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TUNNEL CONSTRUCTION GUIDELINES
07 - GUIDANCE NOTE ON THE USE OF TUNNEL AND CAVERN ROCK MASS CLASSIFICATIONS IN HONG KONG

1. INTRODUCTION

Rock mass classifications are an empirical, indirect method of determining rock mass parameters for engineering applications. They can be applied to the design and construction of sub-surface excavations (shafts, tunnels and caverns), slopes and foundations. The main aim is to provide quantitative data for design, enable better engineering judgment and provide more effective project communication. The major advantages and disadvantages are summarised in **Table 1**.

Advantage	Disadvantage
Few parameters are required and they are applicable to a range of situations.	Considerable experience and knowledge needed to assess engineering parameters and support strategies appropriate for site specific conditions.
They provide a direct and rapid guidance for support design.	Other influences affecting the engineering performance may not be included; such as the relative orientation, spacing and persistence of discontinuities and rock type.

Table 1: Advantages and Disadvantages

The main classifications for sub-surface excavations, which have the widest range of engineering applications and are established in Hong Kong (HK), include the:

- Rock Mass Rating (RMR) System (Bieniawski, 1989)
- Norwegian Geotechnical Institute (NGI) Q System (NGI, 2013).

This Tunnel Construction Guidance (TCG) document outlines the use of rock mass classification for tunnel and cavern design and construction, emphasising its use in HK. Attention is given to the suitable adoption of these classifications with an aim to ensure best practice. As the "Q" system has been specified in many HK contracts and significant experience has been gained in its use, attention is given to this classification.

2. HISTORY

The first recorded division of a rock mass into observable parameters used for tunnel support was provided by Ritter, 1879. Classifications using multiple parameters determined from project case studies were then developed with different project aims, such as the shorter term support for mines. As a result an emphasis on different parameters was given in successive classifications. Selected earlier developments are summarised below (Rocscience, 2014):

Terzaghi (1946)
The rock loading assessed in this system was carried out using steel sets. The parameters used to derive loads were from descriptive classifications determined from the rock mass behaviour.

Lauffer (1958)
Stand-up time was provided for unsupported tunnel lengths, defined by the distance from the excavation front to the nearest support, based on rock mass quality.

Wickham et al (1972)
This classification increased the range of parameters, using geological, geometrical, groundwater inflow and joint conditions, to assess support using steel sets and more commonly used shotcrete.

3. ROCK MASS CLASSIFICATIONS IN HONG KONG

3.1 General

RMR (Bieniawski, 1973); Q system (Barton et al, 1974)
As these classifications were based on numerous international and local case history examples, and have a wide range of uses and applications, they have traditionally been used in HK. The Q system is preferred due to the following limitations of the RMR:

- Application to rock masses with one dominant discontinuity set, such as sedimentary rocks, which is generally not encountered in HK;
- The support assessment is based on 10m-wide horseshoe-shaped tunnels, with vertical stresses below 25 MPa;
- Deficiencies for design spans greater than 15m;
- Modern support measures, such as fibre reinforced shotcrete, are not included;
- the RMR "may be useful for weak rock but is little use for hard rock common in HK" (GEO, 1992).

Other classifications used in HK and a summary of the input parameters (GEO, 2007) include:
Geological Strength Index (GSI) & Hoek/Brown Strength Criterion – (Hoek et al; 2002 and Hoek & Brown; 1997)

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The GSI is determined from the degree of rock mass interlock and the discontinuity condition. It can be used to determine the Unconfined Compressive Strength (UCS) in combination with the Deformation Modulus, and UCS and excavation disturbance factor when used with the Hoek / Brown strength Criterion (m & v).

This classification has been used for input to numerical analyses to assess temporary support and excavation strategies in HK in poorer ground conditions.

IMS System (McFeat Smith, 1986 and GEO, 2007)

Used for design of underground excavation support systems and Tunnel Boring Machine (TBM) performance. Input parameters are weathering grade, discontinuity spacing, orientation and water inflow.

3.2 RMR SYSTEM

The RMR has values from 0 to 100 assessed from the UCS of intact rock; Rock Quality Designation (RQD); groundwater condition and the discontinuity spacing and orientation relative to the excavation. It is used to determine underground excavation support design, rock mass deformability (GEO, 2006) and stand-up time. More recently the stand-up time has been cross referenced to the Q system, see **Figure 1** (Barton & Bieniawski, 2008).

