CHAPTER IV

ROCKMASS CLASSIFICATION SCHEMES AND GEOMETRY

In a classification system empirical relations between rock mass properties and the behaviour of the rock mass in relation to a particular engineering application, are combined to give a method of designing engineering structures in or on a rock mass (Hack, 1997).

Theoretically, a proper description or geotechnical calculation to determine the behaviour of a rock mass should include all properties in a rock mass including all spatial variations of the properties. This is unrealistic and is also not possible without disassembling the rock mass. Therefore the standard procedure is to divide a rock mass into homogeneous geotechnical units. In practice, such homogeneity is seldom found and material and discontinuity properties vary within the unit.

The Tunneling Quality Index (Q-System) proposed by Barton et al (1974) and the Rock Mass Rating (RMR) classification system proposed by Bieniawski (1973) are the two most commonly used rock mass classification systems. Both are designed to assess factors which influence the stability of an underground excavation

Both methods incorporate geological, geometric and design/engineering parameters in arriving at a quantitative value of their rock mass quality. The similarities between RMR and Q stem from the use of identical, or very similar, parameters in calculating the final rock mass quality rating. The differences between the two systems lie in the different weightings given to similar parameters and in the use of distinct parameters in one or the other scheme.

The RMR uses compressive strength directly while Q only considers strength as it relates to in situ stress in competent rock. Both schemes deal with the geology and geometry of the rock mass, but in slightly different ways. Both consider ground water, and both include some component of rock material strength.

Some estimate of orientation can be incorporated into Q using a guideline presented by Barton et al (1974) : 'the parameters Jr and Ja should relate to the surface most likely to allow failure to initiate'. The greatest difference between the two systems is the lack of a stress parameter in the RMR system.

However various other researchers in the field of rock mechanics have either proposed an alternative system or modify an existing rock mass classification scheme. The literature survey that follows concludes that all classification systems are based on a specific set of parameters or problems, and that their applicability is therefore limited.

The problems experienced by the various researchers do not necessarily advocate that his specific system can be applied generally. Below are examples of such classifications systems, checklists and pure observation ability in the field. It was necessary to conduct research into the various rockmass classification systems to confirm a suitable systems which could relate to the Impala problem. The following will assist the reader to objectively understand the complexity of the Impala problem, which lies in the quest to design the optimum support system for a typical shallow platinum hard rock off-reef tunnel.

The detailed analysis of the rockmass surrounding a tunnel developed 20 years ago, which until today has not been supported, and a tunnel which currently is being developed, which is extremely unstable due to rockmass structure rather than poor rockmass will further fine tune the focus on the problem.

4.1 Identification of keyblocks shapes and sizes

Stability problems in blocky, jointed rock are often associated with gravity falls of blocks from the hangingwall and sidewalls. Rock stresses at relatively shallow depths are generally too low to have a significant effect upon this failure process which is controlled by the three dimensional geometry of the excavation and the rock structure.

A simple identification of keyblocks in tunnels is intended to assess the long-term stability criteria for specific tunnels. It is also virtually impossible to secure a design without the necessary investigation.

4.1.1 Geological Discontinuities in rock

A discontinuity may be defined as a boundary within the rock mass which marks a change in the mass properties and thereby a change in engineering characteristics. This definition includes features such as lithological boundaries, bedding planes, joints and faults.

Joints can usually be seen on an exposed rock surface. They appear as approximately parallel or randomly orientated cracks separated by as little as several centimeters or by as much as 10m or more. One set of joints commonly forms parallel to bedding and there are usually at least two other sets in other directions (SIMRAC, 1994).

Igneous rocks, as in the case study, have irregular jointing systems with three or more sets. Rocks that have been deformed by folding often contain roughly parallel seams of sheared and crushed rock produced by interlayer slip or minor fault development. Such features are sometimes observed around the potholes of the Bushveld Complex rocks. Faults that may off-set all other crossing structures may also occur in the rock surrounding the tunnel. Thus there is a full range of weaknesses in rock masses with a statistical distribution of spacings and orientations at all scales.

The importance of weaknesses stems from the special properties that such features superimpose on rock. Basically, the rockmass becomes weaker and sometimes highly anisotropic, which create a variety of potential problems. Figure 4.1 shows how blocks might fall from the hangingwall of a tunnel due to intersecting joints.

