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AN EVALUATION OF THE RELIABILITY OF BOREHOLE CORE DATA IN ENGINEERING ROCK MASS CLASSIFICATION SYSTEMS

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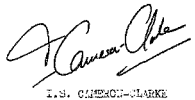
A Dissertation Submitted to the Faculty of Engineering,
University of the Witwatersrand, Johannesburg
for the Degree of Master of Science in Engineering.

Johannesburg 1978.

DECLARATION

This is to certify that this dissertation is my own work and has not been submitted to any other University.



I.S. CAMERON-CLARKE

ABSTRACT

The concept of classifying rock masses for engineering purposes has been used for a number of years. Modern classification systems are generally based on clearly defined geological parameters, which are rated individually to give a final rock quality number for a particular rock mass condition. The Geomechanics Classification System, developed at the Council for Scientific and Industrial Research in Pretoria, and the Norwegian Geotechnical Institute's Q-System, are examples of modern engineering rock mass classification systems.

One of the most important roles of engineering geology in underground rock engineering, is the evaluation of the rock mass condition during the site investigation stage of any project. In some cases the only information available for this purpose may be from borehole cores. Engineering rock mass classification systems have, however, been developed mainly from in situ observations and case history studies, and it is not always clear whether the required parameters can be reliably measured from bore cores alone. The aim of this project is to try to determine whether such parameters can be measured from bore cores, and to assess the reliability of rock mass classifications based thereon. The Geomechanics and Norwegian Geotechnical Institute's Classification Systems were selected for this purpose.

Three small diameter tunnels with adequate borehole coverage were available for examination. Two of the tunnels are situated in the Johannesburg area and were excavated through lavas of the Ventersdorp Supergroup, and quartzite and shale of the Witwatersrand Supergroup, respectively. The third tunnel is located near Paarl in the Cape Province and was excavated through the Cape Granites and Malmesbury sediments.

Borehole cores from the vicinity of each tunnel were logged in detail with particular attention being devoted to the parameters required by both classification systems. The rock mass at each site was classified from this data. The underground intersection points of each borehole were then visited and the classification parameters measured from detail line joint surveys. The rock mass at each site was again classified, using the in situ data.

The two classification systems were first evaluated separately. The in situ classifications were assumed to be truly representative of the rock mass condition at each site and were compared with their associated borehole results in order to establish the reliability of the latter. The

/borehole .....

boreholes and in situ ratings for the individual parameters, and the support predictions and estimated stand-up times and unsupported spans in each case, were also compared. The results from the two systems were then also compared.

For both the Geomechanics and the Norwegian Geotechnical Institute's Classification Systems, the bore cores generally indicated poorer rock mass conditions than were actually encountered underground. The correlations between the borehole and in situ classification values, and the unsupported spans and stand-up times estimated therefrom were better for the Norwegian System than for the Geomechanics System.

The borehole classifications from both systems are not sufficiently reliable for detailed design purposes. They could, nevertheless, serve as a basis for defining general rock classes and likely support requirements expected within a planned underground excavation.

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1. INTRODUCTION

1.1 Introduction

In the modern world of increased population growth and trends towards urbanization, natural resources such as raw materials, power and land space are becoming increasingly in demand as commodities essential to man's continued development. As easily workable, near surface, ore deposits become depleted it is becoming necessary to mine at greater depths. Fuel and power demands require increased exploitation of fuel minerals, which, although often at shallow depths, must sometimes, for ecological or economic reasons, be mined by underground methods. Subsurface chambers are being used to store waste material from nuclear power plants, and in major urban centres lack of land space and environmental considerations are making it necessary to locate basic utilities such as sewage systems, electricity supply networks and transport services beneath the surface. Factories and even sports centres are sometimes constructed underground[1], as are power plants, petroleum storage facilities and military installations.

The progress rate, cost and support requirements of underground excavations are directly related to the general rock quality and ground water conditions encountered[2]. For their optimum design and construction a thorough knowledge of these factors is required. It is therefore advantageous for engineers involved in underground construction to have access to data which provides a reliable and meaningful picture of the subsurface rock conditions in the vicinity of a planned excavation. The required data is related to the engineering geological and rock mechanics characteristics of the rock mass, and is obtained from boreholes, surface outcrops, geophysical and airphoto surveys, laboratory and in situ tests, and stress analyses. Of these methods, core drilling is the only one which can provide direct information of the rock at the required depth prior to excavation. Other methods usually require some interpretation when projecting results to the planned excavation level, but are nevertheless invaluable in building up a composite picture of the engineering geological conditions of a site. Boreholes can also be used for in situ testing, geophysical logging and the observation of in situ conditions with T.V. borehole cameras[3].

Owing to the generally competent nature of rock masses at depths associated with deep level mining, local stress fields and the physical

/deformation .....

deformation characteristics of the rocks are usually of greater significance to the stability of underground excavations than the surface roughness and alteration characteristics of the joints in the rock. In low and medium stress environments, as are usually associated with excavations for civil engineering purposes, the engineering geological behaviour of the rock mass is controlled largely by these latter characteristics. It is also this class of excavation for which the above mentioned investigation techniques are most often used.

The concept of classi ing rock masses for engineering purposes, particularly for the evaluation of underground support requirements, has been used for a number of years, and numerous systems have been developed to cater for local conditions at specific projects. The aim of these systems has generally been to describe the engineering characteristics of the rock mass in quantitative terms useful to both design engineers and contractors, and to indicate, with some degree of certainty, the expected behaviour of the rock mass at the excavation level. Owing to the complex nature of rock masses, however, this quantification process has often been very subjective and empirical. The need for a standardized classification system which can be applied, and understood, by all the disciplines involved in rock engineering problems, and which is relevant to all geological environments, has resulted in efforts to develope numerical expressions based on engineering geological parameters which are related to observed support requirements and rock mass behaviour patterns. This has been attempted largely from case history studies and in situ examinations of existing excavations.

For an engineering rock mass classification system to be useful in the design and pre-construction phases of an underground excavation, the relevant parameters must be obtainable during the pre-construction site investigation. They must also be measureable with sufficient accuracy to provide a *reliable classification of the rock mass*. In areas where surface rock exposures may be minimal or non-existent, such as urban centres or where thick overburden occurs, investigation techniques other than core drilling may not be able to provide sufficiently specific data to allow the rock mass to be classified. In other areas, the depth of cover might preclude the reliable projection of surface observations to the planned excavation level. In cases like these the parameters relevant to a particular engineering rock mass classification system may only be obtainable from bore cores.

The pertinent points which need to be established, therefore, are:

/(i) .....

(i) whether the required parameters can be measured in borecores and, if so,

(ii) whether the results based thereon are sufficiently reliable to be useful.

It is the aim of this dissertation to try to provide answers to these questions.

1.2 Scope of the Work Undertaken

Two rock mass classification systems, developed by the Geomechanics Division of the South African Council for Scientific and Industrial Research, and the Norwegian Geotechnical Institute respectively, were selected for this study. These two systems represent some of the most recent concepts in rock classification for engineering purposes.

Three small diameter tunnels with adequate borehole coverage were available for examination. They are all located in low to medium stress environments, but display a variety of geological conditions. Two of the tunnels are situated in the Johannesburg area and have been excavated through hard, competent and relatively massive lavas of the Ventersdorp Supergroup, and the stratified and laminated weathered quartzites and shales of the Witwatersrand Supergroup respectively. The third tunnel is located close to Paarl in the Cape Province and was excavated through the Cape Granites, and the quartzitic sandstones of the Malmesbury Beds.

Borehole cores from the vicinity of each excavation were examined and logged in detail, with particular attention being devoted to an evaluation of the number of joint sets and the condition of the joint surfaces. The other parameters relevant to the two classification systems were also evaluated as far as possible from the borecores. The rock mass at each site was then classified according to the method outlined for each classification system, using these results.

The intersection points of the boreholes were then visited underground and by means of detail line joint surveys and visual estimates the various classification parameters for the two systems were again assessed. The rock mass at each site was classified again using the in situ data. The support measures adopted at each site underground were also recorded.

For evaluation purposes it was assumed that the in situ classifications for each system were truly representative of the rock mass conditions at each site. The in situ results were compared with the borehole

/results .....

results in order to establish the reliability of the latter. Comparisons were also made between the borehole and in situ ratings of individual parameters in order to explain any discrepancies noted between the borehole and in situ classifications. Comparisons between the support predictions in each case were also undertaken. Lastly the results from each classification system were compared.

1.3 The Development of Engineering Rock Mass Classification Systems

For application to engineering, rock mass classification systems should not only express the relevant characteristics of the rock mass in meaningful terminology, but should also relate to actual engineering problems. In underground excavations, the stability of the rock mass in terms of its stand-up time, the excavation dimensions, and its behaviour with respect to installed supports, is a major consideration. Those characteristics of the rock mass which influence the excavation method and equipment to be used are also important to the design engineer and contractor.

Before discussing the development of rock mass classification systems, it is pertinent to differentiate between the terms 'rock material' and 'rock mass'. The former refers to the continuous body of rock between joints and faults etc. which consists of one lithological unit[4]. The latter refers to an entire structural unit of rock including any joints or faults which may be present and can include different lithologies. In an underground excavation the behaviour of the rock mass is more important than the rock material, but it is nevertheless of interest to briefly examine some aspects of rock material classification in order to illustrate different approaches which are pertinent to general concepts of classifying rock masses.

1.3.1 Rock Material Classification

One of the basic elements of any geological rock classification is the 'rock type'. A purely geological description of a rock material will usually include reference to its colour, texture and mineralogy, and will be adequate to classify it into its relevant lithological group. Different lithologies often display similar mechanical characteristics and it has been possible to group them accordingly. An example of this type of lithologic grouping is that proposed by Handin[5], Table 1.1.

Classification of rock material in terms of mechanical properties

/alone .....

TABLE 1.1

LITHOLOGICAL CLASSIFICATION OF ROCK

Type	Rock Material
1	Unfoliated igneous and metamorphic rocks, quartzite, highly silicacemented sandstone;
2	Slate and highly indurated shale;
3	Dolomite;
4	Moderately well cemented sandstone;
5	Limestone;
6	Schist, shale, mudstone, and poorly indurated siltstone;
7	Salt and gypsum.

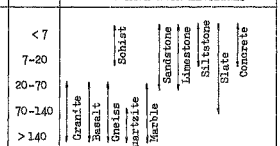
After: Handin, 1966

TABLE 1.2

CLASSIFICATION OF ROCK MATERIALS

BASED ON UNCONFINED COMPRESSIVE STRENGTH

Uniaxial compressive strength			Ranges for some common rock materials
Term	Symbols	MPa	
Very weak	VW	< 7	
Weak	W	7-20	
Medium strong	MS	20-70	
Strong	S	70-140	
Very strong	VS	> 140	



After: Stapledon, 1968

alone, such as the uniaxial compressive strength and the modulus of elasticity[6] have been developed, but a combination of the lithologic and mechanical classification., as suggested by Stapledon[7], is probably the most useful, Table 1.2. Discontinuities between the rock material can also be classified, in either descriptive terminology[8], or quantitatively as suggested by Goodman[9]. This latter classification is based on stress-displacement characteristics for different types of discontinuity surfaces, Table 1.3 and Fig. 1.1.

In terms of their application to real problems these classifications are of limited practical value. However, they provide a better understanding of the individual parameters which influence the behaviour of the rock mass as a whole and are combined, together with other relevant parameters, for example the ground water conditions and number of joint sets, to form the bases for rock mass classification systems.

1.3.2 Rock Mass Classification

So many different methods have been developed for classifying rock masses for engineering purposes that it is impossible to describe them all. Only the more important systems, which illustrate the development of classification systems for underground excavations are therefore discussed here.

In 1946, Terzaghi[10] proposed a system of rock mass classification based on rock loads expected to act on steel supports in tunnels. He used the term 'rock load' to indicate the height of the rock mass which would tend to drop out of the roof of the excavation if left unsupported, and expressed it in terms of the tunnel width (B) and height (H). He described seven rock conditions, Table 1.4, and estimated the rock loads appropriate to each, Table 1.5. The rock conditions were based on the structural integrity and deformation characteristics of the rock mass. Lithological differences were not considered except where implied by structural differences. Generalized support recommendations were also given for each rock condition.

Although of limited application to modern tunnelling, where more use is made of shotcrete and rockbolts for support, the concept of relating observed rock phenomena to then practised technology has been adopted by many classification systems since. A major criticism of the system was made by Coates[11] and has been repeated by many authors without comment. The basis of the criticism was that the system gave different rock classifications for the same rock in different size tunnels, even

/though .....

TABLE 1.3

CLASSIFICATION OF DISCONTINUITIES

Type	Type of Surfaces	Peak *Displacement* mm	Tangential Stiffness MPa/m	Ratio: Peak Strength to Residual Strength
1	Healed joints and incipient joints	0,3	$5,9 \times 10^3$	3,0
2	Clean smooth fractures			
	polished	<0,1	5,9x10	1,1
	unpolished (rough saw cut)	0,2	1,3x10	1,2
3	Clean, rough fractures			
	disturbed samples	2,0	1,3x10	1,75
	undisturbed samples	8,0	1,0x10	1,75
4	Filled joints, sheared zones, shale partings, and smooth bedding			
	dry or slightly moist	1,2	2,2x10	1,5
	wet; thin seam	2,6	1,5x10	0,6
	wet; thick seam	3,0	8,0	1,0

After: Goodman, 1970

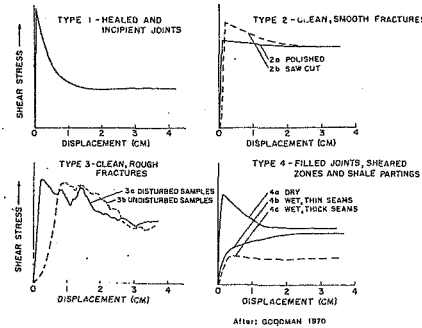


FIG. 1.1 Typical Stress-displacement Relationships in Shear for Various Types of Discontinuities

TABLE 1.4

ROCK MASS CLASSIFICATION FOR TUNNELLING

Intact rock contains neither joints nor hair cracks. Hence if it breaks it breaks across sound rock. On account of the injury to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as spalling condition. Hard, intact rocks may also be encountered in the popping conditions involving the spontaneous and violent detachment of rock slabs from sides or roof.

Stratified rock consists of individual strata with little or no resistance against separation along the boundaries between strata. The strata may or may not be weakened by transverse joints. In such rock, the spalling condition is quite common.

Moderately jointed rock contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. In rocks of this type both the spalling and the popping condition may be encountered.

Blocky and seamy rock consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock vertical walls may require support.

Crushed but chemically intact rock has the character of a crusher run. If most or all of the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.

Squeezing rock slowly advances into the tunnel without perceptible volume increase. Prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or of clay minerals with a low swelling capacity.

Swelling rock advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks which contain clay minerals such as montmorillonite, with a high swelling capacity.

In practice there are no sharp boundaries between these rock categories and the properties of the rocks indicated by each one of these terms can vary between wide limits.

After: Terzaghi, 1946

TABLE 1.5

ROCK MASS CLASSIFICATION FOR TUNNELLING

	Rock condition	Rock load P* (metres)	Remarks
1.	Hard and intact	zero	Light lining, required only if spalling or popping occurs
2.	Hard stratified or schistose	0 to 0,5B	Light support
3.	Massive, moderately jointed	0 to 0,25B	Load change erratically from point to point
4.	Moderately blocky and seamy	0,25B to 0,35(B+H)	No side pressure
5.	Very blocky and seamy	(0,35 to 1,10) (B+H)	Little or no side pressure
6.	Completely crushed but chemically intact	1,10 (B+H)	Considerable side pressure. Softening effect of seepage towards bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs.
7.	Squeezing rock, moderate depth	(1,10 to 2,10) (B+H)	Heavy side pressure, invert struts required.
8.	Squeezing rock, great depth	(2,10 to 4,50) (B+H)	Circular ribs are recommended.
9.	Swelling rock	Up to 80m irrespective of value of (B+H)	Circular ribs required. In extreme cases use yielding support.

After: Terzaghi, 1946

*Rock load P is given as a function of the tunnel width B and tunnel height H, both in metres, at a depth of more than 1,5 (B+H). The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 can be reduced by fifty per cent.

though a smaller tunnel might require no support, while a larger one might need support. It was Coates' contention that the same rock should have the same classification regardless of the nature of the engineering structure.

The present author agrees with this contention but feels that Terzaghi's classification largely fulfilled this requirement. For a particular rock condition, it is the rock load which varies according to the excavation dimensions; the rock condition does not vary appreciably no matter what the excavation size. No allowance is, however, made for *different support requirements in different size tunnels with similar* rock conditions, and it is possibly this aspect to which Coates was referring.

The concept of a bridge action period, or, the time which the unsupported section of the tunnel is stable without support, was apparently first suggested by Rabcewicz[12]. This idea was extended further by Lauffer[13] who introduced the criterion of the effective span of the unsupported rock mass. Lauffer defined seven rock types (designated A-G), using similar structural characteristics to Terzaghi, Table 1.6. For each rock type he gave the expected bridge action period and effective unsupported span together with support recommendations, Table 1.7. The classification system was also presented graphically, Fig. 1.2. The supports covered by Lauffer include shotcrete, rockbolting and both steel and timber, making the classification system more applicable to modern tunnelling methods. However, it remains very empirical in its approach. Bieniawski's[14] criticism of the system was based on the fact that the bridge action period and effective unsupported span are difficult to establish. It appears to the present writer, however, that it was not intended that these parameters should be measured, and that they were given, rather, as *typical values representative of each rock type.* Lauffer's classification system was modified by Underwood[15] to include typical Unconfined Compressive Strength values for each rock type together with their anticipated Rock Quality Designation.

The Rock Quality Designation (RQD) was developed by Deere[16] for describing the rock quality from bore cores. It is a modified core recovery and is given by the per cent of NX diameter core recovered in 10 centimetre, or greater, lengths. The rock quality classification proposed by Deere using the RQD is presented in Table 1.8.

/Table 1.6 .....

TABLE 1.6

ROCK DESIGNATION FOR ROCK MASS CLASSIFICATION

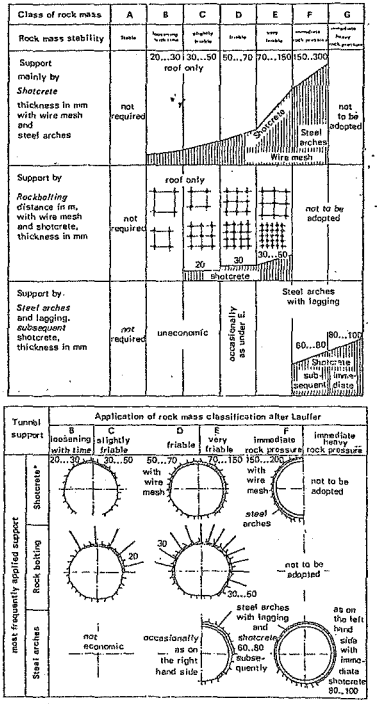
Rock Type	Rock Quality
A	Solid, high strength, widely spaced joints
B	Massive, medium strength
C	Foliated or stratified, high strength moderately jointed
D	Blocky, medium strength, closely jointed
E	Blocky and seamy, low strength, closely bedded and jointed
F	Very blocky and seamy, low strength
G	Squeezing and swelling, very low strength, highly fractured and sheared

After: Lauffer, 1960

TABLE 1.7: ROCK MASS CLASSIFICATION FOR ROCK TUNNELLING DEPENDING ON SELECTED FACTORS

Designation	Class of rock and support measures (timber only)	Bridge action period and effective unsupported span	Shotcrete	Rock bolting	Steel supports remaining in the permanent lining
A	Stable	20 years 4,0 m	not required	not required	not required
B	Loosening with time(overhead protection)	6 months 4,0 m	thickness 2 to 3 cm, only in the arch	spacing 1,5 to 2 m, combined with wire mesh, only in the arch	uneconomic
C	Slightly friable (roof support)	1 week 3,0 m	thickness 3 to 5 cm, only in the arch	spaced at 1,0 to 1,5 m, only in the arch, applying either wire mesh or subsequent shotcreting in a thickness of 2 cm	uneconomic
D	Friable (light timber support)	5 hours 1,5 m	thickness 5 to 7 cm, combined with wire mesh, mainly in the arch	spaced at 0,7 to 1,0 m, mainly in the arch, applying both wire mesh and subsequent shotcreting in a thickness of 3 cm	occasionally, in the same way as under E
E	Very friable (medium-heavy timber support)	20 minutes 0,8 m	thickness 7 to 15 cm, combined with wire mesh	spacing 0,5 to 1,2 m; applicable only if rock bolting possible at all, and after installation of temporary roof support	steel or concrete lagging on steel arches
F	Immediate rock pressure (forepoling without face support)	2 minutes 0,4 m	thickness of 15 to 20 cm, combined with wire mesh and steel arches (if required also the face is shotcreted)	not to be adopted	steel lagging on strutted steel arches and subsequent shotcreting
G	Immediate heavy rock pressure (forepoling with face support)	10 seconds 0,15 m	not to be adopted	not to be adopted	steel lagging on strutted steel arches and immediate shotcreting

After: Lauffer, 1960



* *thickness in mm*

After: Lauffer

FIG. 1.2 The Application of Rock Bolting, Shotcrete and Steel Arches as Tunnel Support

TABLE 1.8

ROCK QUALITY CLASSIFICATION

Rock Quality	RQD (per cent)	Approximate Tunnelman's Classification
Excellent	90 - 100	Intact
Good	75 - 90	Massive, moderately jointed
Fair	50 - 75	Blocky and seamy
Poor	25 - 50	Shattered, very blocky and seamy
Very poor	0 - 25	Crushed

After: Deere, 1968

The RQD was subsequently used by Deere et al[17] as a basis to provide guidelines for the selection of support systems for 20 ft (6,1 m) to 40 ft (12,2 m) diameter rock tunnels, Table 1.9. Six rock qualities were used, for which suggested construction methods, rock loads, and alternative support systems were given. The guidelines given by Deere et al, were based on the experience of those authors and on the results of field measurements. They reflect United States tunnelling technology in 1969[17]. Although the number of rock categories differ for this and Lauffer's classification system, it is interesting to note the general similarities in rockbolt spacing and shotcrete thickness given by each.

The RQD is a rapid and easy method of evaluating the rock mass quality from borecores, and has become internationally accepted as such. Unfortunately it is very sensitive to the drilling quality, and if this is poor it could give completely erroneous results. Its use as a basis for evaluating support requirements in underground excavations introduced the idea of using site investigation data for this purpose.

A different approach to rock classification for tunnelling was proposed by Ikeda[18] from his studies on about seventy Japanese tunnels. He outlined seven principal factors to be considered for the classification of rock conditions, viz.:

1. Rock Quality
2. Degree of crushing
3. Weathering state

/4. .....

TABLE 1.9

GUIDELINES FOR SELECTION OF PRIMARY SUPPORT FOR 6 M TO 12 M TUNNELS IN ROCK

Rock Quality	Construction Method	Alternative Support Systems: Steel Sets: Rock Load (ft = Tunnel Width)	Steel Sets: Weight of Sets	Steel Sets: Spacing[c]	Rock Bolts[a] (Conditional use in poor and very poor rock): Spacing of Pattern Bolts	Rock Bolts[a] (Conditional use in poor and very poor rock): Additional Requirements and Anchorage Limitations[a]	Shotcrete[b] (Conditional use in poor and very poor rock): Total Thickness: Crown	Shotcrete[b] (Conditional use in poor and very poor rock): Total Thickness: Sides	Shotcrete[b] (Conditional use in poor and very poor rock): Additional Support[b]
Excellent[d] RQD > 90	Boring machine	(0.0 to 0.2)B	Light	None to occasional	None to occasional	Rare	None to occasional local application	None	None
	Drilling and blasting	(0.0 to 0.2)B	Light	None to occasional	None to occasional	Rare	None to occasional local application 2 to 3 in.	None	None
Good[d] RQD = 75 to 90	Boring machine	(0.0 to 0.4)B	Light	Occasional to 5 to 6 ft	Occasional to 5 to 6 ft	Occasional mesh and straps	Local application 2 to 3 in.	None	None
	Drilling and blasting	(0.2 to 0.6)B	Light	5 to 6 ft	5 to 6 ft	Occasional mesh or straps	Local application 2 to 3 in.	None	None
Fair RQD = 50 to 75	Boring machine	(0.4 to 1.0)B	Light to medium	5 to 6 ft	4 to 6 ft	Mesh and straps as required	2 to 4 in.	None	Provide for rock bolts
	Drilling and blasting	(0.6 to 1.3)B	Light to medium	4 to 5 ft	5 to 6 ft	Mesh and straps as required	4 in. or more	4 in. or more	Provide for rock bolts
Poor RQD = 25 to 50	Boring machine	(1.0 to 1.6)B	Medium circular	3 to 4 ft	3 to 5 ft	Anchorage may be hard to obtain. Consider-able mesh and straps required.	4 to 6 in.	4 to 6 in.	Rock bolts as re-quired (~4-6 ft cr.)
	Drilling and blasting	(1.3 to 2.0)B	Medium to heavy circular	2 to 4 ft	2 to 4 ft	Anchorage may be hard to obtain. Consider-able mesh and straps required.	6 in. or more	6 in. or more	Rock bolts as required (~4-6 ft cr.)
Very poor RQD < 25 (Excluding squeezing and swelling ground)	Boring machine	(1.6 to 2.3)B	Medium to heavy circular	2 ft	2 to 4 ft	Anchorage may be im-possible. 100 percent mesh and straps re-quired.	6 in. or more on whole section		Medium sets as re-quired
	Drilling and blast-ing	(2.0 to 2.8)B	Heavy circular	2 ft	3 ft	Anchorage may be im-possible. 100 percent mesh and straps re-quired.	6 in. or more on whole section		Medium to heavy sets as re-quired
Very poor, squeezing or swelling ground	Both methods	up to 250 ft	Very heavy circular	2 ft	2 to 3 ft	Anchorage may be im-possible. 100 per-cent mesh and straps required.	6 in. or more on whole section		Heavy sets as re-quired

Note: Table reflects 1969 technology in the United States. Groundwater conditions and the details of jointing and weathering should be considered in conjunction with these guidelines particularly in the poorer quality rock. See Deere et al. [2] for discussion of use and limitations of the guidelines for specific situations.

[a]Bolt diameter = 1 in., length = ⅓ to ½ tunnel width. It may be difficult or impossible to obtain anchorage with mechanically anchored rock bolts in poor and very poor rock. Grouted anchors may also be unsatisfactory in very wet tunnels.

[b]Because shotcrete experience is limited, only general guidelines are given for support in the poorer quality rock.

[c]Lagging requirements for steel sets will usually be minimal in excellent rock and will range from up to 25 percent in good rock to 100 percent in very poor rock.

[d]In good and excellent quality rock, the support requirement will in general be minimal but will be dependent on final geometry, tunnel direction, and relative orientations of joints and tunnel.

After: Deere et al, 1970

4. Physical properties of the rock
5. Expansiveness of the rock
6. Water inflow
7. Depth below ground surface.

Six different classes of rock quality were defined, with each class consisting of lithologies which Ikeda considered to be equivalent as structural materials. Rock conditions were defined for each rock quality using the different velocities of elastic waves in the rock. Modifications were possible to account for ground water inflow and weathering. Suggested excavation methods, powder quantities, steel arch spacings and concrete lining thickness were provided together with comments on the suitability of Rock Tunnel Boring Machines for each rock condition.

Although this system is undoubtedly relevant to the conditions from which it was developed, it does not seem to have much general application. The rock qualities, which form the basis of the system, appear to be related to local geological conditions and are not necessarily relevant elsewhere. This approach, however, displays an extremely good understanding of the local engineering geology and should perhaps serve as an example to engineering geologists elsewhere. The use of elastic wave velocity as a method for describing the rock condition is interesting and is used, together with other physical properties (i.e. density, Poisson's number, shear strength, tensile strength and friction angle, etc.) to calculate the powder quantity, and the stresses acting on the tunnel supports under plastic and competent rock conditions. A similar approach was used by Leeney[19] for rock mass classification from other road and rail tunnels in Japan.

The Rock Structure Rating (RSR) and Rib Ratio (RR) ground support prediction model advanced by Wickham et al[20] possibly represents the first major attempt to standardise rock mass classifications. Initially it was based on studies of thirty-three tunnels but was subsequently updated with the inclusion of data from twenty additional projects[21].

The factors affecting the rock structure were grouped into two general categories, viz. geologic and construction. In the former, the rock type, discontinuities, joint pattern and ground water were considered and in the latter the size of tunnel, drive direction and excavation method were included. Three parameters (A, B and C) were defined from combinations of these factors. For each parameter, weighted values were assigned to each set of factors representing a particular rock condition, Table 1.10. These parameters and weighting values were selected

/after .....

TABLE 1.10

RSR CLASSIFICATION SYSTEM: ROCK STRUCTURE RATING TABLES

Rock structure rating. PARAMETER "A". General Area Geology. Max. value 30

Basic Rock Type					Geological Structure			
	Hard	*Med.*	*Soft*	*De-comp*	Massive	Slightly faulted or folded	Moderately faulted or folded	Intensely faulted or folded
Igneous	1	2	3	4				
Metamorphic	1	2	3	4				
Sedimentary	2	3	4	4				
Type 1					30	22	15	9
Type 2					27	20	13	8
Type 3					24	18	12	7
Type 4					19	15	10	6

Rock Structure Rating. PARAMETER "B". Joint Pattern. Direction of Drive. Max value 45



	Strike ⊥ to axis					Strike ∥ to axis		
	Direction of drive					Direction of drive		
	Both	With dip		Against dip		*Both*		
	Dip of prominent joints					Dip of prominent joints		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
1 Very closely jointed	9	11	13	10	12	9	9	7
2 Closely jointed	13	16	19	15	17	14	14	11
3 *Moderately jointed*	23	24	28	19	22	23	23	19
4 Moderate to blocky	30	32	36	25	28	30	28	24
5 *Blocky to massive*	36	38	40	33	35	36	34	28
6 Massive	40	43	45	37	40	40	38	34

Notes: Flat 0–20°; Dipping 20°–50°; Vertical 50°–90°.

Rock Structure Rating. PARAMETER "C". Ground water. Joint Condition. Max. value 25

Anticipated water inflow (GPM/1000ft)	Sum of parameters A+B					
	13–44			45–75		
	Joint condition					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight (<200 gpm)	19	15	9	23	19	14
Moderate (200–1000 gpm)	15	11	7	21	16	12
Heavy (>1000 gpm)	10	8	6	18	14	10

Joint Condition: Good – Tight or Cemented; Fair - Slightly Weathered or Altered. Poor – Severely Weathered, Altered, or Open. After: Wickham et al, 1976

after consideration of the typical geological information available in the pre-construction period; types of geological investigations used and reliability of developed data; most important geologic factors to be considered with respect to effect on rock structure; methods of measuring the qualitative and quantitative properties of each factor; and the relative effect on ground support requirements. Only the most recent values for each parameter[21] are presented. The RSR-value for a particular rock mass condition is the numerical sum of parameters A, B and C with increasing values reflecting increasing rock competency with respect to its need for structural support.

The Rib Ratio (RR) concept was developed in order to correlate RSR-values with actual support installations. A datum condition was derived from Terzaghi's[10] formula for determining roof loads in loose sand below the water table, and using tables presented by Proctor and White[22], theoretical rib spacings were determined for it. The ratio of the theoretical datum rib spacing to the actual rib spacing, multiplied by 100, is called the Rib Ratio. The RSR/RR relationship presented in Table 1.11, is based on nearly 200 different sections of fifty-three tunnels[21] as are the correlations between RSR, tunnel size and rock load, Table 1.12, and RSR, tunnel size and rib spacing for different rib sizes, Table 1.13.

The majority of the tunnels studied for developing the RSR classification system were supported with steel ribs. Insufficient data was available for reasonable correlations to be made between RSR-values and rockbolt spacing or shotcrete thickness. Empirical relationships based on available data, were suggested however. These are presented in Fig. 1.3, together with rib spacings for different rib sizes. An RSR adjustment factor was also suggested for machine bored tunnels, Fig. 1.4.

The RSR rock mass classification system represents an important advance because it introduces the idea of assigning weighted values to various relevant parameters, in order to obtain a total rating which describes the rock mass condition in terms of its support requirement. It lacks flexibility, however, in that it is developed essentially from steel rib supported tunnels only and is thus difficult to relate to modern support methods. The descriptions used for the rock type, geological structure and joint conditions are very subjective. This is not conducive to obtaining consistent results. In general terms, however, it was found[21] that predicted supports correlated reasonably well with actual installed supports. It has been suggested[23] that rock type descriptions used in future developments of the RSR concept

/should .....

TABLE 1.11

RSR VALUES AND RIB RATIOS

RSR	19	25	30	35	40	45	50	55	60	65	70	75	80
RR.	100	80	67	55	46	37	30	24	18	13	8	4	0

After: Wickham et al, 1976

TABLE 1.12

ROCK STRUCTURE RATING, ROCK LOAD AND TUNNEL DIAMETER

Tunnel Diameter (D) (ft)	*(Wr) Rock load on tunnel arch (Klf)*											
	0·5	1·0	1·5	2·0	3·0	4·0	5·0	6·0	7·0	8·0	9·0	10·0
	Corresponding values of rock structure ratings (RSR)											
10	62·5	49·9	40·2	32·7	21·6	13·8						
12	63·0	53·7	44·7	37·5	26·6	18·7						
14	66·9	56·6	48·3	41·4	30·8	22·9	16·8					
16	68·3	59·0	51·2	44·7	34·6	26·6	20·4	15·5				
18	69·5	61·0	53·7	47·6	37·6	29·9	23·8	18·8				
20	70·4	62·5	55	49·9	40·2	32·7	26·6	21·6	17·4			
22	71·3	63·9	57·5	51·9	42·7	35·3	29·3	24·3	20·1	16·4		
24	72·0	65·0	59·0	53·7	44·7	37·5	31·5	26·6	22·3	18·7		
26	72·6	66·1	60·3	55·3	46·7	39·6	33·8	28·8	24·6	20·9	17·7	
28	73·0	66·9	61·5	56·6	48·3	41·4	35·7	30·8	26·6	22·9	19·7	16·8
30	73·4	67·7	62·4	57·8	49·8	43·1	37·4	32·6	28·4	24·7	21·5	18·6

After: Wickham et al, 1976

TABLE 1.13

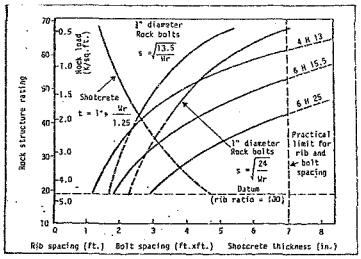
RIB SPACING (FEET) BASED ON RSR AND TUNNEL DIAMETER

RR	RSR	*10 ft diameter*			*12 ft diameter*		
		417·7	4H13	6H15·5	4H13	6H15·5	6H20
100	19	1·16	2·01	3·19	1·51	2·37	3·02
80	25	1·45	2·51	3·99	1·89	2·96	3·78
67	30	1·73	3·00	4·76	2·25	3·54	4·51
55	35	2·11	3·65	5·80	2·75	4·31	5·49
46	40	2·52	4·37	6·93	3·28	5·15	6·57
37	45	3·14	5·43	8·62	4·08	6·41	8·16
30	50	3·87	6·70		5·03	7·90	
24	55	4·82	8·38		6·29		
18	60	6·44			8·39		
13	65	8·92					

RR	RSR	*14 ft diameter*			*16 ft diameter*		
		4H13	6H15·5	6H25	6H15·5	6H25	8W31
100	19	1·16	1·81	2·86	1·42	2·25	3·24
80	25	1·45	2·26	3·58	1·78	2·81	4·05
67	30	1·73	2·70	4·27	2·12	3·36	4·84
55	35	2·11	3·29	5·20	2·58	4·09	5·89
46	40	2·52	3·93	6·22	3·09	4·89	7·04
37	45	3·14	4·89	7·73	3·84	6·08	9·76
30	50	3·87	6·03	9·53	4·73	7·50	
24	55	4·83	7·54		5·92	9·38	
18	60	6·44	10·05		7·89		
13	65	8·92			10·92		

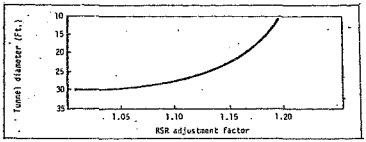
RR	RSR	*18 ft diameter*			*20 ft diameter*		
		6H15·5	6H25	8W40	6H20	8W31	8W48
100	19	1·14	1·81	3·37	1·20	2·14	3·34
80	25	1·42	2·26	4·21	1·50	2·68	4·18
67	30	1·70	2·70	5·03	1·79	3·19	4·99
55	35	2·07	3·29	6·13	2·18	3·89	6·07
46	40	2·47	3·93	7·33	2·61	4·65	7·26
37	45	3·08	4·89	9·11	3·24	5·78	9·03
30	50	3·80	6·03		4·00	7·13	
24	55	4·75	7·54		5·00	8·91	
18	60	6·33	10·05		6·67		
13	65	8·77			9·23		

After: Wickham et al, 1976



After: Wickham

FIG. 1.3 Support Requirement Chart for a 4,2 m Diameter Tunnel



After: Wickham

FIG. 1.4 RSR Adjustment for Tunnel Boring Machine Operation

should relate to commonly used strength terminology.

The Geomechanics and Norwegian Geotechnical Institute's (N.G.I.) rock mass classification systems both represent further advances in the concepts already discussed. Although both systems were developed independently, they display some basic similarities. The Geomechanics System represents a synthesis of previous ideas, while the N.G.I. System, although drawing on previous concepts, is quite unique as a rock mass classification system. Both these systems are discussed in more detail in the following sections.

1.4 The Geomechanics System

The Geomechanics Classification System of Jointed Rock Masses was first proposed by Bieniawski in 1973[14]. The main purpose of the System was to combine the best features of the various classification systems which had been developed up to that time in order to "ensure effective communication between the engineer and the geologist and between the designer and the contractor." The Geomechanics System follows the same approach as that developed by Wickham et al[20, 21] in that it is based on weighted parameters which are assigned rating values related to their respective conditions. These values are totalled to give a Rock Mass Rating (RMR) for the rock mass as a whole. After its initial inception, several modifications were made to the Geomechanics Classification System[24, 25]. These included changes to the number of parameters used, changes to the rating values for the individual parameters, and changes to the RMR-values defining the various rock classes.

The requirements of a rock mass classification system were defined by Bieniawski[14] as follows:

i) It should divide a rock mass into groups of similar behaviour;

ii) it should provide a good basis for understanding the characteristics of the rock mass;

iii) it should facilitate the planning and design of structures in rock by yielding quantitative data required for the solution of real engineering problems;

/iv) .....

iv) it should provide a common basis for effective communication among all persons concerned with a geomechanics problem.

Based on these requirements and on the premise that the design of an engineering structure in rock requires prior site investigation, it was decided[14] that the classification parameters should be obtainable during the site exploration. The following parameters were therefore incorporated into the system:

1. Rock Quality Designation (RQD);
2. State of weathering;
3. Uniaxial Compressive Strength of intact rock;
4. Spacing of joints or bedding;
5. Strike and dip orientations;
6. Separation of joints;
7. Continuity of joints;
8. Ground water inflow.

Each of these parameters was defined in categories related to the general engineering quality of the rock. Parameter 2, the state of weathering of the rock, was subsequently combined with Parameter 3, the uniaxial compressive strength, to give a parameter called the 'strength of intact rock material.' The weathering parameter was retained on the Input Data Form, Table 1.14, specially designed for use with the Geomechanics System because it was considered to be a useful geological identification parameter[24]. The parameters used to describe the separation and continuity of joints were also combined into one parameter, the 'condition of joints', which also included an assessment of the joint roughness and the quality of the joint wall rock. The parameters finally selected are described below.

Rock Quality Designation (RQD)

The relationship between the numerical RQD values and the rock quality, as proposed by Deere[16], was used (refer Table 1.8). Bieniawski[14] comments that the limitations of the RQD are that it disregards the influence of joint condition, continuity and gouge. In the first two cases this is undoubtedly correct, but it is felt by the present writer that the third case, i.e. gouge, is indirectly accounted for in terms of

/core .....

TABLE 1.14

GEOMECHANICS CLASSIFICATION SYSTEM: INPUT DATA FORM

Name of project:

Site of survey: STRUCTURAL REGION

Conducted by:

Date:

ROCK TYPE AND ORIGIN

DRILL CORE QUALITY R.Q.D.

Very good quality: 90 - 100%

Good quality: 75 - 90%

Fair quality: 50 - 75%

Poor quality: 25 - 50%

Very poor quality: <25%

NOTE: R.Q.D. = Rock Quality Designation in accordance with the method of Deere

WEATHERING

Unweathered

Slightly weathered

Moderately weathered

Highly weathered

Completely weathered

GROUND WATER

INFLOW per 10 m of tunnel length: .....litres/minute

or

WATER PRESSURE MPa .....

or

GENERAL CONDITIONS (completely dry, moist only, water under pressure, severe problems)

.....

STRENGTH OF INTACT ROCK MATERIAL

Designation	Uniaxial compressive strength	Point-load strength index
Very high:	Over 200 MPa	>8 MPa
High:	100 - 200 MPa	4 - 8 MPa
Medium	50 - 100 MPa	2 - 4 MPa
	25 - 50 MPa	1 - 2 MPa
Very low:	10 - 25 MPa	<1 MPa
	3 - 10 MPa	
	1 - 3 MPa	

SPACING OF JOINTS

	Set 1	Set 2	Set 3	Set 4
Very wide:	Over 3 m			
Wide:	1 - 3 m			
Moderately close:	0.3 - 1 m			
Close:	50 - 300 mm			
Very close:	<50 mm			

NOTE: These values are obtained from a joint survey and not from borehole logs.

Provide data for each joint set.

STRIKE AND DIP ORIENTATIONS

Set 1 Strike: ... (average) (from ..... to .....) Dip: ... (weight) (direction)

Set 2 Strike: ... (from ..... to .....) Dip:

Set 3 Strike: ... (from ..... to .....) Dip:

Set 4 Strike: ... (from ..... to .....) Dip:

NOTE: Provide data for each joint set, Refer all directions to magnetic north

CONDITION OF JOINTS

	Set 1	Set 2	Set 3	Set 4
CONTINUITY				
Not continuous, no gouge				
with gouge				
Continuous, no gouge				
with gouge				
SEPARATION				
Very tight joints: Less than 0,1 mm				
Tight joints: 0,1 - 1 mm				
Moderately open joints: 1 - 5 mm				
Open joints: More than 5 mm				
ROUGHNESS				
Very rough surfaces:				
Rough surfaces:				
Slightly rough surfaces:				
Smooth surfaces				
Slickensided surfaces:				
JOINT WALL ROCK				
Hard rock				
Medium hard rock				
Soft rock				

NOTE: Provide data for each joint set.

MAJOR FAULTS OR FOLDS

Describe major faults and folds specifying their locality, nature and orientations.

GENERAL REMARKS AND ADDITIONAL DATA

If gouge is present specify its type, thickness, consistency and consistency.

Describe waviness of joints.

Assess regional stresses.

NOTE: The data on this form constitute the minimum released for engineering design. The geologist should, however, supply any further information which he considers relevant.

After: BIENIAWSKI, 1974

core loss, since the RQD is actually a modified co. e recovery record.

Strength of Intact Rock Material

Bieniawski[14] showed that a number of classifications for rock material strength have been proposed but decided that the one suggested by Deere and Miller[6] was most realistic and convenient. After slight modifications, in order to conform to the SI metric system of units[14], and after taking cognizance of further subdivisions for the low strength (< 25 MPa) rocks suggested by Wilson[26], the following classification was adopted:

TABLE 1.15

STRENGTH CLASSIFICATION OF INTACT ROCK

Uniaxial Compressive Strength (MPa)	Description	Point-load Strength Index (MPa)
> 200	Very High Strength	> 8
100 - 200	High Strength	4 - 8
50 - 100	Medium Strength	2 - 4
25 - 50	Low Strength	1 - 2
10 - 25	Very Low Strength	< 1
3 - 10		
1 - 3		

After: Bieniawski, 1974

Spacing of Joints

The joint spacing classification proposed by Deere[27] was recommended for use in the Geomechanics Classification System, and is reproduced in Table 1.16.

/Table 1.16 .....

TABLE 1.16

CLASSIFICATION FOR JOINT SPACING

Spacing	Description	Rock Mass Grading
> 3 m	Very wide	Solid
1 - 3 m	Wide	Massive
0,3 - 1 m	Moderately close	Blocky/seamy
50 - 300 mm	Close	Fractured
< 50 mm	Very close	Crushed and shattered

After: Deere, 1968

Condition of Joints

The condition of the joints was described with reference to their continuity, roughness, separation and wall rock quality. The continuity was described as either non-continuous or continuous, with or without gouge. Laubscher[28] based his assessment of joint continuity on the joint length relative to the excavation diameter, i.e. "A joint is continuous if its length is, greater than one diameter of the excavation, or three metres."

Joint separation was defined in terms of, 0 mm; <1 mm; 1 - 5 mm; >5 mm, while roughness was, very rough; slightly rough; or slickensided. Joint wall rock was described as either hard or soft. Full descriptions for each category can be seen in Table 1.19.

Ground Water

Ground water conditions were described generally, and in terms of inflow per ten metres of tunnel length. On the suggestion of Pells[29] class divisions related to the pore water pressure, r_u, were also included in this parameter. Bieniawski[24] points out, however, that sufficient information to calculate the pore water pressure is often not available and in such cases the general conditions are found practicable. The

/different .....

different categories are given in Table 1.19.

Joint Orientation

In the initial proposal of the Geomechanics Classification System the effect of strike and dip orientations on the rock mass behaviour were described in terms of either 'favourable' or 'unfavourable'. After comment by various members of the South African Section of the Association of Engineering Geologists[30] this was clarified[24], with slight modifications, along similar lines to those proposed by Wickham et al[20, 21] for tunnelling in rock, Table 1.17.

TABLE 1.17

THE EFFECT OF JOINT STRIKE AND DIP ORIENTATIONS FOR TUNNELLING IN ROCK

Strike Perpendicular to Tunnel Axis				Strike Parallel to Tunnel Axis		Dip 0°-20° (irrespective of strike)
Drive with Dip		Drive Against Dip				
Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	
Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	Fair	Unfavourable

After: Bieniawski, 1974

As already mentioned, the Geomechanics System has been modified in several ways since its initial inception. These modifications are illustrated in Tables 1.18 - 1.21. Bieniawski[14] noted that the ratings assigned to different groups of parameters represented different percentages of the total rating values. The following groups, based on the type or origin of the data, were used:

Parameters 1 and 2 : Data obtained from borecores.
Parameter 3 : Intact Rock Strength.
Parameters 4 and 9 : Field data on Joints.
Parameters 5, 6 and 7 : Combined Ground Water Effect.

Bieniawski also noted that the percentages change as the rock deteriorates

/from .....

TABLE 1.18

PARAMETERS AND RATING VALUES INITIALLY PROPOSED FOR THE GEOMECHANICS CLASSIFICATION SYSTEM

A. CLASSIFICATION PARAMETERS AND INDIVIDUAL RATINGS

Item	Parameter	Class No. and its Description				
		1	2	3	4	5
		Very Good	Good	Fair	Poor	Very Poor
1	RQD%	90-100 16	75-90 14	50-75 12	25-50 7	< 25 3
2	Weathering	Unweathered 9	Slightly Weathered 7	Moderately Weathered 5	Highly weathered 3	Completely Weathered 1
3	Intact rock strength (MPa)	> 200 10	100-200 5	50-100 2	25-50 1	< 25 0
4	Spacing of joints	> 3 m 30	1-3 m 25	0,3-1 m 20	50-300 mm 10	< 50 mm 5
5	Separation of joints	<0,1 mm 5	<0,1 mm 5	0,1-1 mm 4	1-5 mm 3	> 5 mm 1
6	Continuity of joints	Not continuous 5	Not continuous 5	Continuous no gouge 3	Continuous with gouge 0	Continuous with gouge 0
7	Ground water inflow (per 10m of adit)	None 10	None 10	Slight <25 l/min 8	Moderate 25-125 l/min 5	Heavy >125 l/min 2
8	Strike and dip orientation *	Very Favourable 15	Favourable 13	Fair 10	Unfavourable 5	Very Unfavourable 3

* - Ratings for Tunnels only shown.

B. TOTAL RATINGS FOR ROCK MASS CLASSES

Class No.	1	2	3	4	5
Total Rating	100-90	90-70	70-50	50-25	<25

Data from Bieniawski, 1973

TABLE 1.18

PARAMETERS AND RATING VALUES INITIALLY PROPOSED FOR THE GEOMECHANICS CLASSIFICATION SYSTEM

A. CLASSIFICATION PARAMETERS AND INDIVIDUAL RATINGS

Item	Parameter	Class No. and its Description				
		1	2	3	4	5
		Very Good	Good	Fair	Poor	Very Poor
1	RQD%	90-100 16	75-90 14	50-75 12	25-50 7	< 25 3
2	Weathering	Unweathered 9	Slightly Weathered 7	Moderately Weathered 5	Highly Weathered 3	Completely Weathered 1
3	Intact rock strength (MPa)	> 200 10	100-200 5	50-100 2	25-50 1	< 25 0
4	Spacing of joints	> 3 m 30	1-3 m 25	0,3-1 m 20	50-300 mm 10	< 50 mm 5
5	Separation of joints	<0,1 mm 5	<0,1 mm 5	0,1-1 mm 4	1-5 mm 3	> 5 mm 1
6	Continuity of joints	Not continuous 5	Not continuous 5	Continuous no gouge 3	Continuous with gouge 0	Continuous with gouge 0
7	Ground water inflow (per 10m of adit)	None 10	None 10	Slight <25 l/min 8	Moderate 25-125 l/min 5	Heavy >125 l/min 2
8	Strike and dip orientation x	Very Favourable 15	Favourable 13	Fair 10	Unfavourable 5	Very Unfavourable 3

x - Ratings for Tunnels only shown.

B. TOTAL RATINGS FOR ROCK MASS CLASSES

Class No.	1	2	3	4	5
Total Rating	100-90	90-70	70-50	50-25	< 25

Data from Bieniawski, 1973

TABLE 1.19

GEOMECHANICS CLASSIFICATION SYSTEM: REVISED PARAMETERS AND RATING VALUES

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

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

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

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

D. MEANING OF ROCK MASS CLASSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 5 m span	6 months for 4 m span	1 week for 3 m span	5 hours for 1,5 m span	10 minutes for 0.5 m span
Cohesion of the rock mass	> 300 kPa	200 - 300 kPa	150 - 200 kPa	100 - 150 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	40° - 45°	35° - 40°	30° - 35°	< 30°

After: Bieniawski 1974

TABLE 1.20

RATING ALLOCATION INITIALLY PROPOSED FOR THE GEOMECHANICS CLASSIFICATION SYSTEM

Rock Class		1	2	3	4	5
Total Rating		100-90	90-70	70-50	50-25	<25
Parameter Group						
RQD and Weathering	Rating Percent	25 25%	21 25%	17 26,5%	10 29,5%	4 26,5%
Intact Rock Strength	Rating Percent	10 10%	5 6%	2 3%	1 3%	0 0%
Spacing and Orientation	Rating Percent	45 45%	38 45%	30 47%	15 44%	8 53%
Separation, Continuity and Ground Water	Rating Percent	20 20%	20 24%	15 23%	8 23,5%	3 20%

TABLE 1.21

REVISED RATING ALLOCATION FOR THE GEOMECHANICS CLASSIFICATION SYSTEM

Rock Class		1	2	3	4	5
Total Rating		100-81	80-61	60-41	40-21	<20
Parameter Group						
RQD	Rating Percent	20 20%	17 21%	13 24%	8 36%	3 -
Strength of Intact Rock	Rating Percent	15 15%	12 15%	7 13%	4 18%	2 -
Spacing and Orientation	Rating Percent	30 30%	23 28%	15 28%	0 0%	-7 -
Joint Condition and Ground Water	Rating Percent	35 35%	30 36,5%	19 35%	10 45%	0 -

from Class 1 to Class 5.

The percentages of the total rating values first proposed for each group are shown in Table 1.20. The highest percentage has been allocated to the group comprising the Spacing and Orientation parameters, whilst the lowest has been assigned to the Intact Rock Strength. The other two groups have been allocated approximately equ_ percentages of the total ratings.

