.

6.1 Comments and conclusions

From the five chapters presented, the following comments and conclusions are made regarding the applicability of the MBR classification system in comparison to the NGI system on the evidence from the field studies.

The MBR system essentially uses four categories of parameters, namely, strength of intact rock, discontinuity density (RQD and discontinuity spacing), discontinuity and groundwater conditions. It is upon these parameters that the system bases its initial support recommendations. These four basic geological parameters are easy to measure and did not pose any problems during the mapping exercise. However, the mean spacing appeared to have an advantage over RQD because it provided a more quantitative description of the rock fragment size. The fracture per metre was not only easy to visualize but also easy to measure.

Amongst the three first stage parameter adjustments; development adjustment, blasting and induced stress

effects were difficult to assess than fracture orientation. Blasting damage adjustment proved difficult to quantify. The use of qualitative terms like "slightly or moderately damaged" on which the adjustment is based is a matter of personal opinion, and can differ depending on the person making the assessment. In the 480m drive for instance, blasting damage in both the Argillaceous and Aeolian Quartzites is described as moderate even though drives differ in their strata characteristics and intact rock strengths (refer to Table 5.8 and 5.10). This was difficult to avoid because there were no other definitive characteristics on which distinction could have been based upon.

The NGI system basically uses similar parameters as the MBR except that they are expressed and used in different ways to quantify the rock mass. The NGI does not directly use strength of intact rock and orientation of structural features. Both systems, however, use stress, but in different ways. The MBR uses stress as an adjustment whereas the NGI uses it as a direct input parameter. Both systems include RQD, joint spacing and condition and groundwater condition. The fact that the NGI does not include adjustments means that the system was designed only to cater for initial stabilization of single isolated tunnels. The MBR system, on the other

single isolated tunnels. The MBR system, on the other hand, takes into account deterioration of ground which would result if a nearby orebody was mined or an adjacent excavation was developed. This consideration is taken into account by applying adjustments to the initial MBR rating. The MBR also recognizes the need for further support (permanent) after the initial stabilization of the drive.

Despite some limitations in the application of some input parameters, it is concluded that the MBR classification system has an advantage over the NGI system in one major respect, that of its applicability to mining. From the support recommendations in Figure 5.15 it is observed that the MBR takes into account effects of production on stability of development and hence prescribes additional support in form of spot bolting for the 245m and 300m drives. The NGI on the other hand does not take into consideration the effects of caved orebodies, blasting and size of block or panel to be mined. Generally, for single tunnels both classifications give reasonably similar predictions as it can be seen from the 300m and 480m drives support predictions (Table 5.15). Both classification systems clearly define their parameters and these are measurable in the field. The MBR goes further by actually specifying input data sheets for the minimum geological

These data sheets were found very useful and are reproduced in Table B21 and B22 in Appendix B.

6.2 Recommendations

Having assessed the ground conditions in the 245m, 300m, and 480m drives at Baluba Mine using the MBR and NGI classification systems, the following recommendations are made regarding the adoption of the MBR system.

The MBR can be adopted as long as the following recommendations are considered.

1. It is recommended that blasting damage adjustment be quantified. This can be done by undertaking a small pre-production blast vibration effect survey in all the Baluba rock formations. The degree of blast damage can then be determined by the number of new fractures and extent of fractured zones formed and the behaviour of other natural formed geological structures such as bedding, shear zones and joints. The new fractures and their extent around excavations can also be determined by drilling cored holes into the roof and sides of drives. A measure such as a Modified Fracture Frequency (MFF) that will include both blast induced and natural fractures can be used as the basis for reducing the

initial fracture rating.

2. Rock falls are often the cause of injuries and fatalities in Zambian Mines. The most favourable condition for rock falls is the presence of three or more intersecting discontinuity sets. Generally the problem is initiated by instability of individual blocks, roughly polygonal, defined by intersecting joints or joints with bedding planes in an exposed drive roof or walls. In this respect, to further enhance the data base generated by the MBR classification system, it is recommended to introduce a complete hazard analysis of drives. The procedure would comprise the compilation of level maps and sections on which potentially weak or hazardous areas would be shown. In addition, discontinuity stereograms revealing the blocks likely to fall-out or slide-out would be plotted. With such information the use of bolting can be planned and installation carried-out effectively rather than waste support resources on competent areas.

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APPENDIX A

Table A1 Support measures for rock masses giving "exceptionally", "extremely good", "very good", and "good" tunnel stability. (Q range: 1000-10)

SUPPORT CATEGORY	Q	CONDITONAL FACTORS			P Kg/cm2 (approx)	SPAN/ESR (m)	TYPE OF SUPPORT	NOTE
		RQD/Jn/Jr/Ja		SPAN/ESR (m)				
1*	1000-400	-	-	-	<0.01	20-40	sb(utq)	-
2*	1000-40	-	-	-	<0.01	30-60	sb(utq)	-
3*	1000-40	-	-	-	<0.01	46-80	sb(utq)	-
4*	1000-40	-	-	-	<0.01	65-100	sb(utq)	-
5*	400-100	-	-	-	0.05	12-30	sb(utq)	-
6*	400-100	-	-	-	0.05	19-45	sb(utq)	-
7*	400-100	-	-	-	0.05	30-65	sb(utq)	-
8*	400-100	-	-	-	0.05	48-88	sb(utq)	-
9	100-40	>20 <20	- -	- -	0.25	8.5-19	sb(utq) B (itq) 2.5-3m	- -
10	100-40	>30 <30	- -	- -	0.25	14-30	B (utq) 2-3m B (utq) 1.5-2m	- -
11*	100-40	>30 <30	- -	- -	0.25	23-48	B (tq) 2-3m B (tq) 1.5-2m +clm	- - -
12*	100-40	>30 <30	- -	- -	0.25	40-72	B (tq) 2-3m B (tq) 1.5-2m +clm	- - -
13	40-10	>10 >10 <10 <10	>1.5 <1.5 >1.5 <1.5	- - - -	0.5	5-14	sb(utq) B (utq) 1.5-2m B (utq) 1.5-2m B (utq) 1.5-2m +S 2-3 cm	I I I I
14	40-10	>10 <10 -	- - -	>15 >15 <15	0.5	9-23	B (tq) 1.5-2m +clm B (tq) 1.5-2m +S(mr) 5-10 cm B (utq) 1.5-2m +clm	I, II I, II I, III
15	40-10	>10 <10	- -	- -	0.5	15-40	B (tq) 1.5-2 m +clm B (tq) 1.5-2 m +S(mr) 5-10 cm	I, II, IV I, II, IV
16* see note XII	40-10	>15 <15	- -	- -	0.5	30-65	B (tq) 1.5-2 m +clm B (tq) 1.5-2 m +S(mr) 10-15cm	I, II, IV I, II, IV

Continued..... from page 97

- * Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

The type of support to be used in category 1 to 8 will depend on the blasting technique. Smooth wall blasting and thorough barring-down may remove the need for support. Rough-wall blasting may result in the need for single applications of shotcrete, especially where the excavation height is > 25 m. Future case records should differentiate categories 1 to 8.

KEY TO SUPPORT TABLES

sb = spot bolting
B = systematic bolting
(utq) = untensined, grouted
for competent rock masses, grouted
post-tensined in very poor quality
rock masses (see Note XI)
S = shotcrete
(mr) = mesh reinforced
clm = chain link mesh
(sr) = steel reinforced
CCA = cast concrete arch

Bolt spacings are given in metres (m)
Shotcrete, or cast concrete arch thickness
is given in centimetres (cm).

Table A2 Support measures for rock masses giving "fair" and "poor" tunnel stability. (Q range: 10-1).