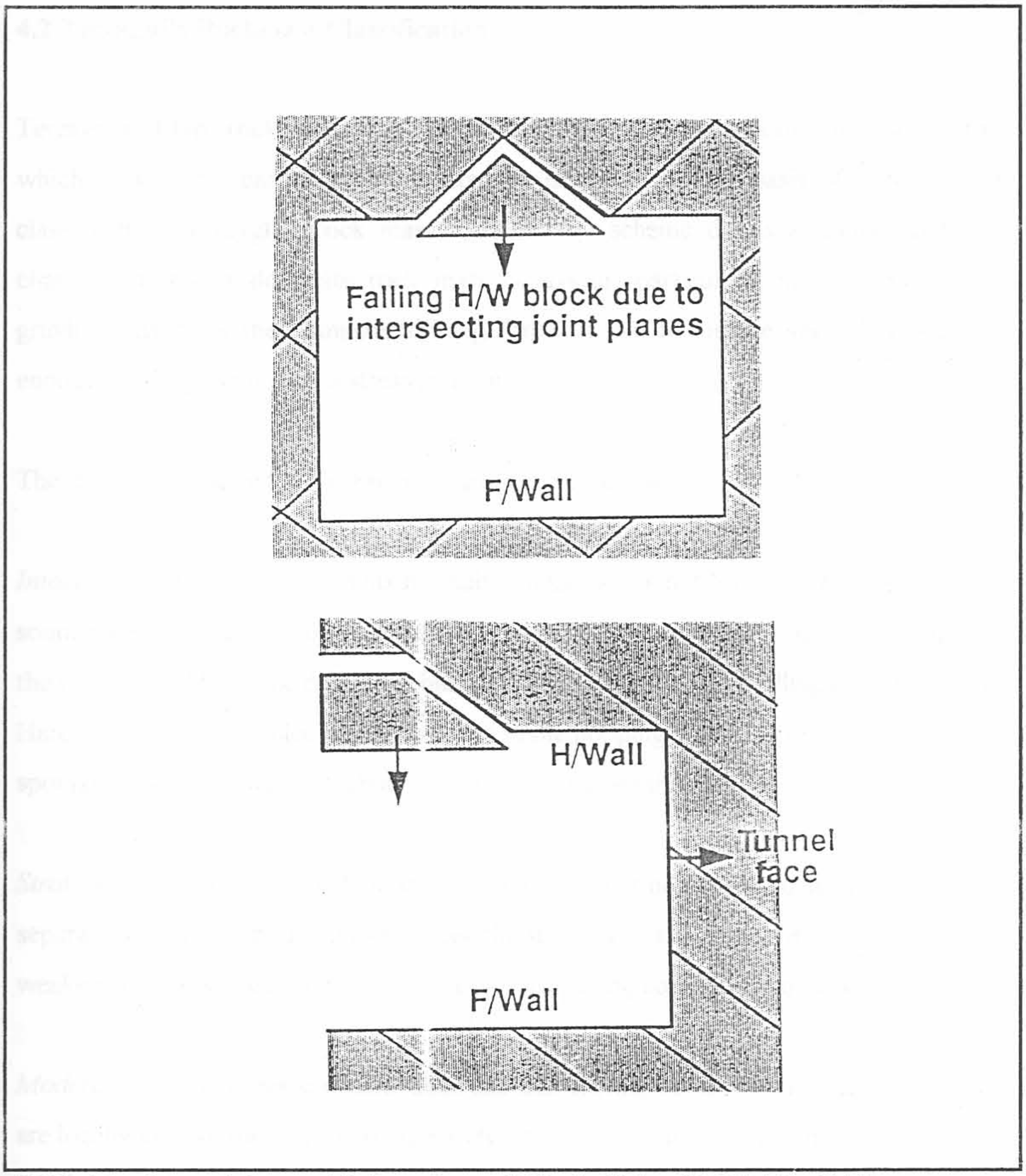


FIG. 4.1 - Potential unstable hangingwall block (After SIMRAC, 1994)

4.2 Terzaghi's Rockmass Classification

Terzaghi's (1946) rock mass classification is applied to the design of tunnel support in which rock loads, carried by steel sets, are estimated on the basis of a descriptive classification. Terzaghi's rock mass classification scheme draws attention to those characteristics that dominate rock mass behaviour, particularly in situations where gravity constitutes the dominant driving force. It can further be applied to shallow enough workings that in-situ stress is not important.

The factors considered in Terzaghi's system are as follows :-

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

Stratified rock consists of individual strata with little or no resistance against separation along the boundaries between the strata. The strata may or may not be weakened by traversed 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.

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 lateral support.

Crushed but chemically intact rock has the character of crusher run. If most or all 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.

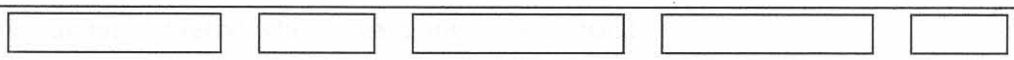
Squeezing rock slowly advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic particles of micaceous minerals or clay minerals with a low swelling capacity.

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

Terzaghi's system is quick and easy to apply but it leaves a lot of factors out that would be important in mining. It is used to design support using sets, which are generally not used in the mining industry. The system is therefore not considered appropriate for platinum mining because of the limited detailed analysis of the jointed rockmass.

4.3 Rock Quality Designation Index (RQD)

Deere (1964) developed the RQD index to provide a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact core pieces longer than 100mm in the total length of core. The core should be at least 50mm in diameter and should be drilled with a double barrel diamond drilling equipment. Figure 4.2 illustrates how the rock quality designation index is determined.



L = 38cm L = 17cm L = 20cm L = 15cm L = 9cm

Total L = 99cm

$$RQD = \frac{\text{Sum length of core pieces} > 10\text{cm}}{\text{Total length of core run}} \times 100\%$$

$$= 90\text{cm} / 99\text{cm} \times 100\%$$

$$= 91\%$$

FIG. 4.2 - Procedure for measurement and calculation of RQD (After Deere, 1964)

Palmstrom ***(1982)*** suggested that, when no core is available but discontinuity traces are visible in surface exposures or exploration audits, the RQD may be estimated from the number of discontinuities per unit volume.

$$RQD = 115 - 3.3 * Jv \qquad (4.1)$$

Where *Jv* is the sum of the number of joints per unit length for all joint (discontinuity) sets known as the volumetric joint count.

RQD is intended to represent the rock mass quality in situ. Thus the most important use of RQD is as a component of the RMR and Q rock mass classifications which is discussed later.

- The RQD support criteria system has limitations in areas where the joints contain thin clay fillings or weathered material. Such a case might occur in near surface rock where weathering or seepage has produced clay which reduces the frictional resistance along joint boundaries. This would result in unstable rock although the joints may be widely spaced and the RQD high.
- The RQD does not take direct account of other factors such as joint orientation which must influence the behaviour of a rock mass around an underground opening.
- It does not provide an adequate indication of the range of behaviour patterns which may be encountered when excavating underground.

4.4 Rock Structure Rating (RSR)

The RSR (Wickham et al., 1972) is a quantitative method for describing the quality of a rock mass and for selecting appropriate support on the basis of a Rock Structure Rating classification. Most of the case histories, used in the development of this system, were for relatively small tunnels supported by means of steel sets, although historically this system was the first to make reference to shotcrete support.

The RSR system is the first system to demonstrate the logic involved in developing a quasi-quantitative rock mass classification system.

The following are the parameters considered in the RSR :-

i. *Parameter A, Geology :* General appraisal of geological structure on the basis of (see Table 4.1) :

a) Rock type origin (igneous, metamorphic,, sedimentary).

b) Rock hardness (hard, medium, soft, decomposed).

c) Geologic structure (massive, slightly faulted/folded, moderately faulted/folded, intensely faulted/folded)

TABLE 4.1 - Rock Structure Rating - Parameter A - General Area Geology

	Basic Rock Type							
	Hard	Medium	Soft	Decomposed	Geological Structure			
					Massive	Slightly Folded or Faulted	Moderately Folded or Faulted	Intensively Folded or Faulted
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

ii. *Parameter B, Geometry :* Effect of discontinuity pattern with respect to the direction of the tunnel drive on the basis of (see Table 4.2) :

a) Joint spacing.

b) Joint orientation (strike and dip).

c) Direction of tunnel drive.