Allocating the highest percentage rating to the joint spacing and joint orientation data does not seem realistic to the present writer considering that this information cannot always be obtained from in situ field measurements as stipulated and must therefore, in many cases, be estimated from borecores alone. Even if sufficient rock exposure does exist at surface within reasonable proximity of a proposed tunnel, the degree of reliability with which one can extrapolate the joint data to tunnel level is questionable. Although it is realized that these ratings have been developed from case history studies and are thus related to the rock mass behaviour rather than to the methods used to obtain the data, it is nevertheless considered that for a classification system to be useful in practise, this type of problem should, when possible, be accounted for. The ratings for joint spacing apply to rock masses having three joint sets and conservative assessment is therefore obtained when only one or two joint sets are present.

The rating allocations for the revised System are presented in Table 1.21. They are more evenly distributed with the Joint Condition and Ground Water group having the highest allocation. A comparison of the changes made is of some interest.

Initially the Strength parameter in Rock Class 5 had a rating allocation of zero per cent while the Spacing and Orientation group constituted 53 per cent of the total. According to the revised System, the latter group has a Zero or negative rating in Classes 4 and 5 respectively, while the Strength parameter has an allocation of up to 18 per cent. Another point is that the percentage rating for the Strength of Intact Rock Material increases with a decrease in rock strength. This is also contrary to the initially proposed rating allocation, but is consistent with the idea that the rock strength is more significant to the rock mass behaviour in the lower rock strength range. The most important point that these observations illustrate is the way a rock classification system of this type can be updated to bring it into line with current theory and practice.

/The procedure .....

The procedure to be adopted when using the Geomechanics Classification System was outlined by Bieniawski[14] and is summarised below:

1. The engineering geologist divides the rock mass into regions and provides the required input data for each.

2. The rock mass is classified and rating values assigned to the various parameters. These are then added to give the total rating and the rock class is established.

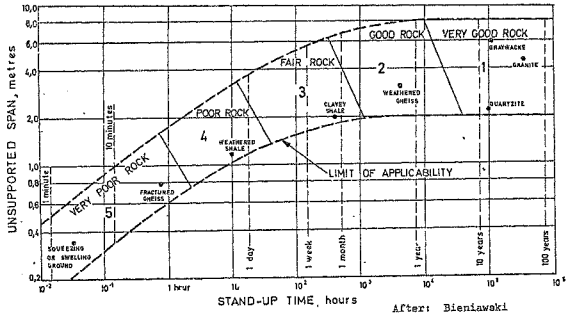
3. The average stand-up time and active unsupported span are determined for the relevant rock mass class.

4. Primary tunnel support is selected.

The initial[14], second[31] and latest[25] proposed relationships between the Unsupported Span, Stand-up Time, RMR-values and Rock Classes are illustrated in Figs. 1.5, 1.6 and 1.7 respectively. In the second proposal the zone of applicability was widened by raising the upper limit to incorporate new data. In the latest relationship, the rock class boundaries are in the same position with respect to the Stand-up Time and Unsupported Span as in the previous proposals, but the RMR-values related to these boundaries are different. Figure 1.7 also shows the relationship for output data from other classification systems. It should be noted that when using this diagram for evaluating the Stand-up Time for a particular rock mass condition, it is not sufficient to use the rock class alone since this would give too wide a variation in the result. The actual RMR-value should be used since, "... for any one value of RMR there is only one given Stand-up Time..."[32]. The approximate ranges of Stand-up Time associated with each Rock Class are listed in Table 1.22.

/Table 1.22 .....

9	Stand-up time	10 years	6 months	1 week	5 hours	10 min
10	Active unsupported span	5m	4m	3m	1,5m	0,5m



After: Bieniawski

FIG. 1.5 Geomechanics Classification System; Initially Proposed Relationship between the Stand-up Time and Unsupported Span

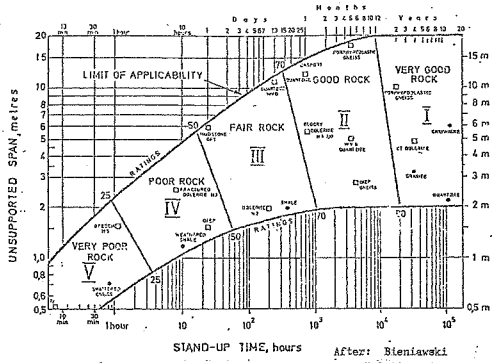


FIG. 1.6 Geomechanics System; Second Proposed Relationship between Unsupported Span and Stand-up Time

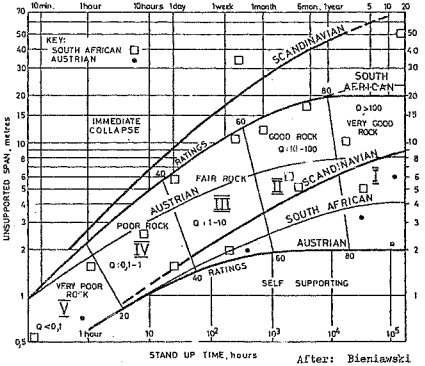


FIG. 1.7 Geomechanics System; Finally Proposed Relationship between Unsupported Span and Stand-up Time

TABLE 1.22

GEOMECHANICS CLASSIFICATION SYSTEM;

STAND-UP TIMES RELEVANT TO EACH ROCK CLASS

Rock Class	Approximate Stand-up Time
Class 1 RMR: 100 - 81	> 2 years
Class 2 RMR: 80 - 61	1 month - 2 years
Class 3 RMR: 60 - 41	2 days - 1 month
Class 4 RMR: 40 - 21	2 hours - 2 days
Class 5 RMR: < 20	< 2 hours

The guidelines for primary support selection, relevant to the different Rock Classes are presented in Tables 1.23 and 1.24. When the Geomechanics System was introduced[14] these guidelines were given as alternative support systems using either rockbolts, shotcrete or steel arches. The most recent recommendations[25] include excavation and support guidelines and for the latter, appear to suggest that combinations of rockbolts, shotcrete and arches may be applicable. The support recommendations are very similar in both cases although some modifications have been made in the latter case. The guidelines are for tunnels of ten metres span but allow for variations in rock bolt lengths for different tunnel diameters, using the criteria of bolt length being a third to half the tunnel diameter[32]. An abbreviated form of the latest support recommendations, Table 1.25, has been compiled for the present study

/using .....

TABLE 1.23

GEOMECHANICS CLASSIFICATION SYSTEM: GUIDELINES FOR THE SELECTION OF PRIMARY TUNNEL SUPPORTS FOR TUNNELS 5-12 M DIAMETER CONSTRUCTED BY DRILL AND BLAST

Rock mass class	Average stand-up time at unsupported span	ALTERNATIVE SUPPORT SYSTEMS						
		ROCKBOLTS*		SHOTCRETE			STEEL SETS	
		Spacing	Additional support	Crown	Sides	Additional support	Type	Spacing
1	10 years 5 m	GENERALLY NOT REQUIRED						
2	6 months 4 m	1,5–2,0 m	Occasional wire mesh in crown	50 mm	Nil	Nil	Uneconomic	
3	1 week 3 m	1,0–1,5 m	Wire mesh, plus 50 mm shotcrete in crown as required	100 mm	50 mm	Occasional wire mesh and rock bolts, if necessary	Light sets	1,5–2,0 m
4	5 hours 1,5 m	0,5–1,0 m	Wire mesh, plus 50–90 mm shotcrete in crown and sides	150 mm	100 mm	Wire mesh and 3 m rockbolts at 1,5 m spacing	Medium sets plus 50 mm shotcrete	0,7–1,5 m
5	10 min 0,5 m	Not recommended		200 mm	150 mm	Wire mesh, rockbolts and light steel sets. Seal face. Close invert.	Heavy sets with lagging, immediately 80 mm shotcrete	0,7 m

*Bolt diameter 25 mm, length ⅓ tunnel width. Resin bonded fully.

After: BIENIAWSKI, 1973

TABLE 1:24

GEOMECHANICS CLASSIFICATION SYSTEM; GUIDE FOR EXCAVATION AND SUPPORT IN ROCK TUNNELS
Shape: Horseshoe; Width: 10m; Vertical Stress: Below 25 MPa; Construction: Drill and Blast

Rock mass class	Excavation	Support		
		Rockbolts (20 mm dia., fully bonded)	Shotcrete	Steel sets
Very good rock I RMR: 81-100	Full face. 3 m advance	Generally no support required except for occasional spot bolting		
Good rock II RMR: 61-80	Full face. 1,0-1,5 m advance Complete support 20 m from face.	Locally bolts in crown 3 m long, spaced 2,5 m with occasional wire mesh.	50 mm in crown where required.	None
Fair rock III RMR: 41-60	Top heading and bench 1,5 - 3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1,5 m - 2 m in crown and walls with wire mesh in crown.	50 - 100 mm in crown and 30 mm in sides.	None
Poor rock IV RMR: 21-40	Top heading and bench 1,0 - 1,5 m advance in top heading. Install support concurrently with excavation - 10 m from face.	Systematic bolts 4 - 5 m long, spaced 1 - 1,5 m in crown and walls with wire mesh.	100 - 150 mm in crown and 100 mm in sides.	Light ribs spaced 1,5 m where required.
Very poor rock V RMR: < 20	Multiple drifts. 0,5 - 1,5 m advance in top heading. Instal support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5 - 6 m long, spaced 1 - 1,5 m in crown and walls with wire mesh. Bolt invert.	150 - 200 mm in crown, 150 mm in sides and 50 mm on face.	Medium to heavy ribs spaced 0,75 m with steel lagging and forepoling if required. Close invert.

After: BIENIAWSKI, 1976

TABLE 1.25

GEOMECHANICS CLASSIFICATION SYSTEM: SUPPORT RECOMMENDATIONS
(Abbreviated from Bieniawski, 1976)

Rock Mass Class	Support	
Very Good Rock 1 RMR: 81-100	Generally no support required except for occasional spot bolting.	
Good Rock 2 RMR: 61-80	B S	2,5 m + clm (occasional) 50 mm (in crown if required)
Fair Rock 3 RMR: 41-60	B S	1,5 - 2 m + clm (crown) 50-100 mm (crown), 30 mm (sides)
Poor Rock 4 RMR: 21-40	B S	1-1,5 m + clm 100-150 mm (crown), 100 mm (sides) 1,5 m (if required)
Very Poor Rock 5 RMR: <20	B S M.R.-H.R.	1-1,5 m + clm (close invert) 150-200 mm (crown), 150 mm (sides), 50 mm (face) 0,75 m + steel lagging

B - Rockbolts (20 mm diam. fully bonded)
clm - Wire mesh
S - Shotcrete
L.R.,M.R.,H.R. - Light, Medium and Heavy Steel sets

using terms similar to the N.G.I. System. This was done to simplify the presentation of results later in the text.

The Geomechanics Classification System applies only to rock engineering[32] and since it was introduced has been applied quite extensively in this country to problems of underground construction. General and detailed comparisons have been made between predicted and encountered rock mass conditions and support requirements. These are discussed later, as is the application of this classification system to problems of design and contract specifications.

1.5 The N.G.I. Rock Mass Classification System

The engineering classification of rock masses for designing tunnel supports, which has become known as the N.G.I. or Q-Classification System, was developed independently of the Geomechanics System by Barton, Lien and Lunde[33] of the Norwegian Geotechnical Institute in Oslo. The classification system was based on the analysis of approximately 200 tunnel case histories which included thirteen igneous rock types, twenty-four metamorphic rock types and nine sedimentary rock types. It describes the rock mass in terms of its quality Q, where Q is calculated from six parameters combined in the following way:

$$Q = \frac{RQD}{Jn} \cdot \frac{Jr}{Ja} \cdot \frac{Jw}{S.R.F.} \quad \ldots\ldots\ldots 1$$

where,

RQD - rock quality designation,
Jn - joint set number,
Jr - joint roughness,
Ja - joint alteration,
Jw - joint water reduction factor,
S.R.F. - stress reduction factor.

Descriptions and ratings for each of these parameters as given by Barton et al[33] are reproduced in Table 1.26.

The parameter ratings are more than just arbitrary scales and are related to the basic properties controlling tunnel stability in rock masses[33]. The quotient, RQD/Jn, is a crude measure of the "block size" or, in other words, the size of the intact sections of rock between joints, while the function $\tan^{-1}(Jr/Ja)^{\circ}$ was found to be an approximation of the

/total .....

TABLE 1.26

N.G.I. CLASSIFICATION SYSTEM: PARAMETERS AND RATING VALUES

1. ROCK QUALITY DESIGNATION (RQD)

A. Very poor	0 - 25
B. Poor	25 - 50
C. Fair	50 - 75
D. Good	75 - 90
E. Excellent	90 - 100

Note: (i) Where RQD is reported or measured as ≤ 10, (including 0) a nominal value of 10 is used to evaluate Q in equation (1).

(ii) RQD intervals of 5, i.e. 100, 95, 90, etc. are sufficiently accurate.

2. JOINT SET NUMBER	(J_n)
A. Massive, no or few joints	0.5-1.0
B. One joint set	2
C. One joint set plus random	3
D. Two joint sets	4
E. Two joint sets plus random	6
F. Three joint sets	9
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

Note: (i) For intersections use ($3.0 \times J_n$)

(ii) For portals use ($2.0 \times J_n$)

3. JOINT ROUGHNESS NUMBER

(a) *Rock wall contact* and (b) *Rock wall contact before 10 cms shear*	(J_r)
A. Discontinuous joints	4
B. Rough or irregular, undulating	3
C. Smooth, undulating	2
D. Slickensided, undulating	1.5
E. Rough or irregular, planar	1.5
F. Smooth, planar	1.0
G. Slickensided, planar	0.5

Note: (i) Descriptions refer to small scale features and intermediate scale features, in that order.

(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

Note: (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.

(iii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are orientated for minimum strength.

After: BARTON et al, 1974

TABLE 1.26
(cont.)

4. JOINT ALTERATION NUMBER	(J_a)	(ϕ_r) (approx.)
(a) *Rock wall contact*		
A. Tightly healed, hard, non-softening, impermeable filling i.e. quartz or epidote	0.75	(-)
B. Unaltered joint walls, surface staining only	1.0	(25-35^0)
C. Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	2.0	(25-30^0)
D. Silty-, or sandy-clay coatings, small clay fraction (non-soft.)	3.0	(20-25^0)
E. Softening or low friction clay mineral coatings, i.e. kaolinite or mica. Also chlorite, talc, gypsum, graphite etc., and small quantities of swelling clays	4.0	(8-16^0)
(b) *Rock wall contact before 10 cms shear*		
F. Sandy particles, clay-free disintegrated rock etc.	4.0	(25-30^0)
G. Strongly over-consolidated non-softening clay mineral fillings (continuous, but <5 mm thickness)	6.0	(16-24^0)
H. Medium or low over-consolidation, softening, clay mineral fillings (continuous but <5 mm thickness)	8.0	(12-16^0)
J. Swelling -clay fillings, i.e. montmorillonite (continuous, but <5 mm thickness). Value of J_a depends on percent of swelling clay-size particles, and access to water etc.	8-12	(6-12^0)
(c) *No rock wall contact when sheared*		
K, Zones or bands of disintegrated		
L, or crushed rock and clay (see		
M. G, H, J for description of clay condition)	6, 8, or 8-12	(6-24^0)
N. Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	5.0	(-)
O, Thick, continuous zones or		
P, bands of clay (see G, H, J for 10, 13,		
R. description of clay condition)	or 13-20	(6-24^0)

5. JOINT WATER REDUCTION FACTOR	(J_w)	Approx. water pres (kg/cm²)
A. Dry excavations or minor inflow, i.e. <5 l/min. locally	1.0	<1
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1-2.5
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5-10
D. Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5-10
E. Exceptionally high inflow or water pressure at blasting, decaying with time	0.2-0.1	>10
F. Exceptionally high inflow or water pressure continuing without noticeable decay	0.1-0.05	>10

Note: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.

(ii) Special problems caused by ice formation are not considered.

TABLE 1.26
(cont.)

6. STRESS REDUCTION FACTOR

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

A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth) 10

B. Single weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤ 50 m) 5

C. Single weakness zones containing clay or chemically disintegrated rock (depth of excavation >50 m) 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 ≤50 m) .. 5.0

F. Single shear zones in competent rock (clay-free) (depth of excavation > 50 m) .. 2.5

G. Loose open joints, heavily jointed or "sugar cube" etc. (any depth) ..... 5.0.

Note: (i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.

(b) *Competent rock, rock stress problems*

	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-.33	0.5-2
L. Mild rock burst (massive rock)	5-2.5	0.33-.16	5-10
M. Heavy rock burst (massive rock)	<2.5	<0.16	10-20

Note: (ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c and σ_t to 0.8 σ_c and 0.8 σ_t. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to 0.6 σ_c and 0.6 σ_t, where: σ_c = unconfined compression strength and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.

(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

(c) *Squeezing rock plastic flow of incompetent rock under the influence of high rock pressure* (SRF)

M. Mild squeezing rock pressure 5-10

O. Heavy squeezing rock pressure 10-20

(d) *Swelling rock chemical swelling activity depending on presence of water*

P. Mild swelling rock pressure 5-10

R. Heavy swelling rock pressure 10-15

TABLE 1.26
(cont.)

ADDITIONAL NOTES:

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in Tables 1 to 6:

1. When borecore is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay-free rock masses (Palmstrom, 1975):

$$RQD = 115 - 3.3\ J_v \text{ (approx.)}$$

where

J_v = total number of joints per m^3

(RQD = 100 for $J_v < 4.5$)

2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistocity, slatey cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as an complete joint set. However, if there are few "joints" visible, or only occasional breaks in bore core due to these features, then it will be more appropriate to count them as "random joints" when evaluating J_n in Table 2.

3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favourably oriented for stability, then a second, less favourably orientated joint set or discontinuity may sometimes be of more significance, and its higher value of J_r/J_a should be used when evaluating Q from equation 1. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.

4. When a rock mass contains clay, the fac or SRF appropriate to loosening loads should be evaluated (Table 6a). In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength (Table 6b). A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in Note (ii), Table 6b.

5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to present or future in situ conditions. A very conservative estimate of strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

TABLE 1.27

N.G.I. CLASSIFICATION SYSTEM: ESTIMATES OF APPARENT SHEAR STRENGTH FROM PARAMETERS Jr AND Ja

a] Rock wall contact	J_r	$\tan^{-1}(J_r/J_a)^\circ$				
		J_a=0.75	1.0	2	3	4
A. Discontinuous joints	4	79°	76°	63°	53°	45°
B. Rough, undulating	3	76°	72°	56°	45°	37°
C. Smooth, undulating	2	69°	63°	45°	34°	27°
D. Slickensided, undulating	1.5	63°	56°	37°	27°	21°
E. Rough, planar	1.5	63°	56°	37°	27°	21°
F. Smooth, planar	1.0	53°	45°	27°	18°	14°
G. Slickensided, planar	0.5	34°	27°	14°	9.5°	7.1°

b] Rock wall contact when sheared	J_r	$\tan^{-1}(J_r/J_a)^\circ$			
		J_a=4	6	8	12
A. Discontinuous joints	4	45°	34°	27°	18°
B. Rough, undulating	3	37°	27°	21°	14°
C. Smooth, undulating	2	27°	18°	14°	9.5°
D. Slickensided, undulating	1.5	21°	14°	11°	7.1°
E. Rough, planar	1.5	21°	14°	11°	7.1°
F. Smooth, planar	1.0	14°	9.5°	7.1°	4.7°
G. Slickensided, planar	0.5	7°	4.7°	3.6°	2.4°

c] No rock wall contact when sheared	J_r	$\tan^{-1}(J_r/J_a)^\circ$		
		J_a=6	8	12
Disintegrated or crushed rock and clay	1.0	9.5°	7.1°	4.7°
Bands of silty- or sandy-clay	1.0	J_a=5		
		11°		
Thick continuous bands of clay	1.0	J_a=10	13	20
		5.7°	4.4°	2.9°

After: BARTON et al., 1974

total friction angles measured and predicted for the various combinations of wall roughness and alteration. Values of $\tan^{-1}(Jr/Ja)^{o}$ for the three categories of wall rock contact given in Table 1.26 are reproduced in Table 1.27.

The quotient, Jw/S.R.F. is an empirical factor describing the "active stresses"[33] and consists of two stress parameters. The parameter Jw is a measure of water pressure, which reduces the shear strength of joints by reducing the effective normal stress and the parameter S.R.F. is a measure of, (1) loosening loads in shear zones and clay bearing rocks, (2) rock stress in competent rock, (3) squeezing or swelling loads in plastic rock.

Barton et al note that amongst other parameters, the joint orientation would probably improve the accuracy of the classification system. These authors found, however, that the parameters Jn, Jr and Ja were more important than orientation because the number of joint sets determines the degree of freedom for block movement, and the frictional and dilational characteristics (Ja and Jr) can vary more than the gravitational component of unfavourably orientated joints.

As implied by its designation, the main purpose of this classification system is the design of tunnel support systems. The Q-value of a rock mass can vary from 0,001 to 1000, and has been expressed in ranges related to the rock mass quality, Table 1.28.

TABLE 1.28

N.G.I. CLASSIFICATION SYSTEM: Q-VALUE RANGES AND RELATED ROCK QUALITY DESCRIPTIONS

Q-range	Rock Quality
1000 - 400	Exceptional
400 - 100	Extremely Good
100 - 40	Very Good
40 - 10	Good
10 - 4,0	Fair
4,0 - 1,0	Poor
1,0 - 0,1	Very Poor
0,1 - 0,01	Extremely Poor
0,01 - 0,001	Exceptionally Poor

/The respective .....

The respective Q-ranges are logarithmic in form with the upper Q-value for each class being a multiple of its lower value. The multiple is variable and is equal to 10,0 for the rock classes with Q-values from 0,001 to 1,0 and either 2,5 or 4,0 for rock classes with Q-values of 1,0 to 1000.

The rock mass quality was correlated with tunnel support measures giving thirty-eight support categories, which in addition to being related to the defined Q-ranges, are also related to other conditional factors like, 'block-size' (RQD/Jn), inter-block shear strength (Jr/Ja) and the ratio of the excavation span to its Excavation Support Ratio (E.S.R.). The E.S.R. is a value assigned to different types of excavations and reflects the degree of safety and support as determined by the purpose of the excavation and the presence of machinery and personnel. The various types of excavations and their associated Excavation Support Ratios are listed in Table 1.29. The support categories with their recommended support measures and associated conditional factors are reproduced from Barton et al, in Appendix I.

These categories (Nos. 1 - 38) are also recorded on Fig. 1.8, which shows the relationship between Q, the required support and the Equivalent Dimension of an excavation (SPAN/E.S.R.). This diagram was developed after the successive re-analysis of the case histories until a consistent relationship was obtained between the three factors[33]. The lower diagonal in this diagram represents the 'no-support boundary' and can be approximated by the equation:

$$\text{SPAN/ESR} = 2\,Q^{0,4}$$

or,

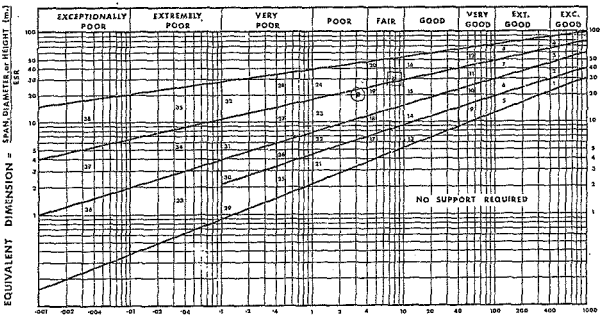
$$\text{SPAN} = 2\,\text{ESR}\;Q^{0,4} \qquad \ldots\ldots\ldots\ 2$$

where the calculated span would be the maximum unsupported dimension for a rock mass with a particular Q-value. Consideration of natural unsupported spans such as the Carlsbad Caverns in the United States suggests that a less conservative relationship may be applicable[34], Fig. 1.9. However until more information becomes available, the more conservative approach given above is used. This relationship can be inverted to estimate the "critical Q-value" of an unsupported rock mass for a given type of excavation with a given span[35],

i.e.

$$Q = \left(\frac{\text{SPAN}}{2\,\text{ESR}}\right)^{2,5} \qquad \ldots\ldots\ldots\ 3$$

/When .....



After: Barton et al, 1974

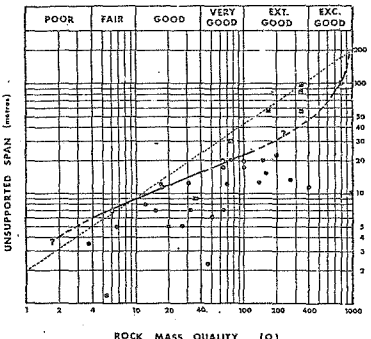
FIG. 1.8 N.G.I. Classification System; Tunnel Support Chart

TABLE 1.29

THE EXCAVATION SUPPORT RATIO (E.S.R.) APPROPRIATE TO A VARIETY OF UNDERGROUND EXCAVATIONS

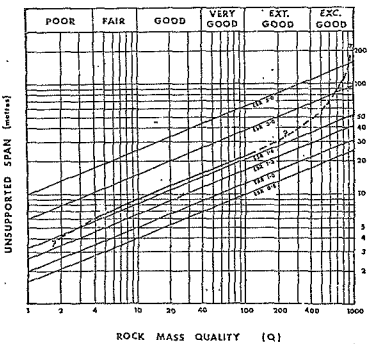
	Type of Excavation	E.S.R.
A.	Temporary mine openings etc.	ca. 3 - 5 ?
B.	Vertical shafts: (i) circular section	ca. 2,5 ?
	(ii) rectangular/square section	ca. 2,0 ?
C.	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks), pilot tunnels, drifts and headings for large excavations etc.	1,6
D.	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels etc. (cylindrical caverns?)	1,3
E.	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections, etc.	1,0
F.	Underground nuclear power stations, railway stations, sports and public facilities, factories, etc.	ca. 0,8 ?

Modified after: Barton, Lien and Lunde, 1974



After: Barton, 1976

FIG. 1.9 N.G.I. System: Maximum Design Span for Permanently Unsupported Man-made Openings



After: Barton, 1976

FIG. 1.10 N.G.I. System: Suggested Design Limits for Various Excavation Types

When equation 2 was evaluated for various E.S.R. values, the assumed safe limits for various types of excavations were obtained[34], Fig. 1.10. A preliminary attempt was also made to correlate the Stand-up Time, Q and the unsupported span, Fig. 1.11, where the latter is considered to be the length from the face to the supported zone or the span itself, whichever is smaller. In order to arrive at realistic safety factors for tunnel design, Barton[34] emphasized the value of case records of failed lengths of tunnels.

In addition to correlating the Q-value with the required support for a rock mass, Barton et al[33] found, from the case history studies, that there was an empirical relationship between the permanent roof support pressure in kg/cm^2, and the rock mass quality.

$$P_{roof} = \left(\frac{2,0}{Jr}\right) Q^{-0,33} \quad \ldots\ldots\ldots 4$$

This relationship was improved with the inclusion of the joint set number (Jn).

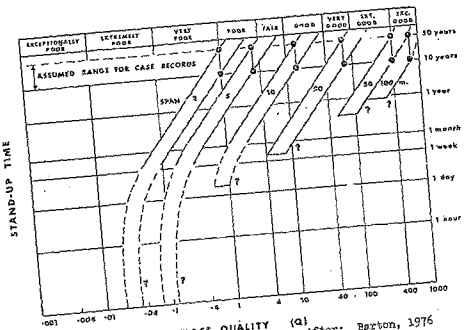
$$P_{roof} = \frac{2.\ Jn^{0,5}\ (Q)^{-0,33}}{3.\ Jr} \quad \ldots\ldots\ldots 5$$

Both equations give the same estimates of roof support pressure when three joint sets are present (Jn = 9), but equation 5 will give lower or higher estimates if less or more than this number of joint sets occurs[33]. Owing to the more favourable position of excavation walls as opposed to roofs, a hypothetical "wall quality" was developed[33] which was related to the Q-value of the rock mass as follows:

Q-value	Recommended 'Wall Quality'
$Q > 10$	5Q
$0,1 < Q < 10$	2,5Q
$Q < 0,1$	1,0Q

It was pointed out by these authors that in exceptional cases of invert swelling, the floor and walls might require lower values of Q than used for the roof.

The N.G.I. Classification System has apparently only been applied to a limited extent in this country. However, Barton[35] gives examples of its use in overseas projects and describes some of the discrepancies between borehole and in situ estimates. These will be discussed in a following section.



After: Barton, 1976

FIG.1.11 N.G.I. System; Preliminary Predictions of the Relationship between Q, Stand-up Time and Unsupported Span

1.6 General Comments about the Geomechanics and N.G.I. Classification Systems

The Geomechanics Rock Mass Classification System represents a synthesis of previous ideas developed mainly for rock tunnelling purposes. Its application has been extended to other rock engineering problems, such as foundations and slopes, although it has apparently not been as well tested in these fields. The rock classes defined for the system are broad, and hence the support recommendations for each class inevitably represent some degree of generalization. The parameters and associated categories have been simply defined and appear to require no more than a basic knowledge of engineering geology for their evaluation.

Although drawing to some extent on previous concepts, the N.G.I. Classification System, which has been developed from an analysis of almost 200 tunnel case records[33], is a relatively unique approach to the problem of rock classification for underground support design. The support recommendations are detailed, and have been related to other factors, including the purpose of the excavation, its dimensions and likely support pressures. The system can be used to evaluate support requirements in both hard and soft ground tunnels. For application to other engineering problems, Barton et al[33] suggest that the six parameters would need to be re-evaluated. They nevertheless feel that the four parameters (RQD, Jn, Jr, Ja) can, with modifications, probably form the basis for many rock mass classification systems. The parameters and related categories are detailed, but have been clearly defined and should pose no problems in their interpretation. They appear, however, to require a reasonably good understanding of engineering geological problems for their evaluation.

1.7 Previous Work

Little work in the same context as the present investigation appears to have been previously undertaken, although comparisons of results from different classification systems have been made.

Tinkler[36] applied the Geomechanics, N.G.I., RQD and RSR classification systems to various engineering problems on the Zambian Copperbelt. Information was obtained from a combination of detailed underground mapping and drillhole information, pilot hole drill cores only and exploration cores. He concluded from the field data that there are considerable

/differences .....

differences between the rock mass quality ratings indicated by each system, and consequently between the predicted spans and support recommendations as well. He recommended the Geomechanics system as being simple and easy to apply in a general mining environment.

Houghton[37] compared the results of three rock mass classification systems, viz. the RSR, RMR and N.G.I. systems, from the Kielder Experimental Tunnel in the United Kingdom and concluded that there was little difference between support predictions for each. He felt that his work confirmed the value of numerical indices for rock quality assessment, and also concluded that the Geomechanics system was practically the easiest to apply.

Payne[38] reviewed the objectives and parameters of rock classification for engineering purposes. Field trials were undertaken and the results discussed with respect to the wider applications of rock classification.

Bergman[39] assessed five methods of subsurface investigation for their reliability of obtaining certain data, and found that with certain prerequisites integral sampling, as developed by Rocha[40], and television examination of boreholes were the most reliable for rock mass evaluation. Conventional core drilling was found to provide ambiguous information which could result in under-rating the rock mass properties.

Tinkler's and Houghton's respective conclusions regarding support predictions from various classification systems are contradictory and indicate the need for further comparative studies. Bergman's conclusion with respect to the information obtained from conventional core drilling emphasizes the value of the present project.

2. METHODS USED TO OBTAIN THE REQUIRED CLASSIFICATION PARAMETERS

The methods used in this study to obtain and evaluate the parameters required by the *Geomechanics* and *N.C.I.* Classification Systems, from the borehole cores and in situ exposures, are described in the following sections. Existing techniques from which these methods have been adapted are also briefly discussed. The main consideration has been to obtain the information relevant to each classification system as simply and as accurately as possible, without unnecessary duplication of effort.

Since a number of the parameters required for both classification systems are related to the jointing in the rock, it is appropriate that some form of joint survey be used. Line surveys, which involve recording information about those joints which intersect a specific line, are suitable for both bore core and in situ measurements. In the first case, the survey line is represented by the borehole, and the required information is obtained from small sections of each joint surface exposed in the core. In the latter case, the line is defined by a tape fixed to the rock face and larger joint surface areas are visible. Additional information can be obtained from the exposed rock adjacent to, or in the vicinity of, the in situ survey line. More reliable joint information can consequently be obtained from in situ surveys than from borehole joint surveys in which certain inferences may be necessary to provide the required parameters. Owing to this, different approaches are needed to evaluate the respective data.

The following sections deal firstly with the type of error common to line joint surveys. The methods used to record the data from the bore cores and in situ exposures are then described, and are followed by discussions related to the evaluation techniques adopted.

2.1 Errors in Scan-Line Joint Surveys and their Correction

Scan-line joint surveys were adopted for recording joint data from both the bore cores and the in situ exposures. This method, which involves measuring joints along uni-directional lines of limited length, can give erroneous estimates of the joint spacing and joint frequency of a particular joint set, if the survey line direction is not at right angles to the joint set direction[41]. This is illustrated in Fig. 2.1, where two hypothetical survey lines of equal length are shown. The one line is

/perpendicular .....

perpendicular to the joint set direction and the joint spacing, d, is the true spacing of that set. The other line intersects the same joint set at an angle of α degrees, and the spacing D is not truly representative of the set. The joint frequencies along each survey line are denoted as N_{90} and N_{α} respectively, where the former is the true frequency along the line at 90 degrees to the joint orientation and the latter is the frequency recorded along the line with the intersection angle, α.

From simple trigonometrical relationships it can be seen that since,

$$N_{90} = L/d \; ; \qquad N_{\alpha} = L/D \; ; \qquad \text{and} \quad D = d/\text{Sin}\alpha,$$

$$\text{then,} \qquad N_{\alpha} = L \, \text{Sin}\alpha/d$$

$$\text{and} \qquad N_{90} = N_{\alpha}/\text{Sin}\alpha.$$

By dividing the number of joints of a particular joint set, recorded along a survey line of known length, by the sine of the angle of intersection between the survey line and the joint set direction, it is thus possible to calculate the true spacing and frequency values for that set[41].

In addition to proposing the above correction, Terzaghi[41] pointed out that a single joint intersected at a low angle ($\alpha = 0^{\circ} - 20^{\circ}$) by a survey line (i.e. in a borehole or tunnel) could either represent an isolated random joint, or could be one member of an important set with a spacing, D, possibly exceeding the survey length L, Fig. 2.2. Neither possibility can be assumed to be correct, however, and meaningful corrections cannot, therefore, be made for values of α between approximately 0 degrees and 20 degrees. Owing to local variations of spacing and continuity displayed by joints in real rock, it is also often impossible to obtain reliable information regarding such joints.

This problem can be overcome by conducting joint surveys along a sufficient variety of directions so that every joint set will be intersected at an angle exceeding 20 degrees, by at least one survey line[41]. The low angle joints measured along any single direction can then be disregarded. This frequently cannot be achieved, however, and it is necessary to rely on data from single direction surveys only. The low angle joints cannot be disregarded in these cases because no other information concerning them will be available.

Although the methods adopted to estimate the number of joint sets and their spacing values from the bore cores and in situ measurements respectively are described in the following sections, it is pertinent at this stage to indicate when the above correction was applied. Stereographic plots,

/supported .....

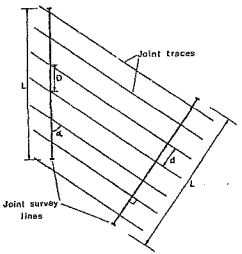


FIG. 2.1 Section along Joint Survey Lines intersecting a Joint Set at β^0 and α^0 respectively.

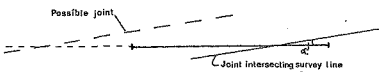


FIG. 2.2 Single Joint intersected by a Survey Line at $\alpha < 20^0$.

supported by additional information from the exposed rock surface adjacent to the survey lines, were used to estimate the number of joint sets from the in situ data. This combination gave reasonable estimates of the joint set number and no correction was used unless surveys along more than one direction were possible.

The joint measurements made on the bore cores could not be orientated with respect to a known point however, and the most suitable method for estimating the number of joint sets was with the aid of histograms showing the number of joints with similar inclination angles. The contrast between the peaks which represented individual joint sets, was emphasized by applying the recommended correction to the data. Although it was realized that this approach enhanced the possibility of introducing errors by indicating non-existent low-angle joint sets, it was considered to be reasonable because they would be unlikely to affect the joint set number significantly. Terzaghi's correction was applied to both the borehole and in situ measurements in order to estimate the spacing values of the different joint sets.

2.2 Borehole Core Logging

Borehole drilling using rotary cored boreholes is currently one of the most popular site investigation techniques in use, and has resulted in the development of specialized forms of recording geological data for engineering applications. These differ in many respects from the methods used for recording geological information for stratigraphic or mineral exploration purposes and emphasize mainly those features influencing the strength characteristics of the rock mass.

The need for standardization and, when possible, quantification in engineering geological borehole logging has been recognized for some time and numerous efforts have been made to meet these requirements[42-47]. Although most parties concerned appear to agree on the essential requirements of a borehole log, differences remain in the actual terminology and format used to record the data. The development of engineering rock mass classification systems using specified parameters for classifying the rock mass greatly assist in standardizing the logging procedure in that the information, and its form, required by the engineer, has been clearly defined.

Three methods of engineering geological borehole logging have recently been proposed in this country[45, 46, 47]. Of these, the system suggested

/by geologists .....

by geologists of the Geological Survey of South Africa[46] is most pertinent to the present study because data relevant to both the Geomechanics and N.G.I. Classification Systems are recorded. The inclusion of an assessment of the number of joint sets present in the rock is a unique and useful feature of this method. However, as indicated by the authors, this parameter is not easily determined from a borehole core and, if no other information source is available, may require some inference or guesswork. This is acceptable provided that some indication is given regarding the possible reliability of the given value; provision for this has unfortunately been omitted. The authors of this borehole log assert that parameters such as the joint continuity, waviness and joint separation cannot be measured from a borehole core alone. The present writer agrees with this contention regarding the continuity, but feels that both the waviness (undulations in this case) and the joint separation, can in certain instances be recorded with reasonable certainty. Information concerning strength and weathering characteristics is presented both verbally and graphically. To the present writer this seems to be an unnecessary duplication.

The C.S.I.R. Geomechanics core log[47] has been specifically designed for use with the Geomechanics Rock Classification System. It is used in conjunction with a joint survey and the Geomechanics data input form and cannot be used alone to classify the rock mass. To the present writer this imposes some rather strict limitations on the practical value of this log because, as already mentioned (Section 1.1), in some cases surface exposures may be absent, or the cover thickness too great for the reliable interpolation of surface data to the tunnel level. In addition, although not a weakness of the log itself, it does not record information relevant to the N.G.I. Classification System and is therefore not really useful to the present study.

The core logging guide proposed by the South African Section of the Association of Engineering Geologists[45] is a general method of describing the parameters commonly of interest in rock engineering. It uses qualitative descriptions dependent on visual inspection and simple mechanical field tests, and is intended as a base upon which to enlarge, if more specific parameters are required. The guide takes cognizance of existing accepted methods of soil and rock description and attempts to remain compatible with these when possible. To the present writer, this coreslogging method is well suited for its intended purpose but is too general for detailed rock mass classification.

/A novel .....

A novel and extremely useful method of core logging for engineering purposes was suggested by Rankilor[42]. The method involves the drawing of all the joints as they occur in the core and the measurement of their inclination angles relative to the long axis of the core. If undertaken carefully, this method provides an unbiased picture of the core from which an assessment of the number of joint sets, their average inclination angles and spacings can be made. The rock description format adopted by Rankilor is similar to that proposed by Franklin et. al.[44]. The former uses the rock name: colour; hardness; lamination; and other, while the latter authors make use of the rock name; weathering state; structure; colour; and texture.

The use of mechanical indices, particularly the Point Load Strength Index, as suggested by Franklin et. al.[44], is becoming increasingly popular in borehole core logging. The Point Load Strength Index eliminates the need for descriptive terms for rock strength. Simple field tests can be used to indicate rock hardness[45] but must be strictly adhered to if they are to be used as indicators of the unconfined compressive strength of the rock. These tests are more suitable for the lower strength rocks where the Point Load Test is less reliable.

The main premise that was considered when selecting the method most suitable for recording the bore core data for the present study, was that the core log should present, as closely as possible, a true picture of the rock core with a minimum amount of description and interpretation being used. The object behind this approach is to enable the rock mass to be classified by anybody from the borehole log alone, without their judgement being affected by the unintentional bias of the original compiler of the log.

The log sheets developed for the pres[illegible]v and presented in Appendix II, have three main sections deal[illegible]tively, from left to right across the sheet, with the rock, the [illegible]n the rock, and lastly with test data, general remarks and drilling data.

The first section includes a depth scale, a legend column and a rock description column. The rock description format is based on those suggested by Franklin et. al.[44] and Rankilor[42] and includes the rock name (i.e. lithological type), colour, texture and weathering state. Any additional information, e.g. indications of shearing or secondary cement, are included in the general remarks column in the third section of the log sheet. The terms used for the description of the texture and

/weathering .....

weathering state have been adopted from the standards suggested by the South African Section of the Association of Engineering Geologists[45], and are reproduced in Appendix IIIA. Two further columns have been included in this section of the log sheet and show graphically the percentage core recovery and Rock Quality Designation (RQD) respectively. Contrary to normal practice in which these values are usually expressed for each core run, they have been calculated here for individual geological units. This was done in order to obtain values related to the rock mass as a whole rather than to parts of it.

The second section of the log sheet includes a depth scale, a joint diagram column and columns for the inclination angles of each joint recorded, and their roughness and alteration respectively. In order to provide as realistic a picture of the core as possible, it was necessary to try to reconstruct it before compiling the joint diagram. Initially an attempt was made to measure both the strike and dip (with respect to an arbitrarily selected line on the core) of each joint so that the number of joint sets could be estimated from a stereographic plot of the data[48]. This was found to generally be possible over relatively short core lengths only because of the difficulty of relating adjacent core runs, and was therefore abandoned. As a result, only the inclination angle of each joint relative to the long axis of the core was recorded and no attempt, except in very obvious cases, was made to relate the individual joint directions to each other. These measurements were made to the nearest 5° by using a graduated clinorule, Plate I.

The roughness and alteration characteristics of the joint surfaces were recorded qualitatively in terms applicable to the classification systems being studied. In order to describe the roughness, a simple differentiation was made between undulating or planar surfaces which were either rough or smooth. Slickensides were recorded whenever apparent. The joint surface alteration, when present, was described in terms of staining and softening or non-softening mineral coatings. Whenever some joint fill was noted an estimate of its thickness was recorded. Abbreviations used to indicate the joint alteration are presented in Appendix IIIA.

Although the terms of description used for the roughness and alteration characteristics of the joint surfaces are derived directly from the categories defined by the N.G.I. Classification System, they were also found to be quite suitable for use with the Geomechanics Classification System. Two additional columns for recording the estimated number of joint sets and the joint spacing were also included on the log sheet. Although

/these .....

these estimates could only be made after the evaluation of the borehole data, they are relevant to both rock classification systems and were included for a convenient assessment of the rock mass from the log sheet.

The third section of the log sheet includes columns for the Point Load Strength Index, which was determined for as many points as possible, any other tests which may have provided a relevant data, a remarks column, and other columns referring to water losses, casing used, core size and method of drilling. Finally another depth scale was provided. Owing to lack of the relevant data, those columns referring to water losses and casing could not be completed.

This method of logging was used over selected sections of the core obtained from boreholes drilled in the vicinity of the tunnels being studied. Ideally, depending on the size of the excavation, it would only be necessary to undertake a detailed assessment of the jointing in the core for a limited distance (e.g. 5 m in these cases) above the planned crown level to a few metres into the invert. The remainder of the core need only be recorded in general terms as suggested by the Association of Engineering Geologists.

In most cases the cores available were in reasonably good condition for the required purpose. In some of the cores from the Bushkoppies Tunnel, however, the required length above the crown level could not be obtained and it was assumed that the available core was representative of the rock mass at the particular site. This was considered to be a reasonably valid assumption in view of the generally homogeneous nature of the rock mass at this tunnel.

2.3 In Situ Measurements

The in situ data for the classification of the rock mass was obtained using a detail line joint survey method adapted from that proposed by Piteau[49], as well as visual estimates and approximate measurements. The detail line joint survey method entails suspending a measuring tape horizontally along the tunnel wall and recording information about every joint which intersects the tape line. The joint features recorded by Piteau are listed below:

/1. .....

1. Co-ordinates (x and y)
2. Elevation (tape)
3. Rock (type, hardness, structure)
4. Strike
5. Dip (angle, direction)
6. Continuity (dip, strike)
7. Gouge (thickness, type, hardness)
8. Roughness
9. Waviness (amplitude and base length)

For the purposes of the present study some of the details of these nine features were excluded from the joint survey as not being relevant to the Geomechanics or N.G.I. Classification Systems. Those excluded were:

1. the x- and y- co-ordinates of the joint being measured and the tape elevation;

2. the strike direction of each joint because it duplicated the information given by the dip and dip direction;

3. the differentiation between strike and dip continuity together with any reference to the actual length of the joint trace visible;

4. the amplitude and wavelength measurements of the surface undulations.

The information obtained from the joint surveys was recorded on joint survey data sheets which are presented in Appendix IIIB.

An outline of the features which were recorded is given below, and detailed descriptions of the various categories (after Piteau) are also included in Appendix IIIA. The descriptions below are related to the columns on the joint survey sheets, numbered from 1 to 14, in which each feature was recorded:

/Column 1 .....

Column 1 : Indicates the position along the tape at which the particular joint was intersected.

Column 2 : Indicates the lithological type of the rock at the joint being measured.

Column 3 : Indicates the hardness of the rock adjacent to the joint.

Column 4 : Indicates the type and origin of the structure being measured, i.e. bedding joint, shear joint, etc.; this parameter cannot always be determined.

Column 5 : Indicates the dip angle.

Column 6 : Indicates the direction of dip.

Column 7 : Indicates the thickness of any joint fill or gouge on the joint being measured.

Column 8 : Indicates the type of alteration, if any, on the joint being measured.

Column 9 : Indicates the hardness, as defined for Column 3, of any notable thickness of fill material along the joint being measured.

Column 10 : Indicates the roughness of the joint surface being measured.

Column 11 : Indicates whether the joint surface is planar (P) or undulating (U).

Column 12 : Indicates whether the joint being measured is discontinuous; continuous out of the face at one end; or continuous out of the face at both ends.

Column 13 : Indicates to what extent water is present on the joint surface being measured.

Column 14 : Provides for general remarks.

The in situ measurements were carried out as close as possible to the underground positions of the relevant boreholes. At the Bushkoppies and Delvers Tunnels the boreholes were drilled from surface, along or adjacent

/to the centre .....

to the centre lines of the planned excavations. At the Du Toitskloof project the holes were drilled from within the tunnel. In all three tunnels the underground positions of the holes were easily located.

The detail line joint survey method was used at the Bushkoppies and Du Toitskloof Tunnels. In the former the surveys were all a standard length of 10,0 m with the survey lines centered as closely as possible about the intersection points of the various boreholes. The jointing patterns in the rock often displayed noticeable local variations over relatively short distances. The 10,0 m survey length was therefore selected being most convenient because, whilst generally furnishing sufficient for analysis, it lessened the possibility of including information which might not have been intersected by the relevant borehole.

The joint measurements at the Du Toitskloof Tunnel were generally carried out along the walls of drilling recesses cut into the side of the tunnel. This made it possible to obtain measurements along two directions orientated approximately perpendicular to each other. The lengths of the survey lines varied with the dimensions of the particular recess, but an effort was made to use similar line lengths for both directions. Owing to access problems due to the presence of underground machinery it was not possible to obtain in situ joint measurements from two of the six locations from which boreholes had been drilled. It was therefore decided to use joint survey data from adjacent localities with similar geological conditions, which had previously been obtained by Messrs. Van Niekerk, Kleyn and Edwards et. al.[50], during their routine tunnel mapping. Detailed maps of the joint patterns at five of the sample locations were also compiled in order to supplement the joint survey data.

The limited space and the continual installation of steel temporary supports, together with the simultaneous construction of the final concrete lining, precluded the use of the detail line joint survey in the Delvers Tunnel. In this case the assessment of the rock mass was based on a visual appraisal and approximate measurements.

2.4 Strength Testing of the Core Samples

The Point Load Strength Index test[51] was used to determine the strength characteristics of the rock cores examined for this project. The test is ideal as a rapid means of evaluating the strength of rock provided it is not too soft[52]. Following Bieniawski's suggestion[53], it was

/included .....

included in the present study, as a routine part of the core logging procedure to replace qualitative rock hardness descriptions. An Engineering Laboratory Equipment machine was used, Plate II, to test the samples.

The testing procedure has been outlined by Broch and Franklin[51]. Three types of point load tests are possible, viz. the diametral test, the axial test, and the irregular lump test. The first two tests apply to testing rock cores, and were found by those authors to yield more consistent results than the last method provided that certain length (L) to diameter (D) ratios of the core specimens were maintained. For the diametral test, cores with L equal to, or greater than, 1,4 x D are suitable for testing, while for the axial test a ratio of $1,1 \pm 0,05$ is required. For bedded or other observably anisotropic rocks, the test should be undertaken in both the strongest and weakest directions of the rock, with the loading directions perpendicular to and parallel with the weakness planes respectively. For both the diametral and axial tests, ten or more tests per sample are recommended.

The Point Load Strength Index for a particular rock sample is obtained from the median value of a set of test results[51]. Broch and Franklin suggest that for standard classification the index Is(50), which is the point load index corrected to a reference core diameter of 50 millimetres, should be quoted. Bieniawski[53], however, feels that the Is(54) index would be more suitable as a standard because NX diameter (54 millimetres) cores are the internationally recommended size for site investigation drilling, standard Rock Quality Designation (RQD) determinations and standard rock strength tests.

A linear relationship has been shown to exist between the point load index and the uniaxial compressive strength of a rock sample[52, 53], and is expressed by the equation:

$$C = K.Is \quad \ldots\ldots\ldots\ldots \quad 6$$

where;

C - Unconfined Compressive Strength of the sample

Is - Point Load Index

K - Variable dependent on the diameter of the core tested.

For NX core, and therefore for any Is(54) index, a value for K equal to 24 has been shown to be a good approximation[53].

The specimens tested from the Bushkoppies and Du Toitskloof Tunnels were NX size cores, while those tested from the Delvers Tunnel were 60 millimetres in diameter. All samples tested were in an air-dry state. The

/number .....

number of tests undertaken for each borehole varied depending on the condition of the core and on the need to maintain intact specimens for later reference by the owners. The diametral test only was used. Several unsuccessful attempts were made to determine the sample anisotropy of the quartzites and quartzitic sandstones from the Delvers and Du Toitskloof Tunnels respectively. The main problem in this respect was that it was extremely difficult to obtain specimens with length to diameter ratios within the narrow limits required for the axial test. Only results from specimens which failed with a tensile fracture between the loading points were considered to be reliable[52].

The mean and median values of each set of point load tests, together with the number of tests performed and the unconfined compressive strengths calculated from the median value are presented in Appendix IV. The Is(54) index value has been adopted for this study. Unconfined compressive strength values, available from previous work, are also tabulated, and have occasionally been used in lieu of the point load test results for the strength characterization of the rock. The results of tests on individual core specimens are shown at the appropriate depths on the borehole log sheets.

2.5 Evaluation of the Borehole Measurements

To assign values to the various parameters specified by the Geomechanics and N.G.I. Classification Systems, the procedure listed below was adopted to evaluate the borehole core measurements. The results for each step have been tabulated separately and are presented in Appendix V.

1. The numbers of joints with similar inclination angles were recorded and corrected for orientational bias (Section 2.1).

2. The results of Step 1 were plotted as histograms and the numbers of joint sets present in each borehole were estimated.

3. The average spacing of the joints in each set was estimated.

/4. .....

4. The numbers of joints in each set with similar surface roughness and alteration characteristics were recorded.

5. The most typical surface roughness and alteration characteristics of each joint set was estimated.

6. Data forms were compiled for each classification system showing the parameters considered to be representative of the rock mass at each borehole location.

7. Calculation sheets were compiled for each classification system using the parameters selected in Step 6. The 'RMR' and 'Q' values for the rock mass at each borehole location were calculated.

The number of joint sets present in each borehole core was estimated by counting joints with similar inclination angles. Ten inclination angle categories were selected, at 10° intervals from 0° to 90°. As previously explained (Section 2.2), the inclination angles of the joints were measured to the nearest 5°. Joints with inclination angles intermediate to the selected categories, i.e. 5°, 15°, 25°, were therefore included in the next highest category.