Support CATEGORY	CONDITIONAL FACTORS				P		Type of support	Note
	Q	RQD/Jn	Jr/Ja	SPAN/BSR (m)	Kq/cm2 (approx)	SPAN/BSR (m)		
17	10-4	>30	-	-	1.0	3.5-9	sb(utq)	I
		>10,<30	-	-			B (utq) 1-1.5 m	I
		<10	-	>6 m			B (utq) 1-1.5 m	I
							+ S 2-3 cm	
		<10	-	<6 m			S 2-3 cm	I
18	10-4	>5	-	>10 m	1.0	7-15	B (tq) 1-1.5 m	I, III
							+clm	
		<5	-	<10 m			B (utq) 1-1.5 m	I
							+clm	
		<5	-	>10 m			B (tq) 1-1.5 m	I, III
19	10-4	-	-	>20 m	1.0	12-29	B (tq) 1-2 m	I, II, IV
							+ S (mr) 10-15cm	
		-	-	<20			B (tq) 1-1.5 m	I, II
							+ S (mr) 5-10 cm	
20* See note XII	10-1	-	-	>35	1.0	24-52	B (tq) 1-2 m	I, V, VI
							+ S (mr) 20-25cm	
				<35			B (tq) 1-2 m	I, II, IV
21	4-1	>12.5	<0.75	-	1.5	2.1-6.5	B (utq) 1 m	I
							+ 2-3 cm	
		<12.5	<0.75	-			S 2.5-5 cm	I
			<0.75	-			B (utq) 1 m	I
22	4-1	>10,<30	>1.0	-	1.5	4.5-11.5	B (utq) 1 m	I
							+ clm	
		<10	>1.0	-			S 2.5-7.5 cm	I
		<30	<1.0	-			B (utq) 1 m	I
							+ S (mr) 2.5-5cm	
23	4-1	-	-	>15m	1.5	8-24	B(tq) 1-1.5m	I,II,IV,VII
		-	-	<15m			+S(mr)10-15cm	
							B(utq)1-1.5m	I
							+S(mr)5-10cm	
24* note XII	4-1	-	-	>30m	1.5	18-46	B(tq)1-1.5m	I, V, VI
		-	-	<30m			+S(mr)15-30cm	
							B(tq)1-1.5m	I, II,IV
							+S(mr)10-15cm	

* Authors estimates of support. Insufficient case records available fo reliable estimation of requirements. (reproduced from Burton et. al., 1974)

Table A3 Support measures for rock masses giving "very poor" tunnel stability. (Q range: 1.0-0)

SUPPORT CATEGORY	Q	CONDITIONAL FACTORS			P Kq/cm (approx.)	SPAN/ESR (m)	TYPE OF SUPPORT	NOTE
		RQD/Jn/Jr/Ja	SPAN/ESR (m)					
25	1.0-0.4	>10	>0.5	-	2.25	1.5-4.2	B (utq) 1 m	I
							+ mr or cla	
		<10	<0.5	-			B (utq) 1m	I
			<0.5	-			+S (mr) 5 cm	
26	0.1-0.01	-	-	-	2.25	3.2-7.5	B (tq) 1 m	VIII, X, XI
							+S(mr) 5-7.5 cm	
							B (utq) 1 m	I, IX
							+ 2.5-5 cm	
27	1.0-0.4	-	-	>12 m	2.25	6-18	B (tq) 1 m	I, IX
							+S(mr) 7.5-10 cm	
		-	-	<12			B (utq) 1 m	I, IX
							+S(mr) 5-7.5 cm	
		-	-	>12			CCA 20-40 cm	VIII, X, XI
							+B (tq) 1m	
		-	-	<12			S (mr) 10-20 cm	VIII, X, XI
							+B (tq) 1 m	
28* see note XII	1.0-0.4	-	-	<30	2.25	15-38	B (tq) 1 m	I, IV, V, IX
							+S(mr) 10-40 cm	
		-	-	>20, <30			B (tq) 1 m	I, II, IV, IX
							+S(mr) 20-30 cm	
				<20			B (tq) 1 m	I, II, IX
							+S(mr) 15-20 cm	
29*	0.4-0.1	>5	>0.25	-	3.0	1.0-3.1	B (utq) 1 m	-
							+S 2-3 cm	
		<5	>0.25	-			B (utq) 1 m	-
							+S (mr) 5 cm	
30	0.4-0.1	-	<0.25	-	3.0	2.2-6	B (tq) 1 m	-
							+S (mr) 5-7.5cm	
							B (utq) 1m	IX
							+S (mr) 5-7.5 cm	IX
31	0.4-0.1	>0.25	-	-	3.0	4-14.5	B (utq) 1m	IX
							+S 2.5-5 cm	
							S (mr) 5-7.5 cm	IX
							B (tq) 1m	
							+S (mr) 5-7.5 cm	VIII, X, XI
		>4	-	-			B (tq) 1 m	IX
							+S(mr) 5-12.5 cm	
		(4, >1.5	-	-			S (mr) 7.5-25 cm	IX
		<1.5					CCA 20-40 cm	IX, XI
							+ (tq) 1m	
		-	-	-			CCA (sr) 30-60 cm	VIII, X, XI
							+B (tq) 1 m	

Continued..... from page 100

32	0.4-0.1	-	-	>20m	3.0	11-34	B (tq) 1 m	II,IV,IX,XI
see							+S(mr) 40-60 cm	
note XII				<20m			B (tq) 1 m	III,IV,IX,
							+S(mr) 20-40 cm	XI
							CCA(sr) 40-120 cm	IV,VIII,X,
							+B(tq) 1 m	XI

* Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

Table A4 Support measures for rock masses giving "extremely poor" and "exceptionally poor" tunnel stability. (Q range: 0.1-0.001)

SUPPORT CATEGORY	Q	CONDITIONAL FACTORS			P	SPAN/ESR	TYPE OF SUPPORT	NOTE
		RQD/Jn/Jr/Ja	SPAN/ESR (m)		Kq/cm2 (APPROX.)	(m)		
33*	0.1-0.01	>2	-	-	6	1.0-3.9	B (tq) 1 m	IX
							+S(mr) 2.5-5 cm	
		<2	-	-			S (mr) 5-10 cm	IX
							S (mr) 7.5-15 cm	VIII, X
34	0.1-0.01	>2	>0.25	-	6	2.0-11	B (tq) 1 m	IX
							+S(mr) 5-7.5 cm	
		<2	>0.25	-			S (mr) 7.5-15 cm	IX
			<0.25				S (mr) 15-25 cm	IX
		-	-	-			CCA(sr) 20-60 cm	VIII, X, XI
							+B (tq) 1 m	XI
35 see note XII	0.1-0.01	-	-	>15 m	6	6.5-28	B (tq) 1 m	III, IX, XI
							+S(mr) 30-100 cm	
		-	-	>15 m			CCA(sr) 60-200 cm	VIII, X, IX
							+B(tq) 1 m	III
		-	-	<15 m			B (tq) 1 m	IX, XI, III
							+S(mr) 20-75 cm	
		-	-	<15 m			CCA(sr) 40-150 cm	VIII, X, XI
							+B(tq) 1 m	III
36*	0.01-0.001	-	-	-	12	1.0-2.0	S (mr) 10-20 cm	IX
							S (mr) 10-20 cm	VIII, X, XI
							+B(tq) 0.5 m	
37	0.01-0.001	-	-	-	12	1.0-6.5	S (mr) 20-60 cm	IX
							S (mr) 20-60 cm	VIII, X, XI
							+B(tq) 0.5-1.0 m	
38* see note XII	0.01-0.001	-	-	>10 m	12	4.0-20	CCA(sr) 100-300 cm	IX
				>10 m			CCA(sr) 100-300 cm	VIII, X, II
							+B(tq) 1 m	XI
		-	-	<10 m			S (mr) 70-200 cm	IX
				<10 m			S (mr) 70-200 cm	VIII, X, III,
							+B(tq) 1 m	XI

* Author's estimates of support. Insufficient case records available for confident prediction of support requirements.

TABLE A5 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 ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q. 2. RQD intervals of 5, i.e. 100, 95, 90 etc are sufficiently accurate.	
B. Poor	25 - 50		
C. Fair	50 - 75		
D. Good	75 - 90		
E. Excellent	90 - 100		
2. JOINT SET NUMBER J_n			
A. Massive, no or few joints	0.5 - 1.0	1. For intersections use $(3.0 \times J_n)$ 2. For portals use $(2.0 \times J_n)$	
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		
G. Three joint sets plus random	12		
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	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3m. 2. $J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided the lineations are orientated for minimum strength.	
B. Rough or Irregular, undulating	3		
C. Smooth, undulating	2		
D. Slickensided, undulating	1.5		
E. Rough or irregular, planar	1.5		
F. Smooth, planar	1.0		
G. Slickensided, planar	0.5		
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		

Table A5 Cont'd

4. JOINT ALTERATION NUMBER	J_a	ϕ_r (approx.)	
<i>a. Rock wall contact.</i>			
A. Tightly healed, hard, non-softening, impermeable filling	0.75	—	
B. Unaltered joint walls, surface staining only	1.0	(25° - 35°)	
C. Slightly altered joint walls non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2.0	(25° - 30°)	1. Values of ϕ_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° - 25°)	
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° - 16°)	
<i>b. Rock wall contact before 10 cms shear.</i>			
F. Sandy particles, clay-free disintegrated rock etc	4.0	(25° - 30°)	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5mm thick)	6.0	(16° - 24°)	
H. Medium or low over-consolidation, softening, clay mineral fillings, (continuous, < 5mm thick)	8.0	(12° - 16°)	
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° - 12°)	
<i>c. No rock wall contact when sheared.</i>			
K. Zones or bands of disintegrated	6.0		
L. or crushed rock and clay (see	8.0		
M. G, H and J for clay conditions)	8.0 - 12.0	(6° - 24°)	
N. Zones or bands of silty- or sandy clay, small clay fraction, (non-softening)	5.0		
Q. Thick, continuous zones or	10.0 - 13.0		
P. bands of clay (see G, H and	13.0 - 20.0	(6° - 24°)	
R. J for clay conditions)			

Table A5 Cont'd.