TABLE 4.2 - Rock Structure Rating - Parameter B - Joint Pattern, direction of Drive

Average Joint Spacing	Strike perpendicular to axis					Strike parallel to axis		
	Direction of drive					Direction of drive		
	Both	With Dip		Against Dip		Either Direction		
	Dip of Prominent Joints [a]					Dip of Prominent Joints		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
1. Very closely jointed, <2 in	9	11	13	10	12	9	9	7
2. Closely jointed, 2-6 in	13	16	19	15	17	14	14	11
3. Moderately jointed, 6-12 in	23	24	28	19	22	32	32	19
4. Moderate to blocky, 1-2 ft	30	32	36	25	28	30	38	24
5. Blocky to massive, 2-4 ft	36	38	40	33	35	36	34	28
6. Massive, >4 ft	40	43	45	37	40	40	38	34

iii) Parameter C : Effect of ground water inflow and joint condition on the basis of (see Table 4.3) :

a) Overall rock mass quality on the basis of A and B combined.

b) Joint condition (good, fair, poor).

c) Amount of water inflow (in liters per minute per 280m of tunnel).

TABLE 4.3 - Rock Structure Rating - Parameter C - Ground Water, Joint Condition

Anticipated Water inflow Gpm/1000 ft of tunnel	Sum of parameters A + B					
	13-44			45-75		
	Joint Condition [b]					
	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	22	7	21	16	12
Heavy, > 1000 gpm	10	8	6	18	14	10

[a] Dip: Flat: 0-20°; dipping: 20-50°; and vertical: 50-90°

[b] Joint condition: good = tight or cemented; fair = slightly weathered or altered; poor = severely weathered, altered or open

Each of the components listed above provide a numerical value of RSR :

$$RSR = A + B + C \tag{4.2}$$

The RSR classification used Imperial units. For a 7,32m diameter tunnel with a RSR value of 62, the predicted support would be 2 inches (50,8mm) of shotcrete and 1 inch (25,4mm) diameter rockbolts spaced at 5 foot (1,524m) center's (See figure 4.3).

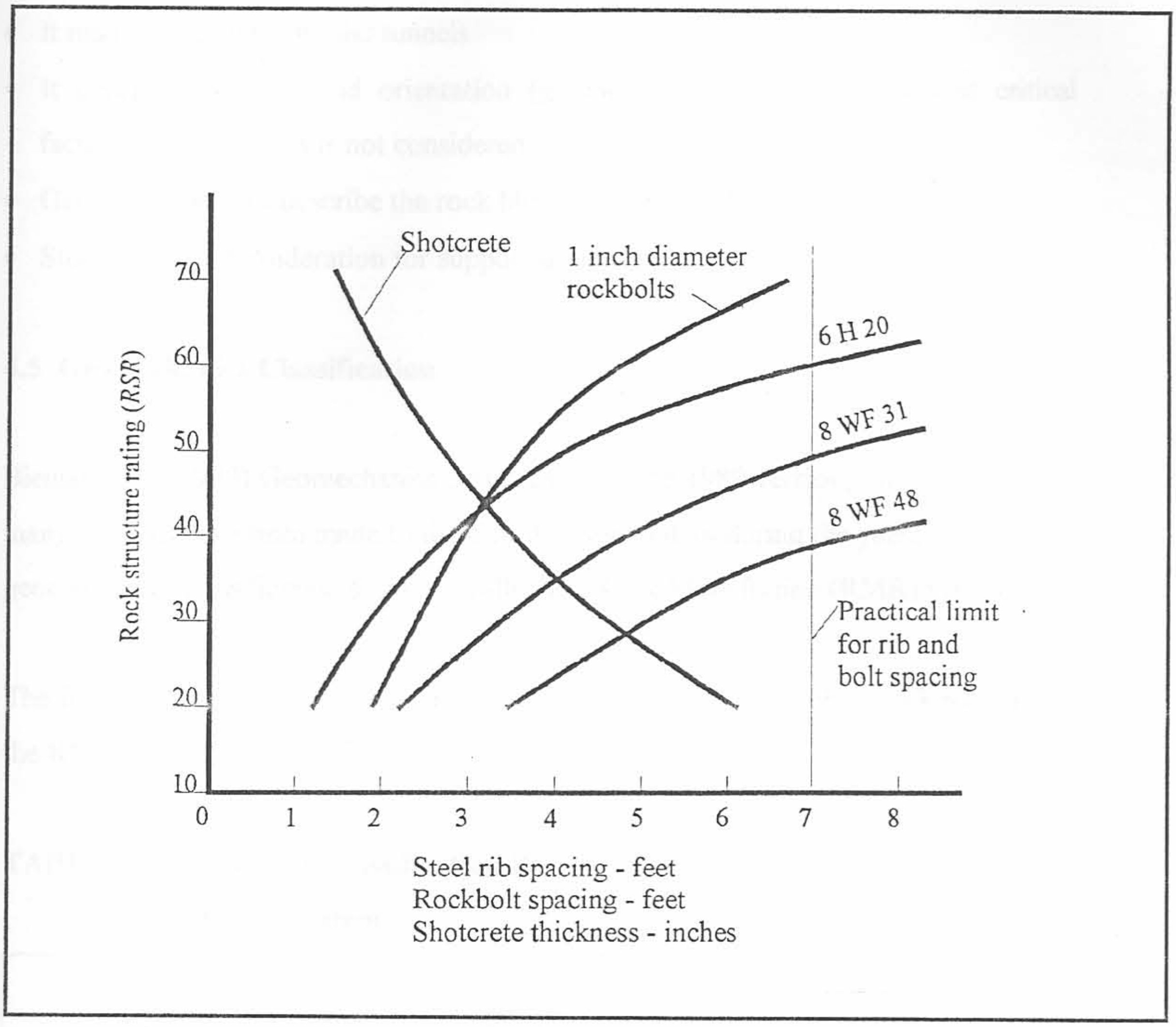


FIG. 4.3 - RSR Support estimates for 24ft (7.3m) Diameter circular tunnel (After Wickham et al, 1972)

The 62 RSR value consists of a hard metamorphic rock which is slightly folded or faulted and moderately jointed, with joints striking perpendicular to the axis of the tunnel, dipping at between 20° and 50°. A moderate water inflow of between 912 and 4560 liters per minute.

The RSR classification system is not widely in use today. However Wickham et al's work played a significant role in the development of other classification systems. This rating system however is not considered for the following reasons :-

- It mainly considers circular tunnels
- It describes jointing and orientation (geometry) however the important critical factor joint roughness is not considered
- Generally does not describe the rock block volume involved
- Steel sets as a consideration for support design

4.5 Geomechanics Classification

Bieniawski's (1989) Geomechanics classification is the 1989 version, for many changes have been made to the classification system during the years. This geomechanics classification system is called the Rock Mass Rating (RMR) system.