The total number of joints in each category was recorded in Row A of the prepared table (refer Appendix V). These values were multiplied by $1/\sin\alpha$ to correct for orientational bias (Section 2.1) and were expressed as percentages of the total corrected number of joints recorded in a particular core. The results were recorded in Rows B and C respectively of the table. The final results recorded in Row C were reproduced as histograms in Step 2. This facilitated visual estimates of the joint set number and the ranges of the inclination angles of each set.

When information from only one borehole is available, the orientational bias correction could introduce significant errors into estimates of the numbers of joint sets present (Section 2.1). Without additional information concerning the joints intersecting the core at low angles it is impossible to determine their importance, and for a conservative approach it must be assumed that they represent joint sets which are pertinent to the classification of the rock mass. This approach must be taken irrespective of how few low angle joints may be intersected by the

/borehole. .....

borehole.

The corrected joint numbers assisted greatly in emphasizing the low angle joint sets on the histograms. They were, however, not used to calculate the joint spacing of those sets with inclination angles between 0° and 20° (Step 3). The spacings of the remaining joint sets, with inclination angles greater than 20°, were obtained by dividing the length of core examined (L) from each borehole, by the corrected number of joints recorded in the angular ranges defined in Step 2.

In Step 4 the numbers of joints in each set with similar surface roughness and alteration characteristics were counted. The roughness was expressed as either undulating or planar, with rough or smooth surfaces. Slickensided joints were not recorded separately but were noted as a number of the undulating or planar joints already recorded. The alteration characteristics were expressed as: closed; no alteration or surface staining only; non-softening mineral coatings; clay or softening mineral coatings; and gouge. The surface characteristics selected in Step 5 as being typical of a particular joint set were those exhibited most frequently by that set.

An estimate of the joint set most critical to the tunnel stability was also given in Step 5. The effect a joint set has on the stability of a tunnel is related to the orientation of the joints with respect to the axis of the tunnel[20, 31], as well as being influenced by the surface condition of the joints. In the case of joint measurements on unoriented borehole cores, only the angle of dip is known, while the strike direction cannot be determined. The criteria outlined by Wickham et.al.[20] and modified by Bieniawski[31] cannot therefore be applied because joints with dip angles greater than 20° can vary from "very unfavourable" to "very favourable" with respect to a tunnel's stability, depending on their strike directions relative to the tunnel axis. Only joints with dips of less than 20° can be considered as consistently "unfavourable", irrespective of their strike directions.

In order to estimate which of the joint sets in a particular borehole was most likely to influence the stability of the tunnel the various inclination angles of the joint sets were expressed as dip angles. This was easily achieved because the majority of boreholes had been drilled either horizontally or vertically. The joint set selected as being most critical was that which occurred most frequently in the borehole. When more than one set had approximately the same frequency, the set with the shallowest dip was selected if its dip angle was less than 20°. If none

/of the .....

of the sets dipped at less than 20°, then the one with the least favourable surface conditions was selected. If a joint set with particularly unfavourable surface conditions (i.e. thick clay gouge) occurred with a noticeably lower frequency than a set with very favourable surface conditions (i.e. undulating and rough, closed) the former was selected as most critical to the stability of the excavation.

The data forms were compiled in Step 6 using the results from Steps 1 - 5 together with RQD estimates and the point load indices. In Step 7 the rating values for each parameter were substituted into the relevant equations, and the RMR and Q-values calculated for each bore core.

2.6 Evaluation of the In Situ Measurements

A similar approach was adopted to evaluate the in situ joint survey measurements as was used for the borehole data. The steps followed are listed below and the results are presented in Appendix VI:

1. The numbers of joint sets were determined with the aid of a stereographic projection.
2. The joint survey data was re-listed from the field sheets, according to the sets determined in Step 1.
3. The intersection angles (α) were determined for each of the joint sets and the numbers of joints in each set corrected for orientational bias. The joint spacings were then determined and the RQD's were calculated.
4. The angular relationship between the tunnel axis and the strike orientations of the various joint sets were determined and their influence on the tunnel stability recorded.

5-8. These four steps were identical to the last four steps used to evaluate the borehole data and included the determination of the typical surface roughness and alteration characteristics of each joint set (Steps 5 and 6) and the compilation of data forms and calculation sheets for each rock classification system (Steps 7 and 8).

/In Step 1 .....

In Step 1 the poles to the joint surfaces measured were plotted on transparent sheets overlaying a 10 cm radius, Schmidt Equal Area stereographic projection. The centre and circumference of the stereonet were marked on the sheets which were then laid over a centimetre square grid so that the marked centre-point of the stereonet coincided with a grid intersection point and the north-south and east-west axes of the stereonet were parallel to the grid lines. The poles were then contoured using the circular counter, Fig. 2.3, method described by Badgely[54] and Wahlström[55], whereby a counter with an area equal to 1 per cent of that of the stereonet (in this case with a radius of 1 cm) was moved across the sheet and the number of poles within the area of the counter were recorded at each node of the grid. A peripheral counter, Fig. 2.3, was used to record the numbers of poles which occurred close to the edge of the stereonet. The number of points recorded at each node were calculated as percentages of the total number of joints measured at a particular locality and the diagrams were contoured at suitable intervals. When more than one joint survey direction was possible (i.e. Du Toitskloof Tunnel), the numbers of poles recorded at each node could be corrected for orientational bias by multiplying by $1/\sin\alpha$. In these cases the corrected numbers themselves were contoured and were not expressed as percentages. The numbers of joint sets at each locality and their respective average orientations were then estimated. The orientation of the tunnel axis together with the direction of advance was also shown on each stereo-plot.

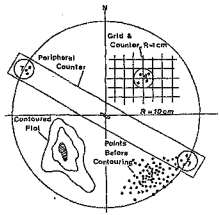


FIG. 2.3 Circular Counter Method of Contouring Stereographic Plots

/After .....

After the analysis of the stereo-plots in Step 1 the joint survey data was grouped and re-listed on new data sheets according to the respective joint sets. This was done to facilitate the comparison between the joints in any particular set and for an easy assessment of the roughness, alteration and water conditions.

In Step 3 the intersection angles (α) between the joint survey lines and the strike directions of the respective joint sets were measured on the stereo-plot. The numbers of joints measured in each set were then multiplied by $1/\sin\alpha$ to correct for orientational bias. The mean spacing of the joints in each set was calculated by dividing the length of the joint survey line (L) by the corrected number of joints. The inverse of the calculated spacing value gave the number of joints per metre.

Whenever possible two independent methods were used to calculate the RQD per cent from the in situ measurements. In the first it was calculated from the expression suggested by Palmström[56] for clay-free rock masses:

$$RQD\% = 115 - 3{,}3\ Jv \text{ (approx.)}$$

where Jv - total number of joints per cubic metre
OR the sum of the number of joints per metre for each joint set.

The second relationship used was proposed by Priest and Hudson[57] and assumes a negative exponential form for the frequency versus spacing curve for most naturally occurring joints.

$$RQD\% = 100\ e^{-0{,}1\lambda}(0{,}1\lambda + 1)$$

where λ - average number of discontinuities per metre.

The latter method could only be used when the in situ data was obtained from detail line joint surveys, and was not suitable when the joint spacing values for individual joint sets were estimated visually, i.e. Delvers Tunnel and Du Toitskloof Tunnel, Locations 4 and 5 (Section 3.4). In these cases the first method was used although it was also not really suitable because clay fillings were quite common on many of the joints.

The influence of the joint orientations on the tunnel stability was estimated from classes defined by the Geomechanics Classification System, which are based on the strike of the joints being parallel or perpendicular to the axis of the tunnel. Few joints were orientated exactly along these directions and it was therefore assumed that joint sets with strike directions intersecting the tunnel axis at angles from 0° to 45° were "parallel", while those with intersection angles greater than 45° were "perpendicular".

/It was .....

It was also necessary to consider the direction of advance of the tunnel when considering those joints striking perpendicular to the tunnel axis.

Steps 1 - 3 were omitted when evaluating the data from the Delvers Tunnel. The numbers of joint sets present at a particular location, their orientations, average spacing values and typical surface conditions were estimated visually and recorded directly and the RQD calculated by the method already described[56]. The dip directions and strike directions of the joint sets were described less accurately than for the other tunnels studied but were found to be adequate for assessing the stability of the tunnel.

The point load test results from the borehole core samples were also used for the in situ classification of the rock masses. The compilation of the data forms and calculation sheets was the same as for the borehole data.

2.7 Evaluation of the Borehole and In Situ Results

In order to have some basis for comparing the in situ and borehole results, the former were assumed to be representative of the true rock mass conditions at each sample site. In addition to comparing these results for each rock mass classification system (i.e. RMR-values for the Geomechanics System and Q-values for the N.G.I. System), the borehole and in situ ratings for the individual parameters were also compared. The ground-water ratings and rock strength data were not evaluated however, because the original data in the former case was not sufficiently reliable, and in the latter the same data was used for both the borehole and in situ calculations. The results were plotted on graphs with the in situ and borehole values along the abscissae and ordinates respectively. The scatter of points was then compared with the ideal linear relationship, $y = x$, where:

y - borehole value

x - in situ value.

The Standard Error of Estimate, S_{yx}, was calculated for each set of results and the 90 per cent confidence limits about the line $y = x$, were shown on each graph. These limits were generally not relevant to the rock classes boundaries or parameter categories defined for each classification system and the results were evaluated instead, with respect to limits related to the differences between the classes or categories. The numbers of points within the selected limits were recorded, together with

/the number .....

the number of points on, above and below the y = x line. These numbers were expressed as percentages of the total number of observations.

The Standard Error of Estimate about a linear regression line is defined by Spiegel[58] as:

$$Syx = \sqrt{\frac{y^2 - c\,y - m\,XY}{N}} \quad \ldots\ldots\ 7$$

where y and x are defined above
c - intercept on the y-axis
m - gradient of the line
N - sample number

For N < 30, N in the above equation is replaced by (N - 2), and owing to the restrictions of the assumed relationship, y = x, the Standard Error of Estimate can be calculated from the equation:

$$Syx = \sqrt{\frac{y^2 - XY}{N - 2}} \quad \ldots\ldots\ 8$$

The Standard Error of Estimate is analogous to the standard deviation[58] and can be used in a similar manner to calculate the confidence limits about an assumed regression line. The 90 per cent confidence limits about y = x are given by:

$$1{,}645\ Syx \quad [58]\ (p.\ 157)$$

Owing to the log-log relationship between the in situ Q-values, Q(INS), and the borehole Q-values, Q(BH), the linear relationship,

$$LOG\ Q(BH) = LOG\ a + b\ LOG\ Q(INS)$$

was applicable[58] (p. 218). To calculate Syx, therefore, LOG Q(BH) and LOG Q(INS) were substituted for y and x respectively in equation 8.

The relationship between RMR and Q for the borehole and in situ results respectively was also assessed. Graphs were prepared with the RMR-values plotted along the ordinates and the Q-values along the abscissae and the best fit linear regression lines were calculated for each case using the method of least squares. Owing to the log-normal relationship between Q and RMR the expression,

$$LOG\ Q = LOG\ a + RMR\ LOG\ b$$

was applicable[58] (p. 218). With the appropriate manipulation this is equivalent to the expression,

$$R=R = \frac{1}{LOG\ b} \cdot LOG\ Q - \frac{LOG\ a}{LOG\ b}$$

The 90 per cent confidence limits about the regression line were also calculated and plotted. The relationships obtained in each case were compared with each other and also with the relationship established by Bieniawski[25].

3. DESCRIPTIONS OF THE PROJECTS STUDIED

3.1 The Bushkoppies Tunnel

3.1.1 Introduction to the Project[59]

The Bushkoppies Tunnel is situated about 15 km to the south of the City of Johannesburg and follows an approximate north west - south east route from the vicinity of the Kliptown Station, beneath Pimville Township, to the future Bushkoppie sewage purification works east of the main highway between Johannesburg and Vanderbijlpark, Fig. 3.1. The tunnel will be used for sewerage transportation and has been designed by the Johannesburg City Engineer's Department as Phase I of a scheme planned to alleviate the overcharged sewers in the Olifantsvlei basin to the west of Johannesburg.

The total length of the tunnel is approximately 6,5 km and it comprises two sections, 3 777 m and 2 766 m long respectively. The latter, known as the west tunnel, consists of an excavation with equal vertical and horizontal dimensions of approximately 2,6 m to 3,0 m, with a flat invert, vertical sidewalls and an arched crown. The former, known as the east tunnel, has a similar cross-section but larger vertical and horizontal dimensions of about 3,30 m and 3,45 m respectively, Fig. 3.2. The tunnel will be lined throughout its length with approximately 300 mm of concrete to bring the completed dimensions of the east and west tunnels to 2,75 m x 2,5 m, and 2,0 m x 2,0 m respectively. The east tunnel is larger than the west because it has been designed to carry the additional flow from Phase II of the scheme. The depth of the tunnel below ground level varies from about 5 m at the portals to 65 m at its deepest point, Fig. 3.3.

Access to the works was from an incline shaft located at the meeting point of the east and west tunnels. The major part of the tunnel was excavated at two headings advancing in south east and north west directions respectively using conventional drill and blast techniques. A third face was also started from the south east portal of the tunnel and was advanced for approximately 300 m through badly jointed and highly weathered rock using hand-held pneumatic diggers together with conventional methods.

The bore cores used for this study were obtained from boreholes drilled during the site investigation of the project. In all, twelve sample sites were selected, Fig. 3.4. A variety of rock mass conditions were studied. Unfortunately the borecores which had been obtained from the very poor quality rock in the vicinity of the south east portal had

/deteriorated .....

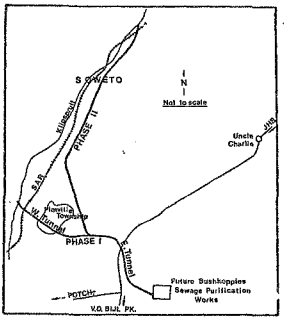


FIG. 3.1 Bushkoppies Tunnel - Locality Map

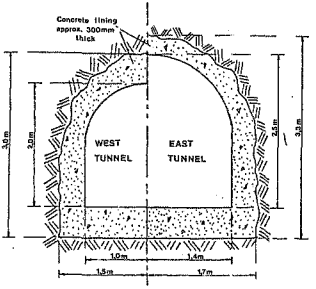


FIG. 3.2 Bushkoppies Tunnel - Cross-Sections through East and West Tunnels

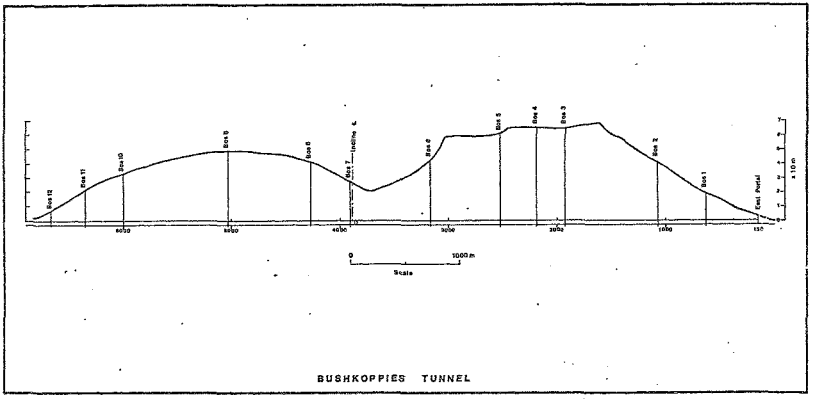


FIG. 3.3 Profile along Tunnel Route

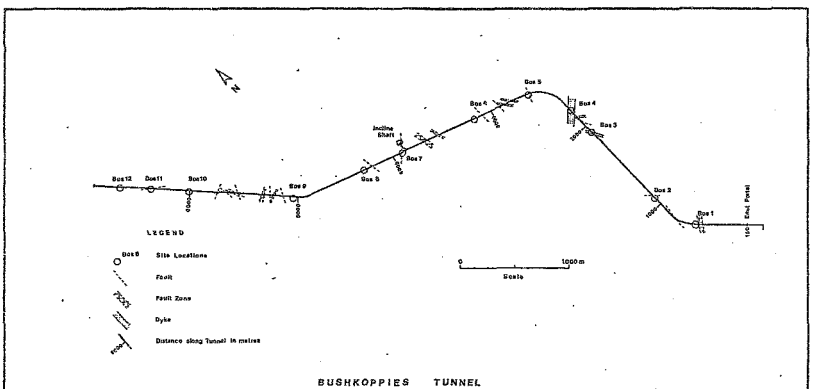


FIG. 3.4 Plan showing Site Locations and Major Geological Features

deteriorated too much to be useful. The sample sites and borehole numbers together with their respective positions in the tunnel and the rock types encountered at each are listed in Table 3.1.

TABLE 3.1

BUSHKOPPIES TUNNEL

LOCATION, NUMBER AND LITHOLOGY OF EACH SAMPLE SITE

Site and Borehole Number	Site Location (in m from S.E. Portal)	Lithology
Bos 1	624	Lava
Bos 2	1076	Lava
Bos 3	1932	Diabase and Lava
Bos 4	2193	Diabase
Bos 5	2621	Lava
Bos 6	3175	Lava
Bos 7	3910	Lava
Bos 8	4295	Lava
Bos 9	5036	Lava
Bos 10	6000	Lava
Bos 11	6357	Lava
Bos 12	6665	Lava

/3.1.2 .....

3.1.2 Regional Geology of the Johannesburg Area

Lithology

Johannesburg is situated on the northern edge of the Witwatersrand sedimentary basin which contains a sequence of argillaceous and arenaceous sediments deposited unconformably, between 2 200 and 2 900 million years ago[60], on a floor of Archean Granites and schist remnants of the Swaziland Supergroup. These sediments are overlain by volcanic and sedimentary rocks of the Ventersdorp, Transvaal and Karroo Supergroups respectively, Fig. 3.5. The succession along a north-south line through central Johannesburg was compiled from Mellor[61, 62] by Pretorius[63], and is presented in Table 3.2.

The Archean Granite occurs to the north of the city. Outcrops in the vicinity of Orange Grove have been described by Horwood[64] as dark grey, biotite granite with some muscovite and Mellor[61, 62] has recorded a variety of amphibole and talc schists and serpentinite remnants within these granitic rocks.

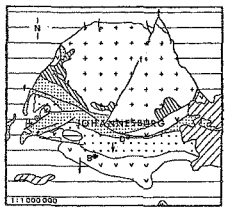
The outcrops of the Witwatersrand Supergroup strike approximately east-west in the vicinity of Johannesburg and assume a gently arcuate shape, concave to the north[63]. The dip is to the south and the dip angle generally decreases stratigraphically upwards in the succession. It has been divided into an Upper Group and a Lower Group.

The Lower Witwatersrand Group consists of the Hospital Hill Formation, the Government Reef Formation and the Jeppestown Formation and contains between 45 per cent and 65 per cent of argillaceous rocks[65]. Petrological work by Fuller[66] has shown that true quartzites occur only in the Hospital Hill Formation, while the quartzitic rocks recorded in the other two subdivisions are sub-graywackes composed of quartz and chlorite with small amounts of muscovite. There is a progressive coarsening of the grain-size of the arenaceous members from the base upwards with those in the Government Reef Formation frequently being gritty[63].

The Upper Witwatersrand Group consists of the Main-Bird Formation and the Kimberley-Elsburg Formation and contains only 10 per cent - 20 per cent of argillaceous horizons[65]. Fuller[66] has indicated that the quartzites of this upper division are in fact hydrothermally altered felspathic quartzites. A similar upward coarsening to that noted in the Lower Witwatersrand is also prevalent.

The volcanic rocks of the Ventersdorp Supergroup occur in two localities in the Johannesburg area. The main one is in the Klipriviersberg

/to the .....



LEGEND

- Karroo Supergroup
- Transvaal Supergroup
- Ventersdorp Supergroup
- Witwatersrand Supergroup (Upper, Lower)
- Ancient Granite & schist remnants
- Fault
- Regional dip direction
- D — Delvers Tunnel
- B — Bushkoppies Tunnel

FIG. 3.5 Regional Geology of the Johannesburg Area

TABLE 3.2

REGIONAL STRATIGRAPHIC SUCCESSION ALONG A NORTH-SOUTH LINE THROUGH THE CENTRAL PORTION OF CENTRAL RAND

Supergroup	Formation	Thickness (m)
Karroo : 61 m	Ecca	61
Transvaal : 1097 m	Pretoria	457
	Dolomite	610
	Black Reef	30
Ventersdorp : 1525 m	Ventersdorp Lavas	1525
Witwatersrand : 7410 m	Kimberley-Elsburg	1860
	Main-Bird	1007
	Jeppestown	1128
	Government Reef	1921
	Hospital Hill	1494
Old Granite	Granite and Gneiss	-
Swaziland : 213 m	Onverwacht	213

Modified after: Pretorius, D.A. 1964

to the south of Johannesburg where the various lava horizons have been referred to collectively[67] as the Klipriviersberg Group. The second less extensive occurrence is present in a downthrown fault block which extends eastward from the centre of Johannesburg[63]. Nicolaysen[60] concluded that these rocks were deposited from volcanic eruptions which took place approximately 2 100 million years ago. The main occurrence to the south of Johannesburg consists of between 1 500 m and 1 700 m of volcanics, which conformably overlie the Witwatersrand Supergroup and unconformably underly the Transvaal Supergroup[67].

The base of the Transvaal Supergroup is marked by the Black Reef, which consists of a scattered pebble-conglomerate underlain by a thin carbonaceous shale and overlain by alternating bands of quartzite and shale[63]. The Malmani Dolomite Formation occurs above the Black Reef, and is composed of dolomitic limestones and cherts. It is followed upwards in the sequence by the Pretoria Formation, which consists broadly of an alternating sequence of quartzites and shales.

The Karroo Supergroup is represented in the Johannesburg area by horizontally disposed sandstones and shales of the Ecca Group. These rocks form an unconformable cover on the Witwatersrand, Ventersdorp and Transvaal Supergroups and become progressively more extensive to the east of Johannesburg[63].

Numerous post-Witwatersrand dykes and sills are present in the Johannesburg area. The oldest and most abundant are considered to be of Ventersdorp age and consist of diabase and epidiorite and are frequently highly altered, sheared and faulted. Other intrusives include those of the Bushveld, Pilansberg and Karroo ages respectively.

The strata of the Witwatersrand Supergroup have suffered low-grade regional metamorphism[68] with the main effects having been the alteration of argillaceous sediments to compacted shales and slates, and the change of the arenaceous members to quartzites with secondary silicification and pyrophyllitization[63]. Local metamorphic effects are common and are often associated with the presence of intrusive rocks.

Structure

The Central Rand forms a portion of the northern limb of the main Witwatersrand synclinorium[63]. The limb dips southward off the Johannesburg-Pretoria granite dome and there is a progressive decrease in the inclination of successively higher strata along a north-south line through Johannesburg. In places the Hospital Hill and Government Reef Formations dip at angles of

/70° .....

70° - 90° while the Jeppestown, Main-Bird and Kimberley horizons dip at 75°, 50° and 40° respectively. A schematic section through the Central Rand[65] is presented in Fig. 3.6

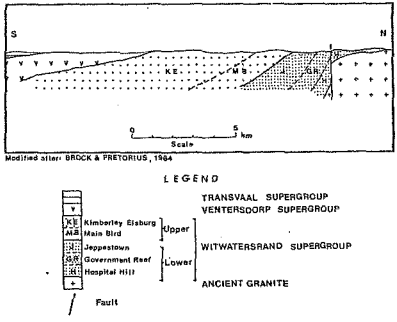


FIG. 3.6 Schematic Section Through the Central Rand

Two approximately perpendicular fold directions have been recognized on the Central Rand. The axes of these folds strike EW and NNW respectively and are both gentle, open structures which, apart from axial-plane cleavage in some shale horizons[63], have had no major influence on the overall structure of the area.

Several longitudinal faults subparallel to the strike of the strata were recognized by Mellor[61] on the Central Rand. The most persistent of these is the Rietfontein fault, which is actually a group of parallel faults dipping north, with a downthrow to the north[62]. They are accompanied by intense crushing and shearing[63]. In the Johannesburg area this displacement has eliminated most of the Government Reef Formation. Reinecke[69] recognized three other groups of major faults, viz. (i) another east-west set accompanied by overthrusting, (ii) a north-west and north-east set with normal displacements, and (iii) a north-south trending set.

/Fault .....

Fault planes are often occupied by dykes and zones of intense mylonitization. The Johannesburg area is also characterized by an east-west striking graben structure containing deeply weathered lavas of the Ventersdorp Supergroup. The boundary faults of this feature are also characterized by intense shearing in the adjacent sediments.

Brock and Pretorius[65] have related these structural features, together with others evident elsewhere, to the geological development of the Witwatersrand basin. The major faults, subparallel to the regional strike; the inwardly decreasing dips of the strata; and the peripheral graben are 'basin edge structures' (op. cit., p.56) which may be expected at the boundary of a sedimentary basin such as the Witwatersrand basin. They are associated with the processes of vertical tectonic adjustment which were active during the deposition of the approximately 7 400 m thick sequence of shallow water sediments and minor volcanic horizons.

3.1.3 Local Geology

The entire tunnel passes through the Klipriviersberg Group of the Ventersdorp Supergroup. A detailed study of these rocks was undertaken by Wyatt[67] who has subdivided them from their base upwards into the Westonaria, Alberton, Orkney and Kibler Park Formations respectively. Generalized descriptions of each Formation are given in Table 3.3.

The tunnel is located in the upper two Formations only, which, together with the Alberton Formation, have been referred to collectively as the upper Klipriviersberg Formations[67]. The general attitude of these rocks is an east-west strike with a shallow (10° - 30°) dip to the south. (Although no positive identification of "bedding" surfaces was made in the tunnel, a flat, south-west dipping joint set appeared to be developed along most of its length and was assumed to represent something similar to a bedding plane.)

The Orkney and Kibler Park Formations are described by Wyatt[67] as being "characterized by the lack of identifiable stratigraphic markers" and are subdivided by him according to the presence or absence of volcanic breccia horizons. The lava type of these upper Formations is characteristic of andesitic basalts and can be classified into three morphological types[67]:

(i) Massive, homogeneous horizons,
(ii) Amygdaloidal horizons,
(iii) Volcanic breccia horizons.

/The massive .....

TABLE 5.3

GENERALIZED GEOLOGICAL DESCRIPTIONS OF THE FORMATIONS COMPRISING THE KLIPRIVIERSBERG GROUP TO THE SOUTH OF JOHANNESBURG

KLIPRIVIERSBERG GROUP	KIBLER PARK FORMATION	Massive, basaltic lava alternating with amygdaloidal and volcanic breccia horizons. Basal volcanic breccia.
	ORKNEY FORMATION	Massive, basaltic lava alternating with amygdaloidal horizons. Volcanic breccia.
	ALBERTON FORMATION	Upper porphyritic zone Porphyritic Marker Mondeor Amygdaloid Massive and amygdaloidal lava, in places containing a porphyritic zone.
	WESTONARIA FORMATION	Meredale Member (lava) Massive basaltic lava with bed of lithic tuff Amphibolite dyke/sill

Modified after: Wyatt, B.A. 1976

The massive flows are greyish-green in colour and have a fine-grained texture. The amygdaloidal horizons contain amygdales varying in diameter from 1 - 100 mm filled by quartz, and lesser amounts of chlorite, clinozoisite and calcite. The volcanic breccias are a characteristic feature of the Klipriviersberg Group and consist of angular to subrounded volcanic fragments, varying in size generally from 10 mm - 100 mm, set in a matrix of finer volcanic material or cemented by secondary minerals such as quartz, chalcedony, calcite and epidote. These breccia horizons are considered[67] to represent ancient flow top breccias. Wyatt concludes that "the bulk of the Klipriviersberg volcanics represent a series of lava flows which were erupted continually without any major interruptions...not unlike modern or recent flood basalts."

Continual logging of the tunnel was not relevant to the problem being studied and detailed examinations of the sample sites only were undertaken. A generalized geological plan has been included, however, Fig. 3.4. Of the three lava types described above only the amygdaloidal and massive horizons were recognized underground. Surface outcrops of the volcanic breccias can, however, be observed above the section of the tunnel between 1 000 m and 2 000 m, and undoubtedly also occur at tunnel level.

In addition to the andesitic lavas, a number of diabase dykes were intersected by the tunnel. The largest of these is approximately 60 m wide and occurs in the vicinity of site number Bos 4. The rock is a typical medium to fine-grained, greenish coloured diabase with fine-grained chill contacts distinguishable from the adjacent lava only after very careful examination. Other narrow dykes were also intersected along the tunnel route. The most significant of these occurs at the location of Bos 3 and consists of a highly sheared chloritic rock with a finely porphyritic texture and a very dark green colour. It is characterized by the development of a fibrous mineral along its boundaries.

A number of faults and fault zones, which appeared to be orientated generally along NW and NNE directions respectively, were intersected in the tunnel. These structures conform generally with the displacements evident on the Black Reef quartzites outcropping approximately 1 - 3 km south of the tunnel route[70]. Localized subhorizontal faulting was evident towards the western end of the tunnel.

3.1.4 Engineering Geology

General Rock Conditions and Support Installations

The Ventersdorp lava exposed during the excavation of the tunnel varied

/from .....

from soft, completely weathered and very closely jointed rock at the east portal, to very hard, almost massive rock along other sections of the tunnel. Over the large proportion of the tunnel the rock is hard, widely to closely jointed, with three or more joint sets and occasional clay filled faults and fault zones, which are associated with more numerous clay coated joints. In general the rock mass provided a good material for the excavation of a small dimension tunnel.

From an engineering geological viewpoint the tunnel can be divided very broadly into four sections. These are:

1. From the eastern portal at 150 m to 624 m,
2. From 624 m to 3 500 m and from 4 000 m to 5 150 m,
3. From 3 500 m to 4 000 m,
4. From 5 150 m to 6 688 m,

and may be described in purely qualitative terms as follows:

1. 150 - 624 m: The weathered rock exposed at the east portal (Plate III) is typical of the type of rock encountered along the first 100 m - 150 m of this section of the tunnel and is related largely to the effects of normal weathering processes; apparently active to about 10 m below the ground surface. Fresher rock was encountered as the excavation advanced, but numerous clay filled joints contributed to the generally unstable nature of the rock mass. Other weathered or partially weathered and closely jointed sections, which appeared to be related to a fairly extensive fault zone, were encountered further in along this part of the tunnel. This was most noticeable at the end of the section. Advance rates were generally slow (18 - 20 m/week)[71] through this section of the tunnel, with hand-held pneumatic diggers used at the start of the portal section, and short (1 - 1,5 m)[71] rounds being broken when the rock hardness necessitated blasting. Heavy support was often required and mesh reinforced shotcrete together with steel arches were used.

2. 624 m - 3 500 m; 4 000 m - 5 150 m: The type of rock along these sections of the tunnel as well as along parts of section 4, is representative of the most common rock condition encountered. With the exception of localized unstable zones caused by minor faults the rock was hard and widely jointed with closed or cemented joints. The excavation outline was generally good with only minor overbreak along more closely jointed sections. Minimum advance

/rates .....

rates of approximately 50 m per week[71] were usually maintained, even though drilling times were often unexpectedly high (2,5 - 3,0 hrs/round)[71] owing to the considerable strength and toughness of the rock. Minimal supports, usually spot-bolting only or none at all, were used. Minor water flows were noted from some of the faults.

3. 3 500 m - 4 000 m: This section of the tunnel is associated with the ground surface depression in the vicinity of the incline shaft. The rock is hard, unweathered and widely jointed, but has a much more blocky appearance than noted along section 2. This is apparently related to the more open nature of the joints, which is indicated by the oxide staining and clay coating on some surfaces. Several major fault zones, containing quite badly sheared rock, and from which moderate water flows were noted, occur within this section. These faults are quite probably the cause of the depressed surface in this area.

   Systematic rock bolting at 1 - 1,5 m intervals was used for support and along one short section, wire mesh was pinned to the crown to prevent the fall-out of small blocks.

4. 5 100 m - 6 688 m: The rock mass along this section of the tunnel is characterized by numerous, intersecting clay filled joints and a number of quite major faults. (The largest of these has been shown on Fig. 3.4.) Most of these features follow the NW, NNE and subhorizontal trends mentioned previously. The rock is hard and widely to closely jointed and weathering, particularly adjacent to the better developed joints and minor faults, becomes more prevalent as the crown depth decreases westward. Moderate water flows were noted in places.

   As indicated above, sections of better quality rock also occur along this part of the tunnel. Advance rates were therefore variable from about 30 m per week through the poorer sections to 40 m, and occasionally 50 m[71], per week where rock similar to that described for 2 above was encountered. Owing to the tendency for the rock mass to loosen as a result of the clay joint fillings it was often necessary to install quite closely spaced rock bolts (0,5 m in some places)[71] to ensure stability. Along particularly bad sections, shotcrete and steel arches were used.

   Overbreak was most noticeable when well-developed subhorizontal

/joints .....

joints were present and a few minor fallouts occurred from the larger fault zones. Owing to the limited width of these features, however, they tended to stabilize themselves 1 - 1,5 m above the crown level.

The presence of clay along the joints was another factor which greatly influenced the advance rates in that it tended to clog the drill holes before the charges could be placed. The drill rods also often jammed under these conditions.

Uniaxial compressive strength tests carried out for the geotechnical investigation yielded a mean value of 366,9 MPa for these rocks[70]. A mean density of 2,89 gm/cc was also calculated.

As indicated above, moderate water inflows occurred from some of the fault zones. In general these flows decreased rapidly after the initial intersection of the fault and at no stage did water inflow pose a problem. An interesting feature was that the flow rate was closely related to the local weather conditions and showed a significant increase after periods of prolonged rainfall. No actual measurements were taken, however.

Rock Conditions at the Sample Sites

The joint spacing descriptions given below have been adopted from those suggested by the South African Section of the Association of Engineering Geologists, and are presented in Appendix IIIA. The stereographic plots of the joint survey data relevant to each of the sites are included in Appendix IIIC.

Pos 1:

The rock consists of hard, widely jointed, dark greenish-grey lava with three, or possibly four major joint sets orientated with dip angles and dip directions as follows:

$68^{\circ}/192^{\circ}$;

$85^{\circ}/306^{\circ}$;

$30^{\circ}/256^{\circ}$;

$85^{\circ}/085^{\circ}$.

Owing to the unfavourable orientation of the $30^{\circ}/256^{\circ}$ joint set with respect to the joint survey line, only one joint was recorded. Its significance as a major joint set was, however, established by the presence of other similarly orientated joints above and below the tape.

/The joint .....

The joint surfaces are generally undulating and rough with reddish coloured clay coatings and oxide staining. Clay gouge, with a maximum thickness of 10 cm, was characteristic of the 30°/256° joint set and resulted in poor rock conditions in the crown of the tunnel, i.e. overbreak or loose blocks, when a joint occurred immediately above the excavation. Randomly spaced rockbolts were generally used as support.

An interesting feature about this site, which emphasized very clearly the danger of interpolation between boreholes, was that within a distance of 2 - 3 m from the borehole intersection point, the rock mass quality had deteriorated to such an extent to warrant the installation of steel arches for support, Plate IV.

Bos 2:

At this site the rock consists of hard to very hard, widely jointed, greenish coloured lava with three main joint sets, characterized by closed, rough and undulating or planar surfaces. The dips and dip directions of these joint sets are as follows:

90°/315°;
90°/243°;
20°/243°.

No supports were installed at this site and the outline of the excavation was good.

Bos 3:

The in situ rock conditions at this site are very different to those indicated by the borehole core. In general the rock consists of hard, widely jointed, dark green lava which has been intruded by a narrow (0,5 - 0,7 m wide) dark greenish-black dyke of medium-hard to soft rock and with close to very close joints, Plate V. Four main joint sets occur at this site. Their orientations are as follows:

90°/218°;
85°/123°;
25°/251°;
90°/136°.

The joint surfaces in the lava are planar or undulating and may be either rough or smooth. Most joints are closed but staining and clay or chlorite coatings do occur. These conditions are generally applicable to the

/rock .....

rock mass at this site.

Within and adjacent to the dyke, the joints are undulating, smooth and slickensided. Joint alteration products consist of soft, chloritic material and a fibrous amphibole mineral which is typically developed along the contact joints. This latter feature, together with the distinctive fine-grained, porphyritic texture of the dyke rock, enabled the same dyke to be recognized in other parts of the tunnel.

No support was used at this site.

Bos 4:

The rock consists of very hard, very widely jointed, greyish-green coloured, medium to fine-grained diabase. The main joint sets are orientated as follows:

$70^{\circ}/000^{\circ}$;

$80^{\circ}/344^{\circ}$;

$80^{\circ}/236^{\circ}$.

The joint surfaces are planar, rough and closed.

Owing to a chill contact zone in the diabase, it was very difficult to pinpoint the lava-diabase contact. A careful examination of the rock, however, revealed a closed but continuous joint which was assumed to be the contact.

No supports were installed at this site and the excavation outline was good.

Bos 5:

At this site the rock consists of hard, widely jointed, greenish coloured lava with three joint sets orientated as follows:

$25^{\circ}/198^{\circ}$;

$90^{\circ}/317^{\circ}$;

$70^{\circ}/113^{\circ}$.

The joint surfaces are undulating or planar and rough, and are usually closed or coated with non-softening minerals such as quartz or calcite.

This site is situated a few metres west of a fault initially predicted from the site investigation studies[70]. The fault is a narrow feature (0,25 - 0,5 m) and crosses the tunnel at a high angle with a strike of approximately 022° from true north. Moderate water flow was evident when the fault was first intersected, but this soon decreased. The fault had no effect on the stability of the tunnel and no supports were installed.

/The tunnel .....

The tunnel outline was generally good.

Bos 6:

The rock consists of hard, widely to very widely jointed greenish lava with three main joint sets with undulating or planar and rough surfaces. Most joints are closed and cemented or coated with calcite and quartz. The dips and directions of the respective sets are as follows:

$90^{\circ}/046^{\circ}$;

$90^{\circ}/116^{\circ}$;

$37^{\circ}/000^{\circ}$.

No supports were installed but slight overbreak was evident in the crown of the excavation. The $37^{\circ}/000^{\circ}$ joint set appeared to be the main contributor to this.

Bos 7:

The rock consists of hard, widely jointed, greyish-green lava with four well-developed joint sets giving the rock mass a blocky appearance. The orientations of the joints are as follows:

$84^{\circ}/238^{\circ}$;

$85^{\circ}/302^{\circ}$;

$45^{\circ}/228^{\circ}$;

$90^{\circ}/006^{\circ}$.

Most joint surfaces are planar and either rough or smooth. The joint alteration varies from closed to iron stained and occasional thin clay coatings. Minor seepage was noted in places. Owing to the blocky nature of the rock mass, the tunnel outline was poor. Systematically spaced rockbolts at 1 - 1,5 m intervals were installed as support.

Bos 8:

Hard to very hard, greenish lava occurs at this site. Four joint sets, both widely and closely spaced, with the following orientations are present:

$85^{\circ}/132^{\circ}$;

$80^{\circ}/240^{\circ}$;

$75^{\circ}/030^{\circ}$;

$33^{\circ}/180^{\circ}$.

/The joint .....

The joint surfaces are planar and rough or smooth and with the exception of the last set listed above, are closed or display iron staining only. The 33°/180° set has minor clay coatings in places. One particular joint with a similar attitude contains 5 - 10 cm of chlorite mica and shows evidence of having acted as a minor shear plane with approximately 15 cm of movement along it. This is, however, a very localized phenomenon and has no influence on the stability of the rock mass. Minor seepage was noted. The excavation outline was generally good and spot bolting only was used as support.

Pos 9:

At this site the rock consists of very hard, very widely jointed, greenish lava with three joint sets orientated as follows:

90°/319°;

75°/250°;

76°/099°.

The surfaces of the joints are undulating and rough when exposed, but are generally closed and silica cemented giving the rock a massive appearance. The excavation outline at this locality was very good and no support was necessary.

Pos 10:

The rock is hard, greenish lava, generally widely jointed but with occasional very closely jointed zones. Three main joint sets which exhibit planar and rough surfaces most frequently are present, and iron staining and clay coatings are common. The orientations of these sets are as follows:

90°/277°;

47°/168°;

80°/168°.

Fillings of clayey sand, up to 5 mm thick, occur along the 90°/277° joint set and localized zones of weathered and discoloured rock are frequently associated with this and the 80°/168° set. The rock has a blocky appearance and overbreak in the crown of the tunnel appeared to be related to the 47°/168° joints. No supports were used at this site. Minor seepage was evident.

/Pos 11 .....

Bos 11:

At this site the rock is hard, widely jointed greenish lava with four main joint sets resulting in a blocky appearance. The dips and dip directions of the joints are as follows:

$90^{\circ}/156^{\circ}$;

$90^{\circ}/004^{\circ}$;

$10^{\circ}/247^{\circ}$;

$90^{\circ}/083^{\circ}$.

The surfaces of the joints are planar and rough or smooth, with staining and clay coatings. One of the $10^{\circ}/247^{\circ}$ joints, exposed at the invert level, contains a wide zone of clay and broken rock together with vein quartz and micaceous material, Plate VI. This feature possibly represents a localized fault plane. This particular joint rose gradually up to the crown level of the tunnel over a distance of approximately 40 m where it became a major weakness plane and resulted in overbreak in the crown. Minor seepage was evident and at the site itself spotbolting only was used as support.

Bos 12:

The rock is hard, widely jointed greenish lava with four main joint sets orientated as follows:

$85^{\circ}/272^{\circ}$;

$90^{\circ}/106^{\circ}$;

$85^{\circ}/250^{\circ}$;

$80^{\circ}/010^{\circ}$.

The joint surfaces are generally planar and rough with clay coating and staining being common. In addition to the joint sets indicated, a sub-horizontal joint containing clay gouge up to 5 cm thick also occurs in the vicinity of this site and, as at other localities, has contributed significantly to the crown overbreak. It has, however, not been included in the assessment of the rock mass conditions at the site itself. Occasional discoloured weathered zones occur adjacent to some of the joints.

The rock mass has a blocky appearance and the excavation outline was poor. Minor seepage was evident. No supports were installed at this site.

/3.2 .....

3.2 The Delvers Tunnel

3.2.1 Introduction to the Project

The Delvers Tunnel is situated in the central area of the city of Johannesburg and follows a route from the corner of Troye and Marshall Streets, westward along Marshall Street to Delvers Street, northward along Delvers Street to Bree Street and then westward along Bree Street to its intersection with Von Brandis Street, Fig. 3.7. The total length of the tunnel is 1,2 km and it was planned by the Electricity and City Engineers Departments of the City of Johannesburg as part of an expansion program to house feeder and distribution cables for a new central city electricity substation[72].

The tunnel is close to the ground surface and the cover depth to its crown varies from 2,5 m to approximately 11 m[72]. The shape selected was that of an inverted "U" with vertical sidewalls and equal vertical and horizontal finished dimensions of 2,0 m. The tunnel is lined throughout with concrete approximately 300 mm thick, Fig. 3.8.

A vertical access shaft was constructed at the intersection of Kerk and Delvers Streets and two headings were advanced simultaneously in north and south directions respectively. Excavation methods included both conventional drill and blast techniques and hand-held pneumatic diggers and temporary supports, usually in the form of steel arches spaced approximately 0,75 - 1,0 m apart, were installed as close to the face as possible. Continuous lagging was necessary along some of the very soft rock and highly jointed sections of the tunnel. The concrete lining operation followed some 30 m behind the advancing face.

Six sample sites were studied in this tunnel, Fig. 3.9. The borecores examined were obtained from boreholes drilled during the preconstruction geotechnical investigation and the site selection was based on the quality of the cores recovered. The softer materials recovered from those boreholes which intersected residual horizons or weathered shales had deteriorated too greatly to be of any use. The details of the sample sites are listed in Table 3.4.

3.2.2 Regional Geology

The regional geological setting and structure of the Johannesburg area has been already described in 3.1.2 and need not be repeated.

/Table 3.4 .....

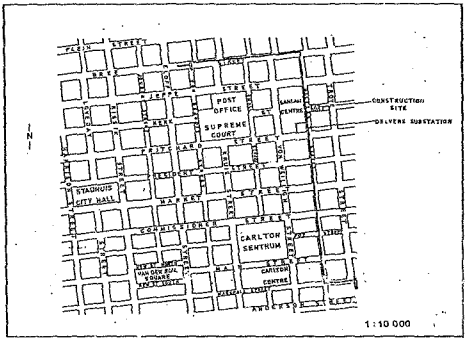


FIG. 3.7 Delvers Tunnel - Locality Map

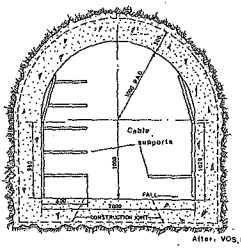


FIG. 3.8 Delvers Tunnel - Cross-section

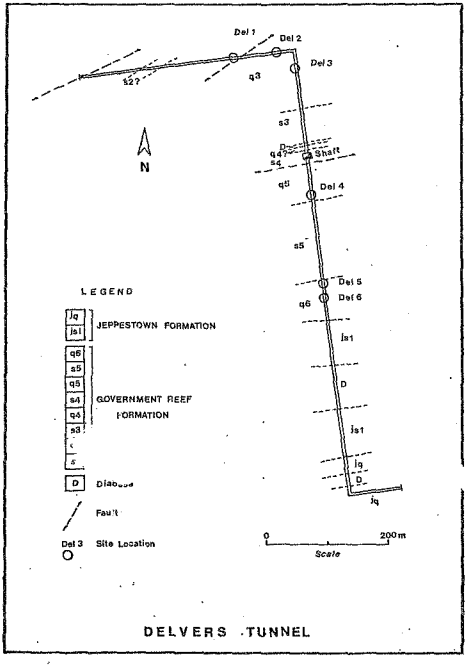


FIG. 3.9 Plan showing Site Locations and General Geology

TABLE 3.4

DELVERS TUNNEL
LOCATION, NUMBER AND LITHOLOGY OF EACH SAMPLE SITE

Sample Site No.	Site Location (in m from N end of tunnel)	Lithology
Del 1	255	Sheared Argillaceous Quartzite
Del 2	330	Sheared Argillaceous Quartzite
Del 3	385	Quartzite with Argillaceous intercalations
Del 4	590	Quartzite
Del 5	735	Quartzite
Del 6	760	Quartzite with Argillaceous intercalations

3.2.3 Local Geology

The Delvers Tunnel passes through alternating arenaceous and argillaceous strata of the Government Reef and Jeppestown Formations of the Witwatersrand Supergroup and, for a very short distance at its northern end, through the sheared contact fault zone at the southern edge of the graben previously described. A generalized geological record of the tunnel was compiled during its excavation and is presented in Fig. 3.9. Although continuous logging of the geology was not really pertinent to the problem being studied, it was nevertheless considered worthwhile because it provides engineering geological information for any future work planned in the area, as well as updating the existing geological map of Johannesburg compiled by J.H. de Beer[73].

In order to be consistent with existing terminology, the nomenclature for the horizons intersected by the tunnel were adopted from de Beer. These are presented in Table 3.5 with their approximate locations along the tunnel route, measured from zero metres at its northern end. The geology along the tunnel route is discussed briefly below and a profile and geological section along the tunnel route are presented in Fig. 3.10.

/Table 3.5 .....

TABLE 3.5

DELVERS TUNNEL
GEOLOGY ALONG THE TUNNEL ROUTE

Horizon		Location along Tunnel (from zero metres at the north end of the tunnel)
SHEARED FAULT ZONE		0 - 38 (approx.)
GOVERNMENT REEF FORMATION		
Quartzite	(q3)	38 - 450
Shale	(s3)	450 - 495
Diabase	(D)	495 - 503
Quartzite	(q4)	503 - 512
Shale	(s4)	512 - 535
Quartzite	(q5)	535 - 600
Shale	(s5)	600 - 732
Quartzite	(q6)	732 - 788
JEPPESTOWN FORMATION		
Arenaceous Shale	(jsl)	788 - 870
Diabase	(D)	870 - 929
Arenaceous Shale	(jsl)	929 - 1018
Quartzite	(jq)	1018 - 1044
Diabase	(D)	1044 - 1058
Quartzite	(jq)	1058 - 1180

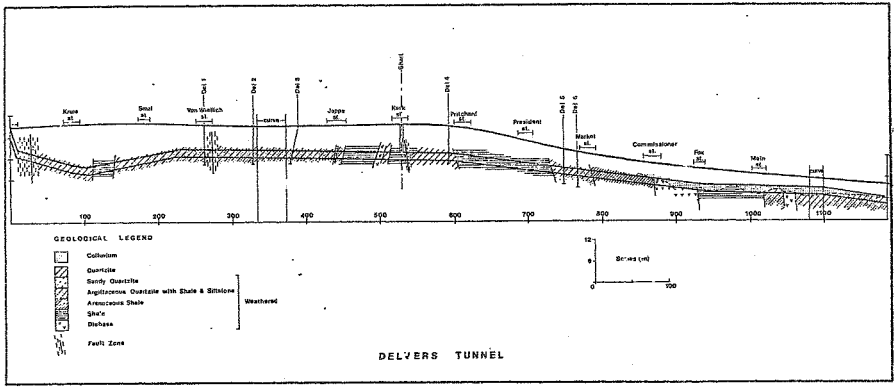


FIG. 3.10 Profile and Geological Section along Tunnel Route

Sheared Fault Zone

The rocks exposed along this section of the tunnel consist of highly sheared remnants of weathered quartzite, argillaceous quartzite, shale and lava. The finer-grained rocks cannot be differentiated with any real certainty, but the yellowish colour of some of the fragments, compared to the typical pink and red coloured argillaceous sediments, suggested that they are probably lava. The dominant structure follows a NE-SW direction and the fault appears to be vertical.

Government Reef Formation

Quartzite (q3) - This horizon is located to the south of the Ventersdorp graben and consists of alternating bands of quartzite, talcose argillaceous quartzite, and subordinate shale and siltstone. It has been subjected to intense shearing parallel to the regional strike and the rocks vary from very soft to moderately hard, are closely jointed and weathered, and contain talc, chlorite and clay minerals along joint surfaces and within the rock itself. The dominant joint direction strikes east-west and dips steeply to the south. Occasional subhorizontal joints also occur.

De Beer has indicated that shales previously reported within this horizon are in fact extremely sheared quartzite. This was confirmed in the tunnel, where examples were encountered in which the original quartzite was only evident after a careful examination of the rock. A very clear example of this is adjacent to the fault at 273 m. Shale bands were also noted, however, and could be recognized by their tendency to be less fractured than the quartzitic horizons. A particularly obvious shale horizon was noted from 112 m - 141 m and may actually be part of the S2 shale which could have been squeezed out along the fault zone rather than terminating abruptly against it as shown by de Beer.

Shale (s3) - The s3 horizon consists of shale and arenaceous shale. The northern contact appears to conform with the underlying q3 horizon and dips at approximately 75°N. At tunnel level the rock is soft and closely jointed with residual zones of clayey silt. The dominant joints are parallel to the strike and may be either bedding planes or shear surfaces. The southern contact of this horizon is marked by a narrow diabase intrusion which consists of residual sandy clayey silt.

Quartzite (q4) - A narrow horizon of hard and moderately jointed

/quartzite .....

quartzite occurs from 503 m to 512 m. This has been tentatively correlated with de Beer's q4 quartzite.

Shale (s4) - This horizon is similar to the s3 horizon described above. The southern contact is a silicified fault zone with an apparent downthrow to the north. The presence of this fault was noted by de Beer although he was unable to locate it exactly. His opinion that the diabase intrusion is related to the fault is probably correct and the narrowness of the quartzite horizon between the diabase and this horizon suggests that the fault is more complex than indicated.

Quartzite (q5) - *This horizon represents the most competent rock type* intersected by the tunnel. It consists of grey, medium-grained and glassy in places, unweathered and moderately jointed, hard rock quartzite.

Shale (s5) - At tunnel level this horizon is soft, dark red and highly jointed with occasional zones, particularly towards the southern contact, of green, medium hard more slaty rock.

Quartzite (qt) - This horizon consists of a dominant argillaceous quartzite component with bands of siltstone and shale. It is generally soft to very soft, maroon and grey coloured, weathered, friable and moderately jointed. The shale and siltstone horizons are often associated with residual zones of soft, clayey sand.

Jeppestown Formation

De Beer has indicated that the sediments of the Jeppestown Formation are "characterized by their general fineness of texture and the ease with which they yield to weathering." Along that part of the tunnel where these rocks were encountered, they were usually completely weathered to residual materials and were often only exposed close to the invert level of the tunnel with the major part of the excavation in the overlying colluvium.

Arenaceous Shale (jsl) - As shown on de Beer's map, this horizon occurs in two sections divided by a major diabase intrusion. The northern section consists of yellow, pink and grey, dense and jointed, clayey silty sand with micaceous horizons and pockets of firm clay, probably derived from the overlying colluvium. To the south of the diabase the material is softer and contains a higher percentage clay. The colluvium is also exposed along this section. The strike of the strata is east-west

/and the .....

and the dip vertical.