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			
<i>a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.</i>			
		SRF	
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)		10.0	1. Reduce these values of SRF by 25 - 50% if the relevant shear zones only influence but do not intersect the excavation.
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	2. For strongly anisotropic virgin stress field (if measured) : when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
G. Loose open joints, heavily jointed or 'sugar cube' (any depth)		5.0	
<i>b. Competent rock, rock stress problems</i>			
	σ_c/σ_1	σ_t/σ_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
<i>c. Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure</i>			
		SRF	
N. Mild squeezing rock pressure		5-10	
O. Heavy squeezing rock pressure		10-20	
<i>d. Swelling rock, chemical swelling activity depending upon presence of water</i>			
P. Mild swelling rock pressure		5-10	
R. Heavy swelling rock pressure		10-20	

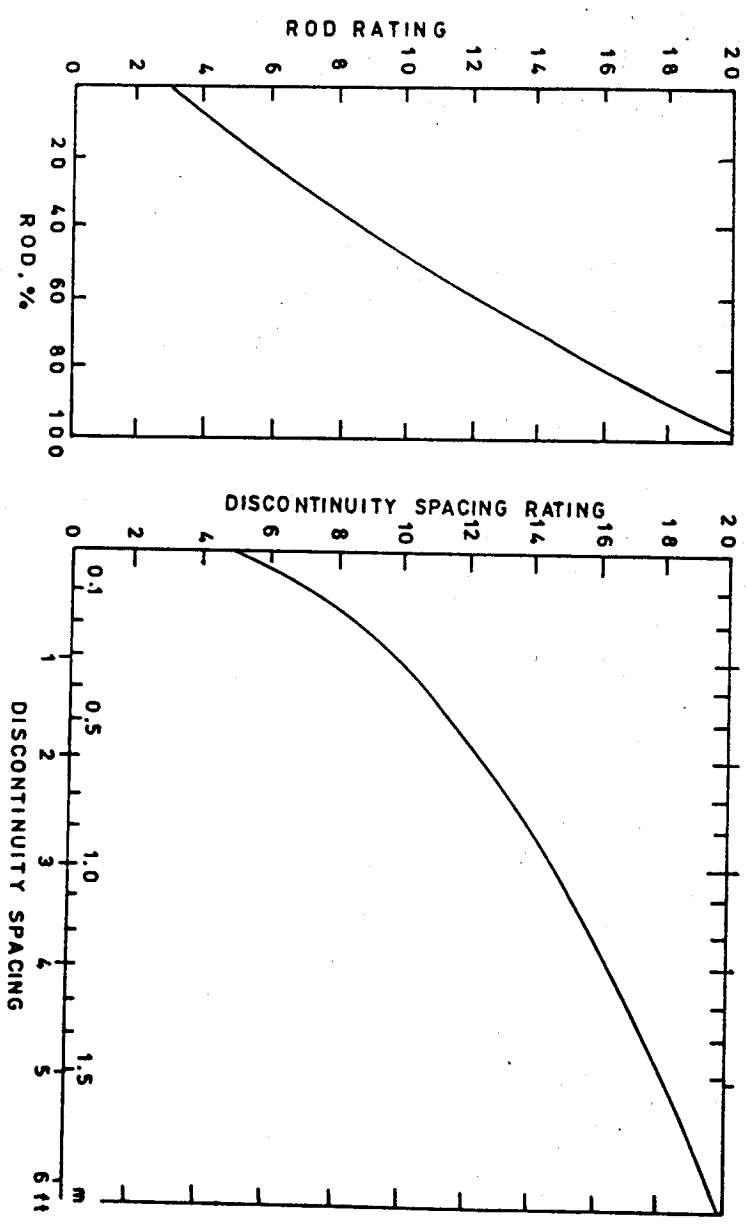


Figure A 4. Ratings for discontinuity density (after Kendorski et al., 1983).

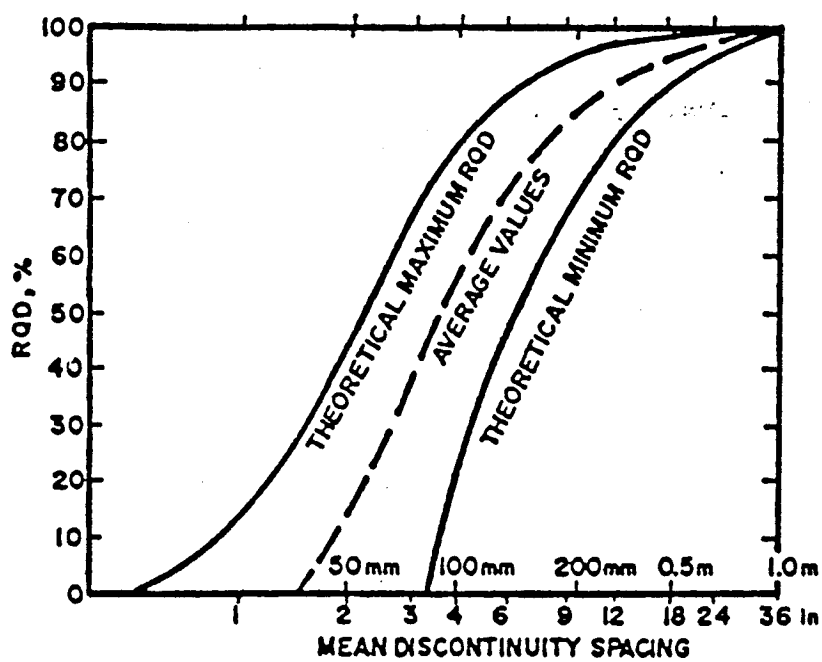


Figure A 5 . Theoretical relationship between RQD and discontinuity spacing (after Priest and Hudson, 1976).

Table A8 Discontinuity condition ratings

Description of discontinuity					
Wall roughness	<i>VR</i>	<i>R-SR</i>	<i>SR</i>	<i>SM-SK</i>	<i>SM</i>
Wall separation	None	Hairline	< 1 mm	1-5 mm	> 5 mm
Joint filling	None	None	Minor clay	Stiff clay, gouge	Soft clay, gouge
Wall weathering	<i>F</i>	<i>SL</i>	<i>SO</i>	<i>SO</i>	<i>VS</i>
Rating	30	25	20	10	0

Roughness:

VR = Very rough (coarse sandpaper);
R = Rough (medium or fine sandpaper);
SR = Smooth to slightly rough;
SM = Smooth but not polished;
SK = Slickensided, shiny;

Weathering (Alteration):

F = Fresh, unweathered, hard;
SL = Slightly weathered, hard;
SO = Softened, strongly weathered;
VS = Very soft or decomposed.

Table A9 . Groundwater condition rating

Condition	Dry	Damp	Wet	Dripping	Flowing
Rating	15	10	7	4	0

Table A10 Fracture orientation rating A_o based on indirect observation of fracture statistics

Strike	Perpendicular				Parallel		Flat dip
Heading direction	With dip		Against dip				
Dip amount	45-90	20-45	45-90	20-45	45-90	20-45	0-20
Adjustment	1.0	0.95	0.90	0.85	0.80	0.90	0.85

Table A11 Fracture orientation rating A_o based on direct observation in drift

Number of fractures defining block	Number of non-vertical faces					
	1	2	3	4	5	6
3	—	0.95	0.80	—	—	—
4	—	0.95	0.85	0.80	—	—
5	1.0	0.95	0.90	0.85	0.80	—
6	1.0	1.0	0.95	0.90	0.85	0.80

Table A12 Blasting damage adjustment A_b

Conditions/method	Applicable term	Adjustment A_b
1. Machine boring	No damage	1.0
2. Controlled blasting	Slight damage	0.94 to 0.97
3. Good conventional blasting	Moderate damage	0.90 to 0.94
4. Poor conventional blasting	Severe damage	0.90 to 0.80 (worst)
5. No experience in this rock	Moderate damage	0.90 (nominal)

Table A13. Major structures adjustment S.
 Adjustments are > 1 if abutment stresses tend to be carried away from the excavation; S = 1 if stresses are concentrated