The following 6 parameters shown in Table 4.4 are used to classify a rock mass using the RMR system.

TABLE 4.4 - Rockmass Classification Parameters for the Rock Mass Rating (RMR) system

1. Uniaxial compressive strength of rock material.
2. Rock Quality Designation (RQD).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities.

In applying this classification system, the rock mass is divided into a number of structural regions and each region is classified separately. The boundaries of the structural regions usually coincide with a major structural feature such as a fault or with a change in rock type.

Guidelines for the selection of support in tunnels in rock for which the value of RMR has been determined and are given in Table 4.6 after using Table 4.5.

TABLE 4.5 - Rock Mass Rating (After Bieniawski, 1989)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

Parameter			Range of values						
1	Strength Of Intact rock Material	Point-load Strength index	>10 MPa	4-10 MPa	2-4 Mpa	1-2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 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 discontinuities		>2m	0.6-2m	200-600mm	60-600mm	<60mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Un-weathered wall rock	Slightly rough surfaces Separation <1mm Slightly weathered walls	Slightly rough surfaces Separation <1mm Highly weathered walls	Slickensided surfaces Or Gouge <5mm thick Or Separation 1-5mm Continuous	Soft gouge >5mm thick or Separation > 5mm Continuous		
	Rating		30	25	20	10	0		
5	Ground Water	Inflow per 10m tunnel length (l/m)	None	<10	10-25	25-125	>125		
		(Joint water press)/(Major principal σ)	0	<0.1	0.1-0.2	0.2-0.5	>0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very Favourably	Favourably	Fair	Unfavourably	Very Unfavourably
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 – 81	80-60	60-41	40-21	<21
Class number	I	II	III	IV	V
Description	Very good rock	Good Rock	Fair Rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand up time	20 yrs for 15m span	1 year for 10m span	1 week for 5m span	10hrs for 2.5m span	30 min for 1m span
Cohesion of rock mass (kPa)	>400	300-400	200-300	100-200	<100
Friction angle of rock mass (deg)	>45	35-45	25-35	15-25	<15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence) Rating	<1m 6	1-3m 4	3-10m 2	10-20m 1	>20m 0
Separation (aperture) Rating	None 6	<0.1mm 5	0.1-1.0mm	1-5mm 1	>5mm 0
Roughness Rating	None 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5mm 4	Hard filling >5mm 2	Soft filling < 5mm 2	Soft filling >5mm 0
Weathering Ratings	Unweathered 6	Slightly Weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELING **

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip – Dip 45-90°	Drive with dip – Dip 20-45°	Dip 45-90°	Dip 20-45°
Very favourable	Favourable	Very favourable	Fair
Drive against dip – Dip 45-90°	Drive against dip – Dip 20-45°	Dip 0-20 – Irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such case use A.4 directly.

** Modified after Wickham et al (1972)

TABLE 4.6 - Guidelines for excavating and support of 10m span tunnels in accordance with RMR system (After Bieniawski, 1989)

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

Cummings et al (1982) and Kendorski et al (1983) have also modified Bieniawski's RMR classification to produce the MBR (modified basic RMR) system for mining. This system was developed for block caving operations in the USA.

It involves the use of different ratings for the original parameters used to determine the value of RMR and the subsequent adjustment of the resulting MBR value to allow for blasting damage, induced stresses, structural features, distance from the cave front and size of the caving block. Support recommendations are for isolated or development drifts as well as for the final support of intersections and drifts.

The above guidelines (Table 4.5) have been published for a 10m span horseshoe shaped tunnel, constructed using drill and blast methods, in a rockmass subjected to a vertical stress <25MPa (equivalent to a depth below surface of 900m). Tables 4.5 and 4.6 have not had a major revision since 1989.

The overall Rock Mass Rating is obtained by adding the values of the ratings determined for the individual parameters. The RMR value may be adjusted for the influence of discontinuity orientation by applying the corrections.

Limitations to the system include the following :-

- Data mainly obtained from civil engineering excavations in sedimentary rocks in S.A.
- Intact rock strength cannot generally be determined with a 10m interval in a tunnel section, this is very impracticable and costly. Drill core of a tunnel section is not always available. There are large variations in rockmass strength in the Bushveld Complex which have not being clearly defined yet
- The system lacks a stress parameter
- Support considerations are limited to a 10m excavation span

4.6 Modifications to RMR for mining

Bieniawski's (1978) Rock Mass Rating (RMR) system was originally based upon case histories drawn from civil engineering. Consequently, the mining industry tended to regard the classification as somewhat conservative and several modifications have been proposed in order to make the classification more relevant to mining applications.

Laubscher (1977) modified Bieniawki's geomechanics classification on the basis of experience gained in a number of chrysotile asbestos mines in Africa. Laubscher and Taylor (1976) and Laubscher and Page (1990) have described a Modified Rock Mass Rating system for mining. This MRMR takes the basic RMR value, as defined by Bieniawski, and adjusts it to account for in situ and induced stresses, stress changes and the effects of blasting and weathering.

In using Laubscher's MRMR system it should be borne in mind that many of the case histories upon which it is based are derived from caving operations. Originally, block caving in asbestos mines in Africa formed the basis for the modifications, but subsequently, other case histories from around the world have been added to the database.

The classification, set out in Table 4.7, uses the same five classification parameters as Bieniawski's scheme but involves differences in detail. Each of the five classes is divided into subclasses, A and B, new ranges and ratings for intact rock strength (IRS in Table 4.7) are used, and the joint spacing and condition of joint parameters are evaluated differently (Brady & Brown, 1985).

The only discontinuities (joints) included in the assessment of RMR are those having trace lengths greater than one excavation diameter or 3m, and those having trace lengths of less than 3m that are intersected by other discontinuities to define blocks of rock. True spacings of the three most closely spaced joint sets present in the rock mass are used in conjunction with Figure 4.4 to obtain a joint spacing rating on a scale of 0 to 30. The way the in which the joint condition rating is influenced by a range of factors are set out in Table 4.7. Before the basic rating for the rock mass is applied, it is adjusted to take account of weathering, field and induced stresses, changes in stress due to mining operations, orientations of blocks with exposed bases and blasting effects (Brady & Brown, 1985).