The diabase intrusion is approximately 60 m wide and consists of reddish-orange, firm, silty clay at the tunnel level. Its width, greater than that suggested by de Beer, may be due to the displacement, reported by de Beer, along the fault associated with the Grahamstown Dyke to the west of the tunnel, or may indicate that the Grahamstown Dyke is located slightly to the east of the position shown on de Beer's map, and has been intersected by the tunnel.

Quartzite (jq) - This horizon was exposed 0,5 - 1,0 m above the invert level of the tunnel and consists of soft, yellowish to pinkish coloured, weathered and friable, moderately jointed medium-grained quartzite. The upper 20 - 30 cm of the horizon is usually characterized by clay-pockets, derived from the overlying colluvium.

A diabase intrusion, recognized during the site investigation(74), also divides this horizon. The position of this body could not be located accurately within the tunnel because only a very small exposure of yellowish-brown, dense sandy silt, which may or may not be residual from diabase, could be observed close to the invert of the tunnel.

Colluvium

The overlying colluvial horizons were first noted in the crown of the tunnel at about 850 m from its northern end. No detailed soil analyses have been undertaken but soil profiles were recorded at intervals of 10 - 30 m. The soil type appears to be reasonably consistent but becomes slightly less cohesive southwards. Variations in the degree of ferruginization and thickness of the pebble marker were also noted. In general the soil profile consists of reddish coloured, firm clayey sand becoming stiffer and more ferruginized with depth and with a pebble marker varying between 0,01 m and 0,1 m in thickness at the base. In the vicinity of the jq quartzite horizon the colluvium contains soft, slightly ferruginized nodules which seem to lower its cohesion.

3.2.4 Engineering Geology

General Rock Conditions and Support Installations

Few problems were encountered during the excavation of this tunnel and an average excavation rate of about 25 metres per week was achieved. Except for minor differences, the temporary support installations were the same

/for the .....

for the full length of the tunnel. These have been discussed briefly already.

For engineering purposes the tunnel may be divided broadly into three main sections defined by differences in the overall rock mass properties with respect to the tunnel, differences in the excavation method and differences in the distance at which the initial arch supports were installed behind the face. The three sections are from 0 - 350 m; 350 - 850 m; and 850 - 1180 m respectively and are discussed briefly below.

0 - 350 m : This section is along that part of the tunnel running parallel to the regional strike and dominant joint directions. The steep dips and the chlorite and clay coated joint surfaces resulted in unstable sidewall conditions with minor failures in places, particularly along the north wall of the tunnel. This was easily controlled by wedging behind the arches. The excavation method was drill and blast and the arches were usually installed within a metre of the face.

350 - 850 m : This section of the tunnel includes that part in which the strike of the dominant structures is perpendicular to the tunnel axis, resulting in more stable rock mass conditions. Although the rock is still heavily jointed in places, the chlorite and clay coatings on the joints become less common as the distance from the graben fault increases. With the exception of the narrow dyke between 495 and 503 m which was excavated by hand, this section was excavated using drill and blast methods. The temporary support was installed approximately 3 - 5 m behind the face.

850 - 1180 m : This section of the tunnel is located in colluvium and residual soils derived from the various geological horizons intersected. Hand-held pneumatic diggers were used for the excavation of the tunnel and owing to the suspected short stand-up time of the ground the arches were installed within a metre of the face. Sections of unstable sandy material, which necessitated the installation of continuous lagging in the crown, were encountered in the vicinity of 1100 m.

Rock Conditions at the Sample Sites

Site Del 1:

The rock exposed at this site consists of medium hard to soft rock, maroon and pink coloured, closely jointed and friable, weathered and sheared, argil[illegible]s quartzite with thin shaly bands. Two major joint sets occur.

/These .....

These are as follows:

1. Shear joints (bedding) - These joints strike approximately east-west and have a vertical or steep southerly dip. Joint surfaces are rough and both planar and undulating, with clay or talc coating. The main through-going surfaces are spaced at about 20 cm intervals although closer more irregular joints are also present.

2. Cross joints - These joints have a subhorizontal attitude, are planar and rough and are characterized by red-clay infilling up to 5 mm thick. An average spacing of about 0,5 - 1,0 m is common, although more closely spaced joints also occur.

The rock at this location appears to soften on exposure to the moist air in the tunnels. Although only two main joint sets were noted the rock could possibly be classified as "highly jointed." Owing to the unfavourable attitude of the shear joints, unstable conditions were prevalent along the sidewalls of the tunnel.

Site Del 2:

At this site the rock consists of soft to hard, micaceous and talcose argillaceous quartzites and quartzites which have been subjected to intense shearing along an east-west direction. Two major joint sets occur with the dominant set related to the shearing. These are as follows:

1. Shear joints (bedding) - An east-west strike and vertical or steep southerly dip is characteristic of this set. Joint surfaces are undulating and irregular although they may be planar in places. Talc coatings are common and bands of sandy clay up to 5 cm thick may occur. The spacing of the joints is approximately 10 cm.

2. Cross joints - These joints are very irregular and may classify as "random" joints. They are undulating and rough when present and are filled with stiff clay gouge.

The less argillaceous varieties of quartzite generally have a more blocky appearance and display more planar joint surfaces.

Site Del 3:

The rock at this site consists of hard, highly jointed, coarse to medium grained quartzite with occasional weathered argillaceous horizons. Joints present are as follows:

/1. Bedding .....

1. Bedding joints - These joints are vertical and strike east-west. They are iron stained, undulating and irregular and usually spaced between 50 and 300 mm.

2. Cross joints - These joints are north-south striking and dip vertically. Surfaces are undulating and rough and iron stained, and spaced at 0,5 - 1,0 m intervals.

3. Cross joints - These joints strike east-west and dip shallowly to the south. They are undulating and rough, stained and spaced at 0,5 - 1,0 m intervals.

Site Del 4:

At this site the rock consists of hard, grey jointed, weathered quartzite, with three main joint directions as well as random joints. The condition of the various sets is as follows:

1. These joints strike north-south and dip approximately 30°W. They are generally undulating and rough with iron staining and occasional zones, up to 10 cm wide, of weathered rock adjacent to them. The spacing is about 30 cms.

2. This joint set also strikes north-south but has a vertical dip. Joint surfaces are planar and rough but sometimes discontinuous. Iron staining is common and the general spacing is approximately 0,5 m.

3. These joints are not very clearly defined. Their attitude appears to be north-west striking, with a dip of about 60° to the south west. Exposed surfaces are undulating and rough and are usually iron stained. The spacing of the joints could not be determined.

Site Del 5:

Maroon, buff and grey coloured, soft to medium-hard rock, weathered, friable, medium-grained quartzite is present at this site. Three main joint sets occur as follows:

1. Bedding joints - These joints strike east-west and dip 80 - 85°N. Undulating and rough, and planar and smooth joint surfaces are present with the latter usually confined to the finer-grained rocks. Most surfaces are iron stained while clayey horizons, derived from the weathering of slightly argillaceous horizons,

/are .....

are also present. A spacing of 0,5 - 1,0 m is average.

2. Cross joints - These joints display an east-west strike and a flat northerly dip. They are planar and rough, iron stained and are usually spaced at 0,5 - 1,0 m.

3. Subparallel joints - These joints strike north-south and have a vertical or steep easterly dip. They are undulating and often filled with firm to stiff, sandy clay gouge with a thickness of 5 - 10 cms. Spacings are usually about 0,5 - 1,0 m.

Site Del 6:

At this site the rock consists of maroon and grey, soft to medium-hard, weathered, friable quartzite with finer bands of shale and siltstone. Three main joint sets are present, together with random joints:

1. Bedding joints - These joints strike east-west and dip steeply to the north ($\pm$ 70°). Surfaces are planar and rough and are coated with red clay up to 5 mm thick. The spacing between the joints varies from 50 mm to 300 mm.

2. Cross joints - These joints strike east-west and dip 20° - 25° south. They are also planar and rough and clay coated up to 5 mm in thickness. Joint spacings vary from 50 - 300 mm with the latter being more common.

3. This joint set is not as well developed as the previous two. The general strike direction is SSW with a dip of about 80°E. Surfaces are usually planar and rough with no apparent coating. The spacing appears to vary from 0,3 m up to 3,0 m.

3.3 The Du Toitskloof Tunnel

3.3.1 Introduction to the Project

The Du Toitskloof Tunnel is situated approximately 50 kilometres to the east of Cape Town, Fig. 3.11. It was planned in order to extend Route 1 of the South African National Freeway System eastwards from Paarl, under the Klein Drakensteinberge, to Worcester. This was considered necessary because there is no economic or suitable location over the mountains for a modern freeway close to the direct Paarl to Worcester line. The planned main tunnel is to be nearly 4 kilometres long and will comprise a horse-shoe

/shaped .....

shaped excavation with a horizontal dimension of approximately 12 m and a height of about 9 m. Its depth below surface will vary from 10 m - 20 m at the portals to approximately 700 m beneath the crest of the mountains. The ultimate aim of the scheme is to have two tunnels, 36 m apart, each carrying unidirectional traffic(75).

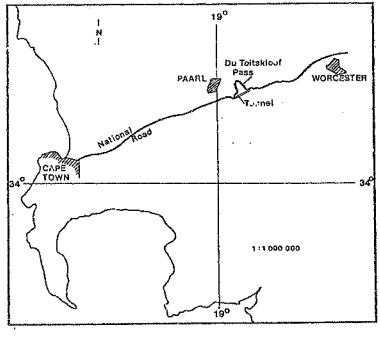


FIG. 3.11 Du Toitskloof Tunnel - Locality Map

/The pilot .....

The pilot tunnel is a very much smaller excavation with equal horizontal and vertical dimensions of approximately 3,0 m, and a horse-shoe shaped cross-section which was varied according to the local rock conditions and support requirements, Fig. 3.12. It is situated along the centre-line of the northernmost future main tunnel and its main purpose is to provide geological and geotechnical data for the final design of the main tunnel[75]. The excavation was undertaken largely by conventional drill and blast methods from two headings advancing inwards from the east and west portals respectively. Forepoling and freezing techniques were used for excavating through the decomposed granite at the west portal[84].

Six sites exposed during the tunnel excavation were selected for the purpose of this study, Fig. 3.13. The sites were chosen primarily on the basis of available borehole cores and the amount of exposure in the tunnel, since shotcrete had been used as a means of support along some of its sections. They were situated in both sheared and unsheared rock and provided examples of a variety of rock conditions. The rock types, corresponding sample site numbers and related boreholes, together with their respective positions in the tunnel measured from the east portal, are listed in Table 3.6.

The bore cores which were used for this study were obtained from ten holes drilled horizontally or vertically from within the tunnel for exploratory and geotechnical information. The boreholes had, in most cases, been drilled only a short time prior to the study and the cores had not been excessively disturbed. They were thus more suitable for the required purpose than the cores from the holes drilled from surface during the initial investigation stages of the project which were, in addition, usually not located close enough to the finally selected tunnel route for a meaningful comparison to be made between the bore cores and the rock conditions in the tunnel.

Each borehole core was evaluated separately for comparison with the in situ measurements. No attempt was made to combine the borehole data at those sample sites from which both vertical and horizontal boreholes had been drilled. Although it was realized that better comparisons would probably have been achieved had this been done, the results obtained would not have been directly related to the other examples studied for this dissertation, where only one borehole core was available at each site.

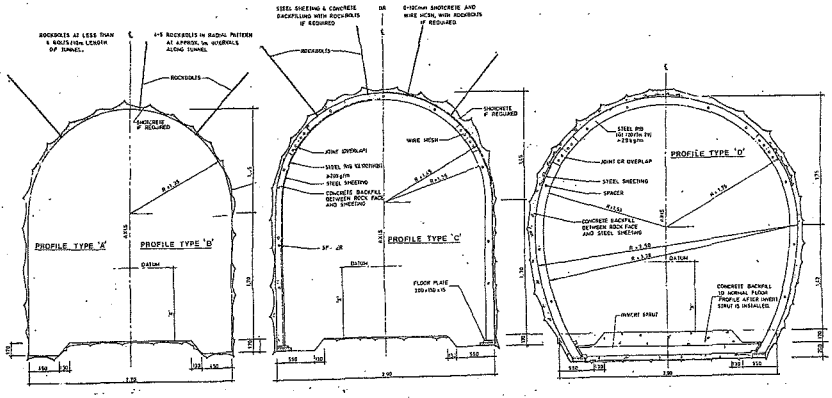
/Table 3.6 .....

TABLE 3.6

DU TOITSKLOOF TUNNEL
LOCATION, NUMBERS AND LITHOLOGY OF THE SAMPLE SITES

Site Number	Borehole Numbers at each Site	Site Location (in m from E. Portal)	Lithology
Dut-1	Dut 1-H * Dut 1-V	383	Quartzitic Sandstone
Dut-2	Dut 2-H Dut 2-V	522	Quartzitic Sandstone
Dut-3	Dut 3-V	633	Weathered and brecciated Quartzitic Sandstone
Dut-4	Dut 4-H Dut 4-V	704	Sheared Granite
Dut-5	Dut 5-V	1005	Granite
Dut-6	Dut 6-H Dut 6-V	1535	Granite

* H - Horizontal
V - Vertical



After : VAN NIEKERK, KLEYN & EDWARDS, et al. 1978

FIG. 3.12 Du Toitskloof Tunnel - Cross-sections and Support Installations for Different Rock Classes

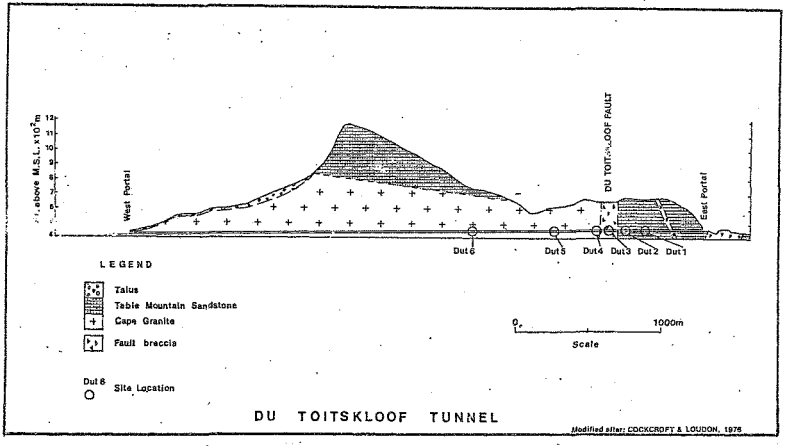


FIG. 3.13 Profile and Geological Section along Tunnel Route

3.3.2 Regional Geology

Lithology

The Du Toitskloof Tunnel occurs in an area of folded and faulted igneous and sedimentary rocks, in which the major structural trends follow a NW to NNW direction, Fig. 3.14. The oldest rocks in the area are the Malmesbury Beds. They have been intruded by the Cape Granites, which are approximately 553 million years[76], and are regarded as late syn-orogenetic intrusions of the Malmesbury orogenic cycle. The Table Mountain sandstones constitute the youngest of the major geological sequences in the area. Younger, bouldery talus deposits occur on the slopes of the mountains, and thick accumulations of alluvium are present in the low-lying drainage basins.

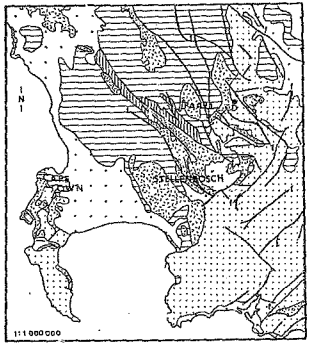
The Malmesbury Beds consist predominantly of micaceous slates and argillaceous quartzites, with occasional limestone bands and local developments of basic lava. They have been highly deformed and occur now as tight NNW striking synclinal and anticlinal folds, which have been intruded by similarly trending, elongate bodies of Cape Granite. Low-grade regional metamorphism has resulted in the formation of phyllites, marbles and quartz-sericite schists, while thermal metamorphic effects are also evident around the aureoles of the intruded plutons[77].

The Cape Granites are variable but consist mainly of coarse-grained, porphyritic biotite-granites with the phenocrysts often consisting of the potassium felspar, microcline[78]. Aplitic and gneisaic varieties are also found.

The Table Mountain Sandstones form the lower unit of the Cape Supergroup and give rise to the most prominent relief features in the area. They have been divided into six principal units[79] which are listed in descending order:

Nardouw	-	Coarse-grained, orthoquartzites
Cedarburg	-	Shale, siltstone and fine-grained sandstone
Pakhuis	-	Glacial mudstones
Peninsula	-	Coarse-grained, orthoquartzites
Graafwater	-	Purple, thinly-bedded sandstone, siltstone and shale
Piekenier	-	Conglomerate and sandstone

/The thickness .....



LEGEND

- Tertiary to Quaternary
- Cape Supergroup
- Klipheuvel Formation
- Cape Granite
- Malmesbury Beds
- Faults
- Du Toitskloof Tunnel

FIG. 3.14 Regional Geology of the Cape Peninsula

The thickness of the Table Mountain Sandstones in the Du Toitskloof area is of the order of 2 000 m[79].

Structure

As already mentioned, the major structural trends in the area follow a NW to NNW direction. This trend has been called the 'Cedarberg direction'[80], and is in contrast to the east-west 'Swartberg direction' which is the main structural direction of the Cape Fold Belt. The boundary between these two structural provinces runs in a south-westerly direction from the vicinity of Worcester and takes the form of a monoclinal fold which De Villiers[81] has called a fold-front. This syntaxis is accompanied by similarly trending faults.

The structures in the Malmesbury Beds, in the area to the west of the syntaxis (Cedarberg direction), are tight and isoclinal, while those in the overlying Table Mountain Sandstones, in the same area, are broad and open. The faulting is more complex and De Villiers[81] has described reverse, tear and normal faults. There appears to be little doubt that many of these faults have had a long and complex history, having been active from pre-Cape times to the post-Cretaceous[80].

Seismicity[32]

On the 29th September, 1969, an earthquake with an estimated magnitude of 6,3 on the Richter Scale occurred in the Ceres-Tulbagh region of the Cape Province. This was followed by a number of after-shock events, which had almost ceased when another quake (magnitude 5,7) occurred on April 14th, 19[illegible]. Prior to September, 1969, the Ceres area had no recorded history of major seismic activity other than very small tremors felt by people on rare occasions.

The main fault in the area is the Worcester fault zone, which has a displacement of over 4 000 m in places. There is, however, no evidence that this zone is still active.

A study of the after-shock events by the Bernard Price Institute of Geophysical Research indicated an approximately vertical, planar fault with a NW strike direction, situated to the NE of Tulbagh and Ceres. The work showed that although the fault plane did not coincide with any physiographic or geological features in the region, its strike was, nevertheless, almost parallel to other major faults in the area.

/3.3.3 .....

3.3.3 Local Geology

The planned tunnel route passes through both the Cape Granites and Table Mountain Sandstones, Fig. 3.13. The Du Toitskloof fault, with a downthrow of an unknown magnitude on its eastern side, is situated approximately 600 m to 700 m west of the eastern portal of the tunnel, and forms the contact between these two geological units at tunnel level. Bouldery talus deposits occur on the lower slopes of the Klein Drakensteinberge in the vicinity of the western portal of the tunnel.

At the time when the study was undertaken, the tunnel excavation had reached about 2 200 m in from the east portal and approximately 100 - 150 m in from the west portal. The following discussion is based largely on observations made along that section of the eastern tunnel which had been excavated up to the time when the study was completed.

Table Mountain Sandstones

These rocks are generally hard, pale brown and cream, and occasionally grey, quartzitic sandstones and quartzites, with shallow, southerly to north-easterly dips and very pronounced jointing (Plate VII). Truswell[68] refers to these rocks as orthoquartzites, indicating that they have a quartz content of 90 - 95 per cent[83]. The grey varieties, although apparently containing more impurities than the lighter rocks, are harder and seem to be more siliceous.

A noticeable deterioration in the rock condition occurs as the shear zone adjacent to the Du Toitskloof fault is approached. The change is essentially a physical one and the brecciated rocks, occurring close to the fault, have been reconstituted to such a degree that, whilst maintaining their competency to some extent, their original joint patterns and bedding planes have been almost totally obliterated. Progressive discolouration is also evident as the fault zone is approached. Iron-oxide staining initially occurs adjacent to joints and shear planes only, but eventually affects the entire rock mass.

With the exception of some of the brecciated material within this shear zone, these rocks are essentially impervious. Water percolation occurs along joints, however, and has resulted in the deposition of oxide coatings on many of these surfaces.

Evidence of shear movement can be observed throughout most of that section of the tunnel which has penetrated the Table Mountain Sandstones. The most obvious of these are slickensided joint planes and subhorizontal,

/clay-filled .....

clay-filled zones of crushed rock. These latter features seem to be due to movement along pre-existing bedding planes. In addition, microfracturing is evident in the more competent horizons while intense local fractures i - ometimes developed at the intersection of major joint planes.

The Cape Granites

The granitic rocks through which the tunnel has been excavated consist mainly of grey or pink coarse-grained, porphyritic rocks with occasional finer-grained and gneissic varieties. Aplite zones, with similar colour variations, and dark pink to almost red coloured varieties of granite are present in some parts of the tunnel.

Joints are well developed throughout the rock mass. They are very often tight structures with quartz or calcite cement being common. Faults, containing clay gouge up to 0,5 m thick, together with chloritized and brecciated zones, were intersected in the eastern tunnel in the vicinity of the 2 000 m mark. Maximum water flows of 10 litres per minute were noted from these faulted areas(84).

Jointing in the aplites is noticeably different to that in the coarse-grained rocks. The joints are planar and generally quite closely spaced, giving the rock a pronounced blocky appearance. The joints are generally tight or closed, however.

Although the granite adjacent to the Du Toitskloof fault zone was found to be moderately hard on exposure, deterioration started soon afterwards, and the rock became soft and friable(84). This was possibly due to the absorption of water by the clay minerals produced from the argillization of the rock. Further away from the fault, the rock displayed evidence of intense shearing, and chlorite-coated microfractures, together with chlorite and clay-filled shear planes, were common.

At the west portal the granite has been decomposed by weathering and now consists of a yellowish-brown and cream coloured material with a dense to very dense consistency. Water flows of approximately 250 litres per minute were encountered from this material(84).

3.3.4 Engineering Geolog

General Rock Conditions and Support Installations

The Table Mountain Sandstones presented no particular problems to the excavation of the pilot tunnel. The rock is well jointed (Plate VIII)

/with .....

with iron staining and occasional clayey infillings on many of the joint surfaces. A reasonable excavation shape was achieved although overbreak was evident in places, particularly where a well-developed subhorizontal joint set occurred in conjunction with a vertical joint. Shotcrete and rock bolts were used to stabilize some sections of the tunnel passing through this rock type. Adjacent to the Du Toitskloof fault the rock is brecciated and softer. Shotcrete and steel arches were installed along the worst sections. Uniaxial Compressive Strengths of 25 to 274 MPa have been measured for these rocks(85).

The Cape Granites provided good tunnelling conditions when not associated with major faults and little support was usually required. In the vicinity of the major structures, viz. the Du Toitskloof fault and the fault zone intersected approximately 2 000 m into the eastern tunnel from the portal, soft, moist, sheared and altered granite occurs. This material often has very poor self-supporting characteristics and heavy support in the form of steel arches, continuous lagging and concrete backfill was used. In places it was necessary to apply shotcrete to the face immediately after the blast, as a temporary support.

Slickensided joints striking subparallel to the tunnel axis are evident along some sections passing through the granites. These joints have apparently caused the gradual slabbing of the sidewalls of the tunnel and were stabilized with grouted rock dowels. Mean Uniaxial Compressive Strength values of 105 MPa have been measured for the unweathered and unsheared granites, while slightly lower mean values of 94 MPa have been obtained from samples of gneissic granite(50).

Completely decomposed granite occurs at the western portal of the tunnel. Ground freezing techniques were used to excavate part of this section of the tunnel.

Rock Conditions at the Sites

Stereographic plots of the in-situ joint survey data and sketch diagrams of the major joint patterns at each site described below are presented in Appendix IIIC.

Dat 1:

The rock consists of hard, jointed, pale grey streaked reddish-brown quartzitic sandstone. The joint survey undertaken in both E-W and N-S directions indicated three possible joint sets with dips and dip directions of:

/28°/205° .....

$28^{\circ}/209^{\circ}$;

$60^{\circ}/327^{\circ}$;

$80^{\circ}/027^{\circ}$.

A fourth set orientated at $60^{\circ}/008^{\circ}$ was also suggested. Mapping confirmed the presence of these four joint sets.

Joint surfaces are both planar and undulating and iron staining is common. Occasional clayey sand joint fillings also occur.

Out 2:

The rock at this site consists of hard, jointed, pale grey quartzitic sandstone with a band of dark grey quartzite and elongate lenses of medium-hard, highly jointed and weathered yellowish-brown, quartzitic sandstone. Joint measurements, undertaken along the south-east and west walls of the drilling cubby indicated three main joint orientations and two subsidiary sets. The dip and dip directions of the main sets are:

$30^{\circ}/124^{\circ}$;

$70^{\circ}/214^{\circ}$;

$45^{\circ}/249^{\circ}$;

and of the subsidiary sets are:

$60^{\circ}/278^{\circ}$;

$80^{\circ}/000^{\circ}$.

Mapping confirmed the presence of the main joint sets as well as the latter of the subsidiary sets.

Joint surfaces are usually rough and both planar and undulating. Many display no alteration or iron staining only, while a filling of crushed rock and clayey sand is common to the $30^{\circ}/124^{\circ}$ joint set.

Out 3:

This site is situated within the immediate zone of influence of the Du Toitskloof fault, Fig. 3.12, and the rock consists of soft to hard jointed yellow, brown and grey, weathered quartzitic sandstone. Although generally re-healed the rock is fractured and microfractured and has been subjected to intense brecciation during its history. Zones of grit-size quartzite fragments in a recemented but weathered matrix of hard, crushed and oxidized quartzite are common.

The joint survey was undertaken along the south and east walls of the drilling cubby and indicated one main joint set with two possible

/subsidiary .....

subsidiary sets. The orientation of these are:

$90^{\circ}/204^{\circ}$;

$58^{\circ}/065^{\circ}$;

$75^{\circ}/227^{\circ}$.

Mapping of the recess also suggested only one main joint set although a few very obvious joints striking subparallel to the tunnel also occur. Clayey coatings occur on many of the joint surfaces, which are again both undulating and planar but smoother, in general, than those at sites 1 and 2.

Out 4:

This site is situated to the west of the Du Toitskloof fault, Fig. 3.12, and the rock, which consists of medium-hard, highly jointed and sheared, greenish coloured granite, has been affected by the movement along it.

Joint measurements were hampered by the electrical and pumping equipment installed at the site and it was necessary to obtain the required information from the results of a joint survey undertaken approximately 32 m to the east(50). Three joint sets, which correlated reasonably well with mapping at the site, were indicated by this survey. The dip and dip directions of the respective sets are as follows:

$80^{\circ}/317^{\circ}$;

$60^{\circ}/217^{\circ}$;

$70^{\circ}/088^{\circ}$.

Most joint surfaces are planar and smooth and chlorite and clay coatings are common.

Out 5:

This site is situated at a 5 m widening in the tunnel used as a siding for the Tremac drilling rig and for hoppers awaiting removal to or from the face. A joint survey was not possible with the machinery in the siding and joint measurements obtained from a previous survey were used instead(50). The rock at this site consists of hard, jointed, coarse-grained, grey porphyritic granite.

The joint survey data was unfortunately too limited to be really useful and showed a wide scatter when plotted on the stereonet. Four possible joint sets were outlined, however. The dip and dip directions of these are as follows:

$/80^{\circ}-90^{\circ}$ .....

80°-$90^{\circ}/205^{\circ}$;

70°-$85^{\circ}/305^{\circ}$;

$75^{\circ}/265^{\circ}$;

$40^{\circ}/051^{\circ}$.

Mapping, as undertaken at the other sites, was also not practical and a verbal description of the rock mass had to suffice. Three main joint sets could be described:

- Sub-horizontal;
- Vertical, striking WNW;
- Vertical, striking N to NE.

Comparison with the stereoplot showed that the subhorizontal joints were not recorded at all by the joint survey; the WNW striking set correlates with either the No.2 or No.4 sets, and the N to NE striking joints can be related to either of the sets No.3 and 3A shown on the stereoplot.

Most joint surfaces at this site are planar and undulating and smooth with slickensides and are either closed or recemented with calcite cement.

Dut 6:

The rock at this site is hard, moderately jointed, coarse-grained, grey porphyritic granite. The joint survey was undertaken along the north and west walls of the drilling recess and indicated two main joint sets and a possible third set with respective orientations as follows:

$70^{\circ}/029^{\circ}$;

$90^{\circ}/111^{\circ}$;

$45^{\circ}/303^{\circ}$.

The two main sets were confirmed by mapping while a different subsidiary set orientated at approximately $90^{\circ}/250^{\circ}$ was indicated. Rough, planar and undulating joint surfaces are most common with no alteration or cement and thin coatings of quartz and calcite.

4. PRESENTATION OF RESULTS

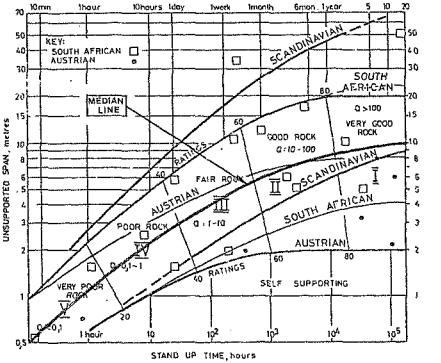
The results calculated from the in situ and borehole measurements, according to the procedure outlined in Chapter 2 for each classification system, are presented in Tables 4.1 to 4.4. These results represent the net output for each system.

For the Geomechanics Classification System the results presented are, the RMR-values at each sample site, and the rock class, stand-up time and active unsupported span. The two latter values were estimated from the diagram presented by Bieniawski[25], using a median line through the zone representing the limits for South African experience, Fig. 4.1. The support recommendations for each rock class are also tabulated.

The results presented for the N.G.I. Classification System are, the Q-values, limiting-span, stand-up time, and support categories and recommendations for each site. The limiting-span was calculated from the relationship proposed by Barton et al[33] (equation 2, Sect.1.5) and the stand-up time was estimated from the diagram given by the same author for that purpose, Fig. 1.2. The results for each of the steps used to obtain the final RMR and Q-values are presented in Appendices V and VI.

To evaluate the results each classification system was first considered separately, with comparisons between the in situ and borehole results. When possible, the in situ and borehole ratings for individual parameters were also compared.

The standard errors of estimate about the lines representing the ideal relationship (Borehole Result = In Situ Result) are listed in Table 4.5 for each parameter evaluated, together with the values for the 90 per cent confidence limits calculated from the standard error. These limits were generally unrelated to the rock classes and parameter categories outlined for each classification system and more significant limits were selected. These are also presented in Table 4.5.



Modified after: Bieniawski, 1976

FIG. 4.1 Geomechanics Classification System; Median Line used to Estimate the Stand-up Time and Unsupported Span

TABLE 4.1

GEOMECHANICS CLASSIFICATION SYSTEM

BORE CORE RESULTS

Sample Site No.	RMR	Rock Class	Stand-up Time	Active Unsupported Span(m)	Support
Bos 1	44	3	2-5 dys	3	B 1,5-2 m + clm (crw) S 50-100 mm (crw), 30 mm (sd)
Bos 2	75	2	6-8 mths	8	B 2,5 m + clm (occ.) S 50 mm (crw if reqd.)
Bos 3	38	4	1 dy	2,5	B 1-1,5 m + clm S 100-150 mm (crw) 100 mm (sd) L.R. 1,5 m (if reqd.)
Bos 4	72	2	3-5 mths	7,5	B 2,5 m + clm (occ.) S 50 mm (crw if reqd.)
Bos 5	74	2	6-8 mths	8	"
Bos 6	77	2	9-11 mths	8,5	"
Bos 7	58	3	2-4 wks	5,5	B 1,5-2 m + clm (crw) S 50-100 mm (crw) 30 mm (sd)
Bos 8	68	2	2-4 mths	7	B 2,5 m + clm (occ.) S 50 mm (crw if reqd.)
Bos 9	80	2	12-15 mths	8,5	"
Bos 10	45	3	3-4 dys	3,5	B 1,5-2 m + clm (crw) S 50-100 mm (crw) 30 mm (sd)
Bos 11	57	3	2-3 wks	5	"
Bos 12	68	2	2-4 mths	7	B 2,5 m + clm (occ.) S 50 mm (crw if reqd.)
Del 1	28	4	5-10 hrs	2	B 1-1,5 m + clm S 100-150 mm (crw) 100 mm (sd) L.R. 1,5 m (if rqd.)

TABLE 4.1

GEOMECHANICS CLASSIFICATION SYSTEM

BORE CORE RESULTS

Sample Site No.	RMR	Rock Class	Stand-up Time	Active Unsupported Span(m)	Support
Del 2	51	3	6-7 dys	4	B 1,5-2 m + clm (crw) S 50-100 mm (crw) 30 mm (sd)
Del 3	40	4	1-2 dys	3	"
Del 4	66	2	2-3 mths	6,5	B 2,5 m + clm (occ.) S 50 mm (crw if reqd.)
Del 5	33	4	10-12 mths	2	"
Del 6	36	4	15-20 hrs	2,5	
Dut 1-H	56	3	2-3 wks	5	B 1,5-2 m + clm (crw) S 50-100 mm (crw) 30 mm (sd)
Dut 1-V	47	3	4-5 dys	4	"
Dut 2-H	50	3	5-7 dys	4	"
Dut 2-V	54	3	1-2 wks	4,5	"
Dut 3-V	30	4	8-10 hrs	2	B 1-1,5 m + clm S 100-150 mm (crw) 100 mm (sd) L.R. 1,5 m (if reqd.)
Dut 4-H	45	3	4-5 dys	3,5	B 1,5-2 m + clm (crw) S 50-100 mm (crw) 30 mm (sd)
Dut 4-V	46	3	4-5 dys	4	
Dut 5-V	64	2	5-6 wks	6	B 2,5 m + clm (occ.) S 50 mm (crw if reqd.)
Dut 6-H	69	2	3-4 mths	7	"
Dut 6-V	74	2	6-7 mths	8	"

TABLE 4.2

GEOMECHANICS CLASSIFICATION SYSTEM

IN SITU RESULTS

Sample Site No.	RMR	Rock Class	Stand-up Time	Active Unsupported Span(m)	Support
Bos 1	65	2	5-6 wks	6,5	B 2,5 m + clm (occ.) S 50 mm (crw if reqd.)
Bos 2	75	2	6-8 mths	8	"
Bos 3	74	2	6-8 mths	8	"
Bos 4	81	1	12-18 mths	9	SB (if reqd.)
Bos 5	84	1	2-3 yrs	9	"
Bos 6	82	1	1-2 yrs	9	"
Bos 7	75	2	6-8 mths	8	B 2,5 m + clm (occ.) S 50 mm (crw if reqd.)
Bos 8	71	2	4-5 mths	7	"
Bos 9	79	2	12-15 mths	8,5	"
Bos 10	70	2	3-4 mths	7	"
Bos 11	84	1	2-3 yrs	9	SB (if reqd.)
Bos 12	87	1	4 yrs	9	"
Del 1	41	3	1-2 dys	3	B 1,5-2 m + clm (crw) S 50-100 mm (crw) 30 mm (sd)
Del 2	32	4	8-10 hrs	2	B 1-1,5 m + clm S 100-150 mm (crw) 200 mm (sd) L.R. 1,5 m (if reqd.)
Del 3	43	3	2-3 dys	3	B 1,5-2 m + clm (crw) S 50-100 mm (crw) 30 mm (sd)
Del 4	74	2	6-7 mths	8	B 2,5 m + clm (occ.) S 50 mm (crw if reqd.)

TABLE 4.2

GEOMECHANICS CLASSIFICATION SYSTEM

IN SITU RESULTS

Sample Site No.	RMR	Rock Class	Stand-up Time	Active Unsupported Span(m)	Support
Del 5	51	3	6-7 dys	4	B 1,5-2 m + clm (crw) S 50-100 mm (crw) 30 mm (sd)
Del 6	44	3	2-3 dys	3,5	"
Dut 1	61	2	3-4 wks	6	B 2,5 m + clm (occ.) S 50 mm (crw if reqd.)
Dut 2	70	2	3-4 mths	7	"
Dut 3	44	3	2-3 dys	3,5	B 1,5-2 m + clm (crw) S 50-100 mm (crw) 30 mm (sd)
Dut 4	37	4	20-24 hrs	2,5	B 1-1,5 m + clm S 100-150 mm (crw) 100 mm (sd) L.R. 1,5 m (if reqd.)
Dut 5	46	3	3-4 dys	3,5	B 1,5-2 m + clm (crw) S 50-100 mm (crw) 30 mm (sd)
Dut 6	81	1	1-2 dys	9	SB (if reqd.)

TABLE 4.3

N.G.I. CLASSIFICATION SYSTEM

BORE CORE RESULTS

Sample Site No.	Q	Limiting Span (m)	Stand-up Time	Support Category	Recommended Permanent Support
Bos 1	0,07	1,1	1 dy-1 wk	34	S(mr) 7,5-15 cm
Bos 2	2,38	4,5	>10 y	21	S 2,5-5 cm
Bos 3	0,02	0,7	<1 h	34	S(mr) 15-25 cm
Bos 4	26,67	11,9	>10 y	13	B(utg) 1,5-2 m
Bos 5	6,25	6,7	>10 y	17	S 2-3 cm
Bos 6	0,33	2,1	1-6 mth	29	B(utg) 1 m + S 2-3 cm
Bos 7	0,13	1,4	1 wk-1 mth	29	B(utg) 1 m + S(mr) 5 cm
Bos 8	0,80	2,9	5-10 y	25	B(utg) 1 m + S(mr) 5 cm
Bos 9	200,0	26,6	>10 y	5	sb(utg)
Bos 10	0,63	2,7	1-5 y	25	B(utg) 1 m + S(mr) 5 cm
Bos 11	0,13	1,4	1 wk-1 mth	29	B(utg) 1 m + S(mr) 5 cm
Bos 12	2,67	4,7	>10 y	21	S 2,5-5 cm
Del 1	0,08	1,2	1 dy-1 wk	33	S(mr) 5-10 cm
Del 2	0,18	1,6	1 wk-1 mth	29	B(utg) 1 m + S(mr) 5 cm
Del 3	0,32	2,0	1-6 mth	29	B(utg) 1 m + S(mr) 5 cm
Del 4	1,76	4,0	>10 y	21	B(utg) 1 m
Del 5	0,012	0,5	<1 h	33	S(mr) 5-10 cm
Del 6	0,27	1,9	1-6 mth	29	B(utg) 1 m + S(mr) 5 cm

TABLE 4.3

N.G.I. CLASSIFICATION SYSTEM

BORE CORE RESULTS

Sample Site No.	Q	Limiting Span (m)	Stand-up Time	Support Category	Recommended Permanent Support
Dut 1-H	1,2	3,4	5-10 y	21	B(utg) 1 m
Dut 1-V	0,9	3,1	5-10 y	25	B(utg) 1 m + S(mr) 5 cm
Dut 2-H	2,04	4,3	>10 y	21	B(utg) 1 m
Dut 2-V	0,41	2,2	6-12 mth	25	B(tg) 1 m + S(mr) 5 cm
Dut 3-V	1,1	3,3	5-10 y	21	S 2,5-5 cm
Dut 4-H	1,6	4,0	>10 y	21	B(utg) 1 m
Dut 4-V	0,09	1,2	1dy-1 wk	33	S(mr) 5-10 cm
Dut 5-V	7,7	7,2	>10 y	17	S 2-3 cm
Dut 6-H	89,7	19,3	>10 y	9	sb(utg)
Dut 6-V	33,2	13,0	>10 y	13	B(utg) 1,5-2 m

TABLE 4.4

N.G.I. CLASSIFICATION SYSTEM

IN SITU RESULTS

Sample Site No.	Q	Limiting Span (m)	Stand-up Time	Support Category	Recommended Permanent Support
Bos 1	0,14	1,5	1 wk-1 mth	29	B(tg) 1 m + S(mr) 5 cm
Bos 2	6,06	6,6	>10 y	17	S 2-3 cm
Bos 3	1,23	3,5	5-10 y	21	S 2,5-5 cm
Bos 4	11,08	8,4	>10 y	13	B(utg) 1,5-2 m + S 2-3 cm
Bos 5	32,32	12,9	>10 y	13	B(utg) 1,5-2 m
Bos 6	6,66	6,8	>10 y	17	S 2-3 cm
Bos 7	0,98	3,2	5-10 y	25	B(utg) 1 m + S(mr) 5 cm
Bos 8	2,91	4,9	>10 y	21	B(utg) 1 m
Bos 9	16,66	9,9	>10 y	13	B(utg) 1,5-2 m
Bos 10	0,83	3,0	5-10 y	25	B(tg) 1 m + S(mr) 5 cm
Bos 11	0,44	2,3	6-12 mth	25	B(tg) 1 m + S(mr) 5 cm
Bos 12	2,0	4,2	>10 y	21	B(utg) 1 m
Del 1	0,58	2,6	1-2 y	25	B(tg) 1 m + S(mr) 5 cm
Del 2	2,05	4,3	>10 y	21	B(utg) 1 m + S 2-3 cm
Del 3	2,99	5,0	>10 y	21	B(utg) 1 m
Del 4	4,75	6,0	>10 y	17	S 2-3 cm
Del 5	1,02	3,2	5-10 y	21	S 2,5-5 cm
Del 6	0,22	1,7	1 wk-1 mth	29	B(utg) 1 m + S 2-3 cm
Dut 1	1,20	3,4	5-10 y	21	S 2,5-5 cm
Dut 2	1,09	3,3	5-10 y	21	S 2,5-5 cm
Dut 3	1,02	3,2	5-10 y	21	S 2,5-5 cm
Dut 4	0,47	2,4	6-12 mth	25	B(utg) 1 m + S(mr) 5 cm
Dut 5	9,74	8,0	>10 y	17	S 2-3 cm
Dut 6	33,32	13,0	>10 y	13	B(utg) 1,5-2 m

TABLE 4.5

STANDARD ERRORS OF ESTIMATE (Syx), PER CENT DISTRIBUTION OF RESULTS WITH RESPECT TO (BH) = (INS) AND LIMITING VALUES SELECTED AS MOST SIGNIFICANT

Parameter	± Syx	90% C.L. (1,645Syx)	% (BH)=(INS)	% (BH)<(INS)	% (BH)>(INS)	Selected Limit(±)
RMR	22,2	36,5	7	14	79	19
Joint Space	5,0	8,3	11	25	64	5
Joint Condition	3,2	5,2	29	21	50	5
Joint Orientation	3,1	5,1	11	18	71	2
Q	0,78	Log (BH) = Log (INS) ± 1,29	7	29	64	Log (BH)= Log (INS) ± 1
Jn	2,33	3,8	43	14	43	3
Jr	1,22	2,0	39	47	14	1
Ja	1,58	2,6	43	36	21	1
SRF	3,0	4,9	68	28	4	2,5
RQD	32,75	53,87	4	7	89	25

4.1 Geomechanics Classification System

4.1.1 Rock Mass Rating

The borehole and in situ values for each site are plotted in Fig. 4.2. The 90 per cent confidence limits about the ideal line, RMR(BH) = RMR(INS), are also shown and are given by the relationship,

$$RMR(BH) = RMR(INS) \pm 36{,}5.$$

It is important to consider the relevance of these limits with respect to the various rock classes and related stand-up times.

The output from the Geomechanics System, when applied to rock tunnels, is the RMR-value of the rock mass, its rock class, stand-up time and active unsupported span. The latter two results, which for practical purposes are the most important for indicating the quality of the rock mass, are determined from the RMR-value. The rock class serves only as a general indicator of the nature of the rock mass, although it also gives an approximate idea of the range of stand-up time which might be anticipated. The support recommendations are also based on the rock class. To compare the in situ and borehole results for a particular site in practical terms, one should therefore compare the RMR-values rather than the rock classes.

However, owing to the logarithmic relationship between the stand-up time and RMR, the time ranges given for each rock class are variable (see Table 1.19), and if one used the RMR-value as a basis for comparison, it would be necessary to select limits which would be related to the stand-up time for a particular rock class as well as to practical aspects of excavation and support installation. This would be difficult because present knowledge relating stand-up time to rock mass conditions is very general, and any attempt to create RMR limits, other than those already used for the rock classes, would be arbitrary and unrealistic. It is therefore necessary to use the defined rock classes as bases for comparison.

The five rock classes for the Geomechanics Classification System are defined by the following ranges of RMR-values:

Class 1	RMR 81 - 100
Class 2	RMR 61 - 80
Class 3	RMR 41 - 60
Class 4	RMR 21 - 40
Class 5	RMR < 20

It is evident that although a difference of only one between the borehole and in situ RMR-values for a particular site can give different rock classes for the site, a difference of up to nineteen can be tolerated without

/changing .....

changing the rock class. For example, assuming that the in situ RMR-value always represents the true rock mass condition, then a difference of only one, between this value and the RMR-value calculated from the borehole measurements at the same site, can give a different rock class to that actually obtaining at the site. At the same time differences of up to nineteen can give the same rock class for both the borehole and in situ measurements. In addition, in situ and borehole RMR differences of between one and twenty can never result in more than one rock class difference, although one class difference can also be obtained from differences between the RMR-values of up to thirty-nine.

These observations suggest that nineteen, or possibly *twenty*, would probably give more significant limiting conditions than the calculated 90 per cent confidence limits. The relationship,

$$RMR(BH) = RMR(INS) \pm 19,$$

has been selected because as shown above, nineteen gives the cut-off for "no rock class difference" and therefore has more significance than the value *twenty*. These limits are shown in Fig. 4.2 where they can be seen to contain only twenty-one (75 per cent) of the results. This evaluation implies that, within the accuracy of the number of observations made, there is a 75 per cent chance of obtaining a true representation of rock mass condition from a borehole core, or of differing by no more than one rock class from the true condition using this classification system. The likelihood of either possibility occurring can be assessed by examining the rock class differences obtained for each pair of borehole and in situ measurements.

Two histograms have been compiled showing the frequency of the rock class differences for each set of in situ and borehole measurements. The first diagram, Fig. 4.3a, shows the number of class differences only, while the second, Fig. 4.3b, illustrates whether these *differences are positive* or negative with respect to the in situ results (i.e. whether the rock class predicted from the borehole measurements *is* equal to, greater or less than that calculated from the in situ data.) These diagrams show that 18 per cent of the *results give the* same rock class, 75 per cent differ by one rock class only, with 61 per cent and 14 per cent of the borehole *results predicting* one class lower and higher respectively than the in situ results, and 7 per cent differ by two rock classes. Those results differing by two classes all show lower borehole than in situ results. Differences greater than two rock classes were not recorded. In all, 93 per cent of the borehole and in situ results differ by no more than one rock class.

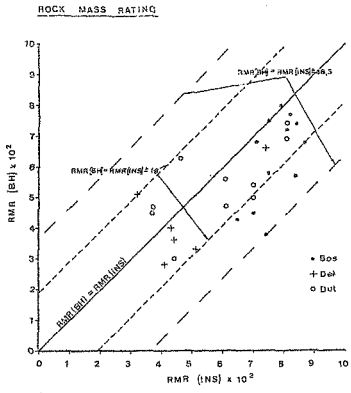


FIG. 4.2 Comparison of Borehole and In Situ RMR-Values

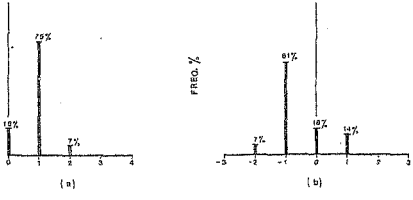


FIG. 4.3 Geomechanics System - Frequency of Rock Class Differences between Borehole and In Situ Results

4.1.2 Joint Spacing

The limits selected for this parameter, and for the other parameters evaluated from the Geomechanics Classification System, are based on the minimum differences between the ratings for each category. This approach was used because the differences between the category ratings for a particular parameter are not always constant and it was important to evaluate whether the borehole and in situ ratings indicated significantly different conditions. These limits are shown on the relevant diagrams (close dashed lines) together with the 90 per cent confidence limits (open dashed lines).

The in situ and borehole ratings for the Joint Spacing are presented in Fig. 4.4 and listed in Table 4.6. Twenty (71 per cent) of the points lie within the limits of $\pm$ 5, while twenty-six (93 per cent) fall within the 90 per cent confidence limits. When considering all the points with respect to the Jsp(BH) = Jsp(INS) line, three (11 per cent) plot on the line, seven (25 per cent) lie above it and eighteen (54 per cent) plot below it. There is thus a tendency for the borehole measurements to give a lower rating than the in situ measurements.

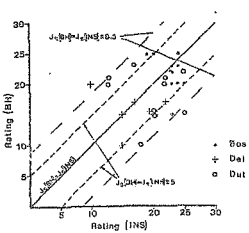


FIG. 4.4 Comparison of Borehole and In Situ Joint Spacing Ratings

TABLE 4.6

GEOMECHANICS CLASSIFICATION SYSTEM
BOREHOLE AND IN SITU JOINT SPACING RATING VALUES

Sample Site No.	Borehole Rating	In Situ Rating
Bos 1	15	21
Bos 2	25	20
Bos 3	20	24
Bos 4	22	23
Bos 5	20	25
Bos 6	23	23
Bos 7	22	23
Bos 8	25	19
Bos 9	20	23
Bos 10	15	20
Bos 11	23	24
Bos 12	25	24
Del 1	10	15
Del 2	20	10
Del 3	17	17
Del 4	17	22
Del 5	15	20
Del 6	15	15
Dut 1-H	15	20
Dut 1-V	15	20
Dut 2-H	20	22
Dut 2-V	21	22
Dut 3-V	10	18
Dut 4-H	20	13
Dut 4-V	21	13
Dut 5-V	23	17
Dut 6-H	15	25
Dut 6-V	22	25

4.1.3 Joint Condition

The ratings for this parameter are presented in Fig. 4.5 and Table 4.7. In this case the 90 per cent confidence limits are almost coincident with the limits selected using the minimum difference between categories. Twenty-two (78 per cent) of the points fall within either of the limits and eight (29 per cent) points plot on the Jo(BH) = Jo(ING) line, while six (21 per cent) and fourteen (50 per cent) lie above and below the line respectively.

These results (i.e. more points on the ideal line; and a more even distribution about the line) suggest a better correlation between the borehole and in situ results, even though the number of points within the selected limits is not much different to the results for both the Rock Mass Rating or the Joint Spacing. Although a tendency for a lower rating from the borehole measurements is again apparent, it is not as marked as for the Rock Mass Rating or the Joint Spacing.

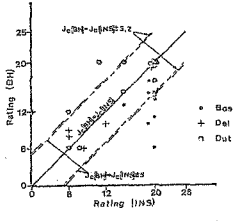


FIG. 4.5 Comparison of Borehole and In Situ Joint Condition Ratings

TABLE 4.7

GEOMECHANICS CLASSIFICATION SYSTEM
BOREHOLE AND IN SITU JOINT CONDITION RATING VALUES

Sample Site No.	Borehole Rating	In Situ Rating
Bos 1	13	15
Bos 2	15	20
Bos 3	6	20
Bos 4	15	20
Bos 5	20	20
Bos 6	20	20
Bos 7	17	19
Bos 8	10	19
Bos 9	20	20
Bos 10	15	19
Bos 11	14	20
Bos 12	11	20
Del 1	9	6
Del 2	12	0
Del 3	10	12
Del 4	20	20
Del 5	8	6
Del 6	6	8
Dut 1-H	20	15
Dut 1-V	15	15
Dut 2-H	15	20
Dut 2-V	17	20
Dut 3-V	6	8
Dut 4-H	6	6
Dut 4-V	12	6
Dut 5-V	20	11
Dut 6-H	20	20
Dut 6-V	20	20

4.1.4 Joint Orientation

The ratings for the Joint Orientation Adjustment are presented in Fig. 4.6 and Table 4.8, together with the relevant limits as described previously. The diagram shows that twenty-seven (96 per cent) of the points fall within the 90 per cent confidence limits, while only seventeen points (61 per cent) plot within the selected limits. Four points (14 per cent) plot on the ideal line, five (18 per cent) plot above it and nineteen (68 per cent) lie below it. If one examines the distribution of points it is quite clear that no real correlation with respect to Jo(BH) = Jo(INS) exists between the borehole and in situ ratings.