Fault strike vs. heading direction	Most nearly perpendicular						Most nearly parallel					
Fault dip direction with respect to works	Towards			Away			Towards			Away		
Amount of dip	Sh	M	St	Sh	M	St	Sh	M	St	Sh	M	St
Distance to nearest fault zone (<i>W</i> = zone width)												
< 0.1 <i>W</i>	0.8	0.75	0.75	–	0.9	0.95	0.75	0.7	0.7	–	0.9	0.95
0.1 <i>W</i> to 1.0 <i>W</i>	0.85	0.8	0.8	0.90	0.9	1.0	0.8	0.75	0.9	0.85	1.0	0.95
1.0 <i>W</i> to 10.0 <i>W</i>	0.95	0.85	0.9	0.95	1.05	1.05	0.9	0.9	0.95	0.90	1.05	1.10
10.0 <i>W</i> to 50.0 <i>W</i>	1.0	0.95	1.0	1.05	1.05	1.0	1.0	1.0	1.0	1.10	1.0	
> 50 <i>W</i>	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.05	1.0	1.0
Within zone	0.85	0.80	0.90	0.85	0.80	0.90	0.80	0.75	0.85	0.80	0.75	0.85
Sh = Shallow (< 30°)	< 0.1 <i>W</i> factor not to be applied for <i>W</i> < 3 m;											
M = Moderate (30° to 60°);	> 50 <i>W</i> factor not to be applied for <i>W</i> > 3 m.											
St = Steep (> 60°);												

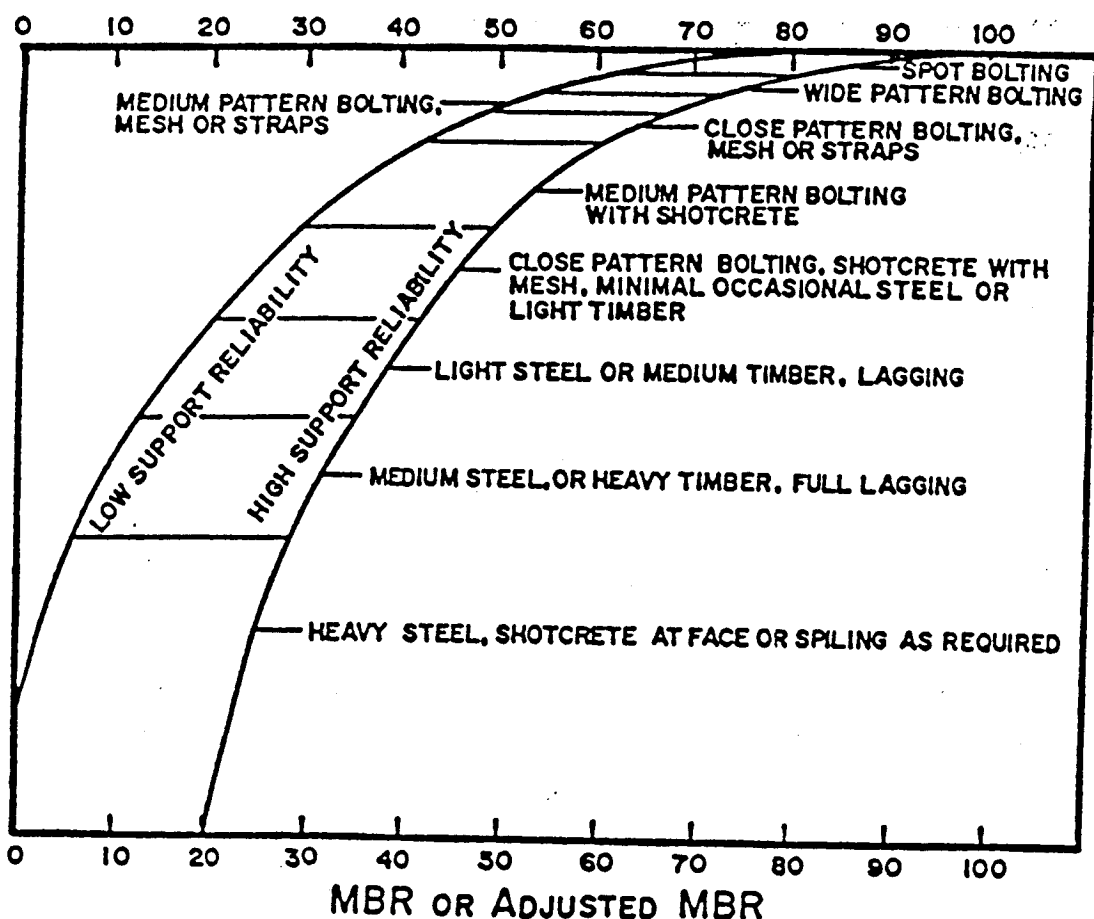


Figure A6 Support chart for isolated drifts or development drifts (after Kendorski et al., 1983).

Explanations of the support types in Figure A6

Spot bolting: Bolting to restrain limited areas or individual blocks of loose rock, primarily for safety.

Wide pattern bolting: Bolt spacing on 1.5 m to 1.8 m, or wider in very large openings.

Medium pattern bolting, with or without mesh or straps: Bolts spaced 0.9 m to 1.5 m, 23 cm wide straps or 100 mm welded wire mesh.

Close pattern bolting, mesh, or straps: Bolt spacing less than 0.9 m, 100 mm welded wire mesh, 0.3 m straps, or chain link.

Medium pattern bolting with shotcrete: Bolts spaced 0.9 m to 1.5 m and 80 mm (nominal) of shotcrete. Light mesh for wet rock to alleviate shotcrete adherence problems.

Close pattern bolting, shotcrete with mesh, minimal occasional steel or light timber: Bolt spacing less than 0.9 m with 100 mm welded wire mesh or chain link throughout, and nominal 100 mm of shotcrete. Localized conditions may require light wide-flange steel-sets or timber sets.

Light steel, medium timber, lagging: Bolting as required for safety at the face – full contact (grouted or Split Set) bolts only. Light wide-flange steel-sets or 0.25 m timber sets spaced 1.5 m, with full crown lagging and rib lagging in squeezing areas.

Medium steel, heavy timber, full lagging: Medium wide-flange steel-sets or 0.3 m timber sets spaced 1.5 m, fully lagged across the crown and ribs. Support to be installed as close to the face as possible.

Heavy steel, shotcrete at face or spiling as required: Heavy wide-flange steel-sets spaced 1.2 m, fully lagged on crown and ribs, carried directly to face. Spilling or shotcreting of face as necessary.

General: Bolting: bolts in spot bolting through close pattern bolting are considered to be 19 mm in diameter, fully grouted or resin-anchored standard rockbolts; mechanical anchors are acceptable in material of MBR > 60. Split-Set use is at the discretion of the operator.

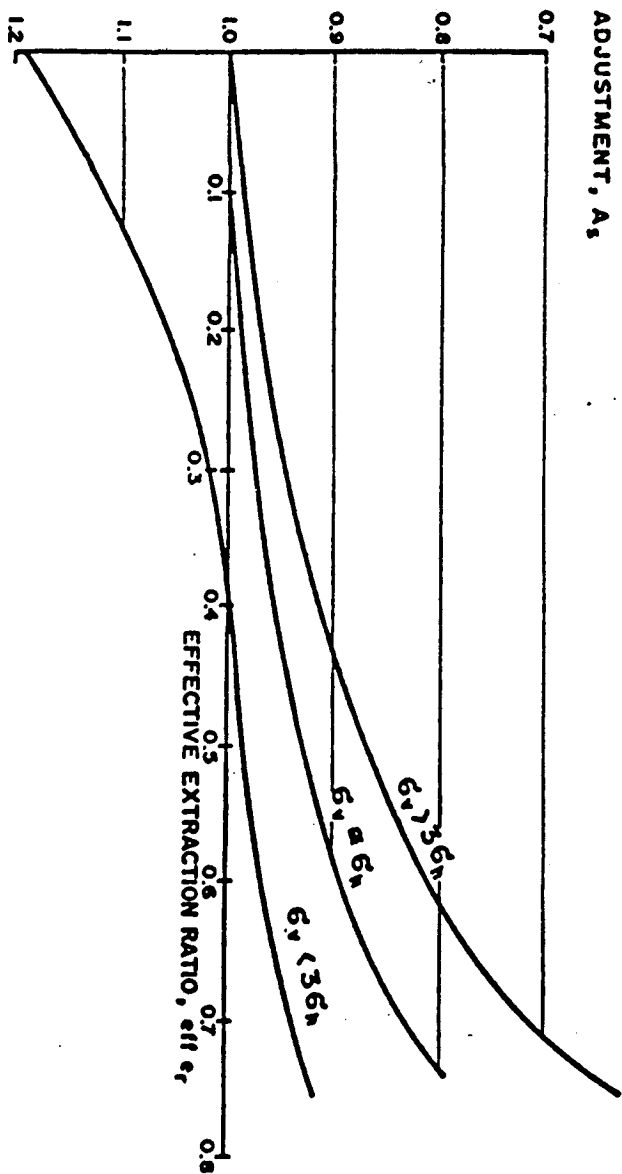


Figure A7 Adjustment A_s for induced stresses due to multiple openings (after Kendorski et al., 1983).