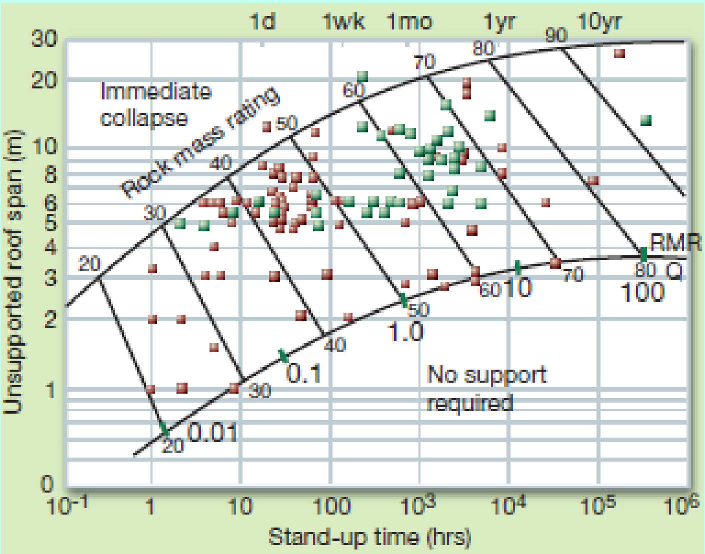

Figure 1: Stand up time, Q and RMR systems

Stand-up time

The stand-up time originated from Lauffer, 1958 and was further developed by Bieniawski, 1989 and Barton & Bieniawski, 2008. It can be estimated from the RMR and Q systems **(Figure 1)** from the "span" defined as the distance from the excavation face to the nearest support. Failure in highly stressed rock mass may occur following a period of stress readjustment around the excavation; this may increase support if a suitable period is allowed. Given the uncertainty of the stand-up period for the typical rock conditions encountered in HK it is prudent to install support after excavation has taken place.

3.3 Q SYSTEM

The most recent Q system update (NGI, 2013) has been based on over one thousand support installation case histories and has values ranging from 0.001 to 1000. Its main uses are for the design of underground excavation support, TBM performance and rock mass deformability (Barton, 2000). It is determined by 6 parameters, namely the RQD, the number of Joint sets (J_n), the Joint roughness (J_r), the Joint alteration (J_a), the Joint water reduction (J_w) and Stress Reduction Factor (SRF), which are combined to represent the:

- degree of jointing (or block size), RQD / J_n;
- Joint friction (inter-block shear strength), J_r / J_a; and
- Active stress (J_w / SRF)

The Q value is calculated using the following formula:

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

The support requirements are presented in **Figure 2**, and can also be cross referenced to RMR (Barton, 1995 and Bieniawski, 1989)

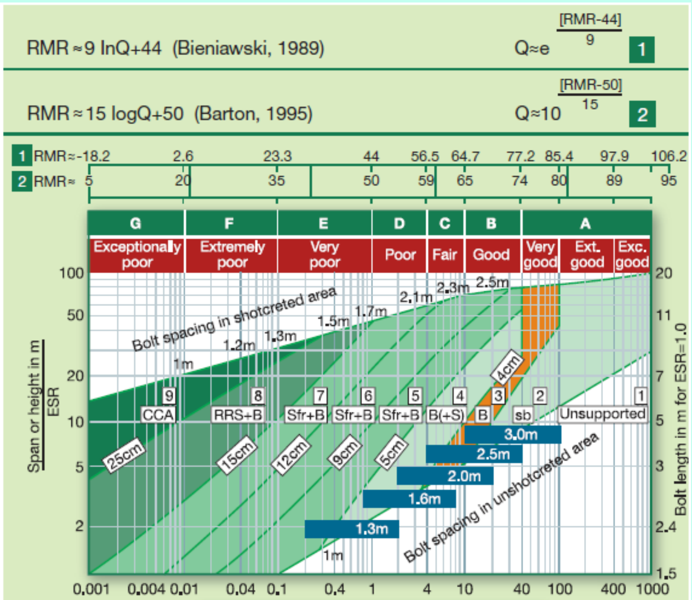

Figure 2: Support chart, Q and RMR systems

(Special thanks to Mr Sandy Mackay, ALYSJ JV, Doha; and Miss Ada Chan, Leighton Asia Ltd, for preparation of this guideline)

Equivalent "dimension"
The Equivalent Dimension, De (Barton et al, 1974), is defined by the "height" or "span" divided by the "Excavation Support Ratio" ("ESR") and is used to determine support in **Figure 2**. The ESR provides additional security to the support installation dependent upon the underground space usage.

Cavern and tunnel wall support
The support chart represents the "permanent" support requirements for a tunnel or cavern roof, termed "Q_r". The support required for the wall, termed "Q_w", can be modified dependent upon the quality of the rock condition encountered as summarised in **Table 2**:

Q (Q_r) value	Condition (Fig. 2)	Q_w
>10	Good or better	Q/5
0.1 to 10	Very poor – fair	Q/2.5
<0.1	Extremely poor or worse	Q

Table 2: Modification for Q wall

Portal and intersections
At portals and intersections within a cavern and tunnel the potential for the joint sets (J_n) to be exposed is increased and needs to be factored as follows:

- portal - J_n*2
- intersection – J_n*3

The distance of the support along the main excavation is half the diameter or span from the intersection (D/2).