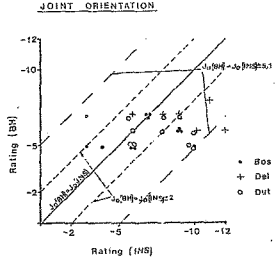


FIG. 4.6 Comparison of Borehole and In Situ Joint Orientation Ratings

TABLE 4.8

GEOMECHANICS CLASSIFICATION SYSTEM
BOREHOLE AND IN SITU JOINT ORIENTATION RATING VALUES

Sample Site No.	Borehole Rating	In Situ Rating
Bos 1	- 6	- 9
Bos 2	- 7	- 7
Bos 3	- 5	- 4
Bos 4	- 7	- 7
Bos 5	- 5	- 3
Bos 6	- 5	- 6
Bos 7	- 6	- 9
Bos 8	- 6	- 9
Bos 9	- 5	- 6
Bos 10	- 7	- 3
Bos 11	- 5	- 10
Bos 12	- 7	- 7
Del 1	- 8	- 11
Del 2	- 6	- 12
Del 3	- 7	- 8
Del 4	- 6	- 10
Del 5	- 7	- 9
Del 6	- 7	- 6
Dut 1-H	- 6	- 10
Dut 1-V	- 5	- 10
Dut 2-H	- 7	- 8
Dut 2-V	- 6	- 8
Dut 3-V	- 5	- 10
Dut 4-H	- 5	- 6
Dut 4-V	- 6	- 6
Dut 5-V	- 6	- 9
Dut 6-H	- 5	- 6
Dut 6-V	- 7	- 6

4.2 Rock Quality Designation

because the RQD is common to both rock mass classification systems it was possible to compare the actual borehole and in situ RQD-values without considering the ratings used in the Geomechanics Classification System. A limiting value of plus or minus 25 per cent was selected because, with the exception of the 'good' or 'excellent' categories, it is the most common range used for differentiating between the various rock qualities.

The results are presented in Fig. 4.7, and listed in Table 4.9. Eleven (39 per cent) of the plotted points lie within the selected limits while eighteen (64 per cent) plot within the 90 per cent confidence limits. One point only (4 per cent) plots on the RQD(BH) = RQD(INS) line while two (7 per cent) and twenty-five (89 per cent) plot above and below it respectively. In general the borehole RQD estimates are much lower than the in situ estimates. Very little real correlation is apparent between the two sets of results, however.

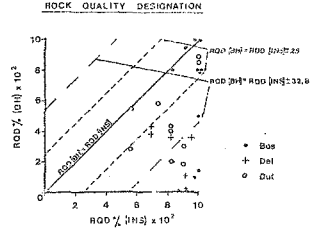


FIG. 4.7 Comparison of Borehole and In Situ RQD Estimates

TABLE 4.9

BOREHOLE AND IN SITU RQD VALUES

Sample Site No.	Borehole Rating	In Situ Rating
Bos 1	10	87
Bos 2	95	91
Bos 3	10	92
Bos 4	80	100
Bos 5	100	97
Bos 6	80	100
Bos 7	10	98
Bos 8	80	82
Bos 9	100	100
Bos 10	50	99
Bos 11	15	100
Bos 12	80	100
Del 1	10	92
Del 2	36	82
Del 3	38	69
Del 4	66	95
Del 5	11	92
Del 6	43	69
Dut 1-H	30	90
Dut 1-V	18	90
Dut 2-H	20	82
Dut 2-V	10	82
Dut 3-V	43	82
Dut 4-H	55	56
Dut 4-V	28	56
Dut 5-V	58	73
Dut 6-H	89	100
Dut 6-V	75	100

4.3 N.C.I. Classification System

4.3.1 Q-Values

The borehole and in situ Q-values are plotted in Fig. 4.8 which also shows the ideal line, Ln Q(BH) = Ln Q(INS), and the 90 per cent confidence limits about this line. These limits are given by the relationship.

$$\text{Ln } Q(BH) = \text{Ln } Q(INS) \pm 1,3.$$

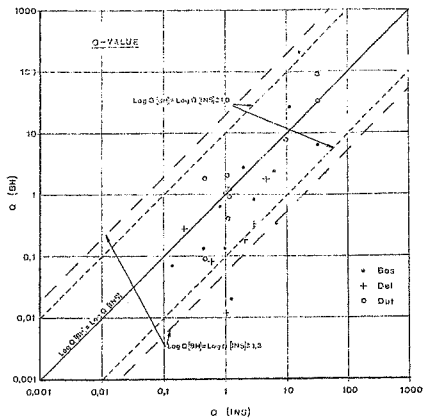


FIG. 4.8 Comparison of Borehole and In Situ Q-values

/An examination .....

An examination of the Q-values defining the rock classes for this classification system shows that the upper Q-value of each class is equal to a multiple of its lower Q-value. The value of the multiple is not consistent for all the rock classes, being 2,5 for Classes 1, 3 and 5; 4,0 for Classes 2, 4 and 6; and 10,0 for Classes 7, 8 and 9 respectively.

In order to evaluate any correlation between the in situ and borehole Q-values, for the full range of possible Q-values (0,001 - 1000), it is necessary to define limits which are relevant to all the rock classes. One possible way to do this is to look for similarities between the support categories in adjacent rock classes and if these exist, to use them as a basis for defining the required limiting Q-values.

If one examines the support recommendations it can be seen that, for low values of SPAN/ESR, they do not vary too greatly for the rock classes representing the better quality rock, i.e. Rock Classes 1 - 5. Considering, therefore, that the tunnels studied were all small diameter excavations with consistent ESR ratings, it is reasonable to combine Rock Classes 1 - 6 into pairs with ranges of Q-values given by multiples of 10,0 similar to the three lower rock classes. This gives six modified rock classes which can be used to evaluate the in situ and borehole data. These modified classes also provide a simpler basis for comparing the N.G.I. results with those from the Geomechanics System, which has five rock classes for defining the rock mass condition. It should be noted that Q-values in adjacent rock classes can differ by multiples up to 100. For example, if we consider the classes given by Q-values of 0,1 - 1,0 and 1,0 - 10,0 respectively, it can be seen that the lower Q-value of the former differs from the upper Q-value of the latter by a multiple of 100. This is relevant to the following discussion.

The limits selected are given by the relationship,

$$\text{Ln } Q(\text{BH}) = \text{Ln } Q(\text{INS}) \pm 1,$$

and include twenty-three (82 per cent) of the results. Within the accuracy of the number of observations, this implies therefore that there is an 82 per cent chance of a borehole Q-value being equal to, or within a range of a multiple of 10,0 of the in situ Q-value from the same site. For lower Q-values ($< 1,0$) this means a variation of one rock class between the borehole and in situ results, while for higher Q-values it could mean a difference of up to two rock classes.

In order to assess the variations between the in situ and borehole rock classes at each site, histograms similar to those prepared for the

/Geomechanics .....

Geomechanics Classification System were used. The six combined rock classes were again adopted in preference to the nine classes originally defined for this system. These diagrams, Figs. 4.9a & b, illustrate that 46 per cent of the borehole results give the same rock class as the in situ results from the same sample site. Forty-seven per cent differ by one rock class with 36 per cent and 11 per cent of the borehole Q-values predicting one rock class lower and higher respectively than the associated in situ Q-values, and 7 per cent differ by two classes. Rock class differences greater than two were not recorded.

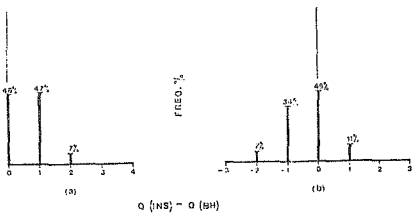


FIG. 4.9 N.G.I. System - Frequency of Rock Class Differences between Borehole and In Situ Results

/4.3.2 .....

4.3.2 Joint Set Number

The limits selected for this parameter represent the most common difference between the ratings for the different numbers of joint sets. In this case, higher ratings refer to poorer rock mass conditions, i.e. the more joint sets present the higher the allocated rating. This is important to remember because in all the cases considered up to now, higher ratings have been related to better rock conditions and points plotting below the ideal relationship line indicated conservative borehole estimates of the true conditions. In this case, however, all points below the Jn(BH)=Jn(INS) line represent examples where fewer joint sets (i.e. better rock conditions) have been estimated from the bore core than the in situ joint survey.

The in situ and borehole ratings are plotted in Fig. 4.10 and listed in Table 4.10. Twenty of the points (71,5 per cent) lie within the selected limits, while twelve (43 per cent) plot on the Jn(BH) = Jn(INS) line and four (14 per cent) and twelve (43 per cent) plot above and below the line respectively. In general, this means that 71,5 per cent of the borehole results do not differ from their related in situ result by more than one category, while there is a tendency for fewer joints to be recognized in the bore cores than were recorded from the in situ survey.

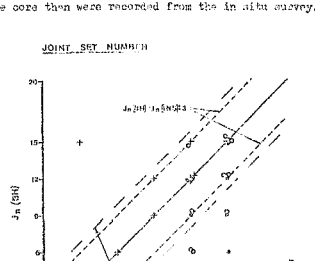


FIG. 4.10 Comparison of Borehole and In Situ Jn Ratings

TABLE 4.10

N.G.I. CLASSIFICATION SYSTEM
JOINT SET NUMBER RATING VALUES

Sample Site No.	Borehole Rating	In Situ Rating
Bos 1	15	15
Bos 2	12	12
Bos 3	6	15
Bos 4	9	12
Bos 5	12	15
Bos 6	12	12
Bos 7	15	15
Bos 8	15	15
Bos 9	2	12
Bos 10	12	12
Bos 11	12	15
Bos 12	9	15
Del 1	6	6
Del 2	15	3
Del 3	9	9
Del 4	15	12
Del 5	12	9
Del 6	12	12
Dut 1-H	15	15
Dut 1-V	12	15
Dut 2-H	12	15
Dut 2-V	15	15
Dut 3-V	6	12
Dut 4-H	6	12
Dut 4-V	15	12
Dut 5-V	9	15
Dut 6-H	4	12
Dut 6-V	9	12

4.3.3 Joint Roughness

The limits selected for this parameter represent the difference between the ratings for the various categories considered to be most relevant in view of the generalized methods used to assess the roughness characteristics of the joint surface. Higher rating values are related to more stable rock mass conditions.

The in situ and borehole ratings are presented in Fig. 4.11 and Table 4.11. Twenty (71,5 per cent) of the results lie on or within the selected limits while eleven (39 per cent) plot on the Jr(BH) = Jr(INS) line and thirteen (47 per cent) and four (14 per cent) plot above and below it respectively. These results suggest that there is a tendency for the borehole estimates of the joint roughness to indicate better rock conditions than the in situ measurements. In addition, 71,5 per cent of the borehole results are within approximately one roughness category of the in situ values.

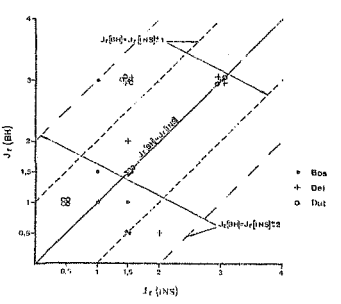


FIG. 4.11 Comparison of borehole and In Situ Jr Ratings

TABLE 4.11

N.G.I. CLASSIFICATION SYSTEM
JOINT ROUGHNESS RATING VALUES

Sample Site No.	Borehole Rating	In Situ Rating
Bos 1	1,5	1,5
Bos 2	1,5	1,5
Bos 3	0,5	1,5
Bos 4	3,0	1,0
Bos 5	3,0	3,0
Bos 6	0,5	1,5
Bos 7	1,0	1,5
Bos 8	1,5	1,0
Bos 9	3,0	1,5
Bos 10	1,5	1,5
Bos 11	1,0	1,0
Bos 12	1,5	1,5
Del 1	2,0	1,5
Del 2	3,0	3,0
Del 3	3,0	3,0
Del 4	3,0	1,5
Del 5	0,5	2,0
Del 6	3,0	1,5
Dut 1-H	1,5	0,5
Dut 1-V	1,5	0,5
Dut 2-H	3,0	1,5
Dut 2-V	3,0	1,5
Dut 3-V	1,5	1,5
Dut 4-H	1,5	0,5
Dut 4-V	1,5	0,5
Dut 5-V	3,0	1,5
Dut 6-H	3,0	3,0
Dut 6-V	3,0	3,0

TABLE 4.11

N.G.I. CLASSIFICATION SYSTEM
JOINT ROUGHNESS RATING VALUES

Sample Site No.	Borehole Rating	In Situ Rating
Bos 1	1,5	1,5
Bos 2	1,5	1,5
Bos 3	0,5	1,5
Bos 4	3,0	1,0
Bos 5	3,0	3,0
Bos 6	0,5	1,5
Bos 7	1,0	1,5
Bos 8	1,5	1,0
Bos 9	3,0	1,5
Bos 10	1,5	1,5
Bos 11	1,0	1,0
Bos 12	1,5	1,5
Del 1	2,0	1,5
Del 2	3,0	3,0
Del 3	3,0	3,0
Del 4	3,0	1,5
Del 5	0,5	2,0
Del 6	3,0	1,5
Dut 1-H	1,5	0,5
Dut 1-V	1,5	0,5
Dut 2-H	3,0	1,5
Dut 2-V	3,0	1,5
Dut 3-V	1,5	1,5
Dut 4-H	1,5	0,5
Dut 4-V	1,5	0,5
Dut 5-V	3,0	1,5
Dut 6-H	3,0	3,0
Dut 6-V	3,0	3,0

4.3.4 Joint Alteration

This parameter is similar to the joint set number in that the higher ratings are related to the poorer alteration conditions. The limiting value was selected because it appeared to be the most significant rating difference within the range of conditions encountered. Wider limits would probably be more appropriate for highly altered or sheared joint surfaces.

The results are presented in Fig. 4.12 and Table 4.12. Once again twenty points (71,5 per cent) lie on or within the appropriate limits while twelve points (43 per cent) plot on the $J_a(BH) = J_a(INS)$ line. Ten (36 per cent) and six (21 per cent) of the points plot above and below the line respectively. The results indicate that 71,5 per cent of the borehole estimates of the joint alteration condition are equal to or no more than one category different from the in situ estimates at the same sites. Although not marked, there is a slight tendency for the borehole estimates to indicate poorer joint alteration conditions than were noted in situ. The correlation between the borehole and in situ evaluation of this parameter is however generally good.

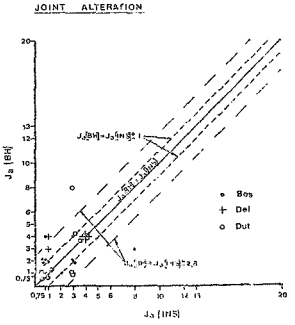


FIG. 4.12 Comparison of Borehole and In Situ Ja Ratings

TABLE 4.12

N.G.I. CLASSIFICATION SYSTEM
JOINT ALTERATION RATING VALUES

Sample Site No.	Borehole Rating	In Situ Rating
Bos 1	3,0	3,0
Bos 2	2,0	0,75
Bos 3	4,0	3,0
Bos 4	1,0	0,75
Bos 5	4,0	0,75
Bos 6	2,0	0,75
Bos 7	1,0	2,0
Bos 8	2,0	0,75
Bos 9	0,75	0,75
Bos 10	2,0	3,0
Bos 11	2,0	3,0
Bos 12	2,0	1,0
Del 1	4,0	4,0
Del 2	4,0	4,0
Del 3	4,0	1,0
Del 4	3,0	1,0
Del 5	4,0	4,0
Del 6	4,0	4,0
Dut 1-H	1,0	1,0
Dut 1-V	1,0	1,0
Dut 2-H	1,0	3,0
Dut 2-V	3,0	3,0
Dut 3-V	4,0	4,0
Dut 4-H	1,0	3,0
Dut 4-V	4,0	3,0
Dut 5-V	1,0	0,75
Dut 6-H	0,75	0,75
Dut 6-V	0,75	0,75

4.3.5 Stress Reduction Factor

The limit selected for this parameter is, as for the other parameters evaluated, the most common difference between the ratings for the various categories. The in situ and borehole ratings for each sample site are shown in Fig. 4.13 and Table 4.13. Twenty-six (93 per cent) of the points lie on or within the selected limits while nineteen points (68 per cent) plot on the SRF(BH) = SRF(INS) line and eight (28 per cent) and one (4 per cent) plot above and below the line respectively. The results indicate that 93 per cent of the borehole ratings are equal to or differ from the in situ ratings by no more than one category. There is also a slight tendency for the borehole estimates to indicate poorer rock mass conditions than were encountered underground.

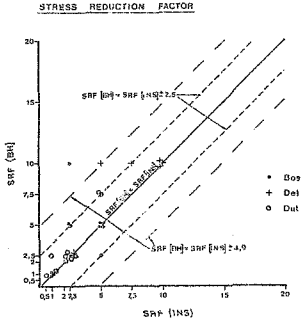


FIG. 4.13 Comparison of Borehole and In Situ SRF Ratings

TABLE 4.13

Q.G.I. CLASSIFICATION SYSTEM
STRESS REDUCTION FACTOR RATING VALUES

Sample Site No.	Borehole Rating	In Situ Rating
Bos 1	5,0	5,0
Bos 2	2,5	2,5
Bos 3	10,0	2,5
Bos 4	1,0	1,0
Bos 5	1,0	1,0
Bos 6	5,0	2,5
Bos 7	5,0	5,0
Bos 8	5,0	2,5
Bos 9	1,0	1,0
Bos 10	5,0	5,0
Bos 11	5,0	5,0
Bos 12	2,5	5,0
Del 1	10,0	10,0
Del 2	10,0	10,0
Del 3	10,0	7,5
Del 4	2,5	2,5
Del 5	10,0	5,0
Del 6	10,0	10,0
Dut 1-H	2,5	2,5
Dut 1-V	2,5	2,5
Dut 2-H	2,5	2,5
Dut 2-V	2,5	2,5
Dut 3-V	2,5	2,5
Dut 4-H	7,5	5,0
Dut 4-V	7,5	5,0
Dut 5-V	2,5	1,0
Dut 6-H	1,0	1,0
Dut 6-V	1,0	1,0

4.4 Comparison of RMR and Q

In order to compare the results obtained from the Geomechanics and N.G.I. Classification Systems, Tables 4.14 and 4.15 showing the Stand-up Times and Unsupported Spans, and the Recommended Support respectively have been compiled. Comparative plots of the RMR and Q-values are presented in Figs. 4.14 and 4.15. The linear regression lines calculated for the borehole and in situ results are given by the expressions,

$$RMR = 4,6 \ LnQ + 55,5$$
$$RMR = 5,0 \ LnQ + 60,8 \text{ respectively.}$$

The 90 per cent confidence limits about the calculated regression lines were found to be $\pm$ 18,57 for the borehole results and $\pm$ 26,85 for the in situ results. The relationship between RMR and Q as given by Bieniawski[25] is also shown on each diagram.

The scatter of points about the regression lines is wider for the in situ values than for the borehole values although the lines themselves are similar. This may be a reflection of the limited number of observations. The scatter is probably too great to indicate any meaningful correlation between the two classification systems. This is also apparent from the accompanying tables.

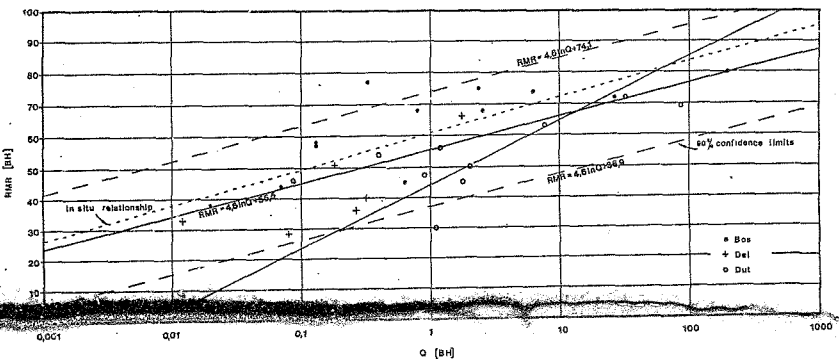


FIG. 4.14 Comparison of Borehole RMR and Q-Values

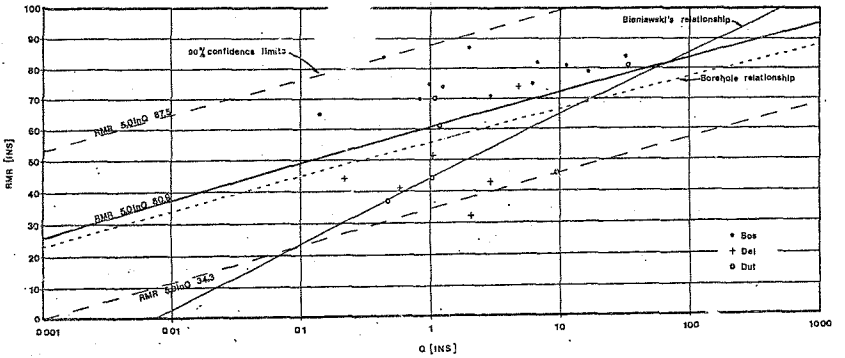


FIG. 4.15 Comparison of In Situ RMR and Q-Values

TABLE 4.14

STAND-UP TIMES AND UNSUPPORTED SPANS PREDICTED FROM BOREHOLE AND IN SITU DATA FOR THE N.G.I. AND GEOMECHANICS CLASSIFICATION SYSTEMS

Sample	Geomechanics Classification System				N.G.I. Classification System			
Site	Borecore Results		In Situ Results		Borecore Results		In Situ Results	
No.	Stand-up Time	Active Unsup. Span	Stand-up Time	Active Unsup. Span	Stand-up Time	Limiting Span	Stand-up Time	Limiting Span
Bos 1	2-3dys	3	5-6wks	6,5	1dy-1wk	1,1	1wk-1mth	1,5
Bos 2	6-8mths	8	6-8mths	8	> 10yrs	4,5	> 10yrs	6,6
Bos 3	1dy	2,5	6-8mths	8	< 1h	0,7	5-10yrs	3,5
Bos 4	3-5mths	7,5	12-18mth	9	> 10yrs	11,9	> 10yrs	8,4
Bos 5	6-8mths	8	2-3yrs	9	> 10yrs	6,7	> 10yrs	12,9
Bos 6	9-11mths	8,5	1-2yrs	9	1-6mths	2,1	> 10yrs	6,8
Bos 7	2-4wks	5,5	6-8mths	8	1wk-1mth	1,4	5-10yrs	3,2
Bos 8	2-4mths	7	4-5mths	7	5-10yrs	2,9	> 10yrs	4,9
Bos 9	12-15mth	8,5	12-15mth	8,5	> 10yrs	26,6	> 10yrs	9,9
Bos 10	3-4dys	3,5	3-4mths	7	1-5yrs	2,7	5-10yrs	3,0
Bos 11	2-3wks	5	2-3yrs	9	1wk-1mth	1,4	6-12mths	2,3
Bos 12	2-4mths	7	4yrs	9	> 10yrs	4,7	> 10yrs	4,2
Del 1	5-10hrs	2	1-2dys	3	1dy-1wk	1,2	1-2yrs	2,6
Del 2	6-7dys	4	8-10hrs	2	1wk-1mth	1,6	> 10yrs	4,3
Del 3	1-2dys	3	2-3dys	3	1-6mths	2,0	> 10yrs	5,0
Del 4	2-3mths	6,5	6-7mths	8	> 10yrs	4,0	> 10yrs	6,0
Del 5	10-12mth	2	6-7dys	4	< 1h	0,5	5-10yrs	3,2
Del 6	15-20mth	2,5	2-3dys	3,5	1-6mths	1,9	1wk-1mth	1,7
Dut 1-H	2-3wks	5	3-4wks	6	5-10yrs	3,4	5-10yrs	3,4
Dut 1-V	4-5dys	4	3-4wks	6	5-10yrs	3,1	5-10yrs	3,4
Dut 2-H	5-7dys	4	3-4mths	7	> 10yrs	4,3	5-10yrs	3,3
Dut 2-V	1-2wks	4,5	3-4mths	7	6-12mths	2,2	5-10yrs	3,3
Dut 3-V	8-10hrs	2	2-3dys	3,5	5-10yrs	3,3	5-10yrs	3,2
Dut 4-H	4-5dys	3,5	20-24hr	2,5	> 10yrs	4,0	6-12mths	2,4
Dut 4-V	4-5dys	4	20-24hr	2,5	1dy-1wk	1,2	6-12mths	2,4
Dut 5-V	5-6wks	6	3-4dys	3,5	> 10yrs	7,2	> 10yrs	8,0
Dut 6-H	3-4mths	7	1-2dys	9	> 10yrs	19,3	> 10yrs	13,0
Dut 6-V	6-7mths	8	1-2dys	9	> 10yrs	13,0	> 10yrs	13,0

TABLE 4.15

ROCK CLASSIFICATION RATINGS FOR THE GEOMECHANICS AND N.G.I. CLASSIFICATION SYSTEMS

SHEET 1/2

SAMPLE SITE NUMBER	LITHOLOGY	GEOMECHANICS CLASSIFICATION SYSTEM						N.G.I. CLASSIFICATION SYSTEM							
		BOREHOLE RESULTS			IN SITU RESULTS			BOREHOLE RESULTS				IN SITU RESULTS			
		RMR	Rock Class	Recommended Support	RMR	Rock Class	Recommended Support	Q	Support Categ.	Recommended Support	Limiting Span	Q	Support Categ.	Recommended Support	Limiting Span
Bos 1	LAVA	44	3	B 1,5-2m + clm(c) S 50-100mm(c) 30mm(s)	65	2	B 2,5m + clm(occ) S 50mm(c.if rqd.)	0,07	34	S(mr) 7,5-15cm	1,1m	0,14	29	B(tg) 1m + S(mr) 5cm	1,5m
Bos 2	LAVA	75	2	B 2,5m + clm(occ) S 50mm(c.if rqd.)	75	2	B 2,5m + clm(occ) S 50mm(c.if rqd.)	2,38	21	S 2,5-5cm	4,5m	6,00	17	S 2-3cm	6,6m
Bos 3	LAVA & DIABASE	38	4	B 1-1,5m + clm S 100-150mm(c) 100cm(s) LR 1,5m (if rqd.)	74	2	B 2,5m + clm(occ) S 50mm(c.if rqd.)	0,02	34	S(mr) 15-25cm	0,7m	1,23	21	S 2,5-5cm	3,5m
Bos 4	DIABASE	72	2	B 2,5 + clm(occ) S 50mm(c.if rqd.)	81	1	sB (if rqd.)	26,67	13	B(utg) 1,5-2m	11,9m	11,08	13	B(utg) 1,5-2m + S 2-5cm	8,4m
Bos 5	LAVA	74	2	B 2,5 + clm(occ) S 50mm(c.if rqd.)	84	1	sB (if rqd.)	6,25	17	S 2-3cm	6,7m	32,32	13	B(utg) 1,5-2m	12,9m
Bos 6	LAVA	77	2	B 2,5m + clm(occ) S 50mm(c.if rqd.)	82	1	sB (if rqd.)	0,33	29	B(utg) 1m + S 5-7cm	2,1m	6,66	17	S 2-3cm	6,6m
Bos 7	LAVA	58	3	B 1,5-2m + clm(c) S 50-100mm(c) 30cm(s)	75	2	B 2,5m + clm(occ) S 50mm(c.if rqd.)	0,13	29	B(utg) 1m + S(mr) 5cm	1,4m	0,98	25	B(utg) 1m + S(mr) 5cm	3,2m
Bos 8	LAVA	68	2	B 2,5m + clm(occ) S 50mm(c.if rqd.	71	2	B 2,5m + clm(occ) S 50mm(c.if rqd.	0,80	25	B(utg) 1m + S(mr) 5cm	2,9m	2,91	21	B(utg) 1m	4,9m
Bos 9	LAVA	80	2	B 2,5m + clm(occ) S 50mm(c.if rqd.	79	2	B 2,5m + clm(occ) S 50mm(c.if tqd.	200,0	5	sB(utg)	26,6m	16,60	13	B(utg) 1,5-2m	9,9m
Bos 10	LAVA	45	3	B 1,5-2m + clm(c) S 50-100mm(c) 30mm(s)	70	2	B 2,5m + clm(occ) S 50mm(c.if rqd.	0,63	25	B(utg) 1m + S(mr) 5cm	2,7m	0,83	25	B(tg) 1m + S(mr) 5cm	3,0m
Bos 11	LAVA	7	3	B 1,5-2m + clm(c) S 50-100mm(c) 30mm(s)	84	1	sB (if rqd.)	0,13	29	B(utg) 1m + S(mr) 5cm	1,4m	0,44	25	B(tg) 1m + S(mr) 5cm	2,5m

TABLE 4.15

ROCK CLASSIFICATION RATINGS FOR THE GEOMECHANICS AND N.G.I. CLASSIFICATION SYSTEMS

SHEET 2/3

SAMPLE SITE NUMBER	LITHOLOGY	GEOMECHANICS CLASSIFICATION SYSTEM						N.G.I CLASSIFICATION SYSTEM							
		BOREHOLE RESULTS			IN SITU RESULTS			BOREHOLE RESULTS				IN SITU RESULTS			
		RMR	Rock Class	Recommended Support	RMR	Rock Class	Recommended Support	Q	Report Categ.	Recommended Support	Limit. Span	Q	Report Categ.	Recommended Support	Limit. Span
Bos 12	LAVA	68	2	B 2,5m + clm(occ) S 50mm(c.if rqd.)	87	1	sB (if rqd.)	2,67	21	S 2,5-5cm	4,7m	2,0	21	B(utg) 1m	4,2m
Del 1	ARGILL. QUARTZITE	28	4	B 1-1,5m + clm S 100-150mm(c) 100mm(s) LR 1,5m (if rqd.)	41	3	B 1,5-2m + clm(c) S 50-100mm(c) 50mm(s)	0,08	35	S(mr) 5-10cm	1,2m	0,58	25	B(utg) 1m + S(mr) 5cm	2,6m
Del 2	ARGILL. QUARTZITE	51	3	B 1,5-2m + clm(c) S 50-100mm(c) 50mm(s)	32	4	B 1-1,5m + clm S 100-150mm(c) 100mm(s) LR 1,5m (if rqd.)	0,18	29	B(utg) 1m + S(mr) 5cm	1,6m	2,05	21	B(utg) 1m + S 2-3cm	4,3m
Del 3	QUARTZITE with ARGILL.br	40	4	B 1-1,5m + clm S 100-150mm(c) 100mm(s) LR 1,5m (if rqd.)	43	3	B 1,5-2m + clm(c) S 50-100mm(c) 50mm(s)	0,32	29	B(utg) 1m + S(mr) 5cm	2,0m	2,99	21	B(utg) 1m	5,0m
Del 4	QUARTZITE	66	2	B 2,5m + clm(occ) S 50mm(c.if rqd.)	74	2	B 2,5m + clm(occ) S 50mm(c.if rqd.)	1,76	21	B(utg) 1m	4,0m	4,75	17	B 2-3cm	6,0m
Del 5	QUARTZITE SHALE & SILTSTONE	33	4	B 1-1,5m + clm S 100-150mm(c) 100mm(s) LR 1,5m (if rqd.)	51	3	B 1,5-2m + clm(c) S 50-100mm(c) 50mm(s)	0,018	33	S(mr) 5-10cm	0,5m	1,02	21	B 2,5-5cm	3,2m
Del 6	QUARTZITE SHALE & SILTSTONE	36	4	B 1-1,5m + clm S 100-150mm(c) 100mm(s) LR 1,5m (if rqd.)	44	3	B 1,5-2m + clm(c) S 50-100mm(c) 50mm(s)	0,27	29	B(utg) 1m + S(mr) 5cm	1,9m	0,22	29	B(utg) 1m + S 2-3cm	1,7m

TABLE 4.15

ROCK CLASSIFICATION RATINGS FOR THE GEOMECHANICS AND N.G.I. CLASSIFICATION SYSTEMS

SAMPLE SITE NUMBER	LITHOLOGY	GEOMECHANICS CLASSIFICATION SYSTEM						N.G.I. CLASSIFICATION SYSTEM					
		BOREHOLE RESULTS			IN SITU RESULTS			BOREHOLE RESULTS				IN SITU RESULTS			
		RMR	Rock Class	Recommended Support	RMR	Rock Class	Recommended Support	Q	Support Categ.	Recommended Support	Limits Span	Q	Support Categ.	Recommended Support	Limits Span
Dut 1-H	QTZTIC. SANDSTONE	56	3	B 1,5-2m + clm(c) S 50-100mm(c) 30mm(s)	61	2	B 2,5m + clm(ccc) S 50mm(c.if rqd.)	1,2	21	B(utg) 1m	3,4m	1,2	21	S 2,5-5cm	3,4m
Dut 1-V	QTZTIC. SANDSTONE	47	3	B 1,5-2m + clm(c) S 50-100mm(c) 30mm(s)				0,9	25	B(utg) 1m + S(mr) 5cm	3,1m				
Dut 2-H	QTZTIC SANDSTONE	50	3	B 1,5-2m + clm(c) S 50-100mm(c) 30mm(s)	70	2	B 2,5m + clm(ccc) S 50mm(c.if rqd.)	2,0	21	B(utg) 1m	4,3m	1,0	21	S 2,5-5cm	3,3m
Dut 2-V	QTZTIC SANDSTONE	54	3	B 1,5-2m + clm(c) S 50-100mm(c) 30mm(s)				0,4	25	B(utg) 1m + S(mr)	2,2m				
Dut 3-V	QTZTIC SANDSTONE weath. & brecc.	30	4	B 1-1,5m + clm S 100-150mm(c) 100mm(s) LR 1,5m (if rqd.)	44	3	B 1,5-2m + clm(c) S 50-100mm(c) 30mm(s)	1,1	21	S 2,5-5cm	3,3m	1,0	21	S 2,5-5cm	3,2m
Dut 4-H	GRANITE sheared	45	3	B 1,5-2m + clm(c) S 50-100mm(c) 30mm(s)	37	4	B 1-1,5m + clm(c) S 100-150mm(c) 100mm(s) LR 1,5m (if rqd.)	1,8	21	B(utg) 1m	4,0m	0,4	25	B(utg) 1m + S(mr) 5cm	2,4m
Dut 4-V	GRANITE sheared	46	3	B 1,5-2m + clm(c) S 50-100mm(c) 30mm(s)				0,0	33	S(mr) 5-10cm	1,2m				
Dut 5-H	GRANITE	63	2	B 2,5m + clm(ccc) S 50mm(c.if rqd.)	46	3	B 1,5-2m + clm(c) S 50-100mm(c) 30mm(s)	7,7	17	S 2-3cm	7,2m	9,7	17	S 2-3cm	8,0m
Dut 6-H	GRANITE	69	2	B 2,5m + clm(ccc) S 50cm(c.if rqd.)	81	1	sB (if rqd.)	89,7	9	sB(utg)	19,3m	33,3	13	B(utg) 1,5-2m	13,0m
Dut 6-V	GRANITE	74	2	B 2,5m + clm(ccc) S 50mm(c.if rqd.)				33,2	13	B(utg) 1,5-2m	13,0m				

5. DISCUSSION OF RESULTS

5.1 Geomechanics Classification System

Although the Geomechanics Classification System has apparently been applied quite extensively to problems of underground excavation in this country, records of results obtained have only been published in a few cases. In addition, few attempts appear to have been made to compare borehole predictions of RMR-values with related values obtained from in situ measurements.

A detailed geotechnical monitoring program of the Overvaal railway tunnel near Ermelo in the Eastern Transvaal was undertaken by the C.S.I.R. during its construction. The tunnel was a horse-shoe shaped excavation with a width and height of 5,5 m and 6 m respectively, and was excavated entirely through dolerite. The program included the classification of the rock mass using the Geomechanics System, and a comparison of the results obtained with the actual stand-up times encountered, and supports installed. Some of the results were published by Bieniawski and Maschek[86] and have been partly reproduced in Table 5.1, together with predicted stand-up times and support recommendations estimated from the RMR-values. The predicted and actual stand-up times show a good correlation, but the predicted support requirements are generally more conservative than the actual support used. The main difference between the predicted and actual support, appears to be related mainly to the use of shotcrete, which was in fact only used at one locality (F5) although recommended for all except locality C9 (refer Table 5.1).

The Geomechanics System was also used for assessing the rock conditions at the Steenbras pump storage scheme near Cape Town[87]. At this project the site investigation data was used to assess the anticipated rock conditions. These varied from Rock Class 2 to Rock Class 5, and when encountered underground generally corresponded to those predicted. In some cases, however, the rock conditions were better than expected[87].

The results from the present study indicate that, although the RMR-values and stand-up times predicted from the borehole measurements differ quite markedly from the in situ values, a reasonably good estimate of the Rock Class can be obtained from a bore core using the methods outlined in Chapter 2. The unsupported spans estimated from the borehole results also correlate quite well with the in situ estimates. In general, the boreholes indicated poorer rock mass conditions than were encountered under-

/ground. .....

TABLE 5.1

RMR-VALUES AND PREDICTED AND ACTUAL STAND-UP TIMES AND SUPPORT REQUIREMENT FOR THE OVERVAAL TUNNEL *

Station	Rock Mass Class and Rating	Predicted Stand-up Time	Recommended Support	Actual Stand-up Time	Actual Support Used (1¼ mm diam. bolts, 1,8 m long tensioned to 50 kN)
F1	2 62	1-2 mths	B 2,5 m + clm(occ.) S 50 mm (in crw if reqd.)	4 mths	B 1,5 m
F2	3 52	1-2 wks	B 1,5 m + clm(crw) S 50-100 mm (crw) 30 mm (sds)	Isolated local falls	B 1,2 m
F3	4 29	8-9 hrs	B 1-1,5 m + clm S 100-150 mm (crw) 100 mm (sds) L.R. 1,5 m (if reqd.)	2 dys	B 1,0 m + clm
F4	4 29	8-9 hrs	B 1-1,5 m + clm S 100-150 mm (crw) 100 mm (sds) L.R. 1,5 m (if reqd.)	8 hrs	B 0,8 m + clm
F5	2 70	3-6 mths	B 2,5 m + clm (occ.) S 50 mm (in crw if reqd.)	No problems	S + SB
C6	2 70	3-6 mths	B 2,5 m + clm(occ.) S 50 mm (in crw if reqd.)	No problems	B 2,0 m
C7	5 15	1 hr	B 1-1,5 m + clm S 150-200 mm (crw) 150 mm (sds) 50 mm (face) MR-RH 0,75 + LAC	1 hr	B 1,0 m M.R 1,5 m Concrete prelining
C9	1 83	1-3 yrs	No support	No falls for 18 mths	No support

* Modified after Bieniawski and Maschek, 1975

ground. The results obtained are discussed in more detail below.

5.1.1 Rock Mass Rating

If one examines the results from the Geomechanics Classification System, two observations can be made. Firstly, there is an apparent discrepancy between the percentages indicated in Figs. 4.2 and 4.3, which show a difference of no more than one rock class between the borehole and in situ RMR-values. Secondly, the borehole measurements tend to predict lower rock classes than do the in situ measurements.

This discrepancy is because a difference of up to thirty-nine between related borehole and in situ RMR-values can be associated with only one rock class difference. More than one class difference is, however, also possible. By selecting a limit of nineteen, as was done to evaluate the results presented in Fig. 4.2, the related borehole and in situ RMR-values, differing by between nineteen and thirty-nine, and corresponding to one rock class difference, were disregarded. The histograms, however, Fig.4.3, recorded the rock classes only, and the results were thus not influenced by the limit of nineteen. This upper limit is nevertheless considered justified for evaluating the results, because any higher limiting value could include rock class differences exceeding one.

With regard to the second point, the tendency for a conservative estimate of the rock mass condition from the borehole data is probably related, to some extent, to each of the classification parameters. Some may however be more relevant than others. The largest deviation from the ideal relationship is shown by results from the Bushkoppies Tunnel, with the most obvious from the Bos 3 site. This latter case is due to the borehole intersection of a narrow, sheared dyke, which was subsequently found to be unrepresentative of the true rock conditions at the site. The other results which deviate quite significantly from the ideal, viz. from sites Bos 1, 10 and 11, are all close to zones of faulting and more intense jointing. These factors could have influenced the assessment of any of the classification parameters. The relationship between the borehole and in situ ratings for those parameters evaluated and their possible bearing on the final RMR-value will be discussed later.

One might expect less variation between the borehole and in situ results in better quality rock (Rock Classes 1 and 2) than in poorer rock. This is not borne out by the present results and could be due to the small number of observations as well as the limited range of rock classes displayed by these observations. At most sites Rock Classes 2 and 3 obtain,

/while .....

while only a few sites are characterized by Rock Classes 1 and 4, and no examples of Rock Class 5 were studied.

The spread of results is consistent with the various lithologies examined. The fine-grained lavas from the Rushkoppies Tunnel generally give high RMR-values, while the weathered sediments from the Delvers Tunnel give lower values. The porphyritic granite from the Du Toitskloof Tunnel also has higher RMR-values, similar to those calculated for the lava. The quartzitic sandstones, and the faulted and sheared rocks associated with the Du Toitskloof Fault, give low RMR-values and display quite significant scatter. Houghton's[37] work at the Kielder Experimental Tunnel in England did not reveal any significant variation between RMR-values calculated for various sedimentary lithologies in a common tectonic environment.

5.1.2 Joint Spacing

The borehole evaluation of the joint spacing generally indicated more closely jointed rock conditions than were actually present at the sites. This is not surprising when one considers the practical problems often related to recovering intact core and in addition, the difficulty sometimes encountered in differentiating between unnatural and natural breaks in bore cores.

The results from the Delvers and Du Toitskloof Tunnels deviate most significantly from the ideal relationship (see Fig. 4.4). No satisfactory explanation can be given for this however, because, for example, the results from sites Del 1 and Del 2, which both represent similar sheared and closely jointed rock mass conditions, display approximately equal but opposite deviations from the $Jsp(_{BH}) = Jsp(INS)$ line. Widely jointed quartzite was present at site Del 4, but also gave a similar deviation from the ideal line. The largest discrepancy at the Du Toitskloof Tunnel was from site Dut 6, where relatively widely jointed rock was encountered (average spacing 1 - 3 m). Borehole Dut 6-H indicated more closely spaced joints with spacings of 50 - 300 mm and 0,3 - 1 m. These results suggest that any discrepancy between the borehole and in situ assessment of this parameter is related to the quality of core examined. In reality one would expect poor core from highly jointed rock and better core from widely jointed rock types. This is evidently not always the case however, and may be due to other factors, such as brittleness, which could cause the more competent rocks to break up when drilled.

The average joint spacing in a rock mass is a function of both the spacing of each joint set and the number of joint sets present. If all

/the sets .....

the sets can be observed, and measured, a good estimate of the average joint spacing should be possible. If, however, as may occur in a borehole, all the joint sets are not intersected, an erroneous estimate would be obtained. In this event, however, the error would tend to be in the opposite sense to that observed, i.e. wider joint spacings would be indicated by the borehole. This factor could contribute to the error, but is evidently less significant than the core quality.

No results from the two lower spacing categories are included, and thus no comment can be made concerning the degree of correlation between the borehole and in situ ratings in each category. With regard to the different lithologies, the lavas are generally more widely jointed than the weathered and sheared sediments. The Du Toitskloof Tunnel results show more scatter, particularly in the case of the more highly jointed quartzitic sandstones and sheared rocks.

This parameter evidently contributes towards the tendency for a conservative borehole RMR-value. Better core quality would obviously improve the results obtained here but could also result in the opposite type of error emerging.

5.1.3 Joint Condition

The reasonably good correlation between the borehole and in situ results for this parameter (see Fig. 4.5) is surprising because the joint continuity cannot be evaluated at all from a bore core, while in poor quality rock, the joint separation estimate tends to be little more than an educated guess. Results from the Bushkoppies Tunnel display the largest deviation from the ideal relationship with the biggest error, as expected, from site Bos 3. The deviations displayed by the results from sites Bos 8 and Bos 12, are both apparently due to incorrect joint separation estimates. At Bos 8, a chlorite and mica filled joint recorded in the core was found to be a localized and insignificant feature when observed in situ. At Bos 12, open joints with quartz and calcite coatings exceeding 1 mm in thickness were recorded in the borecore. The in situ survey revealed that these joints were closed and cemented and had no significant influence on the rock mass condition.

The result from Dut 5 was the only other which showed a noticeable deviation from the ideal relationship. In this case, the error was due to the incorrect evaluation of the joint roughness. The borehole estimate was 'rough' while the in situ survey indicated "undulating and smooth with slickensides."

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Owing to the difficulties involved in assessing the joint condition from a bore core, it is suspected that these results do not give a realistic picture of the joint conditions although the average ratings obtained from the bore core and in situ measurements are similar. The tendency for a conservative estimate from the bore core, which is not as marked as for the Joint Spacing, could be due to a natural tendency to adopt a more conservative approach when faced with an uncertain situation. On the other hand, the slightly better correlation with the ideal relationship, could be due to a tendency to select an average condition when faced with the same problem. These are contradictory possibilities and it seems that they may both have influenced the result to some extent.

The spread of results is similar to that previously noted, with the Joint Condition at the Bushkoppies Tunnel being generally better than at either the Delvers or De Toitskloof Tunnels. The results from the latter two projects are not as clearly separated as previously. This may be due to similar geological conditions, i.e. shearing and faulting, prevailing in both cases.

These results also contribute slightly to a conservative borehole RMR-value. In certain cases, for instance, very obviously slickensided joint surfaces, a better correlation between the borehole and in situ ratings may be achieved. In others the "averaging out" process noted above may be more significant and also result in a better correlation.

5.1.4 Joint Orientation

There is no real correlation between the borehole and in situ joint orientation ratings with respect to the ideal Jo(BH) = Jo(INS) relationship. Because no information regarding the strike direction of the joints with respect to the tunnel directions was available from the bore cores, the joint orientation effect was generally assumed to be "Fair" (see Table 1.17). The problems involved in evaluating this parameter correctly from a borehole core are purely physical, and provided all the relevant joint sets can be recognized in the core, are not related to the core recovery. Various core orientation techniques exist, not all of which are always practical or economic to use. A good knowledge of the local geology together with recognizable markers can also be of considerable use.

The lithologies and rock conditions examined cover a reasonably good range, and the in situ results obtained for this parameter are probably a fair reflection of the average conditions one might expect to encounter

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in nature. The fact that the joint orientations were unfavourable at a large number of the sample sites has resulted in a less favourable assessment being generally related to the in situ measurements than to the borehole measurements. This is accentuated by the heavier negative rating values allocated to the less favourable orientation conditions.

These results also contribute to the conservative borehole RMR-value as noted for the previous two parameters discussed. More sophisticated core orientation techniques could, however, provide a better correlation between the borehole and in situ ratings than obtained here.

5.2 N.G.I. Classification System

Apart from the case records and examples given by Barton et al[33], to illustrate the development and use of the N.G.I. Classification System, very few results of its application in practise appear to have been published. Barton[35, 88] has himself provided some results from a 19 metre span, 31 m high power house in quite massive biotite gneiss at a depth of 100 - 150 metres and from an 8,8 metre span, 4,8 kilometre tailrace tunnel varying from 10 - 20 metres to 150 - 160 metres in depth.

The initial rock classification for the power house was based on geological reports and borehole logs from five boreholes drilled in its vicinity. The average 'Best', 'Medium' and 'Poorest' Q-values relevant to the rock mass at the depth of the excavation were estimated. These mean Q-values were 67, 20 and 1,2 respectively and were based on a maximum number of three joint sets for the 'Poorest' quality rock, two for the 'Medium' quality and one for the 'Best' quality. Joint roughness was estimated as smooth, undulating for the 'Best' and 'Medium' qualities and smooth planar for the 'Poorest' quality, and joint alteration varied from unaltered to chlorite coatings. The joint water reduction factor was assumed to be minor inflow for the 'Best' and 'Medium' qualities and medium inflow for the 'Poorest' quality, while stress reduction factors of medium stress and single clay filled weakness zones (> 50 m depth) were selected respectively. The permanent roof and wall support estimated from these Q-values are reproduced in Table 5.2.

/Table 5.2 .....

TABLE 5.2

PERMANENT SUPPORT ESTIMATES FOR UNDERGROUND POWER STATION BASED ON BOREHOLE DATA*

Q	Roof Arch	Walls
'Best' Q = 67	Untensioned, grouted bolts, 5 m long, c/c 2,0 m	Spot bolts
'Medium' Q = 20	Tensioned, grouted bolts, 5 m long, c/c 1,7 m	Spot bolts
'Poorest' Q = 1,2	Tensioned, grouted bolts, 5 m long, c/c 1,0 m + shotcrete, mesh reinforced 15 cm thick	Tensioned, grouted bolts, 7 m long, c/c 1,4 m + shotcrete, mesh reinforced, 12 cm thick

*After: Barton, 1976

The site was subsequently visited and nine locations were classified in situ. These were averaged to give 'Best', 'Poorer' and 'Worst' zones with Q-values of 39, 11 and 0,6 respectively. It was estimated that 90 per cent of the excavated surface in the power house would be 'Best' quality, less than 10 per cent would be 'Poorer' quality and only 1 - 2 per cent would be 'Worst' quality. The permanent support recommendations for these Q-values are given below in Table 5.3.

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TABLE 5.3

PERMANENT SUPPORT ESTIMATES FOR UNDERGROUND POWER STATION BASED ON IN SITU DATA*

Q	Roof	Walls
'Best' Q = 39	Systematic bolting, c/c 1,7 m with chain link mesh or steel bands	Spot bolts
'Poorer' Q = 11	Systematic bolting, c/c 1,5 m with mesh reinforced shotcrete, 7 cm thick	Systematic bolting, c/c 1,6 m with chain link mesh or steel bands
'Worst' Q = 0,6	Systematic bolting, c/c 1,0 m with mesh reinforced shotcrete, 15 cm thick	Systematic bolting, c/c 1,2 m with mesh reinforced shotcrete, 12 cm thick

*After: Barton, 1976

It can be seen that although the three mean in situ values of Q (39, 11 and 0,6) are each about half the borehole Q-values (67, 20 and 1,2), the discrepancy has a relatively small effect on the recommended permanent support[35]. For this project the borehole Q-estimates were less conservative than the in situ Q-values.

For the initial support predictions[35] of the above mentioned tailrace tunnel, corelogs from fifteen irregularly spaced investigatory boreholes were examined at the tunnel depth. Estimates of 'Best', 'Poorer' and 'Worst' rock qualities were made from each corelog and the following

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Average Q-values for the fifteen holes were obtained:

Best Q = 42

Poorer Q = 12

Worst Q = 1,1

The variations between boreholes was quite marked and minimum Q-values of 0,07 and 0,03 were obtained. These were assumed to represent the quality of weakness zones at tunnel level.

Based on a tunnel span and height of 8,8 m and an ESR value of 1,3 estimates of permanent support were made. These are given below in Table 5.4, together with approximate estimates of the anticipated distribution of each rock quality along the tunnel(88). No evidence of swelling clays was obtained from the borehole information, but if encountered it was suggested that cast concrete arches, 50 centimetres or more in thickness, might be required in those zones(88).

TABLE 5.4

PERMANENT SUPPORT ESTIMATES FOR TAILRACE TUNNEL FROM BOREHOLE DATA*

Q	Roof	Wall	Anticipated extent of zone
'Best' Q = 42	None	None	Majority (60 - 80%)
'Poorer' Q = 12	Systematic bolting, c/c 1,5 m	None	Minority
'Worst' Q = 1,1	Systematic bolting, c/c 1,0 m with mesh reinforced shotcrete, 5 cm thick	Systematic bolting, c/c 1,0 m with shotcrete, 3 cm thick OR Mesh reinforced shotcrete, 5 cm thick (depending on block size)	Less Common
'Weakness Zones' Q = 0,05	Mesh reinforced shotcrete, 20 - 25 cm thick (include invert)		Possibly no more than 25 occurrences

*After: Barton, 1976, 1977

/Based .....

Based on these estimates approximately 2 900 metres to 3 800 metres of the tunnel were anticipated to require no support. Assuming that 75 per cent of the remaining 1 000 metres to 1 900 metres was formed by the 'Poorer' quality rock and 25 per cent by the 'Worst' quality, and that the roof arch consisted of a semi-circle with a radius of 4,4 metres, it can be calculated that approximately 7 500 to 15 500 rock bolts would be required together with sufficient shotcrete and mesh reinforcing to support 250 to 500 metres of tunnel and occasional weakness zones as specified. The actual support installed after approximately 4 500 metres of the tunnel had been excavated constituted 4 900 rock bolts, 650 metres of 5 centimetre thick shotcrete, 400 metres of 10 centimetre thick shotcrete and 200 metres of 15 centimetre thick shotcrete[88]. In addition, 350 metres of 50 centimetre thick, cast concrete arches were used because swelling clay was encountered in some zones on excavation. These results suggest that although the support measures adopted were slightly different to those predicted, the general nature of each was similar. If anything, the borehole classification of the rock would appear to have been slightly conservative.