Limitations to the system include the following :-

- The data is mainly obtained from civil engineering excavations in sedimentary rocks in S.A. and mainly being modified to assess block caving operation
- The intact rock strength cannot generally be determined with a 10m interval in a tunnel section, this is very impracticable and costly. Drill core of a tunnel section not always available. There are large variations in rockmass strength in the Bushveld Complex which have not being clearly defined yet

TABLE 4.7 - Modified geomechanics classification scheme (After Laubscher, 1977)

class	**1**		**2**		**3**		**4**		**5**		
rating	**100-81**		**80-61**		**60-41**		**40-21**		**20-0**		
description	**Very good**		**good**		**fair**		**poor**		**Very poor**		
subclasses	**A**	**B**	**A**	**B**	**A**	**B**	**A**	**B**	**A**	**B**	
item											
1. RQD %	100-91	90-76	75-66	65-56	55-46	45-36	35-26	25-16	15-6	5-0	
Rating	20	18	15	13	11	9	7	5	3	0	
2. IRS, MPa	141-136	135-126	125-111	110-96	95-81	80-66	65-51	50-36	35-21	20-6	5-0
Rating	10	9	8	7	6	5	4	3	2	1	0
3. Joint spacing											
Rating	30..0										
4. condition of joint	45°..................... ....5°										
Rating	30 0										
	Inflow per 10m Length or joint water pressure		0		25l/min			25-125l/min		125l/min	
			0		0.0-0.2			0.2-0.5		0.5	
5. Groundwater	Major principal stress Or										
description	Completely dry		Completely dry		moist only			moderate pressure		severe problems	
rating	10		10		7			4		0	

TABLE 4.8 - Assessment of joint condition - adjustments as combined percentages of total possible rating of 30 (After Laubscher, 1977)

Parameter	Description	Percentage adjustment
Joint expression (large scale)	Wavy uni-directional	90-99
	Curved	80-89
	straight	70-79
Joint expression (small scale)	striated	85-99
	smooth	60-84
	polished	50-59
Alteration zone	Softer than wall rock	70-99
	Coarse hard-sheared	90-99
	Fine hard-sheared	80-89
	Coarse soft-sheared	70-79
	Fine soft-sheared	50-69
	Gouge thickness < irregularities	35-49
	Gouge thickness > irregularities	12-23
	Flowing material > irregularities	0-11

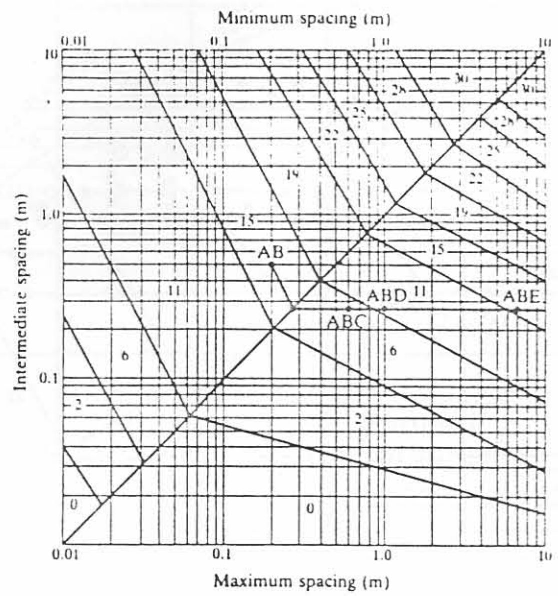


FIG. 4.4 - Joint spacing ratings for multi-joint systems (after Laubscher, 1977)

4.7 Classifications Involving Stand-up Time

Stini (1950) proposed a rock mass classification and discussed many of the adverse conditions which can be encountered in tunneling. He emphasized the importance of structural defects in the rock mass and stressed the need to avoid tunneling parallel to the strike of steeply dipping discontinuities. While both Terzaghi and Stini had discussed time-dependent instability in tunnels, it was Lauffer (1958) who proposed that the stand-up time for an unsupported span is related to the quality of the rock mass in which the span is excavated (Hoek & Brown, 1980).

In a tunnel, the unsupported span is defined as the span of the tunnel or the distance between the face and the nearest support, if this is greater than the tunnel span. The significance of the stand-up time concept is that an increase in the span of the tunnel leads to a significant reduction in the time available for the installation of support (See Figure 4.5 and 4.6).

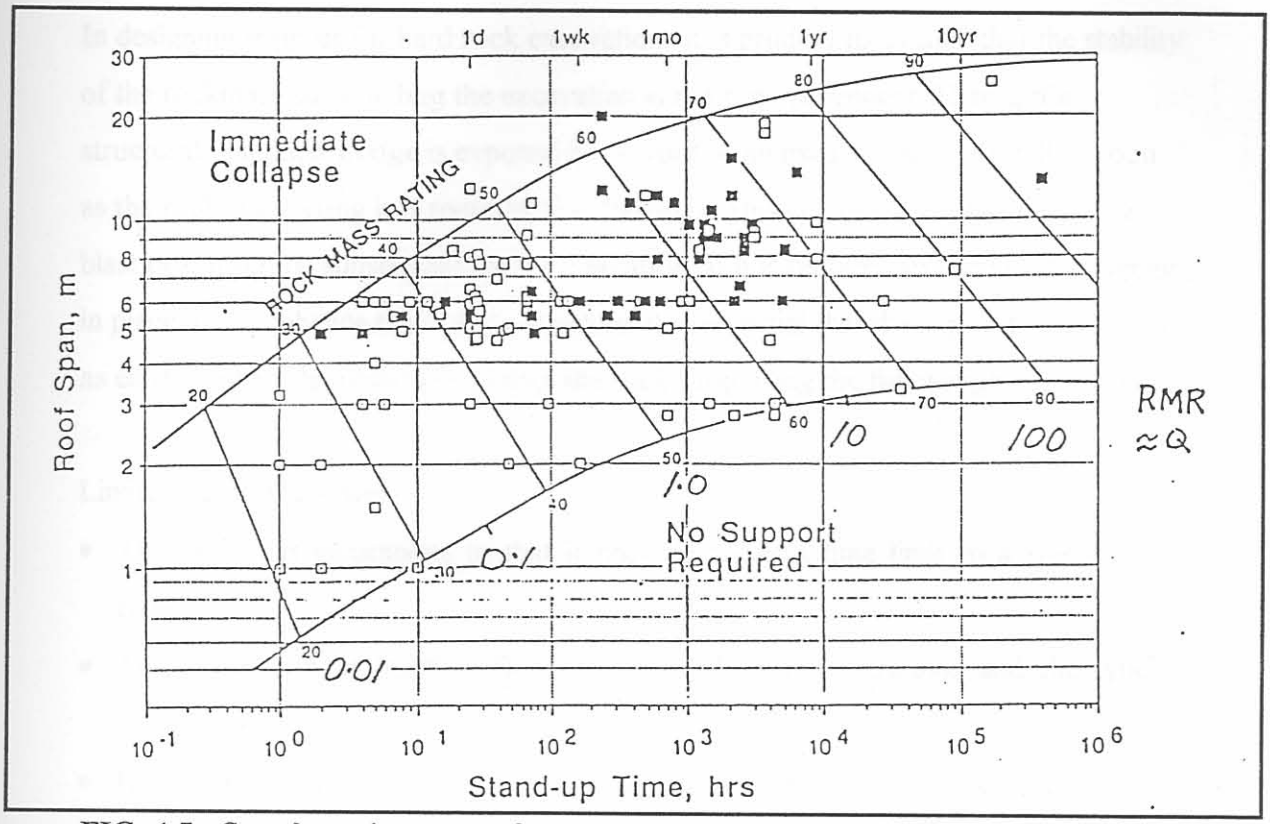


FIG. 4.5 - Stand-up time vs roof span compared to rock quality, RMR & Q value.