Temporary support
Major excavation works can take years to construct often needing extended periods for completion. It is therefore prudent to adopt permanent conditions to asses support.

3.4 INTER-CHANGEABILITY — RMR/Q SYSTEMS

Relationships between the rock mass classifications have been provided **(Figures 1 & 2)**, with the Bieniawski, 1984 equation being preferred. Although the correlations have been widely used and are adequate for crude evaluation, inaccuracies of about 50% or more have been estimated (Palmström & Stille, 2010), resulting from fundamental differences between the systems, such as:

- Case histories, approaches used and application;
- Parameters used, such as UCS for RMR;
- Equations (Barton, 1995 and Bieniawski, 1989).

Suitable judgement is therefore needed for cross correlation and independent assessment recommended for each classification wherever possible.

4. THE Q SYSTEM PARAMETERS

Each Q value input parameter needs thorough understanding for the result to be meaningful. A review of the RQD, J_r, J_a, J_w and SRF is provided below.

4.1 ROCK QUALITY DESIGNATION

The RQD is the degree of fracturing in the rock (Deere, 1963). This is typically calculated from a rock core or excavation scanline using the following formula and a rock core assessment shown in **Figure 3**.

$$\text{RQD} = \frac{\text{Length of core pieces, or blocks,} > 100mm\ length}{\text{Total length of core run or scanline}}$$

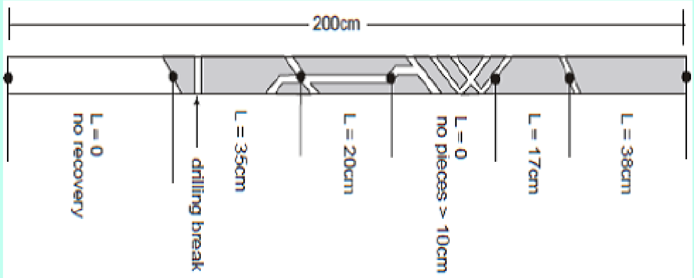

Figure 3: RQD estimated from a rock core

RQD estimates are approximate and are typically shown in 5% intervals, with the lower limit taken to be 10% for Q value estimation. Errors can occur from:

- Tunnel scale effects,
- Scanline and rock core orientation bias, and
- Access difficulties to the unsupported tunnel face;

Choi & Park, 2004, estimate the RQD accuracy measured from scanlines to be about 25%. Mitigation to improve this accuracy have been published as follows:

Hudson & Harrison, 1997 – Fracture Frequency
Use multiple scanlines to estimate the number of rock fractures per metre to estimate the RQD. The drawback is that the discontinuity orientation is not accounted.

Palmström, 2005 – Volumetric Joint Count (J_v)
The Jv, which is the average of the spacing for each observed joint set recorded and distinct joint sets

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identified usually by differences in orientation, aperture and infilling, identified through systematic geological mapping. An example is presented in **Figure 4**.

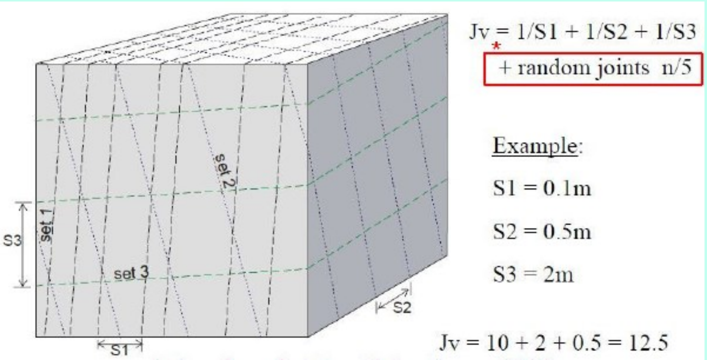


Figure 4: Estimation of J_v

The RQD is estimated from the following formulae, J_v ranges and references, which are broadly applicable to different block shapes, as summarized in **Table 3**.

Reference	block shape	RQD	J_v range
1974 original	Long, flat	115-3.3*J_v	4.5 - 35
2005 update	Cubical, bar	110-2.5*J_v	4 - 44

Table 3: J_v estimation

The accuracy of the RQD estimation using this technique increases with greater RQD values, typical of the more competent granite encountered in HK.