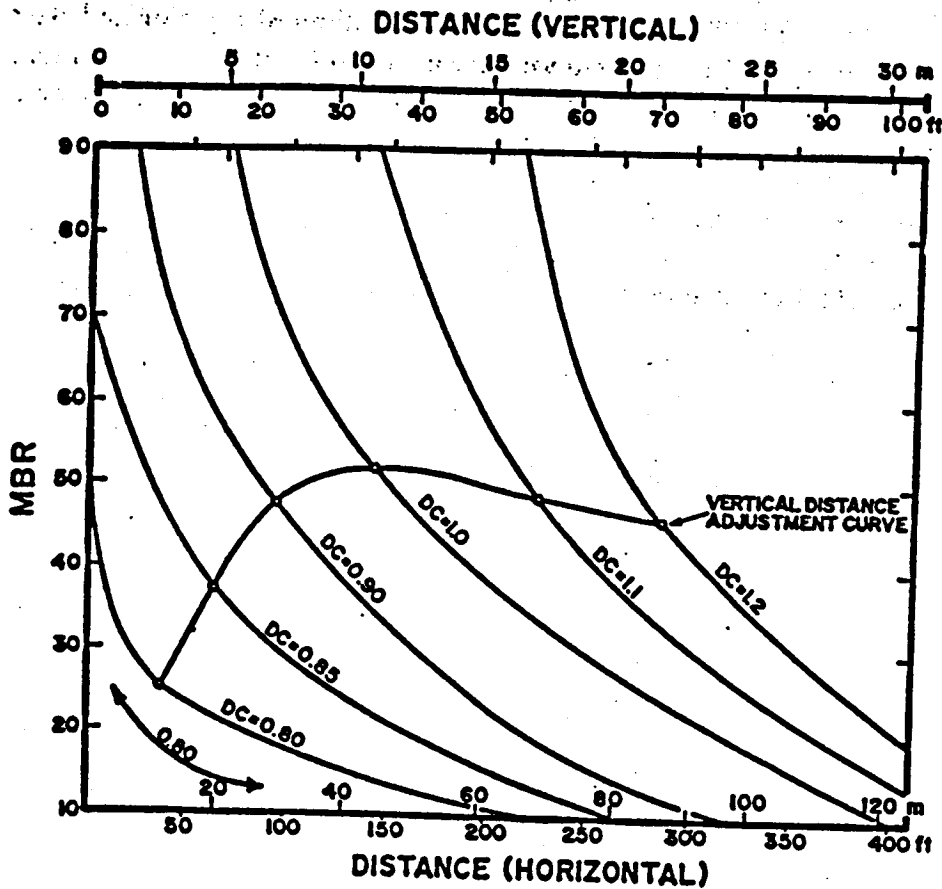


Figure A8 Adjustment DC for distance to cave line (after Kendorski et al., 1983). For drifts beneath the caving area, the vertical distance is projected up to the single vertical distance adjustment curve; the rating is read by interpolating between the multiple curves. For workings horizontally removed from the caving area, the horizontal distance is projected up to the MBR value and the rating is interpolated at that point from the multiple curves. For working both beneath and to the side, ratings are computed both ways, and the lowest value is taken.

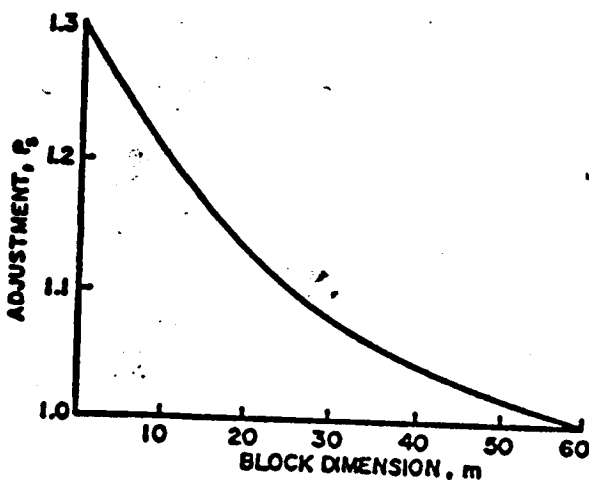


Figure A9 Adjustment PS for block/panel size (after Kendorski et al., 1983).

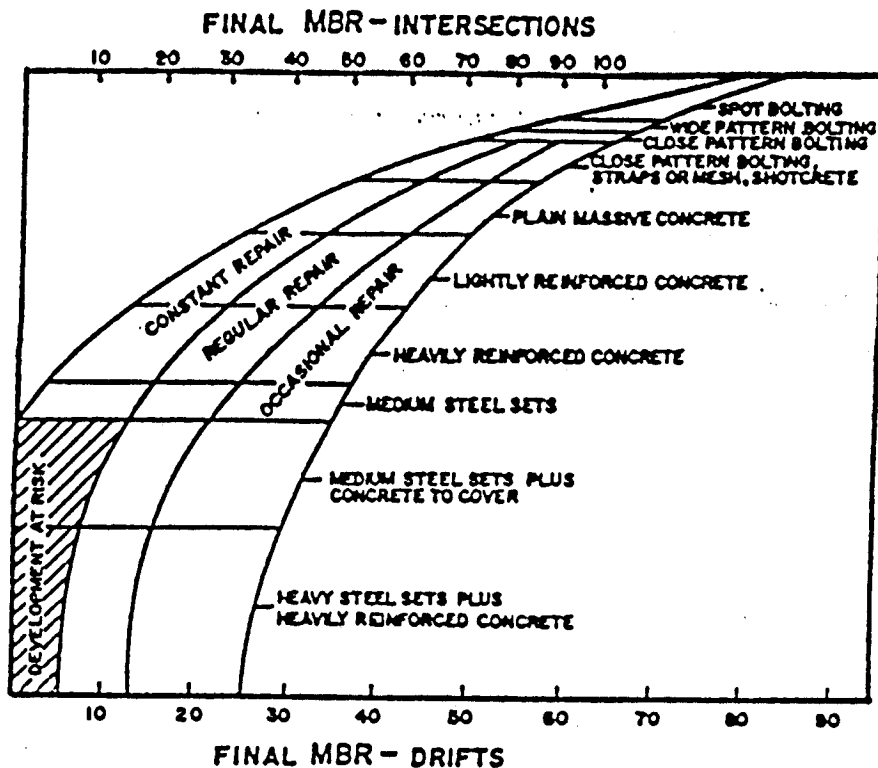


Figure A10 Permanent support chart for production drifts (after Kendorski et al, 1983).

Explanation of the support types in Figure A10

Spot bolting: Bolting to restrain limited areas or individual blocks of loose rock, primarily for safety.

Wide pattern bolting: Bolts on spaced 1.2 m to 1.8 m. May be wider in very large openings, when longer bolts are used.

Close pattern bolting: Bolts spaced less than 0.9 m to 1.2 m (practical limit 0.6 m).

Close pattern bolting, mesh or straps, shotcrete: Close pattern bolts with welded wire mesh or chain link in raveling ground, and nominal 100 mm of shotcrete.

Plain massive concrete: Cast-in-place massive concrete lining, 0.3 m to 0.45 m thick, may be applied over bolts or bolts and mesh when necessary. Prior shotcrete, if not damaged, may be considered part of this concrete thickness. Concrete should have a *minimum as-placed* 28-day compressive strength of 21 MPa (3000 lb/in²).

Lightly reinforced concrete: Massive, cast-in-place concrete lining 0.3 m to 0.45 m thick as above, lightly reinforced with rebars on 0.6 m centers or continuous heavy chain link. Reinforcement mainly in brows, crown, corners, and intersections.

Heavily reinforced concrete: Massive, cast-in-place concrete lining as above, heavily reinforced with rebars on 0.6 m centers or less, on ribs and crown.

Medium steel sets: Medium wide-flange steel-sets on 1.2 m centers, fully lagged on the crown and ribs.

Medium steel sets plus plain concrete to cover: Medium wide-flange steel-sets on 1.2 m centers, with plain, cast-in-place concrete (minimum strength 21 MPa) of sufficient thickness to cover the sets.

Heavy steel sets plus heavily reinforced concrete: Heavy wide-flange steel-sets on 1.2 m centers, with minimum 0.3 m thick heavily reinforced concrete throughout.