The Q-values estimated from the borehole data in the present study were generally lower than the associated in situ Q-values. These results, together with the results obtained for some of the classification parameters, are discussed below. The limiting spans and stand-up times calculated from the borehole and in situ Q-values are generally of the same order, although some marked discrepancies are apparent.

5.2.1 Q-Values

A discrepancy was also noted for this classification system between the borehole and in situ results, differing by one rock class, obtained from the Q(BH) versus Q(INS) graph, and the rock class histograms respectively (see Figs. 4.8 and 4.9). This is because a limiting value of 10,0 was selected for evaluating the graph, even though Q-values in adjacent rock classes can differ by a factor of up to 100, whilst the histograms were based on the rock class differences only without regard for the selected limit.

The limiting value of 10,0 is considered to be useful because it represents the maximum value associated with one rock class difference between related borehole and in situ Q-values. Any limiting value exceeding 10,0 could include results differing by more than one class. Both the histograms and the Q(BH) versus Q(INS) graph indicate a tendency for the borehole Q-values to be either equal to or lower than the associated

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in situ Q-values.

The largest deviations from the ideal relationship are from sites Bos 3 and Del 5. The former result is related to the sheared dyke previously described and the latter appears to be due to the low RQD value obtained from the bore core, together with an over conservative estimate of the borehole SRF rating, as a result of the poor core recovery. Results from sites Del 2, Bos 6 and Bos 9 also plot outside the selected limits. In the first case the error is due to different borehole and in situ RQD estimates, combined with an over estimate of the bore core Joint Set Number (Jn). Errors in the borehole estimates of Jn were also made at the other two sites mentioned.

Of the three projects studied, the results from the Delvers Tunnel show the biggest collective deviation from the $Q(BH) = Q(INS)$ line. The majority of in situ results from this project indicate stand-up times in excess of ten years. These estimates seem somewhat optimistic in view of the weathered state of the rock at most of the sites examined. Although it is realized that the lack of ground water and the small dimensions of the excavation will greatly enhance its stability, the borehole estimates of "weeks to months" seem more realistic. The correlation between the different parameters at this tunnel will be discussed in the following pages.

The spread of results from the lavas, which are relatively homogeneous rocks, suggests that the Q-values are a reasonably good reflection of the various rock mass conditions encountered. The results from the Delvers and Du Toitskloof Tunnels support this view to some extent, with the sheared granite and porphyritic granite, for example, displaying very different results, while the former are very similar to the sheared argillaceous quartzites from the Delvers Tunnel. Some reflection of the different lithologies is also apparent but the number of observations are too few for meaningful comments to be made. From his work at the Kielder Experimental Tunnel, Houghton[37] found very little lithological differentiation between Q-values calculated for sandstone, mudstone and limestone horizons in a common tectonic environment.

The majority of Q-values from this study are between 0,1 and 10,0 and therefore do not cover a large enough range to compare the degree of correlation between the in situ and borehole results over the full range of possible Q-values. One would expect, however, that a better correlation would be obtained for better rock conditions (i.e. higher Q-values).

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5.2.2 Joint Set Number

The confidence with which the number of joint sets present in a rock mass can be estimated, will be related to the method used to record the joints. In the case of a borehole or line survey, the estimated number of sets will be a function of the orientation and spacings of each set with respect to the orientation and length respectively, of the borehole or survey line.

As previously described, the method used to estimate the number of joint sets from the bore cores assumed that all joints with similar dip angles (inclination angles with respect to the core axis) also had similar dip directions. Obviously this is not necessarily correct and could lead to under-estimates of the number of joint sets present. Considering this, together with the possibility that joints can occur which are parallel to the borehole axis, it is not surprising that the joint set number estimated from the bore core tends to be less than the actual number encountered at the site. In this respect the parameter tends, therefore, to oppose rather than contribute to the general tendency for a conservative borehole Q-value.

A large number of random joints were usually recorded from each in situ joint survey. Some of these might be related to other joint sets in the rock mass which were not well defined on the sterographic diagrams, while others are undoubtedly randomly orientated. The question that arises from this is whether an error of this nature would make an appreciable difference to the calculated Q-value. Unfortunately this cannot be answered without analysing the sensitivity of Q with respect to Jn. Because random joints are accounted for in the N.G.I. System, however, it seems that in most cases, particularly if the in situ survey is supported by a visual appraisal of the rock, the error is unlikely to be too great.

The Jn(BH) and Jn(INS) results from the Delvers Tunnel correlate reasonably well. This indicates that the differences between the borehole and in situ Q-values for this project are not caused by this parameter, and although it does not mean that the estimated joint set number in either case is correct, it suggests that both methods of evaluation give a reasonably good estimate of the number of joint sets present.

If one examines the points which plot outside the limits selected for this parameter (see Fig.4.10), it can be seen that they are not necessarily related to poor quality rock conditions. Sites Bus 9 and Dut 6 (Borehole Dut 6-H) are examples of good quality rock masses in which all the joint sets recorded during the in situ survey were not intersected by

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the boreholes. At site Del 2 however, which is located in a sheared rock mass, the borehole estimate of joint sets was greater than found in situ. This could have been due to wavy shear joints appearing to dip in a number of different directions when observed in a bore core. Site Del 1, located in a similar sheared rock did not, however, display a large deviation from the ideal line.

5.2.3 Joint Roughness

The borehole results for this parameter generally indicate equivalent or better joint conditions than do the in situ results (see Fig. 4.11). This is rather unexpected because it implies, for instance, that joint surfaces classified as undulating were actually planar. One would expect the opposite error to be more common, but could probably account for the observed result in a number of different ways. The most likely is, however, possibly that the larger asperities on planar joint surfaces were recognized erroneously as undulations.

Considering the small joint surface area exposed in a rock core, the correlation between the borehole and in situ results is quite good. This is probably due to the relatively simple classification adopted for this parameter. A more random scatter of points may become apparent if more data were evaluated. With respect to the final Q-value, this parameter also tends to oppose rather than contribute to the conservative borehole estimates. The Delvers Tunnel results are scattered and show no particular trend.

5.2.4 Joint Alteration

The reliability of joint alteration estimates from bore cores is allied to the quality of the drilling operation, the core recovery and the care which is taken by the drill operator to record data such as changes in the colour of the flushing water and variations in the bit penetration rate. This information can greatly assist in the assessment of the nature of materials on joint surfaces because it can indicate zones of soft clay which may be totally or partially lost during drilling, and which are particularly important in evaluating the strength characteristics of a rock mass. Obviously, without samples of the clay along the joints, or other records, the swelling characteristics of the material cannot be determined. A general knowledge of the geology can, however, sometimes assist in this regard.

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In the cases studied no drilling information was available to supplement the bore core observations. Although there appears to be a slight tendency for the borehole results to indicate more intense alteration than the in situ evaluation, the general correlation between the two sets of ratings is quite good and does not contribute to the conservative borehole Q-values (see Fig. 4.12). The range of alteration conditions examined was limited however and it is very likely that correlation would be significantly poorer for category (c), 'No wall rock contact when sheared', Table 1.26.

Only two sites deviated significantly from the ideal relationship. At Bos 1, the in situ survey revealed thick clay gouge which had evidently been washed away during the drilling operation and was not recorded in the borehole log. Borehole Rut 2-V evidently intersected several zones of clayey grit above the excavation which were not present at tunnel level. This latter observation illustrates how localized variations in the rock mass can influence its engineering classification.

5.2.5 Stress Reduction Factor

The categories used for this parameter are well defined and are related to the rock mass as a whole rather than to individual joint sets. The correlation between the in situ and borehole results is better than for any of the other parameters evaluated for the N.G.I. System (see Fig. 5.3) and suggests that a general appraisal of this nature can be made with some certainty from a bore core. There is a slight tendency for an unfavourable borehole estimate of this parameter although it is poorly defined and may not be apparent if more data were available. These results, therefore, contribute only slightly to the conservative borehole Q-values.

Two points, Bos 3 and Del 5 respectively, plot outside the selected limits. The first case is to be expected. At the latter site the poor core quality was assumed to indicate the presence of multiple clay-filled weakness zones, while in reality the category relating to single clay-filled weakness zones was found to be more appropriate.

5.3 Rock Quality Designation

The comparison of the borehole and in situ RQD values shows very little correlation with respect to the ideal relationship (see Fig. 4.7). The borehole RQD values vary from 0 - 100 per cent, while the in situ values are all greater than approximately 55 per cent. The results from the

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Bushkoppies and Du Toitskloof Tunnels display a large scatter with some points plotting very close to the ideal line. None of the Delvers Tunnel results fall within the selected limits, however.

As already mentioned out, the measurement of joints along a bore core or joint survey line is the same operation and should theoretically yield similar results for the same location. The main problem, and possibly the most relevant in the present context, is related to the respective directions of each survey. For example, a vertical borehole and a horizontal survey line along a tunnel wall could give entirely different RQD values if the rock mass had a preferential joint direction. The fact that the structurally, more homogeneous or blocky rock types, such as the granite and lava, show a better correlation with the ideal relationship than do those with an inherent structure, tends to support this idea. This could account for the generally poor correlation shown by the Delvers Tunnel results as well as explaining the different RQD values obtained for horizontal and vertical boreholes at the Du Toitskloof Tunnel. The error is most likely to be in the bore core RQD value because the in situ survey can be supplemented by additional data from the surrounding rock exposure.

Other factors such as core quality, and the accuracy of in situ measurements could also have a profound effect on RQD estimates. At the Bushkoppies Tunnel, for instance, a number of the cores had been disturbed during storage movement and were difficult to reconstruct. At the Delvers Tunnel, all the in situ joint spacing values were based on visual estimates alone and may, therefore, have been less reliable than anticipated.

This parameter has made a very definite contribution to the conservative borehole classification values for both systems. However, if the previous discussion is relevant, very little can be done to obtain more accurate RQD estimates from bore cores unless multidirectional drilling is practised.

5.4 Comparison of RMR and Q

The linear regression lines calculated for the comparative plots of the borehole RMR and Q-values and the in situ RMR and Q-values respectively, are very similar (see Figs. 4.14 and 4.15). Although this is possibly coincidental, it could indicate some correlation between the two classification systems. The scatter of points about the lines is large, however, and as pointed out by Barton[89] with reference to similar results presented

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by Bieniawski[25] which also displayed considerable scatter, is probably too great to support the suggestion that a good correlation exists between the two systems. Bieniawski[32] agreed with this viewpoint, but suggested that his results did indicate that some correlation exists.

The relationship given by Bieniawski[25] is very different from those calculated for the present set of results (see Figs. 4.14 and 4.15). Bieniawski's relationship is based on a much larger number of data points than it was possible to obtain from this study, and although the present results are probably representative of the rock mass conditions examined, if any correlation does exist between the two systems, Bieniawski's relationship is more likely to be generally applicable. It would be interesting to attempt to modify Bieniawski's relationship by including the present data, but without more detail of the points used by him this is not possible.

A comparison of the estimates of stand-up time and unsupported span for related results from each classification system is more likely to indicate a possible correlation between the two systems. With a few exceptions the correlation between the two systems, for either borehole or in situ results, is very poor. The differences are so marked in some cases that although the Geomechanics stand-up time is less than the N.G.I. stand-up time, the unsupported span predicted by the former is greater than the limiting span predicted by the latter.

Barton[89] has suggested that quite large differences are apparent in support predictions using both systems. Houghton[37], on the other hand, concluded that there was little difference between support predictions for each system. The present results tend to [illegible] Barton's contention, and although appearing to correlate slightly b[illegible] the stand-up time and unsupported span, are very often quite diffe[illegible]m each other when examined in detail.

Comparing the two classification systems in terms of their practical application, Houghton[37] found the Geomechanics System easiest to apply and felt that the "degree of sophistication" in the N.G.I. System "required considerable expertise." Bieniawski[32] however felt that the N.G.I. System was "relatively easy to apply once the principles were understood." In the present study neither system was found to be particularly difficult to use once an appreciation of the geology of each site had been attained. However, each system was found to have both advantages and disadvantages when applied to a bore core or in situ exposure.

For the classification of a rock mass from a bore core, the pre-

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requisite for both classification systems is good core recovery. If this is achieved a reasonably accurate assessment of the RQD of the rock can be made. In order to obtain average rating values for each parameter pertinent to the Geomechanics System, it is necessary to try to evaluate each joint set present in the rock mass. Some idea of the number of joint sets present is therefore required. This can be achieved visually or by using some form of statistical method such as the one adopted for this study, but will, in most cases, be in error to a greater or lesser degree. Once the number of sets has been defined, their individual spacings can be estimated. The joint continuity, which is relevant to the Joint Condition parameter for this system, cannot be evaluated at all from a bore core and without special core orientation devices the Joint Orientation is likely to be based on extremely generalized information. The water and rock strength parameters are not directly related to the quality of the bore core and are not discussed further.

For the N.G.I. System, it is necessary to estimate the number of joint sets present, as above, together with the roughness and alteration of the set considered to be most likely to cause instability in the tunnel. The joint roughness is not always easy to evaluate, with the greatest difficulty being in differentiating between undulating and planar surfaces. As discussed already the alteration assessment is related to the core recovery and drilling procedure, and with the appropriate information can be evaluated reasonably easily. The most difficult aspect of this system, when classifying the rock from a bore core, is selecting which joint set is most significant to the stability of the excavation. Usually the set with the poorest roughness and alteration characteristics is chosen. The Stress Reduction Factor is a generalized parameter which was not found difficult to estimate provided the data relevant to rock strength and in situ stress was available. The Joint Water parameter, as for the Geomechanics System, is not related to the bore core.

For the in situ classification of a rock mass it is necessary to calculate the RQD from measurements or visual estimates of the joint spacing or joint frequency. Some care is needed, particularly for laminated rocks where all structures are not necessarily actual parting planes. The remaining parameters in both classification systems can be estimated visually and by approximate measurements or from detailed joint surveys. Neither system has any particular advantage over the other in this respect. Rock strength data is more difficult to obtain in this case if no rock core is available, although with the development of the irregular lump test for

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the Point Load Strength Test, this information can be obtained relatively easily. Strength data is not always necessary for classifying the rock mass according to the N.G.I. System. Owing to the generalised breakdown of the Joint Orientation effect, a visual estimate of this parameter for each set is usually quite sufficient.

In summary it can be seen that both classification systems require an estimate of the number of joint sets present in the rock mass. The Geomechanics parameters are possibly slightly more difficult to assess accurately from a bore core, but certain assumptions have to be made in this respect for both systems. Both systems suffer similar minor problems for in situ classification with the RQD estimates being the most problematical in poor rock conditions.

5.5 Rock Classification Systems in Tunnel Designs and Contracts

The question of using rock classification systems for tunnel design and contract purposes was discussed in some detail at the Exploration for Rock Engineering Symposium held in Johannesburg in 1976. Bieniawski[32] felt that a rock classification system is not sufficient for design and should be used mainly to determine general rock conditions during the exploratory phase of a project. If support recommendations were made from the exploratory data, and when possible from in situ data, they should be supported by a monitoring program to check their suitability.

On the contractual aspects, Cockcroft[90] questioned whether a contractor could be reasonably expected to take responsibility for installing primary support designed on this basis. Davies[91] considered that a rock mass classification required judgement and while being suitable as a design tool is not specific enough for the contract situation, and would consequently lead to argument when payment was concerned. Barton agreed with this viewpoint in regard to the N.G.I. Classification System.

Both the Geomechanics and the N.G.I. Classification Systems represent a synthesis of ideas and case records relevant to the problem of adequately describing rock masses for engineering projects like tunnelling. As such they should represent an improvement on the less quantitative methods previously used for this purpose. In general terms the results obtained from evaluating bore cores correlate reasonably well with the in situ results. Discrepancies arise, however, if more detail is required. The results presented in this dissertation lend support to the above arguments

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and suggest that both classification systems are suitable as design tools but neither are suitable, except possibly in general terms, for contractual aspects of tunnelling.

With regard to design, the N.G.I. System is possibly slightly more useful than the Geomechanics System because estimates can be made, with reasonable reliability, of whether the excavations may be stable without support. In addition, the support recommendations given by the N.G.I. System are more specific and appear to be better related to particular rock mass conditions.

A payment system, similar to that described by Cockcroft[90] for the Du Toitskloof Tunnel, based on the support actually installed, would seem to be a most practical solution to the contractual problem. For this purpose, a number of rock classes are defined according to their support requirements, and a cost rate established for each. These are then used as a basis of payment for each section of tunnel excavated through the different rock classes. A rock mass classification could be used to define the respective classes in terms consistent with the anticipated rock mass conditions.

5.6 Comparison of the Predicted and Installed Support

Differing tunnelling practices make it difficult to compare predicted supports with the actual installed support. However, in view of the fact that the prime object of both classification systems is to indicate the support requirements for an excavation, some comment on this aspect is appropriate. The support installed at each site, together with the predicted supports for each classification system are presented in Tables 5.4 - 5.7. For the N.G.I. System, the concept of a limiting span was used to indicate whether or not support was required. Whenever the limiting span exceeded the tunnel dimension, it was assumed that the rock mass could remain unsupported.

Although boreholes along most of the Bushkoppies Tunnel route were examined, the scope of the work undertaken did not allow any interpolation to be made between them. Comments are therefore confined to the sites examined only. The in situ support predictions from the N.G.I. System compare very well with the support installed at each site. The agreement is not as good for the borehole predictions, although some general similarities can be seen. For the Geomechanics System the supports predicted

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from either the borehole or in situ data do not compare well with the installed supports. This is due partly to the fact that the excavation size cannot be accounted for in this system, other than to vary the bolt lengths of the recommended support.

For the Du Toitskloof Tunnel the correlation between the installed support and the support predicted from the N.G.I. System is not as good as for the Bushkoppies Tunnel. Where two boreholes were drilled from a site the more conservative prediction only is shown. The limiting spans calculated for sites Dut 1, 2 and 3 are only slightly greater than the tunnel dimension of 3,0 metres. Assuming the limiting spans to be correct, this could mean that at these sites the tunnel is a border-line case between 'supported' and 'unsupported'. This could account for some of the differences. For the Geomechanics System the predicted supports are more conservative than the installed supports.

The Delvers Tunnel results are not amenable to this type of comparison, because a standardized support system was used along the entire tunnel irrespective of rock conditions. The value of rock bolts in this type of weathered rock, as recommended by both systems, is questioned, however.

TABLE 5.5

BUSHKOPPIES TUNNEL - INSTALLED SUPPORT AND PREDICTED SUPPORT USING THE N.G.I. SYSTEM

Site Number	Support Installed	Support Predicted from In Situ Measurements	Support Predicted from Borehole Measurements
Bos 1	B(t), random, ± 1-2m ave. spacing	B(tg) 1 m + S(mr) 5 cm	S(mr) 2,5-15 cm
Bos 2	None	None	None
Bos 3	None	S 2,5-5 cm	S(mr) 15-25 cm
Bos 4	None	None	None
Bos 5	None	None	None
Bos 6	None	None	B(utg) 1 m + S 2-3 cm
Bos 7	B(t) 1-1,5 m	B(utg) 1 m + S(mr) 5 cm	B(utg) 1 m + S(mr) 5 cm
Bos 8	SB	None	B(utg) 1 m + S(mr) 5 cm
Bos 9	None	None	None
Bos 10	None	B(tg) 1 m + S(mr) 5 cm	B(utg) 1 m + S(mr) 5 cm
Bos 11	SB	B(tg) 1 m + S(mr) 5 cm	B(utg) 1 m + S(mr) 5 cm
Bos 12	None	None	None

TABLE 5.6

BUSHKOPPIES TUNNEL - INSTALLED SUPPORT AND PREDICTED SUPPORT USING THE GEOMECHANICS SYSTEM

Site Number	Support Installed	Support Predicted from In Situ Measurements	Support Predicted from Borehole Measurements
Bos 1	B(t), random, ± 1-2m ave. spacing	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)	B 1,5-2 m + clm (c) S 50-100 mm (c) 30 mm (s)
Bos 2	None	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)
Bos 3	None	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)	B 1-1,5 m + clm S 100-150 mm (c) 100 mm (s) + L.R. 1,5 m
Bos 4	None	SB (if reqd.)	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)
Bos 5	None	SB (if reqd.)	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)
Bos 6	None	SB (if reqd.)	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)
Bos 7	B(t) 1 - 1,5 m	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)	B 1,5-2 m + clm (c) S 50-100 mm (c) 30 mm (s)
Bos 8	SB	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)
Bos 9	None	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)
Bos 10	None	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)	B 1,5-2 m + clm (c) S 100-150 mm (c) 100 mm (s)
Bos 11	SB	SB (if reqd.)	B 1,5-2 m + clm (c) S 100-150 mm (c) 100 mm (s)
Bos 12	None	SB (if reqd.)	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)

TABLE 5.7

DU TOITSKLOOF TUNNEL - INSTALLED SUPPORT AND PREDICTED SUPPORT USING THE N.G.I. SYSTEM

Site Number	Support Installed	Support Predicted from In Situ Measurements	Support Predicted from Borehole Measurements
Dut 1	None	S 2,5 - 5 cm	B(utg) 1 m + S(mr) 5 cm
Dut 2	None	S 2,5 - 5 cm	B(tg) 1 m + S(mr) 5 cm
Dut 3	S(mr) 10 - 15 cm with concrete invert installed later(84)	S 2,5 - 5 cm	S 2,5 - 5 cm
Dut 4	B(tg) 1 - 2 m S 5-10 cm planned for this area(84)	B(utg) 1 m + S(mr) 5 cm	S(mr) 5 - 10 cm
Dut 5	B(tg) 1-2 m	None	None
Dut 6	SB	None	None

TABLE 5.8

DU TOITSKLOOF TUNNEL - INSTALLED SUPPORT AND PREDICTED SUPPORT USING T... GEOMECHANICS SYSTEM

Site Number	Support Installed	Support Predicted from In Situ Measurements	Support Predicted from Borehole Measurements
Dut 1	None	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)	B 1,5-2 m + clm (c) S 50-100 mm (c) 50 mm (s)
Dut 2	None	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)	B 1,5-2 m + clm (c) S 50-100 mm (c) 50 mm (s)
Dut 3	S(mr) 10-15 cm with concrete invert installed later(84)	B 1,5-2 m + clm (c) S 50-100 mm (c) 30 mm (s)	B 1-1,5 m + clm (c) S 100-150 mm (c) 100 mm (s) L.R. 1,5 m (if reqd.)
Dut 4	B(tg) 1-2 m S 5-10 cm planned for this area(84)	B 1-1,5 m + clm (c) S 100-150 mm (c) 100 mm (s) L.R. 1,5 m (if reqd.)	B 1,5-2 m + clm (c) S 50-100 mm (c) 50 mm (s)
Dut 5	B(tg) 1-2 m	B 1,5-2 m + clm (c) S 150-100 mm (c) 50 mm (s)	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)
Dut 6	SB	SB (if reqd.)	B 2,5 m + clm (occ.) S 50 mm (c if reqd.)

6. CONCLUSIONS

The work presented in this dissertation has attempted to evaluate the reliability of borehole core data in engineering rock mass classification systems for underground excavation design. Several geological environments were examined during the fieldwork, and it is felt by the author that the results presented indicate the general trends that may be anticipated for similar rock mass conditions.

For both the Geomechanics and N.G.I. Classification Systems, the borehole cores generally indicated poorer rock mass conditions than were actually encountered underground. Meaningful rock classifications can nevertheless be made from bore core data, and in this respect both systems are best suited for general appraisals of the rock mass. Values for stand-up times and limiting spans, as well as support recommendations, should also be used in general terms only.

6.1 The Geomechanics Classification System

The results of the present study indicate that, for the range of rock mass conditions examined, data from borehole cores can provide reasonably reliable estimates of the Geomechanics Rock Classes to be encountered in planned underground excavations. The borehole RMR-values for the rock mass, however, often differ noticeably from the related in situ RMR-values.

Estimates of the active unsupported spans from the borehole and in situ results respectively, are similar, but the associated stand-up times and support recommendations do not compare as favourably. The results indicate that there is approximately a 60 per cent to 70 per cent chance for a rock class, estimated from bore core data, to be equal to, or within one class of, the true rock condition at a particular site.

The conservative borehole results are a reflection of similar trends shown by all the classification parameters examined. The most important contributors in this regard were, in the present study, the Rock Quality Designation and Joint Spacing. The correct evaluation of both these parameters is related to the core recovery and quality, and can be significantly affected if sufficient care is not taken during the drilling operation.

Both the Joint Condition and Joint Orientation are difficult parameters to assess accurately from borehole cores. Their influence on the

/borehole .....

borehole results is not as marked as the previous two parameters however, owing to the apparent tendency for one to select intermediate conditions when the core is difficult to evaluate. This could also lead to significant errors in the rock mass classification.

The Geomechanics Classification System was found to be simple to apply to both the borehole and in situ classification of rock masses. The generalized descriptions used for the individual parameters assisted in this regard, although the Joint Condition parameter was found to be slightly ambiguous for poorer rock conditions.

6.2 The N.G.I. Classification System

The results obtained for the N.G.I. Classification System indicate that borehole data may be used to reliably classify rock masses with Q-values exceeding approximately 0,1. Rock conditions related to Q-values significantly lower than this figure were seldom encountered during the present study. Approximately 90 per cent of the borehole and in situ results differed by no more than one rock class, although the actual Q-values were often quite different. The rock classifications based on bore core data generally indicated poorer conditions than were encountered underground. Limiting spans and stand-up times calculated from associated borehole and in situ results were generally similar.

The borehole evaluation of the individual classification parameters correlated reasonably well with the in situ results. The Joint Set Number and Joint Roughness indicated better rock conditions than were encountered in situ, while the Joint Alteration and Stress Reduction Factor displayed an opposite trend. The Rock Quality Designation probably affected the present results most significantly with a trend towards conservative borehole Q-values.

In practice the individual parameters defined for this system were found to be simple to use. The Joint Roughness is probably the most difficult parameter to evaluate accurately from a bore core and the Joint Alteration could also be problematical for conditions of intense alteration.

6.3 Comparison of the Geomechanics and N.G.I. Classification Systems

Borehole data can be used with reasonable reliability in both the Geomechanics and N.G.I. engineering rock mass classification systems. The

/correlation .....

correlation between the borehole and in situ classification values is better for the N.G.I. System than for the Geomechanics System, as is the correlation between the Unsupported or Limiting Spans, and Stand-up times estimated from these results.

Once a basic understanding of each system was obtained, together with an appreciation of the procedure necessary to realistically evaluate the individual parameters in each case, neither system was found to be particularly difficult to use. In some instances, however, the very general parameter definitions for the Geomechanics System were found to be less useful than the more precise descriptions given for the N.G.I. System.

Neither classification system displayed a good correlation between the borehole and in situ support predictions, although a better result was again obtained for the N.G.I. System. This appears to be due, to some extent, to the greater number of support categories and more detailed support recommendations, together with the fact that allowance can be made for excavation dimensions.

The borehole classifications from both systems are not sufficiently reliable for detailed design purposes, although in many cases they will represent the best available data prior to excavation. They can nevertheless serve as a basis for defining general rock classes expected to be present within a planned underground excavation, and their likely support requirements. This could be invaluable for cost estimates.

The borehole results are usually applicable to the rock mass in the immediate vicinity of the borehole only. Great care would therefore be necessary if any attempt at extrapolation of the result was envisaged.

6.4 Future Work

The present work, and the previous studies referred to in Chapter 1, have established the usefulness of engineering rock mass classification systems to rock engineering problems. Continued application of these systems and comparison of the results obtained, would help to confirm their value. Additional work in evaluating the reliability of borehole data in other geological environments would also be useful. A study of the application of the N.G.I. Classification System to soft ground tunnelling could also be pertinent.

Sensitivity analyses of the individual classification parameters, to establish their influence on the final classification values, would be

/useful .....

useful to determine the limits of accuracy necessary when assessing the parameters. The extension of the N.G.I. System for use in evaluating other rock engineering problems, such as drillability or excavation method would also be of great value, while studies of the relationship between different rock classes and their geophysical properties would aid in the extrapolation of results between borehole points.

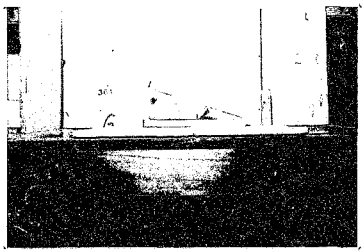

PLATE I
Graduated Clinorule for Measuring Joint Inclination Angles along Borehole Cores.

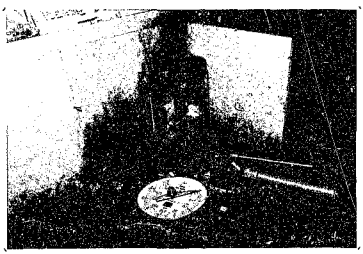

PLATE II
Engineering Laboratory Equipment Point Load Test Machine.

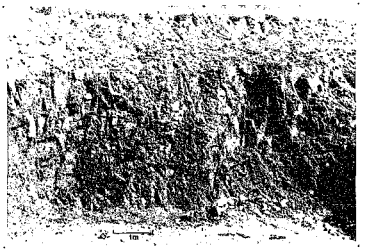

PLATE III

Bushkopples Tunnel, East Portal - Weathered,
Closely Jointed, Lava.

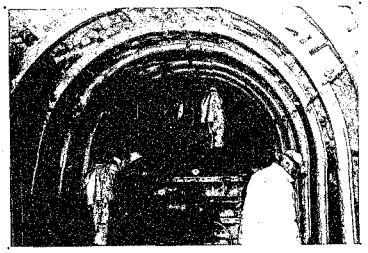

PLATE IV

Bushkopples Tunnel - Steel Arch Supports
Adjacent to Site Bas 1.

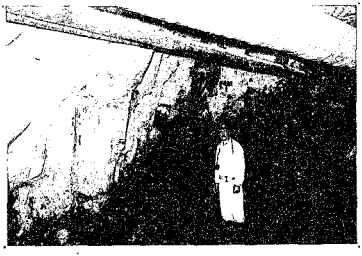

PLATE V

Bushkoppies Tunnel - Narrow Diabase Dyke at Site Bos 3.



PLATE VI

Bushkoppies Tunnel - Flatly Dipping Joint with Clay, Broken Rock, Quartz and Mica Infill, close to Site Bos 11.

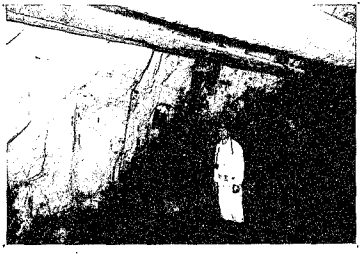

PLATE V

Bushkoppies Tunnel - Narrow Diabase Dyke at Site Bos 3.



PLATE VI

Bushkoppies Tunnel - Flatly Dipping Joint with Clay, Broken Rock, *Quartz and Mica Infill*, close to Site Bos 11.



PLATE VII

Du Toitskloof Tunnel – Table Mountain Sandstone underlain by Cape Granite.
(eastward view from West Portal)

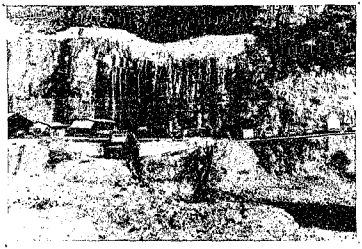

PLATE VIII

Du Toitskloof Tunnel, East Portal – Jointed Table Mountain Sandstone.

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APPENDIX I

NORWEGIAN GEOTECHNICAL INSTITUTE CLASSIFICATION SYSTEM, SUPPORT CATEGORIES

Support Measures for Rock Masses of "Exceptional", "Extremely Good", "Very Good", and "Good" Quality
(Q range: 1000-10)

Support category	Q	Conditional factors RQD/J_n	J_r/J_a	SPAN/ESR (m)	P kg/cm² (approx.)	SPAN/ESR (m)	Type of support	Note see p. 229
1*	1000—400	—	—	—	<0.01	30—40	sb (utg)	—
2*	1000—400	—	—	—	<0.01	30—60	sb (utg)	—
3*	1000—400	—	—	—	<0.01	46—80	sb (utg)	—
4*	1000—400	—	—	—	<0.01	65—100	sb (utg)	—
5*	400—100	—	—	—	0.05	12—30	sb (utg)	—
6*	400—100	—	—	—	0.05	19—45	sb (utg)	—
7*	400—100	—	—	—	0.05	30—65	sb (utg)	—
8*	400—100	—	—	—	0.05	43—80	sb (utg)	—
9	100—40	≧20	—	—	0.25	8.5—19	sb (utg)	—
		<20	—	—			B (utg) 2.5—3 m	—
10	100—40	≧30	—	—	0.25	14—30	B (utg 2—3 m	—
		<30	—	—			B (utg) 1.5—2 m +clm	—
11*	100—40	≧30	—	—	0.25	23—48	B (tg) 2—3 m	—
		<30	—	—			B (tg) 1.5—2 m +clm	—
12*	100—40	≧30	—	—	0.25	40—72	B (tg) 2—3 m	—
		<30	—	—			B (tg) 1.5—2 m +clm	—
13	40—10	≧10	≧1.5	—	0.5	5—14	sb (utg)	I
		≧10	<1.5	—			B (utg) 1.5—2 m	I
		<10	≧1.5	—			B (utg) 1.5—2 m	I
		<10	<1.5	—			B (utg) 1.5—2 m +S 2—3 cm	I
14	40—10	≧10	—	≧15	0.5	9—23	B (tg) 1.5—2 m +clm	I, II
		<10	—	≧15			B (tg) 1.5—2 m +S (mr) 5—10 cm	I, II
		—	—	<15			B (utg) 1.5—2 m +clm	I, III
15	40—10	>10	—	—	0.5	15—40	B (tg) 1.5—2 m +clm	I, II, IV
		≦10	—	—			B (tg) 1.5—2 m +S (mr) 5—10 cm	I, II, IV
16* See note XII	40—10	>15	—	—	0.5	30—65	B (tg) 1.5—2 m +clm	I, V, VI
		≦15	—	—			B (tg) 1.5—2 m +S (mr) 10—15 cm	I, V, VI

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

The type of support to be used in categories 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
(utg) = unmentioned, granted

After: Barton, Lien and Lunde, 1974

Support Measures for Rock Masses of "Fair" and "Poor" Quality
(Q range: 10-1)

Support category	Q	Contributional factors RQD/J_n	J_r/J_a	SPAN/ESR	P Kg/cm² (approx.)	SPAN/ESR (m)	Type of support	Note See p. 229
17	10—4	>30	--	---	1.0	3.5—9	sb (utg)	1
		≧10, ≦30	—	—			B (utg) 1—1.5 m	1
		<10	—	≧6 m			B (utg) 1—1.5 m +S 2—3 cm	1
							S 2—3 cm	1
				<6 m				
18	10—4	>5	—	≧10 m	1.0	7—15	B (tg) 1—1.5 m +clm	I, III
		>5	—	<10 m			B (utg) 1—1.5 m +clm	I
		≦5	—	≧10 m			B (tg) 1—1.5 m +S 2—3 cm	I, III
		≦5	—	<10 m			B (utg) 1—1.5 m +S 2—3 cm	I
19	10—4	—	—	≧20 m	1.0	12—29	B (tg) 1—2 m +S (mr) 10—15 cm	I, II, IV
		—	—	<20 m			B (tg) 1—1.5 m +S (mr) 5—10 cm	I, II
20* See note XII	10—4	—	—	≧35 m	1.0	24—52	B (tg) 1—2 m +S (mr) 20—25 cm	I, V, VI
		—	—	<35 m			B (tg) 1—2 m +S (mr) 10—20 cm	I, II, IV
21	4—1	≧12.5	≦0.75	—	1.5	2.1—6.5	B (utg) 1 m +S 2—3 cm	1
		<12.5	≦0.75	—			S 2.5—5 cm	1
		—	>0.75	—			B (utg) 1 m	1
22	4—1	>10, <30	>1.0	—	1.5	4.5—11.5	B (utg) 1 m+clm	1
		≦10	>1.0	—			S 2.5—7.5 cm	1
		<30	≦1.0	—			B (utg) 1 m +S (mr) 2.5—5cm	1
		≧30	—	—			B (utg) 1m	1
23	4—1	—	—	≧15 m	1.5	8—24	B (tg) 1—1.5 m +S (mr) 10—15cm	I, II, IV, VII
		—	—	<15 m			B (utg) 1—1.5 m +S (mr) 5—10 m	1
24* See note XII	4—1	—	—	≧30 m	1.5	18—46	B (tg) 1—1.5 m +S (mr) 15—30cm	I, V, VI
		—	—	<30 m			B (tg) 1—1.5 m +S (mr) 10—15 cm	I, II, IV

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

(tg) = tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; see Note XI)
S = shotcrete
(mr) = mesh reinforced
clm = chain link mesh
CCA = cast concrete arch
(sr) = steel reinforced

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

After: Barton, Lien and Lunde, 1974

Support Measures for Rock Masses of "Very Poor" Quality (Q range: 1,0-0,1)

Support category	Q	Conditional factors RQD/J_n	J_r/J_a	SPAN/ESR (m)	P kg/cm² (approx.)	SPAN/ESR (m)	Type of support	Note See p. 229
25	1.0—0.4	> 10	> 0.5	—	2.25	1.5—4.2	B (utg) 1 m + mr or clm	I
		≦ 10	> 0.5	—			B (utg) 1 m + S (mr) 5 cm	I
		—	≦ 0.5	—			B (tg) 1 m + S (mr) 5 cm	I
26	1.0—0.4	—	—	—	2.25	3.2—7.5	B (tg) 1 m + S (mr) 5—7.5 cm	VIII, X, XI
		—	—	—			B (utg) 1 m + S 2.5—5 cm	I, IX
27	1.0—0.4	—	—	≧ 12 m	2.25	6—18	B (tg) 1 m + S (mr) 7.5—10 cm	I, IX
		—	—	< 12 m			B (utg) 1 m + S (mr) 5—7.5 cm	I, IX
		—	—	> 12 m			CCA 20—40 cm + B (tg) 1 m	VIII, X, XI
		—	—	< 12 m			S (mr) 10—20 cm + B (tg) 1 m	VIII, X, XI
28* See note XII	1.0—0.4	—	—	≧ 30 m	2.25	15—38	B (tg) 1 m + S (mr) 30—40 cm	I, IV, V, IX
		—	—	≧ 20, < 30			B (tg) 1 m + S (mr) 20—30 cm	I, II, IV, IX
		—	—	< 20 m			B (gc) 1 m + S (mr) 15—20 cm	I, II, IX
		—	—	—			CCA (sr) 30—100 cm + B (tg) 1 m	IV, VIII, X, XI

Support Measures for Rock Masses of "Very Poor" Quality (Q range: 1,0-0,1) Cont.

29*	0.4—0.1	>5	>0.25	—	3.0	1.0—3.1	B (utg) 1 m+S 2—3 cm	—
		≦5	>0.25	—			B (utg) 1 m+S (mr) 5 cm	—
		—	≦0.25	—			B (tg) 1 m+S (mr) 5 cm	—
30	0.4—0.1	≧5	—	—	3.0	2.2—6	B (tg) 1 m+S 2.5—5 cm	IX
		<5	—	—			S (mr) 5—7.5 cm	IX
		—	—	—			B (tg) 1 m +S (mr) 5—7.5 cm	VIII, X, XI
31	0.4—0.1	>4	—	—	3.0	4—14.5	B (tg) 1 m +S (mr) 5—12.5 cm	IX
		≦4, ≧1.5	—	—			S (mr) 7.5—25 cm	IX
		<1.5	—	—			CCA 20—40 cm +B (tg) 1 m	IX, XI
		—	—	—			CCA (sr) 30—50 cm +B (tg) 1 m	VIII, X, XI
32 See note XII	0.4—0.1	—	—	≧20 m	3.0	11—34	B (tg) 1 m +S (mr) 40—60 cm	II, IV, IX, XI
		—	—	<20 m			B (tg) 1 m +S (mr) 20—40 cm	III, IV, IX, XI
		—	—	—			CCA (sr) 40—120 cm +B (tg) 1 m	IV, VIII, X, XI

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

Support Measures for Rock Masses of "Extremely Poor" and "Exceptionally Poor" Quality
(Q range: 0,1-0,001)

Support category	Q	Conditional Factors RQD/J_n	Conditional Factors J_r/J_a	SPAN/ESR (m)	P Kg/cm² (approx.)	SPAN/ESR (m)	Type of support	Note See p. 229
33*	0.1—0.01	≧2	—	—	6	1.0—3.9	B (tg) 1 m +S (mr) 2.5—5 cm	IX
		<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 (tg) 1 m +S (mr) 5—7.5 cm	IX
		<2	≧0.25	—			S (mr) 7.5—15 cm	IX
		—	<0.25	—			S (mr) 15—25 cm	IX
		—	—	—			CCA (sr) 20—60 cm +B (tg) 1 m	VIII, X, XI
35	0.1—0.01	—	—	≧15 m	6	6.5—28	B (tg) 1 m +S (mr) 30—100 cm	II, IX, XI
See note XII		—	—	≧15 m			CCA (sr) 60—200 cm +B (tg) 1 m	VIII, X, XI, II
		—	—	<15 m			B (tg) 1 m +S (mr) 20—75 cm	IX, XI, III
		—	—	<15 m			CCA (sr) 40—150 cm +B (tg) 1 m	VIII, X, XI, III
36*	0.01—0.001	—	—	—	12	1.0—2.0	S (mr) 10—20 cm	IX
		—	—	—			S (mr) 10—20 cm +B (tg) 0.5—1.0 m	VIII, X, XI
37	0.01—0.001	—	—	—	12	1.0—6.5	S (mr) 20—40 cm	IX
		—	—	—			S (mr) 30—60 cm +B (tg) 0.5—1.0 m	VIII, X, XI
38	0.01—0.001	—	—	—	12	4.0—20	CCA (sr) 100—300 cm	IX
See note XIII		—	—	RMV ≧10 m			CCA (sr) 100—300 cm +B (tg) 1 m	VIII, X, II, XI
		—	—	<10 m			S (mr) 70—200 cm	IX
		—	—	<10 m			S (mr) 70—200 cm +B (tg) 1 m	VIII, X, III, XI

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

Supplementary Notes for Support Tables

I. For cases of heavy rock bursting or "popping", tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when "popping" activity ceases.

II. Several bolt lengths often used in same excavation, i. e. 3, 5 and 7 m.

III. Several bolt lengths often used in same excavation, i. e. 2, 3 and 4 m.

IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2—4 m.

V. Several bolt lengths often used in some excavations, i.e. 6, 8 and 10 m.

VI. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4—6 m.

VII. Several of the older generation power stations in this category employ systematic or spot bolting with areas of chain link mesh, and a free concrete arch roof (25—40 cm) as permanent support.

VIII. Cases involving swelling, for instance montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.

IX. Cases not involving swelling clay or squeezing rock.

X. Cases involving squeezing rock. Heavy rigid support is generally used as a permanent support.

XI. According to the authors' experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i. e. > 1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i. e. $RQD/J_n < 1.5$, for example a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete (or shotcrete) arch to reduce the uneven loading on the concrete, but it may not be effective when $RQD/J_n < 1.5$, or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick setting resin anchors in these extremely poor quality rock-masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.

XII. For reasons of safety the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 ($SPAN/ESR > 15$ m only).

XIII. Multiple drift method usually needed during excavation and support of arch, walls and floor in cases of heavy squeezing. Category 38 ($SPAN/ESR > 10$ m only).

APPENDIX II

BOREHOLE LOG SHEETS

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Project: BUSHKOPPIES TUNNEL
Bh No: Bne 1 Inclination: Vertical Location: 624 m
Logged by: [illegible]

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE % REC.	CORE % RQD	SET	JOINT DESCRIPTION: Inclination	Roughness	Alteration	JR	JA	Jointing	J.L.I. Lugg	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Size	Method	Depth
18		LAVA; pale greenish-grey, fine-grained, slightly weathered				90° 80° 5° 80° 85° 90° 60° 0°	Pln. & Sgh. Und. & Sgh. Pln. & Sgh. Und. & Sgh.	Stain Silky oxide coating. Calcite cement Stain.			0,05-0,3	4,25 15,75			Broken core Broken core				18
19		NO CORE								FOUR	0,05-0,3 1-3				CROWN		NX	DIAMOND ROTARY	19
20		LAVA; as above.				45° 85° 85° 45° 80° 85° 85° 85°	Und. Pln. & Sgh. Und. & Sgh. Pln. & Sgh. Und. & Sgh. Pln. & Sgh.	Clayey Stain.				6,0							20
21															INVERT				21
												9,75							

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: BUSHKOPPIES TUNNEL

Bh No: Bos 2 | Inclination: Vertical | Location: 1076 S | Logged by: TROD

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	RQD %	JOINTS No. per m	JOINT DESCRIPTION: Inclination	Roughness	Alteration	Width of Joint mm	Spacing	P.L.T. Is(50)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Return	Core Size	Method	Depth
39					39	40°	Und. & Sgh.	Stain						NB. CORE DISTURBED The original core log suggests that the broken zone recorded from 41,74 to 42,30 was in fact located from 49,50 to 50,00. The core above this zone is therefore assumed to be representative of the tunnel conditions.				39
						35°												
						70°	Pln. & Sgh.			0,2–1								
40		LAVA; greyish-green, fine-grained, unweathered.			40	70°			RANDOM	> 3						NX	DIAMOND	40
						65°												
						70°			THREE	WIDE		UCS.	320 MPa	CROWN			ROTARY	
41					41	8°	Und.	Calcite cement.				?	2860 Kg/m³					41
42					42		Und. & Sgh.	Quartz & calcite cement.						Very broken core; possibly due to poor drilling.				42

SHEET 2/2

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Bh No: 206 7 Inclination: Vertical Location: 1076 a

Project: BUSHKUPPIES TUNNEL
Logged by: ISRM

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	RQD %	JOINTS per m	JOINT DESCRIPTION: Inclination	Roughness	Alteration	No. of sets	Spacing	H.K.I. (MPa)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water loss	Core Dia.	Method	Depth
43		LAVA; greyish-green fine-grained, unweathered.			43							UCS.	348 MPa 2860 kg/m²	INVERT				43
												P		Core previously removed for testing.		NX	DIAMOND	
44					44													44
		C				80°	Fix & Smth	Calcite coating.	TENDS	with RANDOM							ROTARY	
45					45													45

BOREHOLE LOG FOR DETAILED JOINT STUDIES

BH No: BH 3 Inclination (Vertical) Location: 193± m

Project: BUSHKOPPIES TUNNEL

Logged by: EBOS

DEPTH (m)	GEOLOGY	ROCK DESCRIPTION	CORE REC. %	CORE INDEX %	FRACTURE FREQUENCY (per m)	JOINT DESCRIPTION: Inclination	Roughness	Alteration	JOINTING: Wedged	JOINTING: Spacing	PLT Type	OTHER TESTS: Type	OTHER TESTS: Result	REMARKS	DRILLING DATA: Water Loss	DRILLING DATA: Core Size	DRILLING DATA: Method	DRILLING DATA: Depth
						90°	Pln & Rgh.	—										
						70°		Calcite coating										
						0°	Pln & Smth.											
						40°		—										
						85°	Pln & Smth with Slicks.	—										
59		LAVA: Greyish-green, fine-grained, unweathered.				89°	Und & Smth	Calcite coating.										59
						70°	Pln & Smth	—										
						15°	Pln & Rgh.	Calcite covered.										
														Broken core.				
60					60				RMD with	RANDOM	0,2-1					NX	DIAMOND	60
						80°	Und & Smth.	Clay coating						Joint surfaces in the diabase have a black polished appearance suggesting shearing.				
						10°												
						20°												
61		DIABASE; greenish-black, fine to medium-grained and porphyritic, unweathered.			61	10°	Und & Rgh.	Calcite coating									ROTARY	61
						5°												
						30°												
						60°												
						45°								Broken core.				
62					62	35°	Und & Smth				5,0			Below 62,7m the core was *highly fragmented* with *slickensided* and clay coated joint surfaces;				62
						45°	Und & Rgh.	Clay coating			5,0							
						20°	Und & Rgh.				7,5							
						25°	Pln & Smth.	—						CROWN - 64,5 m continued to 71,2m.				
						45°	Pln & Rgh.	—						INVERT - 67,0 m				

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Project: BUSHKOPPIES TUNNEL
Bh No: bos 4 Inclination: Vertical Location: 2193 m
Logged by: ISCU

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	CORE ROD %	CORE DIA.	JOINT DESCRIPTION: Inclination	Roughness	Alteration	INCL. of joint with	Bearing	P.L.S.I.	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: WATER Depth	WATER Loss	Method	Depth
60																		60
							Wnd. & Rgh.	Clay coating.										
61											32,0							61
										0,3-1								
		DIABASE; dark green, medium-grained unweathered.							INNER	0,3-1						NX	ROTARY DIAMOND.	
62						45°	Wnd. & Rgh.	Stained.		1-3	31,0							62
						70°	Rm & Rgh.	Calcite Cement Non-softening MCA material.										
63						40°	Wnd. & Rgh.	Clay coating										63
						20°	Rm & Rgh. with Slick.	Clay coating			28,2			Core previously removed for testing.				

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: BUSHKOPTIES TUNNEL

Bh No: BDT 4 | Inclination: Vertical | Location: 2193 = | Logged by: IDDO

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE RECOVERY %	CORE QUALITY %	WATER LOSS %	JOINT DESCRIPTION: Inclination	Roughness	Alteration	Infilling	Is(50) MPa	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Method	Depth
85		DIABASE; dark-green, medium-grained unweathered.				25°	Wod & Rgh	Wr					CROWN			85
						40°										
86						20°	Wod & Rgh	Stain.								86
						30°										
87						30°							Core previously removed for testing.			87
						85°							INVERT			

SHEARS

NX

ROTARY DIAMOND.

SHEET 1/1

BOREHOLE LOG FOR DETAILED JOINT STUDIES.

Bh No: CCR 2 Inclination: Vertical Location: 2621 E

Project: BUSHOFFIES TUNNEL

Logged by: TSCO

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	RQD %	FRACT. PER METRE	JOINT DESCRIPTION: Orientation	Roughness	Alteration	Infilling	Spacing	W.S.I. (MPa)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Size	Method	Depth
82		LAVA; greyish-green fine-grained and slightly porphyritic, unweathered.	63			90° 70°	Und & Rgh.	Clay coating.						CROWN The original borehole log indicates that the rock mass condition in the crown of the tunnel should be similar to that described here.				82
						70° 30°	Und & Rgh Und.	Clay coating Calcite & Quartz veinlet.										
						70°		Clay coating		1-3								
								Closed.	TERTIARY WITH RANDOM	1-3	4,3							
83			63			70°								Numerous hairline fractures in the rock suggest that it has been affected by shearing.	NX	RODARY	DIAMOND.	83
						80° 5°	Und. & Rgh.	Clay-silty coating Calcite cement.										
						40° 80°		Calcite cement.			3,1			INVERT Broken core.				
84			64															84
											4,1 to 64,6							

BOREHOLE LOG FOR DETAILED JOINT STUDIES.

Project: DUSHKOTIES TUNNEL

Bh No: 208 E | Inclination: Vertical | Location: 3175 E | Logged by: ISRM

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	ROCK QUALITY %	FRACTURE FREQUENCY per m	JOINT DESCRIPTION: Orientation	Roughness	Alteration	Persistence	Spacing	P.L.T. Ia/50	OTHER TESTS: Type	Results	REMARKS	DRILLING DATA: Water Loss	Core Loss	Method	Depth
40		LAVA; greyish green, fine-grained, highly amygdaloidal, unweathered.			40			Quartz cemented.						NB. CORE DISTURBED. The original borehole log indicates that "closely jointed" rock occurs from 38.3m downwards. This suggests that the core recorded in this log fr. 42.0m represents the rock conditions at and above the tunnel level.				40
41					41	60°	Pln.& Sph.		THREE with RANDOM	1-3								41
42					42	6°	Pln.& Sph. with slicks.	Quartz & Calcite coatings.		03-1								42
						40°	Und.& Sph. with slicks.											
						38°	Und.& Sph. with slicks.											
						60°	Pln.& Sph.					UCS.	311 MPa					
43					49		Und.& Irreg.	Closed Calcite stringers.				Q	2870 kg/m³	42.0 to 43.5m: Rock displays slight indications of shearing.				43

NX ROTARY DIAMOND.