4.2 Joint SET NUMBER (J_n)

The J_n is a joint set rating. Examples are shown in **Figure 5**, which presents 1 joint set (J_n=2) and 2 to 3 joint sets (1 to 3) plus 1 random (R), which gives J_n estimates of 9 to 12.

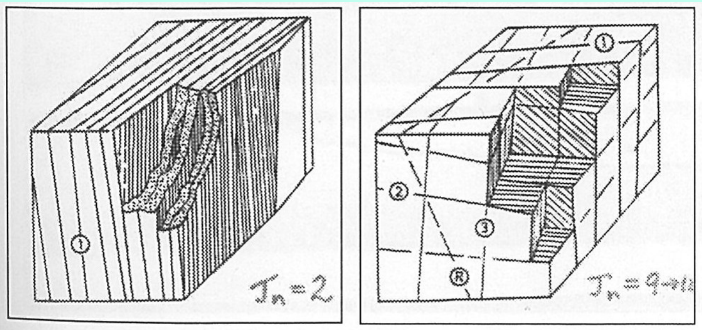


Figure 5: Estimation of J_n

Both the J_n and RQD are evaluated using all available exposures of the tunnel face, wall and, if possible, invert. For increased J_n values; 15 (4 or more joint sets, heavily jointed, "sugar cube") and 20 (crushed rock, earthlike), the behaviour is dependent on the stress levels. Estimation with SRF is therefore needed. As HK generally has low stress levels and wide joint spacing, "crushed rock" behaviour is not common.

4.3 JOINT ROUGHNESS (J_r) AND ALTERATION (J_a)

J_a and J_r provide the "frictional component" in the Q value formula and are determined from the "weakest significant joint set or infilled discontinuity". The shear strength (φ) along discontinuities relate to:

- Rock wall contact (a),
- Rock wall contact when sheared (b) and
- No wall contact when sheared (c).

The φ value °relates to J_r and J_a using:

$$\varphi = \tan - 1\frac{J_r}{J_a}$$

When there is no wall contact J_r is equal to unity and the φ value determined by the infill characteristics. When φ is determined from rock wall contact, either directly or after shear, it is determined by J_r using the Joint Roughness Coefficient (JRC) and Joint Wall Compressive Strength (JCS). These can relate to discontinuity field descriptions (Barton & Choubey, 1977). **Figures 6 to 8** show relationships between JCS, JRC, φ, in-situ conditions (σ & T_n), field descriptions, J_r and J_a.

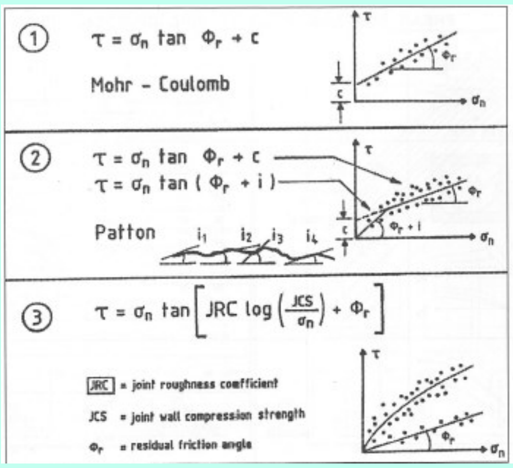
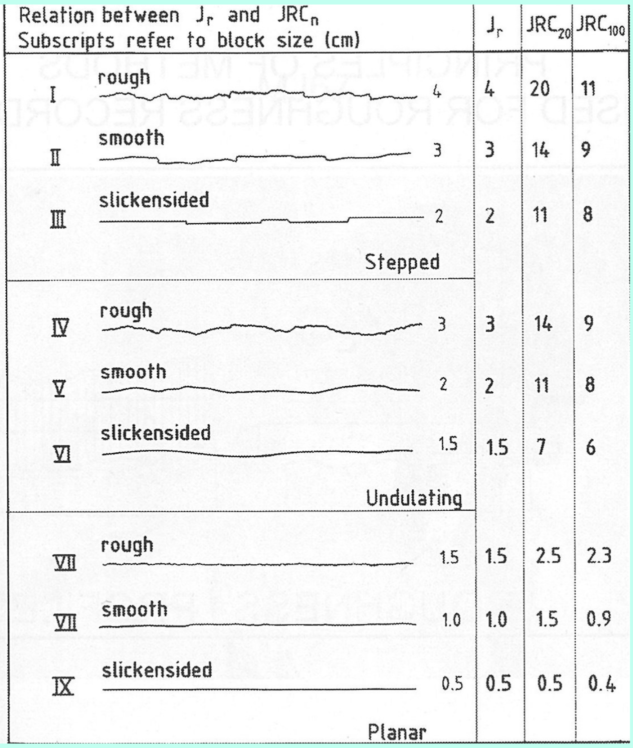