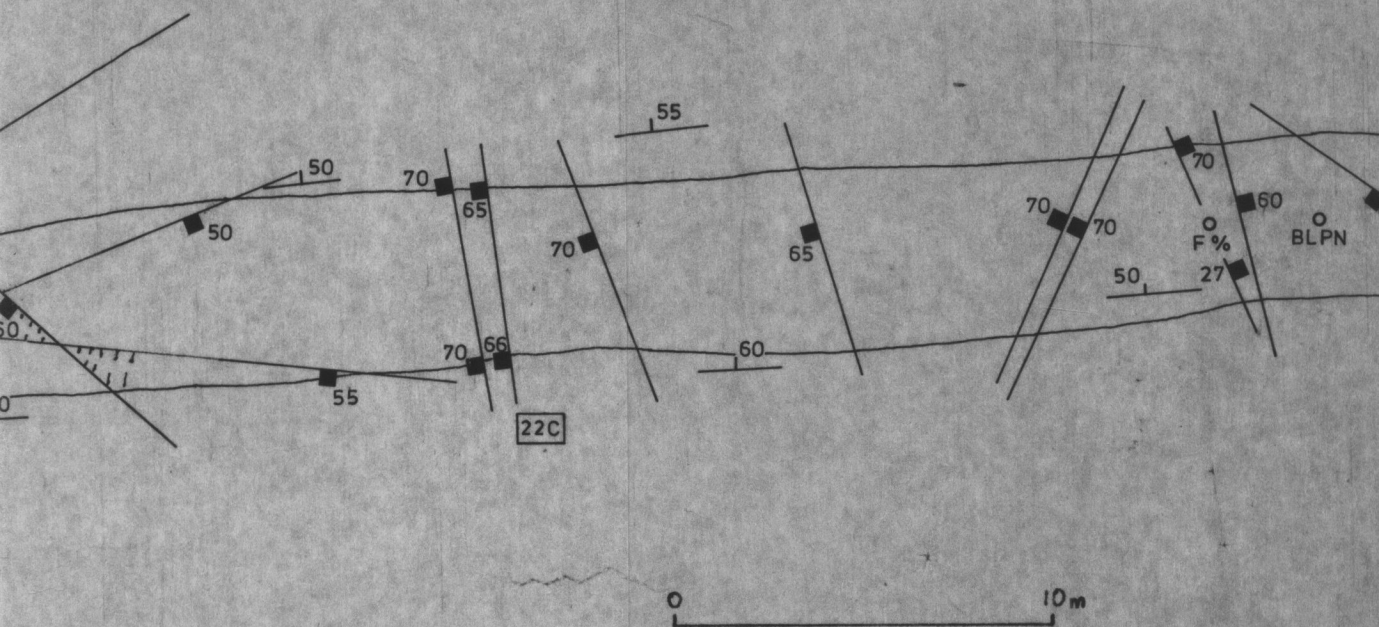
General:

Concrete: it is assumed that proper concrete practice is observed: negligible aggregate segregation, full rock-concrete contact, adequate curing time.

Chain link or steel sets, from development support, are considered reinforcement if the concrete between the sets is also reinforced.

APPENDIX B

SS 28



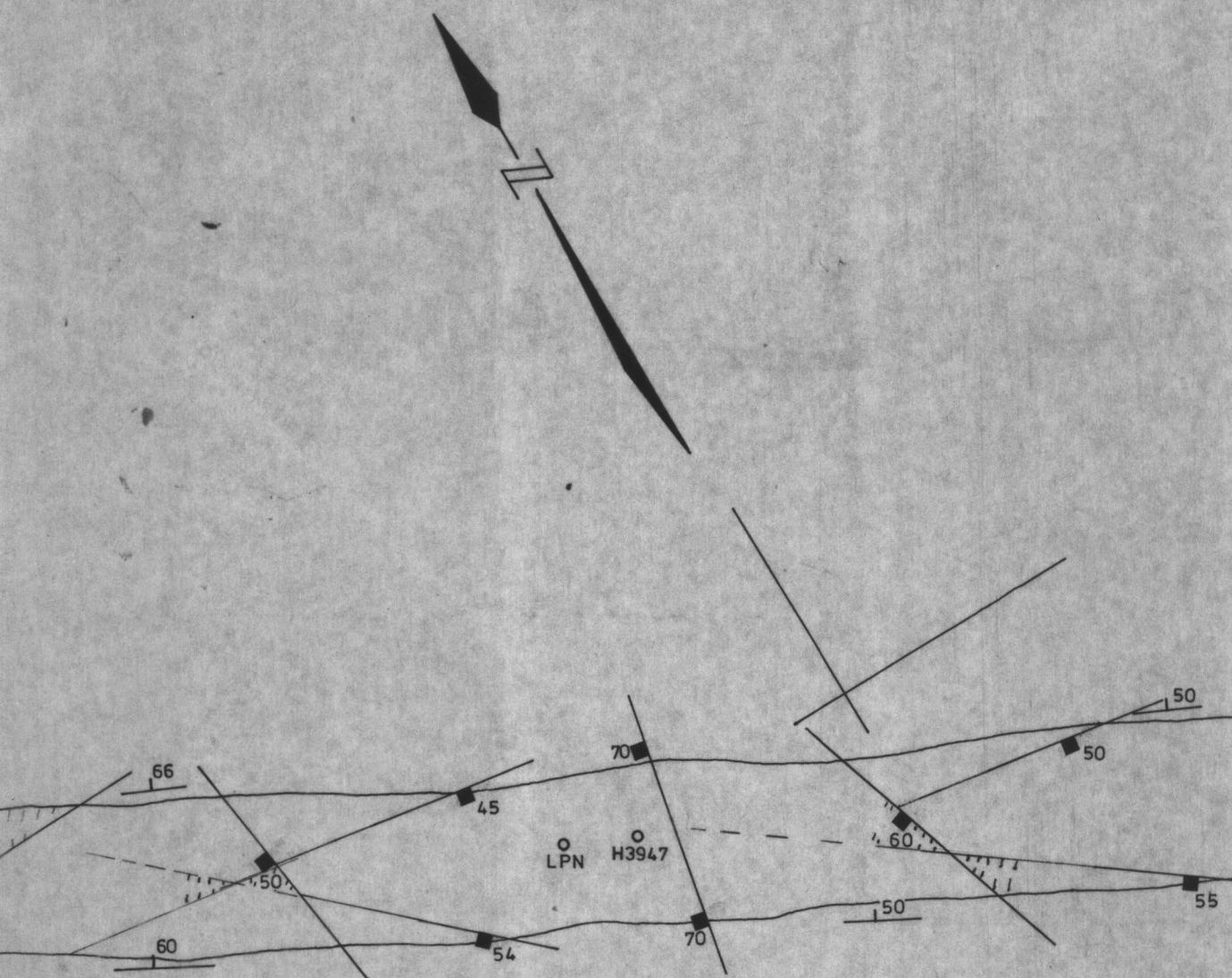
SS 28

BALUBA MINE - 245 metre level - Tramming Haulage

Map showing discontinuities (Bedding and Jointing)

Mapped and drawn by R.M. Kamanga

Scale 1:200



ARGILLACEOUS QUARTZITE

KEY

-  Joint / Dip
-  Bedding / Dip
-  Potential wedge failure area
-  Schmidt hammer test station