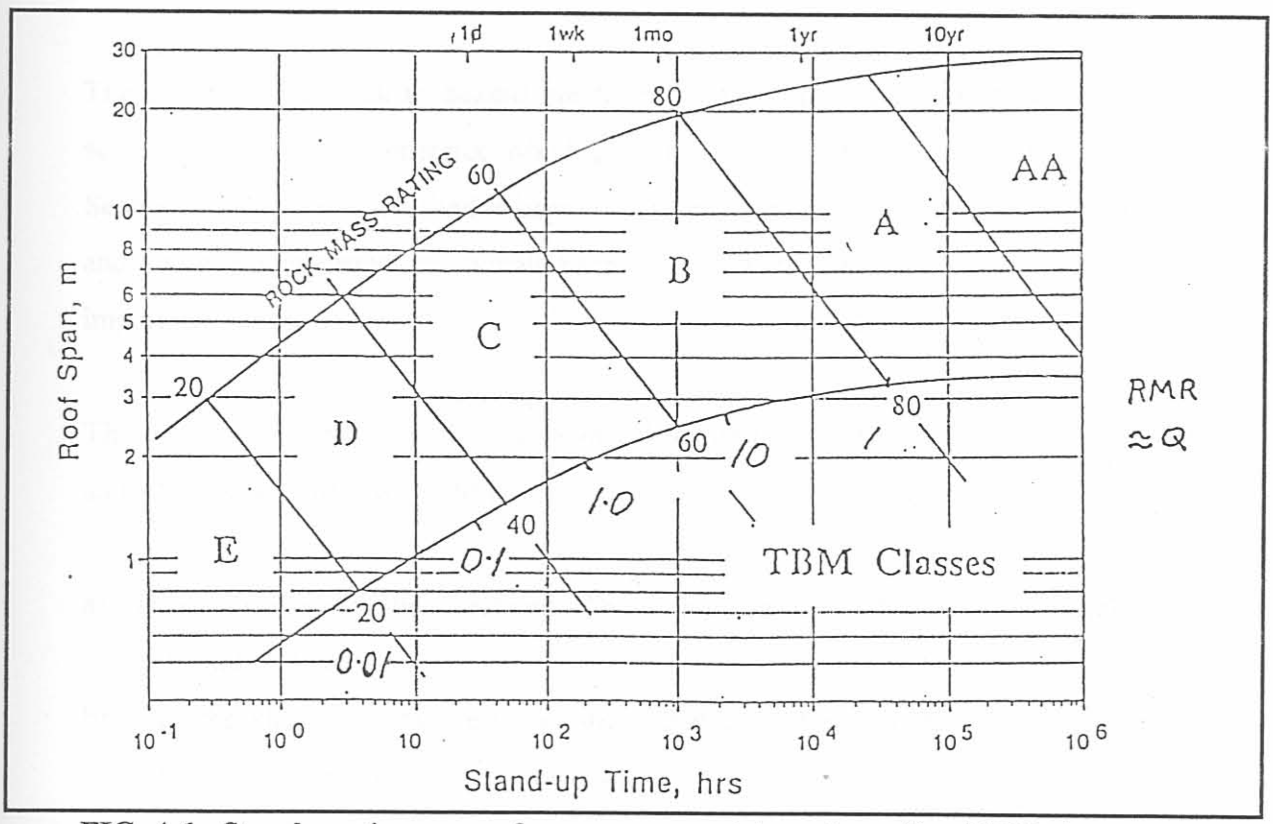


FIG. 4.6 - Stand-up time vs roof span compared to rock quality, TBM classes

In designing support for hard rock excavations it is prudent to assume that the stability of the rockmass surrounding the excavation is not time dependent. Hence, if a structurally defined wedge is exposed in the roof of an excavation, it will fall as soon as the rock supporting it is removed (i.e. barring). This can occur at the time of the blast or during the subsequent scaling operation. If it is required to keep such a wedge in place, or to enhance the margin of safety, it is essential that the support be installed as early as possible, preferably before the rock supporting the full wedge is removed.

Limitations to the system :

- The system is incomplete in that it only describes a time limit to a specific rock class.
- The above can be highly influenced by the blasting operations and the type of explosives used.
- Little consideration is given to the rockmass instability due to rock structure.

4.8 Checklist Methodology

The checklist approach to hazard identification takes place in two phases. Firstly, a series of questions is generated which pertain to a given hazard or group of hazards. Secondly, a consensus method is used to determine individual scores for each question and thereby generating an overall hazard score with which to assess the relative importance of the hazard.

There are a number of limitations to any checklist method that should be recognized and addressed (SIMRAC, 1998) :-

a) There are likely to be omissions. These should become fewer as the checklist is used and updated;
b) The checklist is insensitive to situations that are subject to change and may, after some time, contain irrelevant questions;

c) Perhaps the greatest disadvantage with a checklist is that it tends to put "blinkers" onto the user who becomes disinclined to look beyond checklist items for hazardous situations

Despite not completely satisfying all of the requirements for hazard identification, the checklist approach does provide an appropriate methodology particularly if limited rock engineering resources are available. The methodology is also useful as an initial approach to hazard identification and risk assessment.

The checklist methodology has three immediate benefits (SIMRAC, 1998) :-

i) It allows a hazard-based comparison or two or more activities to be carried out using a simple, arbitrary, but consistent, scoring method
ii) It allows an early identification and ranking of hazards that are relevant to the activity.
iii) It provides information on the nature of incidents which, potentially, can arise from hazard together with their possible cause and consequences.