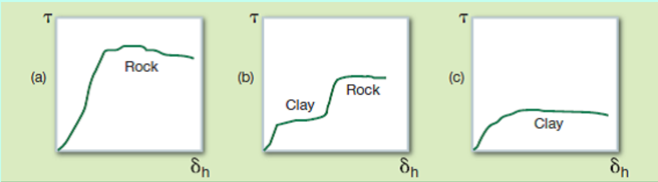
Figure 6: JRC, JCS and J_r against in-situ stress

(Special thanks to Mr Sandy Mackay, ALYSJ JV, Doha; and Miss Ada Chan, Leighton Asia Ltd, for preparation of this guideline)



Relation between J_r and JRC_n Subscripts refer to block size (cm)		J_r	JRC_{20}	JRC_{100}
I rough	4	4	20	11
II smooth	3	3	14	9
III slickensided	2	2	11	8
Stepped				
IV rough	3	3	14	9
V smooth	2	2	11	8
VI slickensided	1.5	1.5	7	6
Undulating				
VII rough	1.5	1.5	2.5	2.3
VIII smooth	1.0	1.0	1.5	0.9
IX slickensided	0.5	0.5	0.5	0.4
Planar				

Figure 7: JRC, JCS and J_r against field description



(a) Rock wall contact	Ja =	0.75	1.0	2	3	4		
(thin coatings)	Jr	tan⁻¹ (Jr/Ja)°						
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	Ja =	4	6	8	12			
(thin fillings)	Jr	tan⁻¹ (Jr/Ja)°						
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	Ja =	5	6	8	12	13	16	20
(thick fillings)	Jr	tan⁻¹ (Jr/Ja)°						
Nominal roughness of discontinuity rock walls	1.0	11.3°	9.5°	7.1°	4.8°	4.4°	3.6°	2.9°

Figure 8: Variability in ɸ value against J_r and J_a

4.4 JOINT WATER INFLOW (J_w)

The J_w is based on the inflow from discontinuities and is assessed using 6 components, ranging from dry to minor, local inflow (<5 litres / minute, J_w=1) to exceptionally high inflow or water pressure continuing without noticeable decay (>10kg/cm2, J_w = 0.1 – 0.05). Examples of J_w are presented in **Figures 9 and 10**.

Figure 9: J_w = 0.66

Figure 10: J_w = 0.5

Crude prediction of J_w can be made ahead of an excavation by relating the depth (0-5, 5-25, 25-250 and >250m) against J_w (1, 0.66, 0.5 and 0.33 respectively). This is provided that hydraulic connectivity, estimated from RQD/J_n, ranges from 0.5 to 25.

4.5 STRESS REDUCTION FACTOR (SRF)

The SRF is derived from 4 main components, including (A): "Weakness zones intersecting excavation", typically faulting affected by depth / stress; (B): Competent rock, rock stress problems". Stress / strength ratio; (C): Squeezing and (D): Swelling, with swelling and squeezing not typically encountered in Hong Kong. The SRF can be determined from stress related observations including depth, weak zone descriptions and in-situ stress measurements. Values typically range from 1, 2.5 and 5, and are often related to:

- (A): Single weakness zones containing clay or chemically disintegrated rock at excavation depths <50m, SRF = 5 and >50m, SRF = 2.5;
- (B): Low stress, near surface, open joints, SRF = 2.5, medium stress, favourable stress condition, SRF = 1

5. CONTRACT CONSIDERATION

Rock Mass Classifications are typically required for all contracts for the Government or the Mass Transit Railway Corporation Limited (MTRCL). The general conditions of contract, such as the Material and Workmanship (M&W) Specification (MTRCL, 2009), and Technical Guidance Note 25 (GEO, 2005) reference the use of the Q system (Barton & Grimstad, 1994).

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Many of the MTRCL and Government Particular Specifications (PS) supplement the Q system with a required numerical analysis, typically below Q values of 0.05 for single tunnels (9m outside diameter, O.D) and 0.13 for twin tracks (15m O.D).

Given the risks ensuring that the correct judgement is made for the excavation support, many specifications, such as those used for the MTRCL state that "a Geologist oversees input to the tunnel support design using the Q system to validate the engineering geological assumptions". The typical requirement for the personnel carrying out the assessment is stated below:

- MTRCL – degree in Geology, related subject and minimum 5 years geological mapping experience;
- Government – such as the Drainage Services Department (DSD), DC/2010/23, Stonecutters Island and Highways Department, HY/2010/15, Central Wanchai Bypass, also require the Geologist to be Chartered with the Geological Society of London.