SS 29

Direction of advance

ARGILLACEOUS QUARTZITE

QUARTZ VEIN
0.2m thick

BR 1754

21

50

67

55

22B

X = 9500

Y = 8500

SS 29

MS 7397

MS 7395

MW-MS Blocky Feldspathic Argillaceous Quartzite



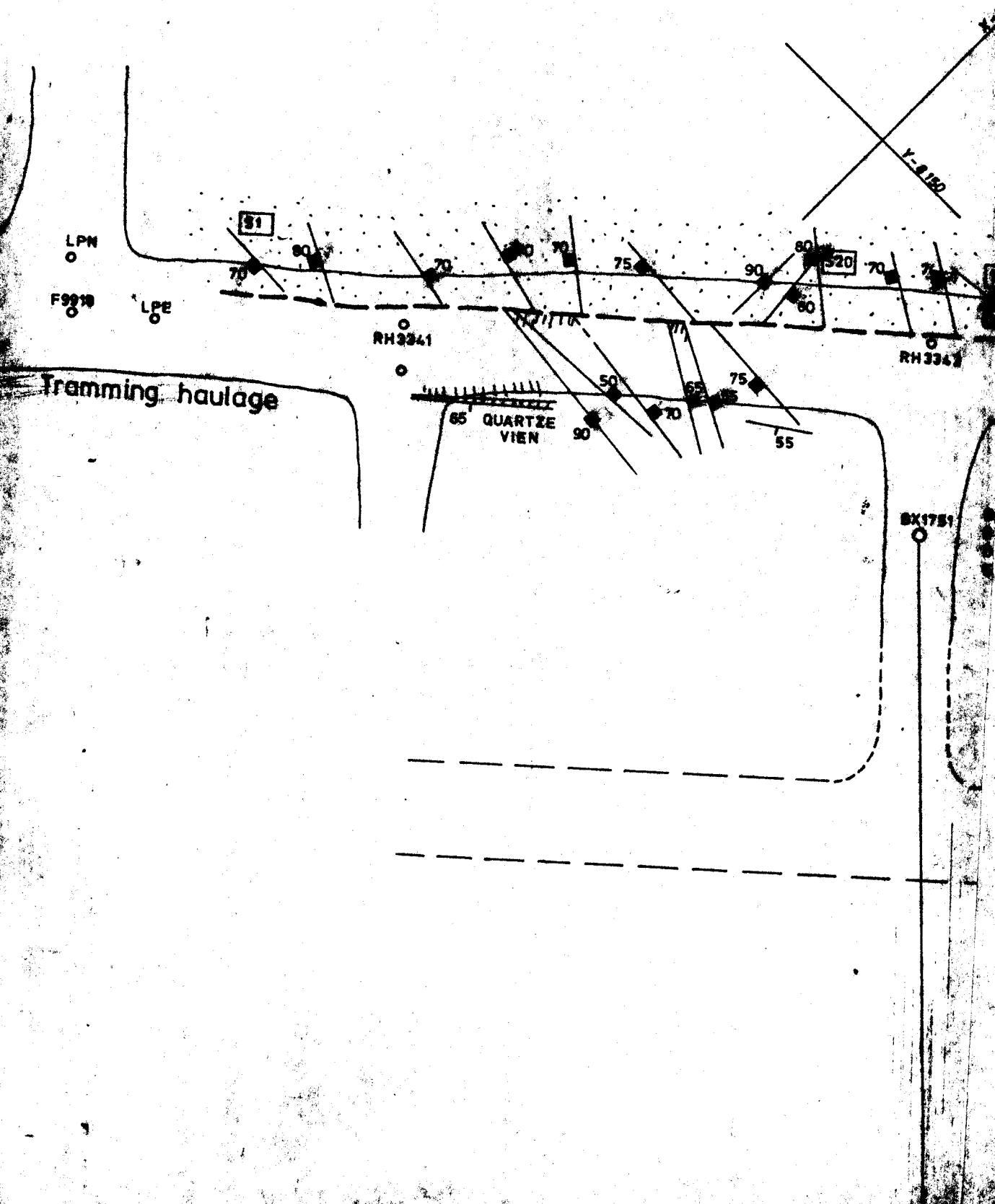


Table B14 DIAMETRAL POINT LOAD STRENGTH TEST RESULTS
ON ARGILLACEOUS QUARTZITE

Borehole No.: Bx1740 Size: 42mm Depth: 26m Level: 245
Inclination (degrees): + 66 Section: SS27

Sample No.	Sample interval	Failure load KN	$Is = P/D^2$ MPa	$Is(50)$ MPa	U.C.S $Is(50) \times 22$ MPa	Mode of failure
AQ 1	0.0-0.5	18.0	10.2	9.5	209.9	IR
AQ 2	0.5-1.0	14.0	7.9	7.3	160.6	IR
AQ 3	1.0-1.5	6.4	3.6	3.3	72.6	W
AQ 4	1.5-2.0	15.5	8.8	8.2	180.4	IR
AQ 5	2.0-2.5	8.5	4.8	4.5	99.0	IR
AQ 6	2.5-3.0	11.8	6.7	6.2	136.4	IR
AQ 7	3.0-3.5	4.0	2.3	2.1	46.2	W
AQ 8	3.5-4.0	14.5	8.2	7.6	167.2	IR
AQ 9	4.0-4.5	19.0	10.8	10.0	220.0	IR
AQ 10	4.5-5.0	15.2	8.6	8.0	176.0	IR
AQ 11	5.0-5.5	15.9	9.0	8.4	184.8	IR

IR = Failure through intact rock

Mean U.C.S = 170.4 MPa

W = Failure along plane of weakness

standard deviation = 21.4 %

U.C.S = Uniaxial compressive strength

Table B15 DIAMETRAL POINT LOAD STRENGTH TEST RESULTS
ON ARGILLACEOUS QUARTZITE.

Borehole No.: Bx1751 Size: 42mm Depth: 50m Level: 480m
Inclination (degrees): 0 Section: MS7453

Sample No.	Sample interval (m)	Failure load KN	$Is = P/D^2$ MPa	$Is(50)$ MPa	U.C.S $Is(50) \times 22$ MPa	Mode of failure
AQ 12	0.0-0.5	9.3	5.3	4.9	107.8	IR
AQ 13	0.5-1.0	3.9	2.2	2.0	44.0	W
AQ 14	1.0-1.5	5.8	3.3	3.1	68.2	W
AQ 15	1.5-2.0	15.1	8.6	7.9	173.8	IR
AQ 16	2.0-2.5	15.1	8.6	7.9	173.8	IR
AQ 17	2.5-3.0	7.1	4.0	3.7	81.4	W
AQ 18	3.0-3.5	6.7	3.8	3.5	77.8	W
AQ 19	3.5-4.0	15.6	8.8	8.1	178.2	IR
AQ 20	4.0-4.5	8.9	5.0	4.6	101.2	IR
AQ 21	4.5-5.0	9.3	5.3	4.9	107.8	IR
AQ 22	5.0-5.5	11.6	6.6	6.1	134.2	IR
AQ 23	5.5-6.0	14.2	8.0	7.4	162.8	IR
AQ 24	6.0-6.5	20.0	11.3	10.4	228.8	IR
AQ 25	6.5-7.0	14.2	8.0	7.4	162.8	IR
AQ 26	7.0-7.5	11.8	6.7	6.2	134.2	IR

U.C.S. = 151.4 MPa

Standard deviation = 25.0 %

Table B16 DIAMETRICAL POINT LOAD STRENGTH TEST RESULTS
ON AEOLIAN QUARTZITE

Re-drilled samples from 480m level

Core size: 47mm

Sample No.	Failure load KN	$I_s = P/D^2$ MPa	$I_s(50)$ MPa	U.C.S $I_s(50) \times 22$ MPa	Mode of failure
AO 1	29.4	13.3	12.9	283.8	IR
AO 2	30.2	13.7	13.3	176.9	IR
AO 3	24.9	11.2	10.9	239.8	IR
AO 4	28.0	12.7	12.4	272.8	IR
AO 5	31.6	14.3	13.9	305.8	IR
AO 6	22.2	10.0	9.7	213.4	IR
AO 7	32.0	14.5	14.1	310.2	IR
AO 8	10.7	4.8	4.7	103.4	W
AO 9	28.9	13.1	12.7	279.4	IR
AO 10	25.4	11.5	11.2	246.4	IR
AO 11	25.8	11.7	11.4	250.8	IR
AO 12	22.1	10.0	9.7	213.4	IR

Mean U.C.S. = 253.9 MPa

Standard deviation = 16.0 %

Table B17 DISCONTINUITY MAPPING DATA - 245m level

Location: 245m level drive west Elevation: 1025m above sea level
Scanline No.1 Scanline elevation: 1025.5m Bearing (degree): 115
Scanline length: 150m Rock type: Argillaceous Quartzite.

Dip direction (degrees)	Dip	Joint set	Joint separation (mm)	Gouge	Roughness	Bedding dip dir/ dip (degrees)
310	55	4	3	clay	R	24/50
317	70	4	<2	clay	S-R	20/50
258	55	2	3	clay	S-R	22/50
262	60	2	2	dolomite	R	0/66
300	70	3	<1	clay	R	30/60
235	60	2	2	dolomite	R	20/52
192	50	RJ	1	dol+clay	R	55/50
192	52	RJ	<1	-	S-R	15/65
280	55	3	<1	dol+clay	S-R	25/50
234	60	2	1	clay	S	20/50
246	60	2	<1	dolomite	S-R	18/57
254	65	2	<1	dolomite	S-R	40/50
160	60	RJ	2	clay	S	20/50
252	65	2	2	clay	S-R	24/50
234	55	2	1	dolomite	R	22/66
300	60	3	<1	clay	-	19/60
293	54	3	3	clay+dol	S-R	19/65
131	55	1	2	dolomite	R	18/60
290	50	3	<1	clay	S	11/55

Table B17 continued.....

290	52	3	2	-	I	23/55
260	60	2	2	clay	I	
180	60	RJ	3	dolomite	S	
113	70	1	<1	dolomite	I	
263	50	2	<1	dolomite	R	
190	45	RJ	3	dol+clay	R	
190	54	RJ	1	dolomite	S-A	
282	70	3	<1	clay	S	
252	60	2	1	clay	S	
190	50	RJ	1	dolomite	S-A	
217	55	RJ	2	dolomite	S-A	
293	70	3	<1	dolomite	S-A	
310	55	4	2	dolomite	S-A	
294	65	3	<1	clay	R	
285	65	3	1	clay	S	
325	70	4	2	clay	R	
247	75	2	1	clay	S	
95	60	1	<1	clay	R	
100	60	1	<1	clay	R	
294	70	3	2	clay	S-A	
283	70	3	<1	dolomite	S-A	
317	70	4	<1	dolomite	S-A	
325	70	4	<1	clay	S	
235	60	2	2	dolomite	S-A	
234	60	2	1-2	clay	S-A	
240	60	2	<1	clay	R	
248	70	2	<1	clay	R	
160	50	1	2	dolomite	R	
95	65	1	1-2	clay	S	
290	65	3	<1	clay	R	
234	65	2	<1	clay	S-A	
180	57	RJ	<1	clay	S-A	
101	50	1	<1	clay	S-A	

S = Smooth R = Rough RJ = Random Joint dol = dolomite
I = Irregular dir = direction

Table B18 DISCONTINUITY MAPPING DATA - 300m level

Location: 300m level drive east Elevation: 974m above sea level
Scanline: No.2 Scanline elevation: 995.5m Bearing: 115
Scanline length: 37m Rock type: Argillaceous Quartzite

Dip direction (degrees)	Dip	Joint set	Joint separation (mm)	Gouge	Roughness	Bedding Dip dir/ dip (degrees)
125	50	1	1-2	clay	R	215/50
110	90	3	1-2	clay	R	215/56
105	90	3	<1	-	-	223/55
287	50	2	2-3	clay	S-A	223/48
118	60	1	<1	dol+clay	S-A	223/65
118	74	1	1-2	clay	R	217/50*

Table B18 continued.....