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: BOSHKOPPIES TUNNEL

Bh No: 203 S | Inclination: Vertical | Location: 3175 m | Logged by: ISOC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE No.	RQD %	DEPTH (m)	DIAM.	JOINT DESCRIPTION: Orientation	Roughness	Alteration	No. of sets	Persistence	I.S.I. Is(50)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Used	Core Rec.	Method	Depth
44		LAVA; greyish-green fine-grained and slightly amygdaloidal, unweathered.			44		30°	Und.& Sph.	—	SEMI- with RANDOM			UCS 254 MPa				NX	ROTARY DIAMOND	44
							60°												
							70°	Fin.& Sph.	Quartz & Calcite cement.										
45					45		50°						P 2880 kg/m³						45
							0°	Und.& Sph. with slicks.											
							45°, 55°	Fin.& Smth. with slicks. Fin.& Smth.											
46					46										CROWN - 43,5 INVERT - 45,0				46

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Bh No: BOR 7 | Inclination: Vertical | Location: 3910 m | Project: BUSHKOPPIES TUNNEL | Logged by: ISCC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC %	CORE RQD %	JOINTS per m	JOINT No.	JOINT DESCRIPTION: Inclination	Roughness	Alteration	Wall Str.	Spacing	P.L.S.: Is50	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water loss	Core size	Method	Depth
26		LAVA: greyish-green fine-grained, unweathered.				26	40°	Fin & Rgh.	Stain.			11,7							26
							20°												
27						27	20°	Und & Rgh.	Calcite cement.	FOUR	0,3-1	13,3					NX	ROTARY DIAMOND.	27
							0°		Stain & clay coating		V 2								
							0°	Fin & Smth.			1-3				CROWN				
							15°	Und & Rgh.	Stain.			12,8							
28						28	25°		Stain & clay coating.			15,2							28
							25°												
							10°	Fin. & Smth.											
							30°		Stain.										
							35°												
29						29	10°												29
							0-5°	Und. & Smth.							INVERT				

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Bh No: BOS 1 | Inclination: Vertical | Location: 3910 m | Project: BUSHKOPPIES TUNNEL | Logged by: ISCO

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	ROD %	FRACT. FREQ.	JOINT DESCRIPTION: Inclination	Roughness	Alteration	No. OF SETS	Spacing	P.L.I. Is(50)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: ROD?	CORE DIA.	Method	Depth
30		LAVA; greyish-green, fine-grained, unweathered.			30	40°	Plan & Rgh.	Stain	FOUR		11,7					NX	ROTARY DIAMOND.	30
						10°	Plan & Smth	Calcite cement.										
31					37		Plan & Rgh.	Stain.										31

SHEET 1/2

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Bh No 308 B | Inclination: Vertical | Location: 4295 S

Project: BUSHKOPPIES TUNNEL

Logged by: ISOC

DEPTH	LEGEND	ROCK DESCRIPTION	CORE Rec %	CORE RQD %	CORE Fract. per m	CORE Hard.	JOINT DESCRIPTION Inclination	Roughness	Alteration	Hist. Rel. Orient.	Spacing	P.L.I. I$_{s(50)}$	OTHER TESTS Type	Result	REMARKS	DRILLING DATA Water loss	Core size	Method	Depth
39							0°												39
							20°	Und & Sph.	Clayey silty coating		1-3								
		LAVA; greyish-green, fine-grained and highly amygdaloidal, unweathered.									1-3	12,2							
40							40°												40
							10°	Und.	Closed.		1-3								
							15°	Pln & Sph.	Calcite cement										
							20°	Pln & Smth.	—								NX	DIAMOND.	
41							25°	Pln & Sph.	With mineral.	FOUR with RANDOM.		12,7						ROTARY	41
							30°	Und & Sph. with Undu.	Calcite coating						CROWN				
							35°	Pln & Sph. with Undu. Pln & Sph.	Mica coating										
42							40°	Pln & Smth. with Undu.	—										42
							5°	Pln.	—										
							30°	Pln & Smth.	—										
							45°												

BOREHOLE LOG FOR DETAILED JOINT STUDIES — Project: BUSHTOPPIES TUNNEL

BH No.: 508 S — Inclination: VERTICAL — Location: 4295 m — Logged by: [illegible]

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE Rec. %	RQD %	Fract. No./m	JOINT DESCRIPTION: Inclination	Roughness	Alteration	INTACT ROCK STR.	Spacing	P.L.I. (MPa)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Dia.	Method	Depth	DEPTH (m)
43						15°	Una. & Rgh. Plan.	Calcite coating, clayey; calcite cemented											43
						25°													
						10°	Und. & Rgh.	Pyrite, Quartz & calcite coating						INVERT					
						20°					10,5 9,3					NX			
		LAVA; greenish-grey fine-grained, unweathered.							FOUR with BARTON								CORE BARREL		
44						8°	Una. & Polish and discontinuous	—			≥17,0			Rock as before but with indications of shearing.					44
45											12,7 @ 45,5								45

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Bh No: Bon 9 Inclination: Vertical Location: 5036 E

Project: MUSHROOMFIELS TUNNEL

Logged by: JSCS

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC %	CORE RQD %	JOINT DESCRIPTION Inclination	Roughness	Alteration	No. of Joints/m	Spacing	P.L.S. (MPa)	OTHER TESTS Type	Result	REMARKS	DRILLING DATA Water Loss	Core Size	Method	Depth
47					20°	Und.	Closed; calcite cement.			11,6							47
						Discont.	Calcite veins.			9,3							
48		LAVA; greyish-green, fine-grained and slightly porphyritic and amygdaloidal; unweathered.			20°	Und.	Closed; calcite cement.	ONE	0,5-1						NX		48
										10,2			CROWN			ROTARY DIAMOND.	
49																	49
										6,8							
50					10°	Und.	Closed; calcite cement.						INVERT				50

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Bh No: 306 TV (Inclination Vertical)
Location: 6000 m
Project: HUGHKOPPIES TUNNEL
Logged by: TGM

DEPTH (m)	GEOLOG	ROCK DESCRIPTION	CORE REC %	CORE RQD %	JOINT DESCRIPTION: Inclination	Roughness	Alterations	No. of sets	Spacing	P.L.T. (qbd)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core	Method	Depth
					30°	Und. & Rgh.	Clayey silt coating & stain.			2,0			CROWN The original log suggests that this zone extends from 30,8 to 34,0m. It has been assumed therefore that similar rock conditions as recorded here occur above crown level.				
					80°	—	5mm thick Quartz vein.										
					30° 10°	Und. & Smth. Pln. & Rgh.	Silky coating & stain										
		LAVA; greyish-green, fine-grained and highly amygdaloidal, moderately weathered.			20°	Und. & Rgh.	Silky coating & occ. stain.										
33					10°	Und. & Smth.	Silky coating										33
					20°	Und. & Rgh.							Numerous leached vugs occur above 34m.				
					70°	Pln. & Rgh.	Stain.										
					20°	Pln.	Closed.	RANDOM	0,05-0,3	2,7							
34					20°	Pln. & Rgh.	Stain, silty coat. & talc. Shear.	THREE with	0,3-1				INVERT				34
					10°	Und. & Rgh.	Stain & silty coating								NX	ROTARY DIAMOND.	
					0°	Und.											
					80°	Und.	Closed.										
35					25												35
					10°	Pln.											
		LAVA; greyish-green, fine-grained, amygdaloidal, unweathered.			20°	Pln. & Rgh.	—										
					20°	Und. & Rgh.	Stain. & Silty coat.										
					75°	Pln. & Smth.	—										
36					36												36
					20°	Und.	Calcite vein.										

525

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: BUSHKOPJIES TUNNEL

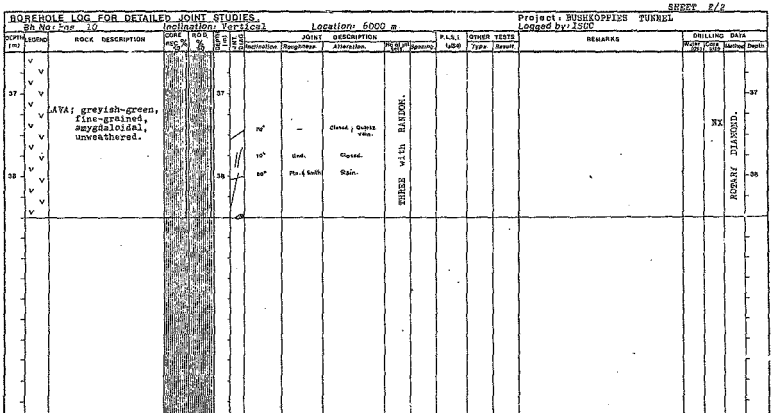
Bh No: 205 K

Inclination: Vertical

Location: 5000 =

Logged by: INC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	ROD %	TCR	SCR	RQD	JOINT DESCRIPTION: Inclination	Roughness	Alteration	Dip/Strike	Spacing	P.L.s.i. Is(50)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Water Pres.	Method	Depth
								70°	Und. & Sph.	Clayey Silt coating & stain.			4,78			CROWN				
								60°	—	4mm thick Quartz vein.						The original log suggests that this zone extends from 30,8 to 34,0m. It has been assumed therefore that similar rock conditions as recorded here occur above crown level.				
								34°	Und. & Smth.											
								18°	Pln. & Sph.	Silty coating & stain.										
		LAVA; greyish-green, fine-grained and highly amygdaloidal, moderately weathered.						30°	Und. & Sph.	Silty coating & Fe stain.										
33							33	10°	Und. & Smth.	Silty coating										33
								20°	Und. & Sph.											
																Numerous leached vugs occur above 34m.				
								70°	Pln. & Sph.	Stain.										
								20°	Pln.	Closed.										
34							34	20°	Pln. & Sph.	Stain, silty coat. & fully filled calcite.	BANDOCK	8,05-92	2,7			INVERT				34
								10°	Und. & Sph.	Stain & silty coating.		92-1						HX		
								0°	Und.											
								20°	Und.	Closed.										
35							35	10°	Pln.		with								ROTARY DIAMOND	35
		LAVA; greyish-green, fine-grained, amygdaloidal, unweathered.						80°	Pln. & Sph.	—										
								20°	Und. & Sph.	Stain & silty coat.										
								10°	Pln. & Smth.	—										
36							36				CHERT									36
								20°	Und.	Calcite vein.										



BOREHOLE LOG FOR DETAILED JOINT STUDIES

SHEET 2/2

Bh No: mx 10 | Inclination: Vertical | Location: 6000 m | Project: BUSHEKOPPIES TUNNEL | Logged by: ISRM

DEPTH (m)	GRAPHIC	ROCK DESCRIPTION	CORE REC %	RQD %	CORE LOSS	JOINT SET	JOINT DESCRIPTION: Orientation	Roughness	Alteration	No. of sets per unit	Spacing	P.L.S.I (MPa)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water (Core loss)	Core Size	Method	Depth
37		LAVA; greyish-green, fine-grained, amygdaloidal, unweathered.			37		N°	–	Closed ; Quartz veins.	THREE with RANDOM.							MX	ROTARY DIAMOND.	37
							10°	Und.	Closed.										
38					38		80°	Plan. & Smth.	Stain.										38

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Bh No: Sos 11 Inclination: Vertical Location: 6357 m

Project: NUSHKOPPIES TUNNEL
Logged by: TSOO.

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	CORE RQD %	CORE DIAM. (mm)	JOINT SPACING (m)	JOINT DESCRIPTION: Roughness	JOINT DESCRIPTION: Alteration	JOINT DESCRIPTION: No. of Joint Sets	JOINT DESCRIPTION: Spacing	PLT: Is50	OTHER TESTS: Type	OTHER TESTS: Result	REMARKS	DRILLING DATA: Water Loss	DRILLING DATA: Core Size	DRILLING DATA: Method	DRILLING DATA: Depth
21						21	10° Undl.	Clayey.										21
							40°	Stain.										
							20° Undl & Rgh	Stain.										
							80° Irreg.	Calcite cement.										
							45° Undl & Rgh	Stain				UCS.	459 MPa					
							20° Plnr & Smth					ρ	2910 kg/m³					
							85°											
							85° Plnr & Smth	Stain	RANDOM	1 – 3				CROWN				
22		LAVA; greyish-green, fine-grained and slightly amygdaloidal, slightly weathered.				22	15°	Stain & silty clay coat.	3 NEAR WITH	0,3 – 1						NX	ROTARY DIAMOND.	22
							10° Undl & Smth											
							15° Plnr & Smth											
								Stain.										
23						23	10° Undl & Smth											23
												UCS.	495 MPa					
							70°					ρ	2950 kg/m³	INVERT				
							80° Plnr & Smth											
						24												24

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Bh No: Sor 12 Inclination: Vertical Location: 6665 a
Project: BUSHKOPPIES TUNNEL.
Logged by: ISOO

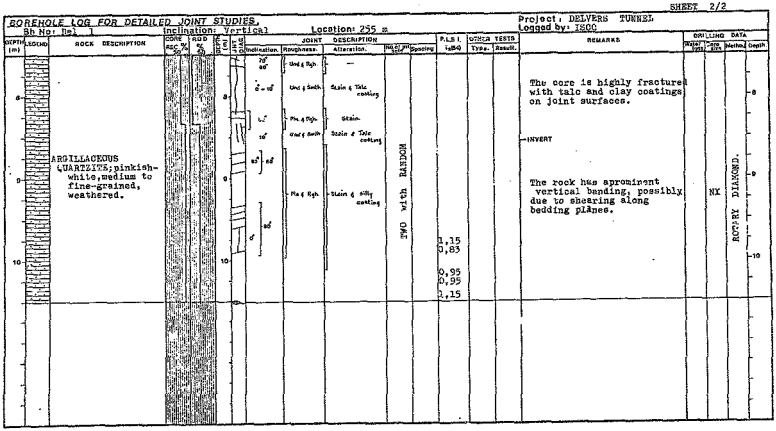
DEPTH (m)	LOGGED	ROCK DESCRIPTION	CORE REC %	CORE RQD %	DEGREE OF WEATHERING	DEGREE OF STRENGTH	JOINT DESCRIPTION Inclination	JOINT DESCRIPTION Roughness	JOINT DESCRIPTION Alteration	Rep of Joints per Metre	Spacing	P.L.T. I50	OTHER TESTS Type	OTHER TESTS Result	REMARKS	DRILLING DATA Water Loss	DRILLING DATA Core Size	DRILLING DATA Machine	DRILLING DATA Depth
6		LAVA; greyish-green, fine-grained, slightly weathered.					20°	Und & Rgh.	Stain & silty clay coating.										6
							20°	Und & Rgh.	Stain & silty clay coating.						Core previously for testing.				
															CROWN				
7											0.2-1								7
											>3								
											1-3								
							90°	Und.	Closed; calcite compact.	SEVERE						NX	DIAMOND	BOYLEY	
8		LAVA; greyish-green, fine-grained, slightly amygdaloidal, slightly weathered.											UCS.	384 MPa 2850 kg/m²					8
															INVERT				
9							85°	Fin & Smth.	Stain.										9

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Bh No: Sag 12
Inclination: Vertical
Location: 6665 m
Project: BUSHKOPPIES TUNNEL
Logged by: JSOO

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	RQD %	FRACTURES PER METRE	JOINT DESCRIPTION: Orientation (to CA)	Roughness	Alteration	JOINT DESCRIPTION: Dip of Joint (to CA)	Spacing	P.L.S.I. Is(50)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Size	Method	Depth
10		LAVA; greyish-green, fine-grained, slightly amygdaloidal and porphyritic, slightly weathered.			10	75°	Plm.	Closed; chlde & Quartz (Fmm)				UCS ?	384 MPa 7870 kg/m³			NX	ROTARY DIAMOND	10
						10°	Und.	Closed; chlde. coated										
11					11	65°	Plm-Undl.	Occ. sulphides on surfaces.	MERGE									11

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Bh No: DH 1 Inclination: Vertical Location: 255 m
Project: DELVERS TUNNEL
Logged by: JSOC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	CORE RQD %	CORE FRAC. No/m	JOINT DESCRIPTION Inclination	Roughness	Alteration	NO. OF JOINTS	SPACING	P.L.S. I (MPa)	OTHER TESTS Type	Result	REMARKS	CORING Run No.	CORING Core Size	CORING Depth
						85°	Und & Rgh	Stain & silty						0,0 to 3,24: Artificial and residual horizons.			
4						40°											4
						80°	Rou & Rgh	Stain & silky coating									
						10°	Und & Rgh										
						45°	Und & Rgh	Stain						Core mainly in fragments but indications of 10° and 80° joints are quite common.			
5		QUARTZITE; maroon, medium-grained and arkosic, weathered.													NX	ROCKY DIAMOND	5
									TWO with RANDOM								
						10°	Fln & Rgh	Stain						Rock is friable.			
6						80°	Und & Rgh										6
						0° - 10°	Und & Smth	Talc coating		0,05-0,2				CROWN			
7		ARGILLACEOUS QUARTZITE; pinkish-white, medium to fine-grained, weathered.				40°	Und & Rgh	Stain									7



BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Bh No: Bs1 1
Inclination: Vertical
Location: 255 m
Project: DELVERS TUNNEL
Logged by: ISRM

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	RQD %	FRACTURE INDEX (No. per m)	JOINT DESCRIPTION: Indication	Roughness	Alteration	Infilling (Gouge)	Spacing	P.L.S.I. (MPa)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Size	Method	Depth
8		ARGILLACEOUS QUARTZITE; pinkish-white, medium to fine-grained, weathered.				70° 60°	Und & Rgh	—						The core is highly fractured with talc and clay coatings on joint surfaces.				8
						0 = 10°	Und & Smth	Stain & Talc coating										
						5°	Pln & Rgh	Stain										
						10°	Und & Smth	Stain & Talc coating						INVERT				
9						85° 85°												9
							Pln & Rgh	Stain & silty coating						The rock has aprominent vertical banding, possibly due to shearing along bedding plAnes.		NX	DIAMOND	
						0° 90°				2MD with RANDOM							ROTARY	
10											0,15 0,83							10
											0,95 0,96							
											1,15							

BOREHOLE LOG FOR DETAILED JOINT STUDIES
Bh No: No 2 Inclination: Vertical Location: 330 m
Project: DELVILLE TUNNEL
Logged by: ICCC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC %	ROD REC %	Fract. per Metre	JOINT DESCRIPTION: Inclination	Roughness	Alteration	No. of Sets	Spacing	PLS I (MPa)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Water Pressure	Hole Size	Depth
1														0,0 to 2,3m; Transported and residual horizons.				1
2		QUARTZITE pale-yellow, coarse-grained completely weathered.												Core in fragments; indications of 0° to 5° joints with clay and talc coatings.				2
3																		3
4		QUARTZITE; buff, coarse-grained, completely weathered.				20° 5° 25° 30° 35° 40°	Med & Righ.	Talc coating Stain Stain Talc coating Stain Talc coating Stain & Talc co. Stain & Silty mat. Stain & Talc Coat.	FOUR with RANDOM	0,3 - 1 0,3 - 1 0,3 - 1	2,0 2,6 1,08 2,1			All core displays indications of intense shearing.		NX	ROTARY DIAMOND.	4
5						10° 15° 5° 15° 10° 10°	— Plan.& Righ. Plan.& Smth. Und.& Righ.	Stain Closed; but with Talc. Talc. coating Stain.			0,5							5
6						10° 20°	Plan.& Righ. Und.& Rgh.	Talc coating Stain & Talc coating										6

BOREHOLE LOG FOR DETAILED JOINT STUDIES.

Bh No: DH 2 Inclination: Vertical Location: 330 m

Project: DELVERS TUNNEL

Logged by: ISOC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	RQD %	FRACTURE FREQ. per metre	W - Y - Y	JOINT DESCRIPTION: Declination	Roughness	Alteration	Hardness	Spacing	POINT LOAD TEST: Is(50) kg/cm²	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Size	Method	Depth
															CROWN				
							80°	Pln & Sph	Stain.			3,8							
7					7		75°								Closed joints at 10° cause stepped surfaces on open joints.				7
							55°	Und & Sph	Stain & Talc coat										
							60°		Stain & Talc coat. Talc coating										
							40°	Pln & Sph											
							70°		Stain & Talc coat.						Rock sheared as before but also with zones of soft talcose material. These possibly represent shear surfaces.				
							60°	Und & Sph	Stain.										
8					8		80°	Pln &	Talc coating			3,5							8
							60°	Und & Sph	Stain								NX		
		QUARTZITE: buff, coarse-grained, completely weathered.					5°	Pln & Rough	Talc coating						INVERT				
							10°												
							80°		Stain & Talc coating	POUR with RAMSON		2,2						ROTARY DIAMOND.	
9					9							2,5							9
								Und & Sph				1,5							
							0°		Stain.			1,08			Core in fragments.				
							30°												
							65°												
10					10							2,2							10

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Bh No: 1267 S
Inclination: 45° North
Location: 355 m
Project: DELVERS TUNNEL
Logged by: ISDC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	CORE RQD %	No. OF FRACT.	WEATHERING GRADE	JOINT DESCRIPTION Orientation.	Roughness.	Alteration.	No. of sets	Spacing	P.L.S. I$_{50}$ MPa	OTHER TESTS Type.	Result.	REMARKS	DRILLING DATA Water Circ.	Core Dia.	Method	Depth
4		QUARTZITE; buff, medium-grained, completely weathered.					45°	Und & Rgh.	Stain & silty clay coating						0,0 to 3,55m: Artificial and residual soils and very soft rock, weathered quartzite.				4
							4	Und & Smth.	Stained & hot with Stain, clay fill										
							30°	Und & Rgh.	Stain										
							30°	Und & Smth. Und & Rgh.	Stain & Hard Talc coating										
							40°	Und & Smth.	Stain & Talc coat.										
							35°	Und & Rgh.	Red clay fill										
							65°		Stain & clay coat		03-1								
5							5	P.R.& Smth.	Talc coating			0,84							5
		ARGILLACEOUS QUARTZITE; buff, medium-grained, highly weathered.									005-03	1,37							
							85°	Und & Rgh.	Clay seam fill.		03-1								
							85°		Stain Stain & clay coating										
6									Talc coating			0,62							6
							85°												
							60°	Und & Smth. Und & Rgh.	Stain & Talc or st.										
							80°	Pln & Smth.	Talc & clay coat Talc coating										
							85°	Und & Rgh.	Stain										
7							85°	Und & Smth.	Talc coating			0,33							7
							85°	Und & Rgh. with Undls.	Quartz coating										
							85°	Und & Rgh.	Stain										

WHERE

NX

ROTARY DIA-CORD

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: DELVERS TUNNEL

Bh No: DeT 3 | Inclination: 45° North | Location: 305 m | Logged by: ISRC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC %	CORE RQD %	CORE FRACT. PER m	JOINT DESCRIPTION: Inclination	Roughness	Alteration	JOINT SET	JOINT SPACING	P.L.I. (MPa)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Gain	Water Loss	Method	Depth
8		ARGILLACEOUS QUARTZITE; buff, medium-grained, highly weathered.				40°	Und & Rgh.	Talc coating										
						60°	Pln & Rgh.	Hard grey coat.			1,25							
						70°	Und & Rgh.	Silky coating			0,84							
						75°	Und. & Smth	Talc coating Red clay coat.										
						10°	Und. & Smth	Talc coating						Broken core.				
						55°	Und & Rgh.	Stain & Talc coat.			0,84							
9						85°	Und & Smth.	Talc coating Stain & Talc coat.			0,75							9
						55°	Und & Rgh.											
						85°	Pln & Rgh.	Stain.										
						85°	Und & Rgh.	Stain & Mica.										
						85°	Und & Smth	Talc coating.			0,42			CROWN				
		ARGILLACEOUS QUARTZITE; as above but coarse-grained.				80°	Und & Rgh.	Red clay coat.								NX	ROTARY DIAMOND	
10						45°	Pln & Smth	—	THREE									10
						70°	Und & Smth	Stain & Mica.										
						80°	Pln & Smth	Talc coating										
						85°	Und & Rgh.	Stain.										
						85°	Und & Smth.	Talc coating										
11						85°	Und & Rgh.	Stain			0,42							11
						50°	Pln & Smth.	Talc coating										
						40°	Und & Rgh.	Stain Talc coating										
						85°	Pln & Smth	Stain Talc coating										
							Und & Rgh.	Stain & Talc coat.										

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: DELYERS TUNNEL

BH No: WC1 1 Inclination: 45° North Location: 385 m

Logged by: JWCC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE DIA. mm	ROD %	WATER LOSS	JOINT DESCRIPTION: Inclination	Roughness	Alteration	No. of Jts/m	Spacing	P.L.S.I. (psi)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Rqd	Core Dia.	Method	Depth
13		ARGILLACEOUS QUARTZITE: as above but medium-grained.				90°	Vwy v Rgh	Stain & Talc coat			1,15			INVERT				
						60°	Plm & Smth	Talc coating										
						15°	Vwy v Rgh	Stain.										
						70°	Plm & Rgh.	Stain & clay coat.										
						45°	Plm & Smth	Stain.										
						25°	Vwy v Rgh	Talc coating										
								Stain & Talc coat.										
						50°	Plm & Rgh.	Talc coating										
						90°	Vwy v Rgh.	Stain.										
						45°	Vwy v Smth	Talc coating										
						25°	Vwy v Smth Undl v Rgh	Stain & Talc coat										
						40°	Vwy v Rgh											
						25°	Undl v Rgh											
						70°	Vwy v Smth	Talc coating										
								Stain.										
14						60°	Undl v Smth	Talc coating	THREE		1,05							
											2,10							

DRILLING DATA: NX; ROBERT DILWORTH

BOREHOLE LOG FOR DETAILED JOINT STUDIES
Bh No: Del 4 (Inclination: Vertical) Location: 590 m
Project: DELVERS TUNNEL
Logged by: IHOD

DEPTH (m)	SCALE	ROCK DESCRIPTION	CORE No.	CORE %	RQD %	JOINT DESCRIPTION Inclination	Roughness	Alteration	No. of JS/m	Spacing	R.L.S.I. (x10)	OTHER TESTS Type	Result	REMARKS	DRILLING DATA Water Loss	Core Run	Method	Depth
						55°	Und & Sph.	Clay & mica coating, Stain & pit.						0,0 to 2,1: Artificial soil and weathered quartzite boulders.				
		QUARTZITE; pinkish-white, medium-grained, slightly weathered.				70°	Pln & Sph.	Stain.			0,84							
						45° 70°	Und & Sph. Pln & Sph.	Stain & mica. Stain, mica & chl.			4,5							
						60°	Und & Sph.	Stain & mica. Stain.			3,8							
3		QUARTZITE; buff, medium-grained, highly-weathered.			3	21°	Pln & Sph.	Stain & Talc coat										3
						75°	Und & Sph.	Stain.						Broken core.				
						25°	Pln & Smth	Stain & Mica.										
						35°	Und & Sph.	Stain. Talc coating & Mica		0,05-0,2								
						0°	Und & Sph.	Stain & fine silty fill (1-3 mm)	FOUR with RANDOM	0,3-1	3,6			Faint slickensides apparent.			ROCKMEC DIAMOND.	
4					4	80°	Und & Sph.	Stain		0,3-1					NX			4
											7,5							
		QUARTZITE; pinkish-whit-, medium to coarse-grained, slightly weathered.			5	45° & 20°	—	Closed but stained			5,8							
						88°	Und & Sph.	Stain & silty coating										
						32°	Und & Sph.	Talc coating			6,9							
						76°	Und & Sph.	Stain						Horizontal slickensides in hard oxide coating.				
6					6						7,4							6
						45° & 20°	—	Closed but stained										

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: DELVERS TUNNEL

Bh No: No 4 Inclination: Vertical Location: 590 m Logged by: ISCO

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	ROCK QUALITY %	WATER LOSS (LUGEONS)	JOINT DESCRIPTION Set/Joint No.	Roughness	Alteration	No. of Sets	Spacing	P.L.S.I. Value	OTHER TESTS Type	Result	REMARKS	DRILLING DATA Water Loss	Water Rtn	Material	Depth
7		QUARTZITE; as before becoming completely weathered below 7,4				30°												7
						60°								CROWN				
						30°		Stain										
8		VEIN QUARTZ and QUARTZITE; COmpletely weathered.				30°	Und & Sph.	Stain & silty coating						Shear zone.				
						60°		Stain.			6,8					NX		
						80°		Stain & silty coat.										8
		QUARTZITE; grey, medium-grained and glassy, unweathered.				45°			POOR with RANDOM									
						30°					13,5							
9						40°	Fln & Sph.										ROBERT DIAMOND	
						Irreg.	—	Non-softening Mica development.										
						25°	Und & Sph.							INVERT				
10																		10

BOREHOLE LOG FOR DETAILED JOINT STUDIES — Project: DELVERS TUNNEL
Bh No: BH 5 — Inclination: Vertical — Location: 735 m — Logged by: ISCC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	RQD %	FRACT. FREQ. per m	JOINT DESCRIPTION: ORIENTATION	JOINT DESCRIPTION: ROUGHNESS	JOINT DESCRIPTION: ALTERATION	NO. OF SETS	SPACING	P.L.S.I.	OTHER TESTS: TYPE	OTHER TESTS: RESULT	REMARKS	DRILLING DATA: WATER LOSS	DRILLING DATA: CORE DIA.	DRILLING DATA: METHOD	DRILLING DATA: DEPTH
0														0,0 to 2,5: Artificial soil and completely weathered quartzite.				
3		QUARTZITE; pink, fine to medium-grained, highly weathered.				55° 55° 70° 49° 5° 90° 15° 80° 40° 35° 45° 85°	Und & Sgh. No & Sgh. Und & Sgh. — Und & Sgh.	Stain & silky coating stain Stain & silky coating Closed Int. filtered Stain & clay coat. Red clay coating (1-2 mm).	SHEARED with RANDOM	0,05-0,2 0,3-1	0,62 1,28 0,75			Prominent banding developed at approximately 20° to core axis.		NX	DIAMOND	3
5		SHALE and SILTSTONE; pink, fine-grained, highly weathered.												Core in fragments with apparent 0° joint set spaced at 50mm. Rock appears to be sheared and has talc coatings on joint surfaces. Crown			ROTARY	5
6						30°	Und & Sgh.	Stain			0,62							6

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: DELVERS TUNNEL

Bh No: Inel 5 Inclination: Vertical Location: 735 m Logged by: ISCO

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE Rec. %	ROD %	RQD (Min)	JOINT DESCRIPTION: Discrimination	Roughness	Alteration	Shape	Spacing	P.L.S.I. (I_s(50))	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Dia	Method	Depth
7		ARGILLICEOUS QUARTZITE:				20°	Una & Rgh.	Clay coating						Shaly bands also present.				7
														INVERT				
8		QUARTZITE: pink to buff, fine to medium-grained, moderately weathered.							SEMES with RANDOM					There is evidence of planar and slickensided, vertical joints with clay coated surfaces; Spacing 80mm Occasional fractures at 20° to 30° also occur.		NX	ROBERT DIAMOND	8
						20°		Stain			2.2 3.5							
9						10°	Una & Rgh.	Silty coating										9
						20°		Stain										
10																		10

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Bh No.: a1 c

Inclination: Vertical

Location: 760 m

Project: DELVERS TUNNEL

Logged by: ISCO

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE Rec. %	CORE Str. %	RQD %	Fracs per m	Weathering	Discontinuities	JOINT DESCRIPTION: Roughness	JOINT DESCRIPTION: Alteration	Weathering Grade	Spacing	P.L.S.I (MPa)	OTHER TESTS: Type.	OTHER TESTS: Result.	REMARKS	DRILLING DATA: Water Loss	DRILLING DATA: Core Dia.	DRILLING DATA: Method	DRILLING DATA: Depth
								85°	Pln. & Smth.	Silky coating						0,0 to 1,75: Evenpsprted and residual soils.				
2						2		85°	Pln. & Rgh.	Stain										2
								85°	Und. & Smth. Pln. & Rgh.	Silky coating Stain										
								75°	Und. & Rgh.							Broken core.				
								85°	Pln. & Rgh.	Stain & silky coating										
3						3		80°	Und. & Rgh.	Thin clay coat (1-3 mm)		0,05-0,2								3
		ARGILLACEOUS QUARTZITE; pink, medium-grained, highly weathered.						0°	Pln. & Smth.	—		0,2-1	1,35					NX	ROTARY DIAMOND	
								70°		Stain & clay coat										
								85°	Und. & Rgh.	Stain										
								65° 155° 200°	Und. & Rgh.	Stain & clay coating Shiny clay coat. Stain.										
4						4		80°	Pln. & Smth.							Broken core.				4
								0°	Und. & Rgh.	Silky clay coat.										
								30° 10°	—	Stain. Chlorid.										
								25°	Und. & Rgh. with slicks.	Silky coating			1,35							
								60°	Und. & Rgh. with slicks.	Stain & clay coat.						—CROWN				
5						5		80°												5
								25° 10°	Und. & Rgh. Pln. & Smth.	Stain.										
								70°	Und. & Rgh. with slicks											

THREE WITH RANDOM.

BOREHOLE LOG FOR DETAILED JOINT STUDIES
Bh No: De1 6 Inclination: Vertical Location: 760 m
Project: DELVERS TUNNEL
Logged by: ISCO

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	RQD %	JOINTS (per m)	JOINT Inclination	JOINT Roughness	JOINT DESCRIPTION Alteration	JOINT Infill	JOINT Spacing	P.L.S.I. (MPa)	OTHER TESTS Type	OTHER TESTS Result	REMARKS	DRILLING DATA Water Loss	DRILLING DATA Core Size	DRILLING DATA Material	DRILLING DATA Depth
6		QUARTZITE; pink streaked brown and white, medium and fine-grained, highly weathered.				10°	Und & Rgh.	Stain			1,55					NX	CORE FROM BOREHOLE	6
						30°					1,90							
						60°	Plan & Rgh.	Stain & clay coating										
						30°												
						75°		Stain										
						90°					2,80							
7						60°	Und & Rgh.							INVERT				7
						0°		Stain & clay coating (1-5 mm)										
						20°	Und & Smth.	Stain & clay coating clay coating			1,55							
						75°		Stain & clay coat.			2,60							
8						75°	Und & Rgh.	Stain										8
						60°												
						30°	Und & Smth.				1,15							
						70°												
						30°		(1-5 mm)										
9						60°	Und & Rgh.	Stain & clay coating						Broken core.				9
						15°	Und & Smth.	(1-5 mm).										
						20°	Und & Rgh.											
10						15°	Plan & Rgh.				1,35							10
						20°												

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Bh No: DJE 1-H Inclination: Horizontal Location: 383 B
Project: N1 SOUTHKLOOF TUNNEL
Logged by: ISCC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE Rec %	CORE RQD %	JOINT DESCRIPTION: Orientation	Roughness	Alteration	JOINT DESCRIPTION: Per m	Type	Aper mm	Roughness	P.L.S.I. I_s(50)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water loss	Core size	Method	Depth
0					70°	Irreg	—					8,2							0
					85°	Und & Rgh	—												
					70°	Pln & Rgh	—												
					45°	Und & Rgh													
					40°	Pln & Rgh													
1					35°	Und & Rgh	Stain												1
					70°	Pln & Rgh						7,1							
					70°	Pln & Rgh with slicks	Silty oxide coating					12,6							
					35°	—	Clayey												
					40°	Pln & Smth	stain			005-03									
2					35°	Und & Rgh	Quartz coating Stain.	FOUR		1 3							NX	ROTARY DIAMOND.	2
		QUARTZITIC SANDSTONE; cream to greyish-white, medium-grained, moderately weathered (with Mudstone inclusions.)			65°							6,0							
					45°	Pln & Rgh and & Rgh	Stain Stained.					7,2							
					70°	Pln & Rgh	Stain & Quartz coat. Stain & Quartz coat.								Broken core; evidence of 70° to 80° joints with staining and quartz coating					3
3													UCS.	299 MPa	Previously removed for testing.				
													P	2640 kg/m³					
4					80°														4
					70°	Und & Rgh	Stain.					5,7							
					80°														

BOREHOLE LOG FOR DETAILED JOINT S[illegible]IES.

Bh No: [illegible] 1-W Inclination: Horizontal Location: [illegible]

Project: DU TOITSKLOOF TUNNEL

Logged by: [illegible]

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE %	ROD %	RQD	FRACTURE INDEX	JOINT DESCRIPTION: Inclination	Roughness	Alteration	No. of sets	Spacing	P.L.S.I.	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: WEIGHT (KG)	CORE SIZE	METHOD	Depth
5							80°	Und & Sph.	Stain						Broken core; The following joints are evident: 5.15 - 5.50: 55°, U + R, Stain & quarts coating.				5
							70°	Pln & Sph.							40°, U + R, — 5.50 - 5.80: 50°, " — 5.80 - 6.20: 80°, " Stain.				
6							65°	Und & Sph. with Bridges	Stain										6
							85°		Closed										
7		QUARTZITIC SANDSTONE; cream to greyish-white, medium-grained, moderately weathered, (with mudstone inclusions.)					70°	Pln & Sph.	—								NX		7
							65°	Und & Sph.	Stained										
							80°	Pln & Sph.											
8							50°	Pln & Sph. Und & Sph.	Stain & Quartz coat. —										8
							85°								Previously removed for testing.				
							70°						P	2640 kg/m²					
							60°	Und & Sph.	Stain.			5,3							
							85°					6,1							
							80°					7,5							

JOINT

BOREHOLE LOG FOR DETAILED JOINT STUDIES. Project: DU TOITSKLOOF TUNNEL
Bh No: 1st 1-H Inclination: Horizontal Location: 383 m Logged by: IBCO

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE Ang.	CORE %	ROD %	DEPTH (m)	JOINT DESCRIPTION Orientation	Morphology	Alteration	RQD/JV	Spacing	P.L.I. (MPa)	OTHER TESTS Type	Result	REMARKS	DRILLING DATA Water Loss	Core Dia	Method	Depth
10		QUARTZITIC SANDSTONE; cream to greyish-white, medium-grained, moderately weathered. (with mudstone inclusions.)				10	10°	Undul. Rgh.	Stain	POOR		14,5			Broken core.		NX	ROTARY DIAMOND.	10
							10°	Plan. Rgh.	Stain & Calcite.			10,7							
							50°	Undul. Rgh.	Stain Oxide & Calcite - Stain			7,4			Broken core.				
							60°	Undul. Rgh.	Stain										
							40°	Plan. Smth.	Quartz coating										

BOREHOLE LOG FOR DETAILED JOINT STUDIES.

Project: DU TOITSKLOOF TUNNEL

BH No: DTJ 1-V Inclination Vertical (up) Location: 383 m Logged by: JGD

DEPTH (m)	SCALE	ROCK DESCRIPTION	CORE REC. %	RQD %	FRACTURES per m	JOINT DESCRIPTION: Inclination	Roughness	Alteration	Infilling	Spacing	P.L.S.I. (MPa)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: WATER LOSS	CORE LOSS	Method	Depth
1		QUARTZITIC SANDSTONE: cream to greyish-white, medium-grained, moderately weathered. (with mudstone inclusions.)				80°, 85°, 45°, 70°, 0°, 30°, 70°, 41°, 10°, 70°, 0°, 40°, 85°, 10°, 20°–45°, 45°, 20°, 10°, 20°	Und & Rgh; Pln & Rgh; Und & Rgh; —; Und & Rgh; Pln & Rgh; Und & Rgh, Und & Smth; Und & Irreg; Und & Rgh.	Quartz coating; Quartz coating; Quartz coating; Closed. Stain & Quartz coat; Stain	MODERATELY WEATHERED	Q3-1; Q10-Q3	9,2; 10,3; 4,2; 2,2; 2,8			Numerous micro-fractures are evident in the core.		MX	ROTARY ALIGNED.	

SHEET 1/4

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Project: ZU VOLKSLOOP TUNNEL
Bh No: WT 7-11 Inclination: Horizontal Location: 522 m Logged by: IGCC

DEPTH (m)	LOGGING	ROCK DESCRIPTION	CORE % REC.	CORE % RQD	JOINT DESCRIPTION: Roughness.	JOINT DESCRIPTION: Alteration.	Aperture (mm)	Spacing	P.L.S.I. (MPa)	OTHER TESTS: Type.	OTHER TESTS: Result.	REMARKS	DRILLING DATA: Water Rate	DRILLING DATA: Core Size	DRILLING DATA: Method	DRILLING DATA: Depth
0					4° Pln. & Smth. 35° Pln. & Rgh. 80° Pln. & Smth.	— — Stain.			5,9							
1		QUARTZITIC SANDSTONE: cream, medium-grained, moderately weathered.			80° Und.	Clayey.		0,3-1				Broken core.				1
					40° Und. & Rgh. 80°	Stain.		0,3-1	7,6	UCS. P	77 MPa. 2690 kg/m³			HX	ROTARY DIAMOND.	
2							SEMES with RANDOM		9,2 9,3			Previously removed for testing.				2
3		QUARTZITIC SANDSTONE: Grey, medium to fine-grained, slightly weathered. (with mudstone inclusions.)			0° Und & Rgh. 80° Pln. & Smth. 80° Und. & Rgh.	Talc coating ~ Stain.				UCS. P	128 MPa. 2730 kg/m³					3
					0° 80° 70° Und & Rgh. Pln. & Smth.	~ ~ Stain.			6,4			Previously removed for testing.				
4		As from 0-2,5m.			40° Und & Rgh. Pln. & Smth.	— —										4

247

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
BH No: Jnt 2-4 Inclination: Horizontal Location: 522 m
Project: DU TOITSKLOOF TUNNEL
Logged by: IGOC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE %	ROD %	JOINTS per m	JOINT DESCRIPTION: Inclination	Roughness	Alteration	Infill/Fill Width	Spacing	P.L.T. Type	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water	Core Size	Machine	Depth
5		QUARTZITIC SANDSTONE; cream to pink with depth, medium-grained, moderately weathered.				70°	Und & Sgh.	Stain.			4,9			Broken core; Indications of 50° joints with Und. & Slicks.				5
6					6	72° 60°	Und & Sgh. Und & Smth.	Stain		SHEAR with RANDOM	3,4 1,3			Broken core; The majority of breaks in this section appear to be machine breaks.		NX	ROTARY DIAMOND.	6
7					7	65° 70°	Und & Sgh. Pln & Smth.	— —										7
8		QUARTZITIC SANDSTONE; grey, medium to fine-grained, slightly weathered.			8	8°	Und & Sgh. Pln & Smth.	Stain —			2,0							8
					9	80°					2,6			Broken core.				
						70°	Und & Sgh.	Stain										

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: DE TOITSKLOOF TUNNEL

Bh No: Tnt 2-N | Inclination: Horizontal | Location: 522 m | Logged by: ISCC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE Rec. %	CORE RQD %	CORE Fract. No.	DEPTH (m)	JOINT SET No.	JOINT DESCRIPTION: Inclination	Roughness	Alteration	No. of joints per metre	Spacing	P.L.S.I. (x50)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Size	Method	Depth
10		QUARTZITIC SANDSTONE; grey, medium to fine-grained, slightly weathered.				10		70°	Und & Sgh.	Stain.	THREE with RANDOM							NX	ROTARY DIAMOND.	10
								80°	Und & discontinuous	Stain										
								70°	Und. & Sgh.	Stain.										
11						11		5°	Pln & Sgh. with stain.	Calcite coating										11
								20°	Pln & Smth.	—										
12						12		45°	Und.	Closed but with clay cark						Very broken core!! Mainly machine breaks.				12
		QUARTZITIC SANDSTONE; cream, medium-grained, moderately weathered.						20°	Pln & Smth.	Clay coating										
13						13		20°	Pln & Sgh.	Stain			3,1							13
								10°	Pln.	Closed.										

SHEET 4/4

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Bh No: Tnl. 2-H
Inclination: Horizontal
Location 622 S
Project: DU TOIT'S KLOOF TUNNEL
Logged by: ISCC

DEPTH (m)	GEOLOGY	ROCK DESCRIPTION	CORE REC %	RQD %	DEPTH (m)	JNT. SET	JOINT DESCRIPTION: Inclination	Roughness	Alteration	Infilling	Spacing	P.L.S.I. Value	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water loss	Core Size	Method	Depth
14		QUARTZITIC SANDSTONE: cream, medium-grained, moderately weathered.			14		20°	Und. & Rgh.	Stain.										
							0°			CHERT with RAXTON		1,7					NX	ROTARY DIAMOND	14
							70°	Pln.& Rgh.	Stain										

250

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Bh No: 1-1 2-7 Inclination: Vertical (0°) Location: 522 m
Project: DU TOITSKLOOF TUNNEL
Logged by: 1500

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE RECOVERY %	ROD %	TCR (m)	SCR (m)	RQD (m)	Inclination	JOINT DESCRIPTION Roughness	Filtration	Hardness	Spacing	P.L.S.I (MPa)	OTHER TESTS Type	Result	REMARKS	DRILLING DATA Water Loss	Core Size	Method	Depth
1		QUARTZITIC SANDSTONE; cream maroon and grey, medium grained, moderately weathered,						0° 20° 0° 30° 20° 60° 60° 70° 45° 50° 40° 15° 25° 30° 70° 20° 55°	Pln & Rgh. Und & Smth Und & Rgh. Pln & Rgh. Und & Rgh. Pln & Smth. Und & Rgh. Pln & Smth. Und & Rgh. Und & Rgh. Und & Rgh. Pln & Smth.	Clouds ; Quartz cemend. — — — Clay & grit — — Stain clay & grit Stain Stain & silty coating Stain	MODE	1-3 0,3-1 0,3-1		1,0 3,5 4,5 4,2 2,5			Broken core; mainly machine breaks. Broken core.		NX	DIAMOND ROTARY	
2		QUARTZITE; dark grey, medium-grained, unweathered.																		
3		As for 0-1,9m.																		
4																				

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BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Bh No: Int 2-V Inclination: Vertical (up) Location: 522 m
Project: DN 201 SKLOOF TUNNEL
Logged by: ISCC

DEPTH (m)	ZONE	ROCK DESCRIPTION	Core Rec. %	RQD %	Defect Type	Defect No.	JOINT DESCRIPTION: Inclination	Roughness	Alteration	Filling Min/Max	Spacing	P.L.I. (MPa)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water loss	Core Dia.	Method	Depth
5		QUARTZITIC SANDSTONE; cream maroon and grey, medium-grained, moderately weathered.				8	75°	Uns. & Smth.	Stain		POOR	6,4 2,3			Numerous closed hairline fractures between 0° and 20°.		HX	DIAMOND	5
							30°	Pln & Rgh.	—			4,9						REGARY	

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: DE SCHYNSKLOOF TUNNEL

Bh No: 327 1-7 Inclination: Vertical (up) Location: 633 m

Logged by: JDDC

DEPTH (m)	GEOL	ROCK DESCRIPTION	CORE Rec %	CORE RQD %	Defects per m	JOINT DESCRIPTION: Inclination	Roughness	Alteration	Width (mm)	Spacing	PLSI $I_{s(50)}$	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Size	Method	Depth
1		QUARTZITIC SANDSTONE; yellowish orange; medium-grained, highly weathered				70°	Und & Rgh.	Stain & clay coating						NB: The rock has been intensely brecciated and re-healed but has been weathered to a virtually massive rock with no indications of the original bedding planes. Open slickensided joints are also present.				
						20°	Und & Rgh.	—										
						40°	Pln. & Rgh.	—										
						20°	Und & Rgh.	—			0,6							1
						0°	Und. & Rgh. with slicks.	— —										
						10°	Und. & Rgh.	—			1,1							
											1,5							
2						15°	Und. (partly closed)	Grey clay	TWO WIDELY RANDOM	0,04-0,2	0,8				NX	ROTARY DIAMOND.		2
						0°	Und. & Rgh. with grad. slicks.	Stain.						The 0° - 5° joints appear to be discontinuous.				
3						20° - 40°	Und.	—										3
											1,7							
											1,9							
4						55°		Stain										4
						50°	Und. & Smth. with slicks											
						40°		Grey clay.			2,2			Hole drilled to 5,3m				

BOREHOLE LOG FOR DETAILED JOINT STUDIES.
Project: DU TOITSKLOOF TUNNEL
Bh No.: 15 4-H Inclination: horizontal Location: 704 m
Logged by: IsRC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE Incr. %	CORE RQD %	JOINT No. per 0,5 m	JOINT DESCRIPTION: Inclination	Roughness	Alteration	No. of Sets	Spacing	P.L.S. I$_{s(50)}$	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Dia.	Method	Depth
0		GRANITE; grey mottled pink and white and speckled black, medium to coarse-grained, slightly weathered.					Und. & Rgh. with slicks.	Quartz coating			1,1			Broken core.				
1					1	23°					0,8							1
							Fln. & Rgh.	—										
						48°	Und. & Smth.	—										
							Fln.	Closed.			1,5							
														NB: Slickensides on joint surfaces indicates shearing. The rock has a foliated appearance in places, which is apparently also due to shearing.				
2					2	38°	Und. & Rgh.	—	TWO with RANDOM	Q3-1	2,3							2
						23°	Und.	Closed		Q3-1					NX		ROTARY DIAMOND.	
						70°	Und. & Rgh. with V-ribs.				0,5							
						32°	Fln. & Smth. with slicks											
3					3	74°	Und. & Rgh. with slicks	Chlorite coating.			0,8							3
						58°												
						59°	Fln. & Rgh. with slicks				2,7							
						70°												
4					4	48°	Fln	Closed.										4
						28°	Und. & Rgh.	Chlorite coating			0,5							

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: DU TOITSKLOOF TUNNEL

Bh No: Tot S-H | Inclination: Horizontal | Location: 704 m | Logged by: ISCO

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE %	ROD %	FRACT. FREQ. (per m)	RQD %	JOINT DESCRIPTION: Inclination	Roughness	Alteration	Nature of Infill	Spacing	P.L.S.I. Type	P.L.S.I. Value	OTHER TESTS: Type	OTHER TESTS: Result	REMARKS	DRILLING DATA: Water Table	DRILLING DATA: Core Size	DRILLING DATA: Method	DRILLING DATA: Depth
5		GRANITE; grey mottled pink and white and speckled black, medium to coarse-grained, slightly weathered			5		60°	Und. & Sligh. wide stepped	Chlorite coating	RANDOM	1,5	1,2				Previously removed.		NX	ROTARY DIAMOND	5
							90°	Und. & Sligh.	—	SMO with	1,5									

SHEET 1/2

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: DU TOITSKLOOF TUNNEL

Bh No: 64 dd | Inclination: Vertical (up) | Location: 704 m | Logged by: ISCO

Depth (m)	Legend	ROCK DESCRIPTION	CORE RQD %	CORE REC %	JOINT DESCRIPTION: Orientation	Roughness	Alteration	No. of Joints	Spacing	PLSI Is(50)	OTHER TEST: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Loss	Anchor	Depth
		GRANITE; pinkish-grey, speckled black; medium-grained, slightly weathered								3,3							
										2,3							
1		GRANITE(porphyritic); pinkish-grey, medium-grained, slightly weathered			1: 30°/N 2: 35° 85°	— Und & Rough	Closed. —			1,3			Core recovered in approx. 30 cm fragments with apparent irregular joints.				1
2					1: 0° 2: 0°	Und & Rgh Und & Rgh Rough Walls				1,4					5%		2
					1: 30°		Chlorite Coating	FOUR	0,1 - 0,3 0,3 - 1	1,0						SUPPORT INSTALLED.	
3		GRANITE; greyish-green streaked black, medium-grained, slightly weathered			1: 30° - 45°								Rock is friable and intensely fractured.				3
					1: 30° - 45°	Und & Rough Rough Walls											
4					1: 0°	Und & Rgh Rough Walls											4
						Fair & Rgh				0,4							

SHEET 1/2

BOREHOLE LOG FOR DETAILED JOINT STUDIES

BH No: DJS 4-F Inclination: Vertical (Up) Location: 704 m

Project: DU TOITSKLOOF TUNNEL
Logged by: ISCW

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC %	RQD %	JOINTS per m	JOINT DESCRIPTION: Inclination	Roughness	Alteration	Coating	Spacing	FOUR	PLI: Is(50)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water loss	Core size	Method	Depth
0		GRANITE; pinkish-grey speckled black, medium-grained, slightly weathered										3,3							
												2,3							
1		GRANITE(porphyritic) pinkish-grey, coarse-grained, slightly weathered			1	20°/40° 35°/60° 40°	— Und & Sooth	Closed. —				1,3			Core recovered in approx. 30 cm fragments with apparent irregular joints.				1
2					2	0°	Und & Rgh-undl wiskr.				1-3	1,4				NX		ROTARY DIAMOND.	2
						50°		Chlorite coating.			G3-1 G3-1	1,2							
3		GRANITE; greyish-green streaked black, medium-grained, slightly weathered			3	20° - 30°									Rock is friable and intensely fractured.				3
						5° - 10°	Und & Sooth with wiskrs.												
4					4	70°	Und & Rgh-undl wiskr.												4
							Plan & Rgh.					0,4							

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BOREHOLE LOG FOR DETAILED JOINT STUDIES
Rh No: Int 4-V
Inclination: Vertical (up) Location: 704 m
Project: DU TOITSKLOOF TUNNEL
Logged by: ISCO

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	RQD %	FRACT. FREQ. (per m)	DIP (°)	JOINT DESCRIPTION: Inclination	Roughness	Alteration	No. of Joints per 100	Spacing	PLS: Is(50)	OTHER TESTS: Type	Result	REMARKS	DRILLING Data: Water Loss	Core Size	Method	Depth
5	+ +	GRANITE; greyish-green, streaked black, medium-grained, slightly weathered			5	45° 70° 60° 0°-5°	Und (Rgh	Chloritic coating.	FOUR			0,6 0,4			Rock intensely fractured with slickensides and chlorite evident on the 0°-5° joints.		NX	ROTARY DIAMOND.	5

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Project: DU TOITSKLOOF TUNNEL

Bh No: tut. 1-V | Inclination: Vertical | Location: 1005 m | Logged by: JUCC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE RECOV. %	RQD %	FRACT. PER METRE	JOINT DESCRIPTION: Indication	Roughness	Alteration	JOINT SETS: No. of sets	JOINT SETS: Spacing	P.L.I. (MPa)	OTHER TESTS: Type	OTHER TESTS: Results	REMARKS	DRILLING DATA: Water Loss	DRILLING DATA: Core Size	DRILLING DATA: Method	DRILLING DATA: Depth
1		GRANITE(porphyritic grey, coarse-grained, unweathered.			1	0° / 45°	Plan & Sph. Und. & Sph.	— Calcite coating	THREE	1-3 / 03-1	0,9 / 2,9			Broken core; 30-40 cm fragments.		NX	ROTARY DIAMOND	1
2					2	30° / 45° / 0°-5° / 85°	Und. & Sph. Und. & Sph. Und. & Sph.	— — Calcite coating						UNJOINTED.				2

SHEET 1/1

BOREHOLE LOG FOR DETAILED JOINT STUDIES

BH No: 015 fwt Inclination: Horizontal Location: 1535 m Project: DU BOISBLOOP TUNNEL Logged by: IGCE

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE Rec. %	ROD Rec. %	DEFECTS per metre	JOINT DESCRIPTION: Inclination	Roughness	Alteration	No. of Sets	Spacing	P.L.I. (MPa)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water Loss	Core Dia.	Method	Depth
1		GRANITE(porphyritic) grey, coarse-grained, unweathered.			1	40°	Slick & Rgh.	Calcite coating			5,0					NX	ROTARY DIAMOND.	1
											6,5							
						70°	Pln.& Rgh.	—										
										0,06-0,3	7,4							
								Stained; Calcite Lining			6,5							
							~		2	0,3-1	6,9							
						15°	Slick & Rgh.	Calcite coating			1,0							
											7,9							
2					2						4,4							2
											7,6							
						80°	Slick & Rgh.	Calcite coating			3,6							
											8,2							

BOREHOLE LOG FOR DETAILED JOINT STUDIES

Bh No.: 41 (—V

Inclination: Vertical

Location: 1535 m

Project: KU ZOTZKLOOF TUNNEL

Logged by: ISOC

DEPTH (m)	LEGEND	ROCK DESCRIPTION	CORE REC. %	CORE RQD %	JOINTS per m	JOINTS per set	JOINT DESCRIPTION: Orientation	Roughness	Alteration	WATER	JOINTS per m	P.L.S. (MPa)	OTHER TESTS: Type	Result	REMARKS	DRILLING DATA: Water loss	Core Size	Method	Depth
1		GRANITE(porphyritic) grey, coarse-grained, unweathered.			1		80°	Und. & Sph.	Calcite cement			6,9					NX	ROTARY DIAMOND.	1
							40°												
							70°	Pln. & Rgh.	—										
							20°					3,9							
							40°				1–3	6,6							
							75°	Und. & Sph.	Calcite & Quartz coating		0,3–1	2,2							
												11,1							
							70°												
2					2		45°	—	Glossy calcite cement.			10,1							2
												9,8							

APPENDIX III A

TERMINOLOGY USED IN THE TEXT AND ON THE JOINT SURVEY SHEETS

TERMS USED IN THE TEXT TO DESCRIBE THE WEATHERING, TEXTURE AND JOINT SPACING

(Adopted from the South African Section of The Association of Engineering Geologists)

WEATHERING:

Unweathered

No visible signs of alteration in the rock material but fracture planes may be stained or discoloured.