To ensure the relevant engineering geological details are predicted ahead of the excavation the contract typically require ground models and as-built geological records to be produced at suitable excavation stages. MTRCL, 2009, requires the ground models and as built records to be produced for every 100m and 500m respectively.

6. PROJECT EXAMPLES IN HONG KONG

Examples of Q value estimation at different project stages and the effect of complicated geology, such as faulting, on the rock mass classification are provided. Given the similar dimensions and end use for the tunnels the Q value ranges were allocated as follows:

- <0.3 - Heavy support, ribs and forepoling;
- 0.3 – 4 - Light support, rock bolts and shotcrete, and
- >4 - No support.

6.1 ROUTE 8, SHATIN HEIGHTS TUNNEL

- This is a twin carriageway, 800m length with an 18m span tunnel, passing through the ShaTin Granite and influenced by fault zones, particularly the Tolo Channel Fault (Sewell et al, 2000, GEO, 2007). The excavation was carried out using drill and blast techniques and temporary support assessed using the Q system. A comparison of the Q values prior to and after (as-built) construction is shown in **Table 4**.

Q value range (%)	<0.3	0.3 - 4	>4
Pre-construction estimate	14.7	60.6	24.7
As-built estimate	7.5	43	49.5

Table 4: Sha Tin Heights, Q value comparison.

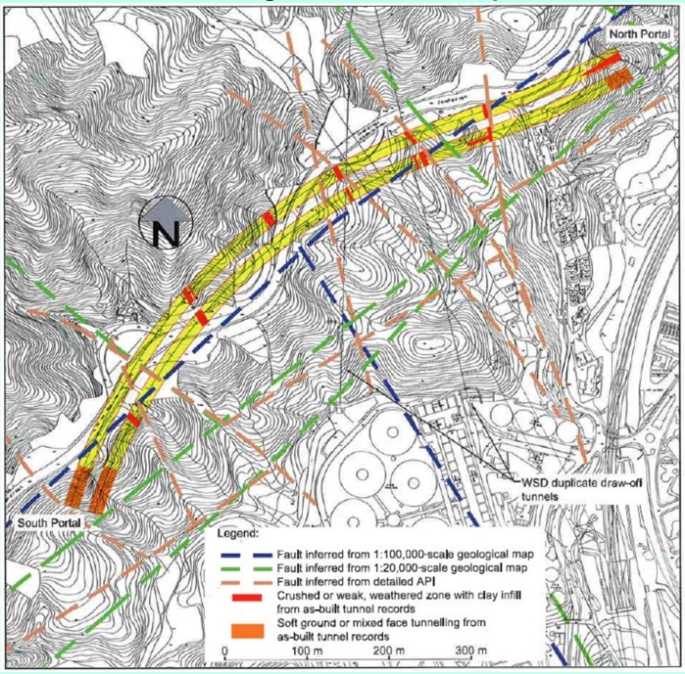

Figure 11: Geological Comparisons – Shatin Heights

The pre-construction estimates were considered conservative due to an over-estimation of the influence of the adverse geological features. This included the rock head being located at a lower elevation in the vicinity of the portals and the presence of faults. The SRF (Q) values were increased accordingly. These geological features were not encountered during excavation. See **Figure 11** for comparisons of the anticipated and as built geological conditions.

6.2 WEST RAIL TAI LAM TUNNEL

The West Rail Tai Lam tunnel is 5.5km length and from SI data was anticipated to encounter the Sham Tseng Fault, with adverse tunnelling conditions along a 40m length, at about 400m below ground surface. Based on the logging for the Tsing Tam to Yau Kom Tau Water Tunnel excavated, running in close proximity to the tunnel alignment, best to worst case ground model predictions were assessed and associated rock mass parameters, using Q, GSI and derived Hoek-Brown

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strength parameters were estimated to anticipate design and construction strategies prior to excavation (GEO, 2007). Selected parameters are summarised in **Table 5**.

Parameter	Worst	Poor	Typical	Best
Q’	0.13	0.17	0.24	0.42
GSI	25	28	31	36
Mi	16	22	23.5	25
v	0.35	0.35	0.33	0.32
Defn (m)	1.3	0.45	0.33	0.26
Strength ratio	0.06	0.1	0.12	0.16

Table 5: West Rail Tai Lam, Q value comparison.

The parameters were used to estimate the constructability from the strain against the strength ratio as shown in **Figure 12** (GEO, 2007).