105	70	1	1-2	clay	S-R
300	70	RJ	<1	clay	R
120	75	1	<1	clay	S
118	60	1	1-2	clay	S-R
115	60	1	<1	clay	R
114	55	1	<1	clay+dol	R
83	70	RJ	1-2	clay	S-R
124	65	1	1-2	clay	S
62	65	R	1-2	clay	S-R
102	65	3	<1	clay+dol	S-R
120	60	1	<1	clay	S
332	70	RJ	<1	clay	S-R
113	65	3	<1	clay	S
287	55	2	2	clay	S-R
118	60	1	1-2	clay	S-R
118	75	1	1-2	clay	S-R
285	55	2	1-2	clay+dol	S-R
119	60	1	<1	clay	S
290	60	2	1-2	clay	R
284	63	2	1-2	clay	S
112	88	3	1-2	clay	S-R
119	55	1	<1	clay	S-R
117	50	1	1-2	clay+dol	R

* shear plane

Table B19 DISCONTINUITY MAPPING DATA - 480m level

Location: 480m level drive east Elevation: 789.0m
Scanline No.: 3 Scanline elevation: 788.5m Bearing: 115
Length of scanline: 150m Rock type: Aeolian Quartzite

Dip direction (degrees)	Dip (degrees)	Joint set	Joint separation (mm)	Roughness	Gouge
313	60	1	<1	S	dolomite
160	75	RJ	<1	S-R	None
280	65	3	<1	S-R	None
313	75	1	1	S	dolomite
312	85	1	1-2	S-R	dolomite
295	70	2	<1	S	None
313	80	1	<1	S	None
290	80	2	<1	S	None
248	55	4	<1	S-R	dolomite
280	85	3	1-2	S	dolomite
250	70	4	<1	S-R	None
280	85	3	<1	S	None
255	70	4	<1	S	None
282	65	3	<1	S	dolomite
312	80	1	1-2	S	dolomite
265	70	3	3	S	dol+clay
245	65	4	<1	S	None

Table B19 continued.....

265	70	3	<1	S	dol+clay
336	55	RJ	<1	S-R	dol+clay
310	70	1	<1	S	none
309	65	1	<1	S	dolomite
135	80	RJ	<1	S	dolomite
290	80	3	<1	S-R	dol+clay
275	80	3	<1	S	dol+clay
280	75	3	<1	S	dol+clay
270	70	3	<1	S	dol+clay
300	65	2	1-2	S-R	dolomite
265	70	3	<1	S	none
270	70	3	<1	S-R	dolomite
290	60	2	<1	S	dolomite
305	70	1	<1	S-R	none
275	75	3	<1	S	none
010	90	RJ	<1	S-R	dolomite
035	85	RJ	<1	S	dolomite
290	80	2	<1	S	dolomite
300	70	2	<1	S	dolomite
320	80	1	<1	S	dolomite
200	89	RJ	<1	S-R	dol+clay
250	50	4	<1	S	none
200	55	RJ	<1	S-R	dolomite
255	50	4	<1	S-R	dolomite
200	80	RJ	<1	S-R	dolomite
290	70	2	<1	S	dolomite
245	50	4	<1	S	none
290	90	2	1-2	S-R	none
250	80	4	2	S-R	dolomite
235	50	RJ	<1	S	dolomite
250	90	4	<1	S	dolomite
180	60	RJ	<1	S	dolomite

Table B20 DISCONTINUITY MAPPING DATA - 480m Level

Location: 480m tramming drive East

Elevation: 789m

Scanline No.: 4

Scanline elevation: 788.5m

Bearing: 115

Scanline length: 125m

Rock type: Argillaceous Quartzite

Dip direction (degrees)	Dip (degrees)	Joint set	Joint seperation (mm)	Roughness	Gouge	Bedding/ dip dir/ dip (degrees)
280	70	2	1-2	R	clay	225/65
300	65	2	1-2	R	clay	240/55
300	50	2	<1	S-R	clay	240/56
150	30	RJ	1-2	S-R	clay	230/60
290	80	2	2	R	clay	260/50
290	85	2	1-2	R	clay	242/40
95	85	1	<1	S-R	clay	230/65
290	90	2	<1	S-R	clay	245/65

Table B20 Continued.....

180	30	RJ	2-3	S-A	clay	235/54
295	80	2	<1	S-A	clay	
310	70	2	1-2	S-A	clay	
290	70	2	1-2	S-A	clay	
100	60	1	<1	A	clay	
99	54	1	<1	A	clay	
308	55	2	<1	A	clay	
120	80	1	<1	A	clay	
121	85	1	<1	A	clay	
95	50	1	1-2	A	clay	
250	90	RJ	<1	A	clay	
122	75	1	1-2	A	clay	
100	80	1	2-3	S-A	clay	

RJ = Random joint S = smooth Joint plane A = rough joint plane

Table B21

Project Name _____ Site of Survey _____ By _____ Date _____

1. Geological Region:	_____		Rock Type	_____		Location	_____	
2. Compressive strength:	Average	_____	Range	_____	Method	_____	Comment	_____
3. Core Recovery:	Interval	_____	Average	_____	Range	_____		
4. RQD:	Interval	_____	Average	_____	Range	_____		
5. Discontinuity Spacing:	Average	_____	Range	_____	Comment	_____		
6. Discontinuity Condition	Wall Roughness	_____	Wall Separation	_____	Joint Filling	_____	Wall Weathering	_____
Most Common	_____	_____	_____	_____	_____	_____	_____	_____
Intermediate	_____	_____	_____	_____	_____	_____	_____	_____
Least Common	_____	_____	_____	_____	_____	_____	_____	_____
Consensus	_____	_____	_____	_____	_____	_____	_____	_____
7. Water Condition	Dry	_____	Damp	_____	Wet	_____	Flowing	_____
8. Fracture Orientations	Strike	_____	Set 1	_____	Set 2	_____	Set 3	_____
	Dip/Dir	_____	_____	_____	_____	_____	_____	_____
	Rank	_____	_____	_____	_____	_____	_____	_____

9. Major Structures	Strike	_____	Dip	_____	Dip Dir	_____	Width	_____	Location/Comment	_____
Name:	_____	_____	_____	_____	_____	_____	_____	_____	Location/Comment	_____
Name:	_____	_____	_____	_____	_____	_____	_____	_____	_____	_____
Name:	_____	_____	_____	_____	_____	_____	_____	_____	_____	_____
10. Stress Field σ_1 :	Direction	_____	Magnitude	_____	Measured?	_____				
σ_3 :	Direction	_____	Magnitude	_____	Measured?	_____				

11. Source of Geological Data _____

Input form: Geological data.

Table B22

Project Name _____ Site of Survey _____ By _____ Date _____

1. Type of Drift(s) _____ 2. Orientation(s) _____ 3. Design Life _____

4. Design Dimensions Width _____ Width variation _____
 Height _____ Height variation _____

5. Drift Spacing (Horizontal) _____ Spacing _____

6. Other openings Type _____ Size _____

6. Extraction Ratio _____

Multiple Openings: Excavated Area _____ Unexcavated _____ e_r _____
 Single Opening: 1:5(width) _____ Excavated _____ Unexcavated _____ e_r _____

7. Distance below undercut - drift floor to undercut floor _____
 drift crown to undercut floor _____

8. Method of Excavation: Machine bored _____ Controlled D & B _____ Conventional D & B _____

9. Excavation conditions: _____

Perimeter Hole Traces _____
 Rib or Crown Looseness _____
 New or Existing Cracks _____
 Overbreak & Barring-Down _____
 Other Criteria _____

10. Intersections, turnouts: Type _____ Location _____ Max Span _____

11. Block Dimensions: Side _____ Orientation _____ End _____ Orientation _____

12. Cave Line Direction _____ Direction of Progress _____

13. Drift Location (in block, with respect to major structures and their dips with respect to cave)

Input Form: Engineering Data

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