4.9 Rockwall condition factor

Wiseman (1979) originally proposed an application of a system of evaluating the conditions of tunnels in a variety of conditions in all 20km of tunnel that was surveyed and analysed. The name of the classification system was given with the 1st edition of the Guide to methods of Ameliorating the Hazards of Rockfalls and Rockbursts (COMRO, 1988). The Rockwall condition factor (RCF) is given below :

$$RCF = (3\sigma_1 - \sigma_3) / F.\ \sigma_c \qquad (4.3)$$

where σ_1 and σ_3 are the major and minor principal stresses within the plane of the excavation cross section; and F is a factor to represent the down grading of σc (the uniaxial compressive strength) for the representative rock mass condition and excavation size. The formulation of the RCF is based on a simple comparison of the maximum induced tangential stress of an assumed circular excavation to the estimated rockmass strength. The empirical relationship between the rockwall conditions factor (RCF) and recommended support systems is based on extensive field studies of Witwatersrand gold mine tunnels which generally have dimensions of 3m by 3m. In general it was found that for RCF < 0.7, good conditions prevailed with minimum support requirements ; for $0.7 <$ RCF < 1.4, average conditions prevailed with moderate support systems requirements ; and for RCF > 1.4, poor ground conditions prevailed with special support requirements.

Empirical relationships have been derived between the RCF and the potentially unstable rock mass thickness for competent rock masses ($F = 1$) due to fracturing. It should be noted that this depth represents the potential unstable block height and will be less than the total depth of fracturing. These guidelines indicate that a RCF = 0.7, the anticipated thickness of unstable rock mass to be supported is approximately 0.7 x the radius of the excavation, and at RCF = 1.4 this thickness is approximately 1.2 x the radius. Under conditions of seismic loading, the increased extent of instability due to the transient dynamic stresses must be considered in the support design.

The above value ranges of the RCF criterion may not apply in Bushveld complex mines where the rock mass is igneous as opposed to the brittle quartzite's encountered in the gold mines (Jager & Ryder, 1999).

4.10 Rock Tunneling Quality Index, Q

Barton et al (1980) of the Norwegian Geotechnical Institute proposed a Tunneling Quality Index (Q) for the determination of rock mass characteristics and tunnel support requirements. The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is defined by :

$$Q = (RQD/Jn) * (Jr/Ja) * (Jw/SRF) \quad (4.4)$$

where RQD is the Rock Quality designation
Jn is the joint set number
Jr is the joint roughness number
Ja is the joint alteration number
Jw is the joint water reduction factor
SRF is the stress reduction factor

The category breakdown of the Q-System is shown in Table 4.9. The first quotient (RQD/Jn), representing the structure of the rockmass, is a crude measure of the block or particle size. The second quotient (Jr/Ja) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favor of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favorable to tunnel stability. When rock joints have thin clay mineral coatings and fillings, the strength is reduced significantly. The rock wall contact, after small shear displacements have occurred, may be a very important factor for preserving the excavation from ultimate failure. Where no rock wall exists, the conditions are extremely unfavourable to tunnel stability. The third quotient (Jw/SRF) consists of two stress parameters. SRF is a measure of :

a) loosening load in the case of an excavation through shear zones and clay bearing rock b) rock stress in competent rock and c) squeezing loads in plastic incompetent rocks.

The above can be seen as a total stress parameter. Jw is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. The above is a complicated empirical factor describing the active stress.

TABLE 4.9 - Classification of individual parameters used in the Tunneling Quality Index Q (After Barton et al 1974)

DESCRIPTION	VALUE	NOTES
1. ROCK QUALITY DESIGNATION	*RQD*	
A. Very poor	0 - 25	1. Where *RQD* is reported or measured as $\leq$ 10 (including 0), a nominal value of 10 is used to evaluate *Q*.
B. Poor	25 - 50	
C. Fair	50 - 75	
D. Good	75 - 90	2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate.
E. Excellent	90 - 100	
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use ($3.0 \times J_n$)
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	2. For portals use ($2.0 \times J_n$)
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact		
b. Rock wall contact before 10 cm shear		
A. Discontinuous joints	4	
B. Rough and irregular, undulating	3	
C. Smooth undulating	2	
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.
E. Rough or irregular, planar	1.5	
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	2. $J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided that the lineations are oriented for minimum strength.
c. No rock wall contact when sheared		
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	
J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)	
4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)
a. Rock wall contact		
A. Tightly healed, hard, non-softening, impermeable filling	0.75	1. Values of ϕr, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25 - 35
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25 - 30
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	20 - 25
E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less)	4.0	8 - 16

TABLE 4.9 - (cont'd) Classification of individual parameters used in the Tunneling Quality Index Q (After Barton et al 1974)

DESCRIPTION	VALUE	NOTES	
4, JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
b. Rock wall contact before 10 cm shear			
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24	
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16	
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0 - 12.0	6 - 12	
c. No rock wall contact when sheared			
K. Zones or bands of disintegrated or crushed	6.0		
L. rock and clay (see G, H and J for clay	8.0		
M. conditions)	8.0 - 12.0	6 - 24	
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0		
O. Thick continuous zones or bands of clay	10.0 - 13.0		
P. & R. (see G.H and J for clay conditions)	6.0 - 24.0		
5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	
6. STRESS REDUCTION FACTOR	*SRF*		
a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated			
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)	10.0	1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation	
B. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth < 50 m)	5.0		
C. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth > 50 m)	2.5		
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5		
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)	5.0		
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)	2.5		
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)	5.0		

TABLE 4.9 - (cont'd) Classification of individual parameters used in the Tunneling Quality Index Q (After Barton et al 1974)

DESCRIPTION	VALUE			NOTES
6. STRESS REDUCTION FACTOR			*SRF*	
b. Competent rock, rock stress problems				
	σ_c/σ_1	σ_t/σ_1		2. For strongly anisotropic virgin stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
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 to wall stability)	10 - 5	0.66 - 0.33	0.5 - 2	
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10 - 20	3. Few case records available where depth of crown below surface is less than span width. Suggest *SRF* increase from 2.5 to 5 for such cases (see H).
c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				
N. 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	

ADDITIONAL NOTES ON THE USE OF THESE TABLES

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

1. When borehole core is unavailable, *RQD* can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to *RQD* for the case of clay free rock masses: $RQD = 115 - 3.3\,J_v$ (approx.), where J_v = total number of joints per m³ ($0 < RQD < 100$ for $35 > J_v > 4.5$).
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of J_r/J_a is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating *Q*. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor *SRF* appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
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 the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