Slightly weathered

Fractures are stained or discoloured and may contain a thin filling of altered material. Discolouration may extend into the rock from the fracture planes to a distance of up to 20 per cent of the fracture spacing (i.e. less than 40 per cent of the rock is discoloured).

Medium weathered

Slight discolouration extends from fracture planes for a distance greater than 20 per cent of the fracture spacing (i.e. generally greater part of the rock). Fractures may contain filling of altered material. The surface of the core is not friable (except in the case of poorly cemented sedimentary rocks) and the original texture of the rock has been preserved. Partial opening of grain boundaries may be observed.

Highly weathered

Discolouration extends throughout the rock. The surface of the core is friable and usually pitted due to washing out of highly altered minerals by drilling water. The original texture of the rock has mainly been preserved but separation of grains has occurred.

Completely weathered

The rock is totally discoloured and the external appearance of the core is that of a soil. Internally the rock texture is partly preserved but grains have completely separated.

/TEXTURE .....

TEXTURE:

Description	Grain Size (mm)	Recognition
Very fine-grained	< 0,06	Individual grains cannot be seen with a hand lens.
Fine-grained	0,06 - 0,2	Just visible as individual grains under a hand lens.
Medium-grained	0,2 - 0,6	Grains clearly visible under hand lens, just visible to the naked eye.
Coarse-grained	0,6 - 2,0	Grains clearly visible to the naked eye.
Very coarse-grained	> 2,0	Grains measureable.

JOINT SPACING:

Description	Spacing (mm)
Very widely jointed	> 1000
Widely jointed	300 - 1000
Medium jointed	100 - 300
Closely jointed	30 - 100
Very closely jointed	10 - 30

ABBREVIATIONS USED FOR THE FEATURES RECORDED FROM THE DETAIL LINE JOINT SURVEYS

Column 1: Distance along the measuring tape given in metres.

Column 2: Rock Type

- Q - Quartzitic Sandstone
- BQ - Brecciated Quartzitic Sandstone
- G - Granite
- L - Lava
- D - Diabase

Column 3: Rock Hardness *

- R1 - Very soft rock - material crumbles under firm blows with a sharp end of a geological pick and can be peeled off with a knife. It is too hard to cut a triaxial sample by hand.
- R2 - Soft rock - can just be scraped and peeled with a knife; indentations 3 mm show in the specimen with firm blows of the pick point.
- R3 - Hard rock - cannot be scraped or peeled with a knife; hand-held specimen can be broken with a hammer end of a geological pick with a single firm blow.
- R4 - Very hard rock - hand-held specimen breaks with hammer end of pick under more than one blow.
- R5 - Very, very hard rock - specimen requires many blows with geological pick to break through intact material.

Column 4: Rock Structure

- BJ - Bedding Joint
- SJ - Shear Joint
- CJ - Cross Joint

Columns 5 and 6: Dip angle and dip direction.

Column 7: Gouge Thickness **

1 0,0 cm
2 0,0 - 0,5 cm
3 0,5 - 2,5 cm
4 2,5 - 5,0 cm
5 > 5,0 (specify)

Column 8: Gouge Type

St - Stain
Cl - Clay
Ch - Chlorite
Ca - Calcite
Sd - Sand
Qz - Quartz
Qz V - Quartz Vein
WR - Weathered Rock
CR - Crushed Rock
DR - Decomposed Rock
SR - Sheared Rock
Mi - Mica
Gr - Grit

Column 9: Gouge Hardness *

S1 - Very soft - easily moulded in fingers; shows distinct heelmarks.

S2 - Soft - moulds in fingers with strong pressure; faint heelmarks.

S3 - Firm - very difficult to mould in fingers; difficult to cut with a hand spade.

S4 - Stiff - cannot be moulded in fingers; cannot be cut with hand spade and requires hand-picking for excavation.

S5 - Very stiff - very tough and difficult to move with hand-pick; requires pneumatic spade for excavation.

Column 10: Surface Roughness *

1 - Slickensided surfaces
2 - Smooth
3 - Defined ridges
4 - Small steps
5 - Very rough

Column 11: Surface Waviness

U - Undulating
P - Planar

Column 12: Joint Continuity *

0 - Both ends can be seen.
1 - One end extends out of face.
2 - Both ends extend out of face.

Column 13: Joint Water **

N - No water.
D - Joint surface feels moist.
S - Surface saturated but no observed flow.
F - Flow.

Column 14: Remarks

Includes any data that cannot be accommodated in the other columns.

* After: Piteau, D., 1970

** After: Jennings, J.E. and MacG. Robertson, A., 1973

APPENDIX XII B

T SURVEY DATA SHEETS

APPENDIX III B

JOINT SURVEY DATA SHEETS

JOINT SURVEY DATA SHEET

Project: MARKOPITS TUNNEL **Location:** Pos 1

Distance along tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
0,95	1	S5		80	121	2	St & Cl	S2	4	U	2	S	
1,1				65	300	2	St & Cl	S2	5	U	2	S	
2,2				90	123	0			4	U	1	D	
2,8				80	305	0			3	P	0	D	
3,7				95	310	2	St & Cl	S2	5	U	0	S	
4,05				90	370	3	St & Cl		3	U	2	S	
6,3				85	110	0			4	U	0	S	SET NO.1
6,6				90	128	0			3	U	0	S	Dip ~ 68°
6,75				90	140	0			4	U	0	S	Direction ~ 192°
7,4				85	320	0			3	U	0	D	
7,6				80	300	3	St & Cl	S2	4	U	2	D	
7,7				90	105	2	St & Cl		4	U	2	S	
7,75				85	277	3	St & Cl		4	U	2	S	
8,7				85	290	2	St & Cl		3	P	2	D	
9,7				50	310	2	St & Cl		3	U	2	S	
1,4				70	156	2	St & Cl	S2	3	U	2	F	
1,8				70	193	2			3	U	2	F	
2,6				60	187	2			3	P	2	D	
3,2				65	196	2			4	U	2	S	
3,8				75	203	2			4	U	0	D	SET NO.2
4,4				60	195	2			3	U	1	S	Dip ~ 85°
5,0				65	207	2			3	U	2	S	Direction ~ 309°
7,1				70	171	2			3	P	2	S	
7,9				80	200	2			3	P	0	S	
3,				30	256	2	Cl	S2	4	P	2	S	SET NO.3 Dip ~ 30° Direction ~ 256°
5,				85	090	2	St & Cl	S2	4	U	2	S	~CCB. SET NO.4
10,0				85	082	2			3	U	2	S	Dip ~ 85° Direction ~ 085°
1,0				50	090	2	St		3	U	0	D	
5,4				60	295	2			3	U	2	S	Random
9,0				80	042	2			4	U	2	S	

JOINT SURVEY DATA SHEET

Project: PUMPING PIPES TUNNEL **Location:** Bay 2

Distance along tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
1.50	1	S5		75	310	1			1	P	0	D	
2.0				85	315	1			2	P	1	D	
2.1				85	325	1			1	P	2	D	
2.35				85	316	1			3	P	2	D	
2.4				80	312	1			3	P	1	S	SET NO.1
2.55				80	120	1			3	P	1	D	Dip - 90°
3.15				55	145	1	St		3	P	1	D	Direction - 315°
4.0				80	135	1	St		2	P	1	D	
4.80				55	327	1			3	U	1	SX	
5.70				80	315	1			2	P	0	N	Poss. Random
6.3				70	327	1			3	U	1	SX	
6.8				45	145	1			3	P	1	SX	
2.2				90	065	1			4	U	0	D	
5.9				80	047	1			3	P	1	S	SET NO.2
6.1				75	250	1			3	P	0	S	Dip - 90°
6.4				90	069	1			3	U	0	D	Direction - 253°
7.9				85	250	1			2	P	1	N	
9.45				83	235	1			2	P	2	N	
1.3				13	215	1			3	P	2	D	
1.8				32	208	1			3	U	0	D	
3.15				22	238	1			3	U	0	D	
5.6				20	265	1			2	U	2	N	Set NO.3
7.0				20	187	1			3	U	2	N	Dip - 20°
7.4				25	227	1			4	U	1	N	Direction - 245°
8.8				25	195	1			2	P	0	N	
10.0				20	245	1			3	U	1	N	
0.0				85	250	1			3	P	2	S	
0.8				85	258	1			2	P	0	S	
1.2				90	025	1			3	U	0	S	
1.35				50	155	1			3	U	1	S	
1.55				90	172	1			3	P	0	D	Random
1.90				90	133	1			2	U	0	D	
3.1				80	220	1			3	U	1	S	
3.3				85	303	1	St		2	P	2	S	
4.28				25	255	1			3	U	2	D	
4.50				80	265	1	St		2	P	1	D	
5.0				55	305	1			3	U	1	D	
5.6				85	220	1			3	P	0	N	
5.95				90	037	1			2	P	0	N	
6.75				80	030	1			3	P	0	N	
6.13				90	180	1			3	P	0	N	
6.55				90	175	1			2	P	2	N	
7.9				85	265	1			2	P	1	N	
7.35				40	218	1			3	U	1	N	
8.1				20	145	1			3	U	2	N	
8.2				90	047	1			4	P	1	N	
8.45				90	085	1			2	U	1	N	
9.0				10	155	1			3	U	0	N	
9.6				15	225	1			2	P	0	N	

JOINT SURVEY DATA SHEET

Project: ROSEMOOR-PIES TUNNEL — Location: Bay 3

Distance along Tube (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dir. Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
0.0	L	R4		80	035	1			2	P	0	W	
0.2				65	210	1			2	U	0	W	
1.1				80	215	1			4	P	2	W	
4.4	D[X]	R2		80	010	2	Ch		2	U	2	D	
5.0				90	070	2	Ch		3	P	2	W	SET NO.1
5.45				90	040	1	St		5	U	1	W	Dip - 90°
8.7	L	R4		85	210	1			2	P	2	D	Direction - 218°
9.4				80	040	1			4	P	1	D	
0.80				55	088	1			5	U	0	W	SET NO.2
1.8				90	120	1			4	U	2	D	Dip - 85°
3.0				60	120	1			3	U	2	W	Direction - 123°
8.1				75	115	1			3	U	2	D	
9.85				90	120	1			1	U	2	W	
3.15	L	R4		25	218	1			2	P	0	W	SET NO.3
5.5	D[X]	R2		25	238	1			3	P	1	D	Dip - 29°
7.6	L	R4		25	255	2			3	P	0	D	Direction - 251°
9.5				20	240	2	Cl		3	P	1	D	
1.65	D[X]	R2		85	320	1			3	P	1	D	M.25 SET NO.4
8.5	L	R4		85	139	3	Ch & Cl		3	U	2	W	Dip - 90°
													Direction - 116°
0.15	L	R2		60	070	1			2	P	0	W	
0.25				75	025	1			3	P	0	W	
0.65				45	080	1			2	P	0	W	
0.70				90	145	1			3	P	0	W	
0.85				85	025	1			3	P	1	W	
1.4				75	200	1			3	U	0	W	
1.6				85	185	1			3	P	1	W	
2.15				80	052	1			3	U	2	W	Random
2.4				80	250	1			3	U	1	W	
2.7				90	090	1			2	U	1	W	
2.9				50	340	1			3	P	2	W	
3.1				90	000	1			2	P	0	W	
3.3				85	340	1	St		3	P	1	W	
3.4				90	017	1			2	P	1	W	
3.5				90	065	1			3	P	2	W	
3.6				90	075	1			2	P	1	W	
3.7				70	310	1			3	U	0	W	
3.75	L	R4		65	175	1			3	U	1	W	
3.80	D[X]	R2		90	060	3	Ch & Cl		3	P	2	D	
4.8				85	130	2	Ch		3	P	2	W	
5.3				80	105	2	Ch		3	U	1	W	
5.8				55	300	1			3	U	2	S	
6.0				85	050	2	Cl		3	P	2	D	
6.2	L	R4		90	200	1			3	U	1	S	
6.5				85	120	2			3	U	1	S	
7.0				90	070	1			4	U	1	D	
7.1				70	270	1			3	U	2	D	
7.9				85	210	1			2	P	1	W	
8.1				80	090	2			3	U	2	D	
9.0				45	275	1			3	U	1	W	
9.70				80	065	1			2	P	0	D	
10.0				75	225	1			3	P	0	W	

X Fine-grained dykeous dyke occur between 3.60 and 6.6 m along survey line.

JOINT SURVEY DATA SHEET

Project: MOKELUMNE TUNNEL **Location:** Box 4

Distance along tape (ft)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
0,52	D	85		73	357	1			3	P	2	W	
0,92				65	000	1			3	P	2	W	SET NO.1
1,73				65	010	1			3	P	0	W	Dip - 70°
2,30				80	005	1			4	P	2	W	Direction - 000°
5,6				55	00.5	1			3	P	2	W	
9,45				70	0.0	1			3	U	2	W	
2,60				80	345	1			3	P	1	W	
3,43				75	340	1			2	P	0	W	SET NO.2 Dip - 80°
4,72				65	345	1			3	P	0	W	Direction - 340°
8,20				80	342	1			3	U	1	W	
0,80				85	090	1			2	U	0	W	
2,05				90	090	1			5	P	0	W	SET NO.3
8,15				80	225	1			2	P	0	W	Dip - 80°
8,9				85	235	1			2	P	2	W	Direction - 230°
9,35				80	235	1			2	P	2	W	
10,0				90	260	1			2	U	0	W	
0,0				55	140	1			3	U	0	W	
0,15				85	265	1			3	U	1	W	
1,45				70	026	1			3	U	2	W	
2,70				50	235	1			3	P	0	W	
2,90				80	187	1			3	U	1	W	
3,35				85	030	1			4	P	0	W	
5,9				60	235	1			3	U	0	W	
6,2				65	320	1			3	P	1	W	Random
4,35				85	345	1			3	P	1	W	
4,55				85	300	1			3	P	2	W	
5,1				90	243	1			3	P	2	W	
5,8				90	345	1			3	U	2	W	
5,35				90	180	1			4	P	2	W	
3,3				90	005	2			3	U	0	W	
6,4				55	260	1			3	U	0	W	
6,6				65	350	1			5	U	1	W	
6,7				85	075	1			4	P	1	W	
6,85				85	162	1			3	P	0	W	
7,2				85	020	1			4	U	1	W	
7,45				90	110	1			3	P	1	W	
7,8				40	260	1			4	U	1	W	
8,0				65	053	1			3	U	2	W	
8,6				90	275	1			4	P	0	W	
9,80				80	280	1			2	U	2	W	
-				40	295	1			3	U	1	W	
-				15	295	1			3	P	0	W	Subhorizontal
-				15	320	1			2	P	0	W	Joints above Tape Line

JOINT SURVEY DATA SHEET

Project: BENGPIPE TUNNEL **Location:** Box 5

Distance along Tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
2.2	L	SS		20	265	1			2	P	0	S	
4.75				55	180	1			4	U	2	W	SET NO.1
5.4				80	190	1			3	P	0	S	Dip - 25°
9.0				25	220	1			4	U	1	N	Direction - 190°
10.0				90	205	1			4	U	2	W	
4.65				85	325	1			3	U	0	N	
6.4				90	315	1			2	U	1	N	SET NO.2
6.8				90	325	1			3	U	2	N	Dip - 90°
7.9				75	140	1			3	P	0	N	Direction - 317°
8.35				80	330	1			3	U	2	W	
9.75				85	135	1			3	P	0	S	
				90	135								
2.3				75	110	2	Ca	R2	3	U	2	N	SET NO.3
5.2				60	215	1			3	U	1	N	Dip - 70°
6.0				75	115	1			3	P	0	N	Direction - 113°
2.65				80	285	3	Ca	R2	3	P	2	N	
2.85				60	285	2			3	P	0	S	
2.95				80	285	2	Ca	R2	3	P	2	N	
0.0				15	250	1			3	U	1	N	
0.3				80	130	1			3	U	0	N	
0.5				70	315	1			3	U	0	N	
0.7				55	145	1			3	U	0	N	
0.75				90	300	1			3	U	1	N	
1.1				80	150	1			3	U	0	N	
1.2				75	150	1			2	P	0	N	Random
1.65				70	095	2	Ca	R2	4	U	2	N	
2.75				85	180	1			2	P	1	N	
3.45				85	260	1			3	P	1	N	
3.6				80	135	1			4	U	0	N	
3.75				45	190	1			3	U	2	N	
4.0				70	235	1			3	P	0	N	
4.2				90	015	1			2	P	1	N	
5.0				40	190	1			4	U	0	N	
5.1				80	305	1			4	U	2	N	
5.65				90	300	1			3	U	1	N	
5.8				90	060	1			2	U	0	N	
6.2				70	025	1			4	P	0	N	
6.6				50	175	1			3	P	0	N	
6.9				70	150	1			3	U	2	N	
7.5				70	355	1			2	U	2	N	
7.8				35	265	1			4	U	0	N	
9.4				85	085	1			2	P	1	N	
9.6				80	115	1			2	P	0	N	
-				70	025								Subhorizontal
-				70	339								Joints above Tape
-				90	220								Line
				40	145								

JOINT SURVEY DATA SHEET

Project: BENBOSPILA TUNNEL — Location: Box 6

Distance along tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
1,1	L	H5		75	050	1			4	U	C	H	
2,5				80	252	1			3	P	I	H	SET NO.1
4,9				85	040	1			3	U	C	H	Dip - 90°
5,45				85	223	1			3	U	O	H	Direction - 046°
6,6				85	045	1			4	U	2	H	
7,58				85	225	1			3	P	2	H	
10,0				80	235	1			3	P	2	H	
3,1				75	345	1			3	U	O	H	
5,2				85	115	2			4	U	1	H	SET NO.2
5,4				85	295	2	Ca	B2	3	U	2	H	Dip - 90°
5,55				80	100	2	Ca	B2	3	U	2	H	Direction - 116°
6,3				60	315	2	Ca	B2	1	U	2	H	
7,3				80	115	1			3	U	1	H	
9,1				90	130	1			3	P	O	H	
3,7				35	000	1			2	U	O	H	SET NO.3
4,95				33	005	1			3	U	1	H	Dip - 33°
													Direction - 030°
0,1				89	086	1			3	P	O	H	
0,2				70	300	1			3	P	O	H	
0,5				70	093	1			3	U	O	H	
0,6				70	128	1			2	U	2	H	
0,7				88	090	1			2	U	O	H	
0,9				85	164	1			3	U	O	H	
2,0				65	054	1			2	P	O	H	
2,8				75	125	1			3	P	1	H	Random
3,4				79	340	1			3	P	O	H	
4,4				55	085	2	Ca	B2	2	P	1	H	
4,5				65	175	1			2	P	O	H	
5,0				85	285	1			3	U	2	H	
5,75				35	135	1			4	U	O	H	
8,3				85	030	1			3	P	1	H	
9,25				85	005	1			4	U	O	H	
9,8				25	055	1			3	P	O	H	

JOINT SURVEY DATA SHEET

Project: SCHEMELPFEIS TUNNEL — **Location:** Sos 7

Distance along tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
0,4	L	R5		80	276	2	J1 J3	C2	5	P	2	M	
2,8				90	245	1	J1		5	P	1	M	SET NO.1
5,2				80	242	1	J1		3	P	1	M	Dip ~ 80°
5,8				82	060	1	J1		2	P	0	M	Direction ~ 298°
5,9				90	264	1	J1		4	U	2	D	
6,2				80	240	1	J1		3	P	1	M	
7,2				85	225	1	J1		3	U	2	M	
8,2				85	227	2	J1 & J2	R3	5	P	2	M	
1,1				85	302	1			2	P	2	M	SET NO.2
1,15				85	302	1			2	P	2	M	Dip ~ 85°
1,95				85	232	1	J1		2	P	0	D	Direction ~ 302°
6,4				80	235	3	J1 J2	C2	2	P	0	D	
2,55				40	230	1			3	U	0	M	SET NO.3
3,9				45	225	1			3	U	0	M	Dip ~ 45°
5,0				45	228	1			5	P	0	M	Direction ~ 228°
4,8				85	190	1			2	P	0	M	SET NO.4
8,5				90	005	2	J1 & J2	R3	3	P	0	D	Dip ~ 90°
9,1				85	005	2			2	P	2	M	Direction ~ 006°
0,15				75	200	2			3	P	0	D	
1,50				65	282	1			3	U	2	M	
1,55				70	262	1	J1		3	U	1	M	
2,2				70	280	1			3	P	0	M	
2,4				70	280	1			3	P	0	M	
2,5				75	323	1			2	P	0	M	
2,9				90	145	1	J1		5	P	1	M	Random
3,05				80	147	1	J1		2	P	0	M	
3,2				75	305	1			2	U	0	M	
3,4				50	155	1			4	P	0	M	
3,55				35	300	1	J1		3	U	0	M	
4,0				60	272	1			2	P	0	M	
5,1				40	115	1			3	U	0	M	
5,3				85	315	1			3	P	1	D	
5,8				70	196	1			3	P	0	M	
7,0				80	328	1			5	P	1	M	
8,0				50	330	1			3	P	1	M	
10,0				70	325	1			5	P	0	M	
"	"	"	"	05	305	1			4	U	2	M	Above tape
"	"	"	"	10	175	3			3		2	M	~ 9,8 m.

JOINT SURVEY DATA SHEET

Project: ELTENTUNNEL — **Location:** Tos 8

Distance along tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
0,25	L	H3		70	N 4	1	St		2	P	0	D	
1,2				75	125	1			2	P	2	M	
1,6				85	125	1			3	P	1	M	
2,95				80	120	1			2	P	0	M	
4,05				70	135	1			2	P	1	M	
4,8				90	140	1			2	P	1	M	SET NO.1
5,2				81	115	1			3	P	1	M	Dip - 85°
5,6				85	125	1			3	P	0	M	Direction - 132°
6,0				80	125	2	Gr V	H5	3	P	2	M	
6,6				90	310	1			3	P	1	M	
7,2				90	125	1			3	U	1	M	
7,8				90	320	1	St		2	P	1	M	
8,8				90	140	1			3	P	1	M	
9,25				85	135	1			3	P	1	M	
9,8				85	125	1			2	P	1	M	
5,7				80	238	1	St		3	U	1	M	SET NO.2
6,25				85	240	1			2	P	0	M	Dip - 83°
6,7				80	240	1			3	P	0	M	Direction - 220°
7,5				80	237	1			3	U	1	M	
0,85				75	110	1			2	P	0	M	
1,4				80	078	1			2	P	1	M	
1,7				80	050	1			2	P	1	M	
2,75				80	025	1			2	P	0	M	
3,2				80	030	1			2	P	0	M	SET NO.3
3,75				80	028	1			2	P	1	M	Dip - 75°
4,0				75	030	1			2	P	1	M	Direction - 050°
4,3				80	038	1			2	P	1	M	
9,0				70	020	1			2	P	2	M	
9,7				80	020	1			2	P	2	M	
10,0				80	016	1			2	P	1	M	
0,0				35	173	3	Gr & Mi	R4	4	P	2	M	SET NO.4
1,1				40	185	2	CR	R2	3	P	2	M	Dip - 33°
2,3				30	185	1			2	P	0	M	Direction - 180°
7,4				35	175	1			3	P	0	M	
0,35				65	100	1			3	P	0	M	
0,55				85	110	2	Gr V	R4	3	P	2	D	
0,6				70	105	1			3	U	0	D	
0,7				70	105	1			3	U	0	M	
3,15				45	150	1			2	P	0	M	
3,85				45	150	1			2	P	1	M	
2,3				50	130	1			3	P	1	M	Random
3,1				80	040	1			2	P	0	M	
3,45				80	100	1			2	P	0	M	
4,25				75	110	1			3	P	1	M	
5,0				90	052	1			2	P	1	M	
5,1				80	343	1			2	P	0	M	
6,35				75	115	1			2	P	1	M	
7,0				35	216	1			3	P	2	M	
8,0				55	062	1			2	P	0	M	
"				15	200	1			3	U	2	M	above tape @ 8,0 m.

JOINT SURVEY DATA SHEET

Project: BUSHKOPPIES TUNNEL **Location:** Box 9

Distance along tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
0,85	L	R5		85	120	1			5	P	O	N	
4,5				85	310	1			4	P	O	S	
4,5				90	315	2	Ca	R3	3	Clay	2	H	SET NO.1
4,6				85	125	2	Ca	R3	5	Clay	O	N	Dip - 90°
5,1				75	340	2	Ca	R3	3	U	1	M	Direction - 315°
5,8				85	320	1			5	P	O	N	
5,8				85	135	2	Ca	R3	5	U	O	N	
7,5				90	322	1	Cru Ca		5	U	1	N	
8,6				85	145	2	Ca	R3	3	U	2	D	
3,2				80	265	1			3	U	O	N	
4,2				70	252	1			5	U	1	N	SET NO.2
5,4				75	242	1			2	P	O	N	Dip - 75°
7,6				85	250	1			5	P	1	M	Direction - 250°
7,8				75	255	1			5	P	1	M	
8,1				80	247	1			5	U	1	M	
0,4				80	104	1			3	U	O	N	
2,0				75	100	1			4	U	1	M	SET NO.3
2,2				60	265	1			5	U	1	N	Dip - 76°
4,75				80	095	1			5	U	1	M	Direction - 099°
10,0				75	090	1			2	P	O	S	
0,6				90	045	1			3	U	1	N	
0,9				30	312	1			3	P	O	N	
1,1				60	075	1			5	P	O	N	
1,75				90	080	1			5	P	O	F	
1,8				65	215	1			4	P	O	S	
2,15				90	075	1			5	P	O	N	Random
2,5				60	265	1			3	U	1	N	
2,95				60	180	1			4	P	1	N	
4,4				25	340	1			4	P	O	S	
4,85													
5,65				90	005	1			3	P	O	N	
6,55				80	075	2	Ca	R3	3	P	1	N	
6,6				60	165	1			4	U	O	N	
9,4				90	030	1			4	P	1	S	Seepage from
10,0				80	210	1			4	U	1	S	fracture in crown.

JOINT SURVEY DATA SHEET

Project: EUSEBE PRES TUNNEL **Location:** Bas 10

Distance along Base (m)	ROCK Type	ROCK Hardness	ROCK Structure	ORIENTATION Dip	ORIENTATION Dip Direction	COUCHE Thickness	COUCHE Type	COUCHE Hardness	SURFACE Roughness	SURFACE Waviness	Continuity	Water	REMARKS
1,4	L	S5		90	090	1	St		4	U	1	D	
1,6				50	055	2	Cl & St	R3	3	U	2	D	
2,3				75	240	1			2	P	0	W	SET NO.1
4,05				80	095	1			3	P	1	D	Dip - 90°
4,4				80	278	1	Cl & Cl	S2	4	P	2	N	Direction - 277°
4,8				90	285	1	St		2	P	0	N	
5,7				85	275	1			3	P	0	S	
7,75				85	265	1			3	P	0	D	
0,6				45	170	2	Cl & St	S2	5	P	1	S	
0,75				50	160	1	St		3	P	2	D	
1,0				45	156	2	St	R4	4	P	2	D	
4,5				40	131	2	Cl	R3	3	P	2	D	SET NO.2
5,0				50	177	2	Cl	S3	4	P	3	D	Dip - 47°
5,3				60	175	2	Cl	S3	3	P	2	D	Direction - 168°
5,5				30	170	1	St		3	P	1	D	
7,1				45	165	2	St	R4	3	P	2	D	
7,55				45	165	1			4		2	D	
0,3				85	170	1	St		3	U	1	S	
1,2				75	165	1	St		3	U	1	D	
2,35				85	168	1	St		3	U	2	N	
2,95				80	165	1			3	P	0	D	SET NO.3
4,55				75	167	1	St		3	P	0	N	Dip - 80°
5,6				75	173	1	St		3	P	0	S	Direction - 168°
5,8				80	162	1	St		3	P	1	D	
6,0				65	177	1	St		3	U	2	D	
6,6				85	162	1	St		4	P	1	D	
0,6				80	260	1	St		4	P	2	N	
2,05				70	090	1	St		3	U	1	S	
2,2				75	085	1	St		3	U	1	D	
2,9				80	145	1	St		3	P	1	D	
3,75				85	075	1	St		3	U	0	S	Random
4,1				65	180	1	St		4	U	2	D	
4,9				75	186	1	St		3	P	0	S	
6,3				65	140	1	St		3	P	0	D	
7,0				75	066	2	St & VR	R3	3	P	2	D	
7,65				30	000	1			2	P	0	D	
7,80				35	075	1	St		2	P	0	S	
8,4				80	262	1	St		3	P	1	D	
8,6				70	116	4	Cl	S3	3	P	2	W	
9,0				15	098	1			3	U	1	D	
9,3				60	290	1			3	P	0	S	
9,5				60	295	1			3	U	0	N	
10,0				90	270	1	St		3	P	1	S	

JOINT SURVEY DATA SHEET

Project: SUNKOPPIES TUNNEL Location: Box 11

Distance along tape (m)	ROCK Type	ROCK Hardness	ROCK Structure	ORIENTATION Dip	ORIENTATION DIP Direction	DOUDE Thickness	DOUDE Type	DOUDE Thickness	SURFACE Roughness	SURFACE Waviness	Continuity	Water	REMARKS
0,1	L	R3		90	245	1	St		3	P	1	M	
1,1				85	338	1	St		3	P	0	M	SET NO.1
1,25				80	151	2	St		3	P	2	M	Dip - 90°
2,35				85	155	1	St		2	P	0	M	Direction - 156°
2,8				85	153	1	St		2	P	0	M	
3,9				85	150	2	Cl & St	S3	2	P	2	D	
4,2				85	340	1			2	P	0	M	
6,15				90	350	2	Cl & St	S3	3	P	2	D	
0,8				85	176	1	St		2	P	0	M	
1,5				90	003	1	St		3	P	2	M	SET NO.2
1,85				90	005	2	St		2	P	0	M	Dip - 90°
2,4				90	010		St		3	P	2	M	Direction - 004°
4,6				85	185	2	Cl & St	S3	3	U	2	D	
6,6				70	188	1	St		2	P	1	C	
9,5				85	006	2	Cl	S3	2	P	1	D	
6,8				5	232	1			3	U	2	D	SET NO.3
				10	250	2			4	P	2	D	Dip - 10°
				10	254	1	Cl & St	S4	4	P	2	D	Direction - 247°
2,4				90	075	1	St		4	P	1	M	
3,2				90	092	1			2	P	1	M	SET NO.4
4,5				80	085	1	St		2	P	0	D	Dip - 90°
6,2				90	090	2	Cl & St	S3	4	P	2	D	Direction - 083°
7,9				85	252	1	St		2	P	1	D	
0,0				55	264	2	Cl & Cb	S4	3	U	2	D	
0,5				8	043	1	St		3	P	1	M	
1,7				75	085	1	St		3	P	2	M	
5,0				60	034	1	St		2	U	2	D	Random
5,9				40	180	1			2	P	2	D	
8,3				45	289	2	Cl	S3	3	P	1	D	
9,0				75	040	1	St		3	P	1	M	
9,7				40	292	2	Cl	S3	2	P	2	D	
10,0				70	070	1			2	P	1	D	

JOINT SURVEY DATA SHEET

Project: KEMPHOPPERS TUNNEL — **Location:** Sos 12

Distance along tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	ORIENTATION: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
0,4	L	R4		80	272	1	St		4	U	1	S	
1,55				80	275	1	Cl & St	S3	4	U	2	D	
1,50				90	272	2	Cl & St	S3	3	U	2	D	SET NO.1
4,4				85	273	1	St		2	P	2	M	Dip - 85°
4,8				85	272	1			2	P	1	M	Direction - 272°
5,2				85	270	2	Cl	S3	3	P	2	M	
5,4				80	265	1			5	P	1	M	
1,37				90	105	2	Cl & St		3	U	0	D	
4,1				95	102	1	C		5	P	1	D	SET NO.2
3,15				95	290	1			5	P	2	M	Dip - 90°
3,7				65	291	1			2	P	0	M	Direction - 106°
5,9				90	292	1			5	U	1	M	
4,0				85	292	1			2	P	1	M	
5,5				85	295	1	St		5	U	0	M	SET NO.3
5,6				80	245	1			2	U	0	M	Dip - 85°
8,0				80	250	1			4	P	1	D	Direction - 290°
10,0				85	250	1	St		5	P	0	M	
1,75				75	010	1	St		3	U	2	D	SET NO.4
4,6				80	010	1	St		4	P	1	M	Dip - 80°
7,5				80	005	1	St		3	P	0	M	Direction - 010°
8,3				85	012	1	St		3	P	2	D	
0,0				90	145	1	St		3	U	0	M	
0,5				85	020	2	Cl & St		3	P	1	D	
0,9				65	135	2	Cl & St		3	C	0	M	
1,0				80	052	1	St		3	P	0	D	
1,3				85	145	2	Cl & St		4	U	1	D	
2,15				75	130	1	St		3	U	1	M	Random
0,6				70	305	1			3	U	2	D	
2,85				45	242	1			3	U	0	M	
3,6				80	297	2	Cl & St	S3	3	P	0	D	
3,75				90	115	1			3	U	0	M	
5,0				80	345	1	St		3	C	1	M	
6,1				60	215	1			4	U	0	M	
7,1				85	050	1			2	P	1	M	
-				05	290	3	Cl	S3	3	U	2	D	Major horizontal Joint 0,5-1,0 m above tape.

JOINT SURVEY DATA SHEET

Project: DUIVITOKLOOF TUNNEL **Location:** Adit 1

Distance along tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
1,82	M	R4	BM	25	217	1	Cl & Sd	S3	4	U	2	D	
2,95				20	190	2	Cl & Sd	S3	3	U	2	S	SET NO.1
2,0[X]				30	220	1			3	P	0	M	Dip - 28°
2,95[X]				30	220	1			3	P	0	M	Direction - 209°
3,10[X]				30	204	1			3	U	0	M	
1,0			CJ	55	327	1	St		5	U	1	M	
1,2				50	324	1			5	U	1	D	SET NO.2
1,95				60	332	1			3	U	0	M	Dip - 60°
2,4				65	320	1	St		3	P	0	M	Direction - 327°
2,55				70	328	1	St		3	U	2	D	
2,90				70	327	1			5	P	0	M	
0,0				85	027	1			4	P	0	M	SET NO.3
0,20[X]				83	023	1	St		3	U	2	M	Dip - 80°
1,80[X]				75	030	1	St		2	P	1	M	Direction - 027°
0,18				85	345	1	St		3	P	0	M	
0,30				55	002	1			4	U	1	M	
0,55				80	000	1	St		3	U	1	M	POSS. SET NO.4
0,78				60	016	1			3	U	1	M	Dip - 60°
1,00				70	010	1	St		3	U	0	M	Direction - 000°
2,40[X]				55	015	1			4	P	0	M	
2,60[X]				80	010	1	St		4	P	2	M	
0,05				85	001	1	St		3	P	0	M	
0,18				80	342	1	St		3	U	2	D	
0,40				75	350	1			3	U	0	M	
2,06				45	008	1			3	P	0	M	
1,20				70	020	1	St		4	U	2	M	Random
0,10				20	135	1	St		5	P	0	M	
1,60				35	237	2	Cl & Sd	S3	4	P	0	D	
1,52				65	300	1	St		3	U	1	M	
1,72				45	334	1			3	P	0	M	
1,25				90	125	1	St		3	P	1	M	
1,80				35	230	1			3	U	0	M	
1,30				20	065	1	St		3	U	1	M	[X] Survey in
0,60[X]				85	197	1	St		2	P	2	M	E-W direction
0,70[X]				30	115	1			3	P	0	M	
1,40[X]				45	297	1			3	P	0	M	
2,15[X]				90	090	1			5	P	0	M	

JOINT SURVEY DATA SHEET

Project: STROTSKLOOF TUNNEL — Location: Dut 2

Distance along Tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
0,0 X	D	St	BF	62	064	1			4	U	2	M	
0,55X				45	112	1			3	U	1	M	SET NO.1
2,20X				30	133	1			4	U	2	M	Dip - 30°
2,70X				25	125	2	Cl & Sd	B3	5	P	2	E	Direction - 124°
1,70				25	070	3	Cl & Gr	B3	3	P	2	D	
1,15X				70	215	1			4	U	0	M	SET NO.2
1,80X				65	211	1	St		3	U	2	D	Dip - 70°
1,35				70	203	1	"	"	3	U	0	K	Direction - 212°
0,80				80	220	1			4	P	0	M	
0,80X				50	255	1	St		3	P	0	M	SET NO.3
1,25X				40	255	1			3	P	0	M	Dip - 45°
													Direction - 245°
1,10X				75	270	1			3	P	0	M	
2,15X				70	264	1			2	P	0	M	JOINT SET
2,65X				50	280	1			2	P	0	M	Dip - 50°
0,70				65	278	1			3	P	0	M	Direction - 278°
0,90				80	280	1			2	P	0	M	
0,35				90	170	1			4	U	1	D	
0,80				90	001	1			3	P	0	D	JOINT SET
0,90				80	358	2	Cl & Sd	B3	4	U	1	D	Dip - 80°
1,20				85	345	1	St		4	U	0	M	Direction - 000°
1,0				70	006	1	St		3	U	1	M	
0,1				75	254	1			3	P	0	D	
0,95				10	052	1	St		3	U	0	D	Random
1,05				80	048	2	Cl & Sd	B3	5	U	1	D	
0,2				65	172	1	St		4	U	2	M	
0,5				90	227	1	St		4	U	2	M	

X Survey in E-W direction.

JOINT SURVEY DATA SHEET

Project: ANDESITE/LAVA TUNNEL Location: Tnl 3

Distance along tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Weathering	Continuity	Water	REMARKS
1,05	S2	S3	CJ	60	050	1			3	F	0	N	SET NO.1
													Dip - 90°
													Direction - 065°
0,4 X			S3	90	200	2	Cl & Sd	S2	1	U	2	D	SET NO.2
1,0 X				90	212	2	Cl & Sd	S2	3	U	0	N	Dip - 90°
0,8				85	200	2	Cl & Sd	S2	2	U	2	N	Direction - 204°
1,30			CJ	70	088	1	St		3	U	1	N	
0,1 X				60	225	2	Cl & Sd	S2	3	U	2	N	POSS. SET NO.3
1,4				85	222	2	Cl & Sd	S2	2	P	2	D	Dip - 75°
0,8 X				85	255	2	Cl & Sd	S2	1	P	2	N	Direction - 227°
0,5 X				85	255	2	Cl & Sd	S2	3	U	0	N	
1,1 X				45	182	1			3	P	0	D	Random
2,2 X				90	112	1			3	P	D	N	
0,23				50	128	1	St		3	U	0	N	
1,40 X				70	001	1	St		3	P	1	N	
1,80				85	340	1			3	U	2	N	

X Survey in E-W direction.

JOINT SURVEY DATA SHEET

Project: DITOTSKLOOF TUNNEL PILOT BORE **Location:** Put 4

Distance along tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
0,10	G	R2		60	320	1	C1	S2	1	P	O	K	SET NO.1
1,80				61	328	2	C1	S1	1	U	2	N	Dip - 80°
0,60				84	307	2	SR	R1	3	U	2	N	Direction - 317°
1,85				70	217	1			2	P	O	N	SET NO.2
2,14				55	214	1			2	P	2	N	Dip - 82°
1,00				84	032	2	C1	S2	1	P	1	N	Direction - 217°
1,50				74	085	2	C1	S2	2	P	O	N	SET NO.3
1,50				70	090	1			1	U	O	N	Dip - 70°
													Direction - 088°
2,15				70	067	1			2	P	4	N	Random
3,00				69	115	3	C1	S3	1	P	1	N	

NB:
Data obtained from
VKS Records

JOINT SURVEY DATA SHEET

Project: WITWATERSKLOOF TUNNEL PILOT BORE **Location:** Dat 5

Distance along tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Wideness	Continuity	Water	REMARKS
0.0	G	R4		75	217	?	Ca	S3	1	U	2	N	SET 20.2
0.78				73	200	1			1	U	0	N	Dip - 80°-90°
1.30				88	027	1			3	P	2	N	Direction - 205°
1.40				83	305	1			3	P	1	N	SET 20.3
2.0				55	300	1			2	P	2	N	Dip - 70°-85°
2.2				75	115	1			3	P	2	N	Direction - 305°
2.7				76	125	2	Ca	S2	1	P	1	N	
0.30				72	275	1			3	P	2	N	SET 20.34
0.60				79	260	1			3	P	1	N	Dip - 75°
													Direction - 265°
0.10				69	050	3	Ca	S3	1	P	2	N	SET 30.4
2.0				90	092	1			3	P	2	N	Dip - 40°
													Direction - 051°
1.80				70	010	1			3	P	1	N	Random

NB:
1) Data obtained from V.K.S. Rec. dia.

2) Data too scattered for significance.

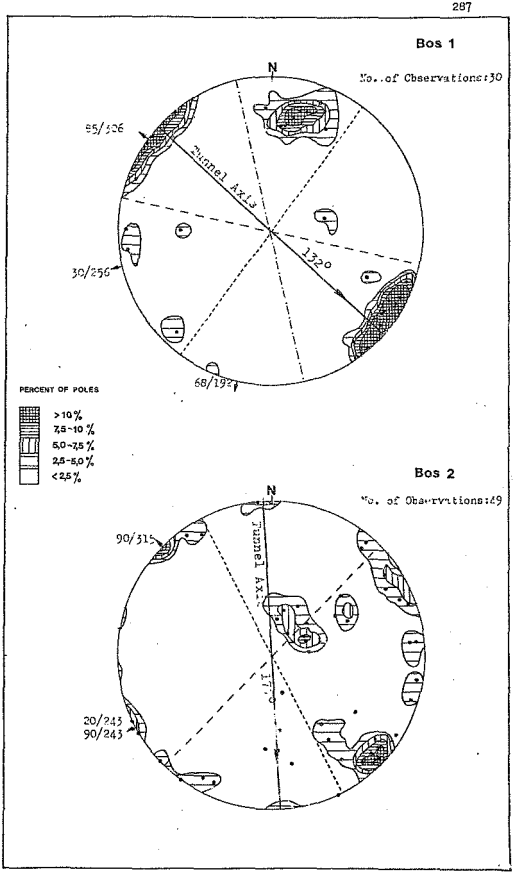
JOINT SURVEY DATA SHEET

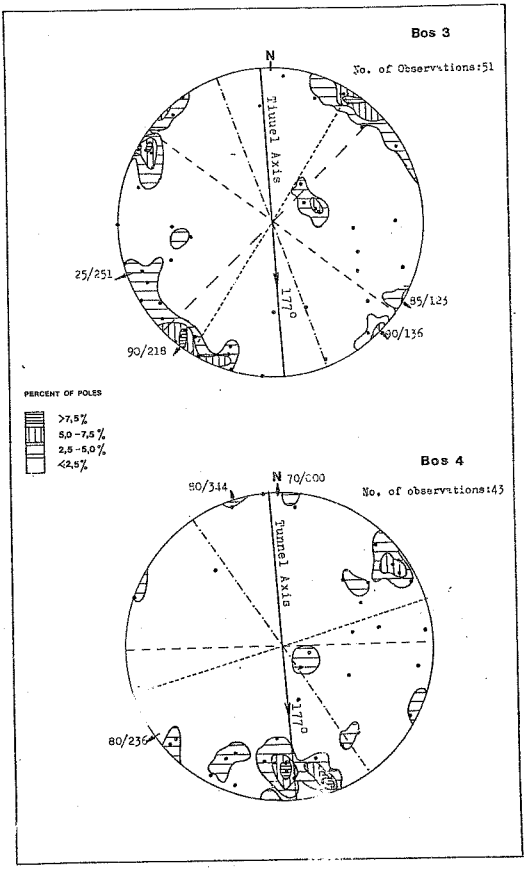
Project: ERTOITSKLOOF TUNNEL **Location:** Cut 6

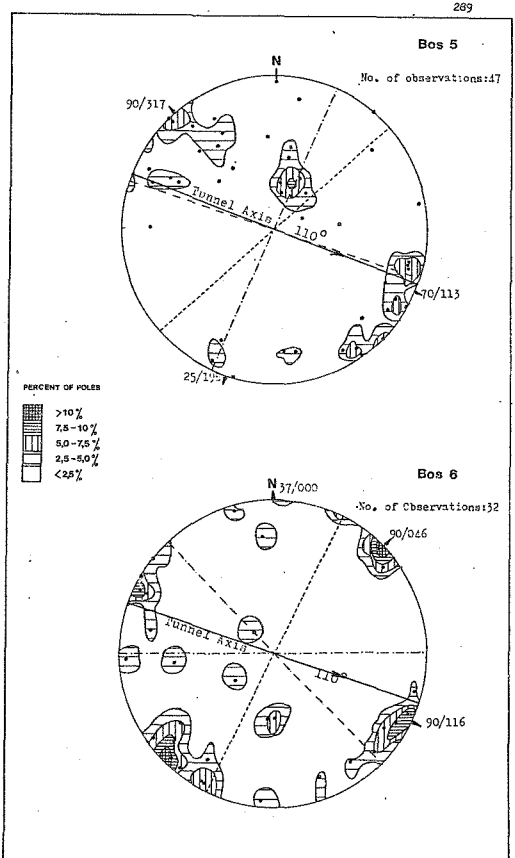
Distance Along Tape (m)	ROCK: Type	ROCK: Hardness	ROCK: Structure	ORIENTATION: Dip	ORIENTATION: Dip Direction	GOUGE: Thickness	GOUGE: Type	GOUGE: Hardness	SURFACE: Roughness	SURFACE: Waviness	Continuity	Water	REMARKS
1,5	D	M		75	025	2	Ca & Cn	H2	4	U	2	F	SET NO.1
2,0				75	035	2	Ca & Cn	H2	3	U	2	N	Dip - 70°
2,3				71	035	2	Ca & Cn	H2	3	U	0	K	Direction - 029°
1,0				90	025	2	Ca	H2	5	U	2	K	
6,9				80	282	1			3	U	1	F	
7,5				85	281	1			4	P	2	N	
7,9				90	300	1			5	U	1	N	SET NO.2
0,4				70	115	1			4	P	1	K	Dip - 50°
0,7				85	294	1			4	P	2	N	Direction - 111°
3,0				90	105	1			3	P	2	N	
2,8				85	290	1			5	U	0	N	
													SET NO.3
1,7				40	300	1			3	P	2	K	Dip - 45°
													Direction - 303°
1,4				80	197	2	Ca	H2	3	U	2	N	
3,0				80	150	2	Ca & Cn	H2	3	U	2	N	
4,8				85	115	1			3	P	2	N	
5,4				85	305	1			3	P	2	N	Random
6,7				85	122	1			2	U	1	N	
4,2				80	230	2	Cn & Qz	H2	3	U	1	N	
8,5				85	035	2	Ca & Cn	H2	3	U	2	F	
0,5				70	071	1			4	U	0	N	
5,6				75	345	2	Ca & Cn	H2	3	U	0	N	
7,2				60	145	1			4	P	0	N	
7,4				74	345	1			3	P	0	N	
5,8-8,5				10	1?0	1			3	U	2	N	Subhorizontal joint 0,7 m. above tape.

APPENDIX III C

STEREOPLOTS OF THE JOINT SURVEY DATA


Bos 1
No. of Observations:30
N
85/326
Tunnel Axis
132°
30/256
68/132
PERCENT OF POLES
>10 %
7,5-10 %
5,0-7,5 %
2,5-5,0 %
<2,5 %
Bos 2
No. of Observations:49
N
90/315
Tunnel Axis
117°
20/243
90/243


Bos 3
N
No. of Observations:51
Tunnel Axis
177°
25/251
85/123
90/136
90/218
PERCENT OF POLES
>7,5%
5,0 - 7,5 %
2,5 - 5,0 %
<2,5%
Bos 4
N 70/000
90/344
No. of observations:43
Tunnel Axis
177°
80/236


Bos 5
N
No. of observations:47
90/317
Tunnel Axis
110°
70/113
25/115
PERCENT OF POLES
>10 %
7,5 - 10 %
5,0 - 7,5 %
2,5 - 5,0 %
<2,5 %
Bos 6
N 37/000
No. of Observations:32
90/026
Tunnel Axis
110°
90/116

Author Cameron-clarke Ian Stuart
Name of thesis An Evaluation Of The Reliability Of Borehole Core Data In Engineering Rock Mass Classification Systems. 1978

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