Thus the rock tunneling quality - Q - can now be considered to be a function of only three parameters which are crude measures of :

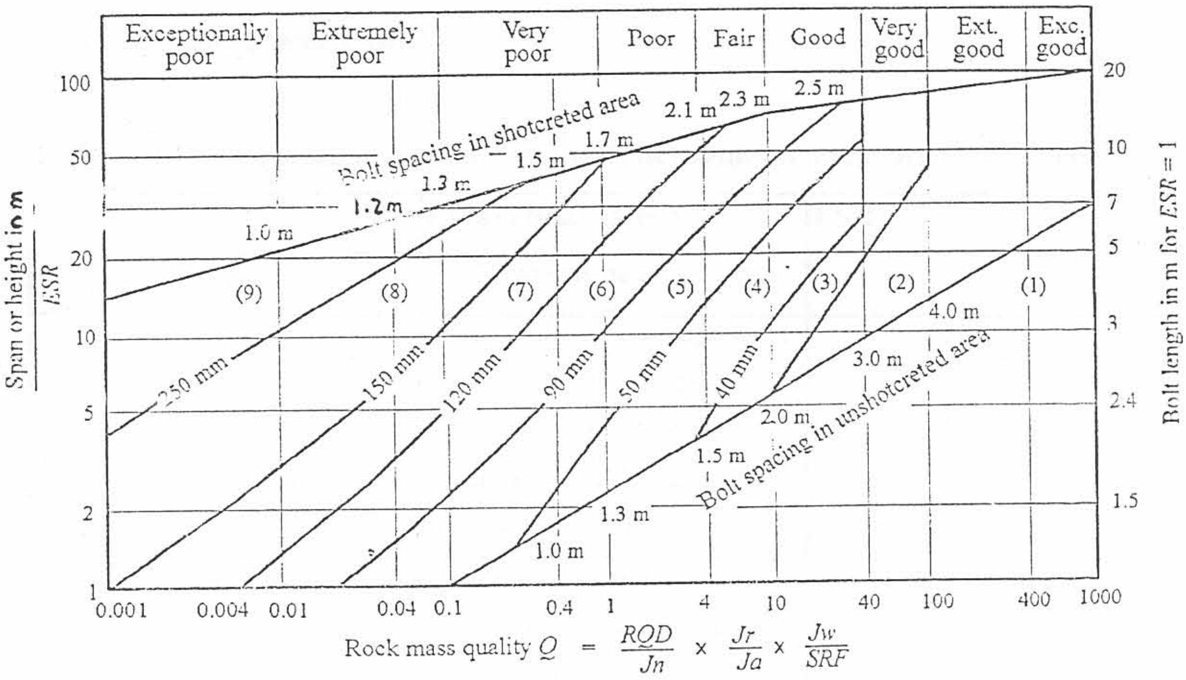
1. Block size (RQD/Jn)
2. Inter-block shear strength (Jr/Ja)
3. Active stress (Jw/SRF)

There are several other parameters that could be added to improve the accuracy of the classification system. One could be the joint orientation. Many case records include the necessary information on structural orientation in relation to excavation axis, but it was not found to be the important general parameter that might be expected.

Many underground observations on Impala will substantiate the above statement. In some tunnels jointing can be found to be parallel with excavation length. However the hangingwall in most cases were found to be intact. It must further be said that the characteristic of the jointing determined the integrity of the hangingwall.

The parameters Jn, Jr and Ja appear to play a more important role than orientation, because the number of joint sets determines the degree of freedom for block movement (if any) and the frictional and dilatational characteristics can vary more than the down-dip gravitational component of unfavourably orientated joints.

Figure 4.7 shows how interrelated the value of the index Q to the stability and support requirements of underground excavations are.



REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting with 40-100 mm unreinforced shotcrete
5) Fibre reinforced shotcrete, 50 - 90 mm, and bolting
6) Fibre reinforced shotcrete, 90 - 120 mm, and bolting
7) Fibre reinforced shotcrete, 120 - 150 mm, and bolting
8) Fibre reinforced shotcrete, > 150 mm, with reinforced ribs of shotcrete and bolting
9) Cast concrete lining

FIG. 4.7 - Estimated support categories based on the Tunneling Quality Index - Q - (After Grimstad & Barton, 1993)

Barton et al (1980) defined an additional parameter which they called the Equivalent Dimensions, De, of the excavation (Figure 18). This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the Excavation Support Ratio,

Thus

$$De = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio ESR}} \qquad (4.5)$$

The value of equivalent support ration (ESR) is related to the intended use of the excavation and to the degree of security that is demanded of the support system installed to maintain the stability of the excavation. Barton et al (1980) suggest the following values (Table 4.10) :

TABLE 4.10 - Equivalent Support Ratio (ESR) values to excavation category

	Excavation Category	ESR
A	Temporary mine opening	3-5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portal intersections	1
E	Underground nuclear power stations, railway stations, sports and public facilities, factories	0.8

The equivalent dimension, De, plotted against the value of Q, is used to define a number of support categories in a chart (Figure 4.7) published in the original paper by Barton et al (1980). This chart has recently been updated by Grimstad and Barton (1993) to reflect the increasing use of steel fibre shotcrete in underground excavation support.

Loset (1992) suggests that, for rocks with 4<Q<30, blasting damage will result in the creation of new 'joints' with a consequent local reduction in the value of Q for the rock surrounding the excavation. He suggests that this can be accounted for by reducing the RQD value for the blast-damaged zone. Barton et al (1980) provide additional information on rockbolt length, maximum unsupported spans and roof support pressures from the excavation width B and the Excavation Support Ratio ESR:

The length (L) of rockbolts can be estimated from the excavation width B and the excavation support ratio ESR:

$$L = \frac{2+0.15B}{ESR} \qquad (4.6)$$

Where,

B = excavation width

ESR = Equivalent Support Ratio

The maximum unsupported span is given by the following relationship :

$$2 * ESR * Q^{0.4} \qquad (4.7)$$

Based upon analysis of case records, Grimstad and Barton (1993) suggest that the relationship between the value of Q and the permanent roof support pressure P_{roof} is estimated from :

$$P_{roof} = \frac{2 * \sqrt{J_n} * Q^{-1/3}}{3 * J_r} \qquad (4.8)$$

When evaluating all the rockmass classification schemes listed above the Q-System fits the profile to the Impala problem best and is a reliable and simple rockmass classification scheme. However as with many other classification schemes the Q-System relates to the civil engineering discipline. However it is opted to take the Q-system and validate it for use on Impala Mine.