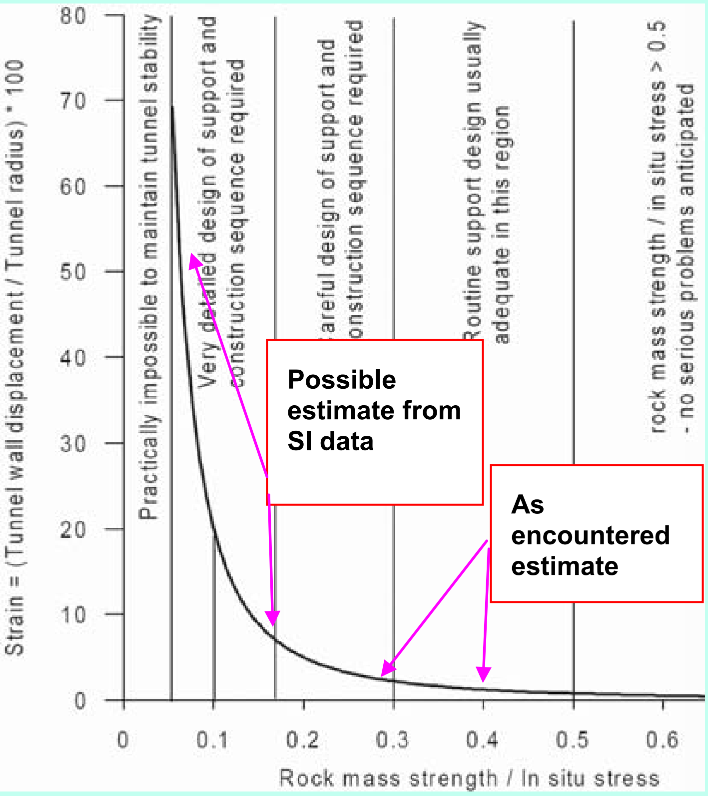

Figure 12: Representation of tunnelling difficulty

The anticipated design and construction approaches were anticipated to be either:

- Full face excavation – face reinforcement using dowels and shotcrete with allowance for stress relaxation prior to temporary support installation;
- Partial face excavation – to control heading stability

As presented in Figure 11 the actual conditions encountered were better than those anticipated.

7. EXCAVATION INFLUENCES ON THE Q VALUE

7.1 OVERBREAK

Excavation over-break is related to the number of joint sets (J_n) and their frictional component ((J_r/J_a), as shown in **Figure 13** (Hoek & Brown, 1997).

Figure 13 – Overbreak example

Although not referenced in the Q value assessment the joint set orientation and the potential for rock block release also effect overbreak. Overbreak may become excessive when the parameter values J_n = 9 and J_r / J_a = 1/4 are exceeded. Mitigation can be carried out by suitable pre-injection grouting.

7.2 GROUTING

Pre-injection grouting improves the rock mass “quality”. This term is used to represent all characteristics influencing rock mass behaviour (GEO, 2007) and differs from rock mass classifications which are limited to the behaviour from the input parameters. Improvements in the Q value parameters are summarised in **Table 6**.

Q para	Effect	Improve From	Improve to
RQD	Increase	30	50
J_n	reduction	9	6
J_r	Increase	1	2
J_a	reduction	2	1
J_w	Increase	0.5	1
SRF	Either	NA	NA

Table 6: Grout injection improvements.

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The increase in the Q value parameters are taken as being "typical" assuming that microfine grout is injected. Using the values given in Table 6 the Q value increases from 0.8 to 17. The improvement in RQD, J_n, J_r, J_a and J_w, result from the sealing of the more "open" joints, typically forming the main joint set. The SRF increase can occur if the excavation is in a 'low stress" environment with little clay infill.

8. OTHER CLASSIFICATION APPLICATION

8.1 TBM Q VALUE

Modifications to the Q value can be used to evaluate influences from the rock mass from a TBM excavation. The main effects are on the:

- Penetration rate (PR) – the rate of disc cutting, and
- Advancement Rate (AR) – advancement of the TBM

The relation between these parameters and QTBM are presented in **Figure 14** (Barton, 2012).

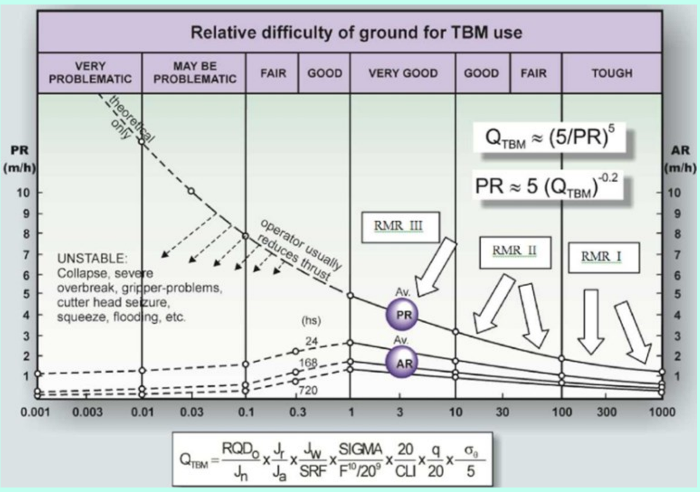

Figure 14: Q_{TBM} assessment

The Q_{TBM} is estimated from the Q parameters RQD, J_n, J_r, J_a, J_w and the SRF with the following considerations:

- RQD - interpreted in the tunnel direction, and
- J_r – taken from orientation most favourable to boring

The other parameters include:

- F – average cutter load (normalized to 20t)
- б – rock mass strength estimate
- CLI – Cutter Life Index (4 – quartzite; 90 – limestone)
- q – quartz content (%)
- б' – Induced radial biaxial stress on face

8.2 DEFORMABILITY

Deformation parameters were originally obtained from the correlation of deformation with Q values from Taiwan projects which gave a simplified relationship between deformation (mm), excavation span (m) and Q (Barton & Grimstad, 1994). This was improved by using a "stress / strain" competency factor as follows:

$$\text{Deformation (mm)} = \frac{\text{SPAN (m)}}{100 * \text{Q}} * \text{SQRT}\frac{6}{\text{UCS}}$$

The deformation and the stress (б) are directional and can be applied vertically or horizontally accordingly. This relationship was used to determine deformation moduli (E) for use in numerical analyses (Bieniawski, 1978 & Barton & Bieniawski, 2008), as presented in **Figure 15**.

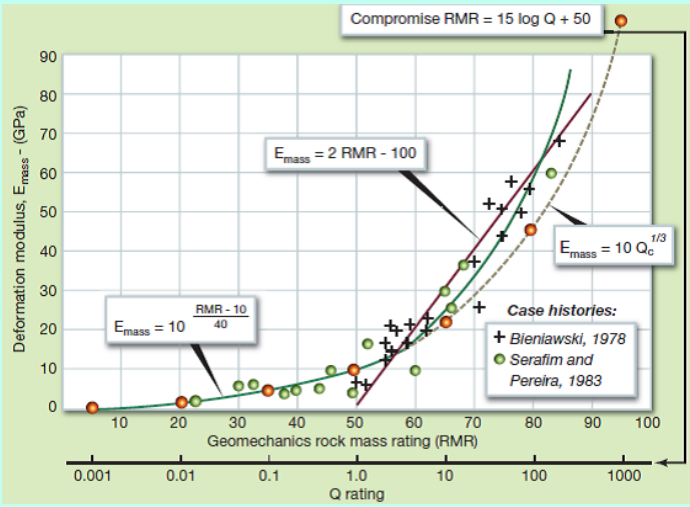

Figure 15: Rock Mass Deformability

9. SUBJECTIVITY

An example of the subjectivity of the rock quality classifications is summarized in **Table 7** (Fookes, 1997, Mackay et al, 2009). This presents findings from independent assessments by engineering geologists (EG). The rock exposure was in Bridport, Devon, UK with strong cleavage, faulting and fracturing, making it difficult to assess. Both EGs had similar qualifications and experience, which included the application of rock mass classifications, and carried out a review of the same boreholes and exposures with sufficient time "to do the job well". The findings varied from a Q value of 0.02 with immediate collapse and requiring 75 to 100mm mesh reinforced shotcrete to a Q value of 5 with one month stand up and no support required.

(Special thanks to Mr Sandy Mackay, ALYSJ JV, Doha; and Miss Ada Chan, Leighton Asia Ltd, for preparation of this guideline)

Para.	EG 1	Estimate	EG 2	Estimate
RQD	0, 10%	Cleavage true discontinuity set.	75%	Cleavage, incipient weakness plane, manifest as fine cracks.
J_n	15		12	
J_r	1	Persistent, smooth planar	2	Impersistent, smooth undulating.
J_a	4	Clay is seen very rarely along joints	1	Joint surfaces are typically discoloured
SRF	5		2.5	
J_w	0.66	Some exposures show a little water seepage.	1	Joints within the rock mass more likely tight and less permeable.
Q	0.022		5	
ESR	4.5m		3.6m	
Stand time		Immediate col-lapse		1 month
Sup.		75 - 100mm mesh reinforced shotcrete		No support

Table 7: Comparison of Q system from 2 EGs.

Estimates from both EGs were the same for J_n, which stated that the “estimate the least favourable joint set over the scale of the excavation considered and for SRF, which stated that “the excavation will be near surface in a relatively low stress regime. The Q system table provides only general data to the lack of case data.”

An example of subjectivity determining RQD is presented in **Figure 16** (Ewan & West, 1983). This shows variability between 6 observers recording joint locations along a scanline survey for the Kielder Aqueduct tunnels.

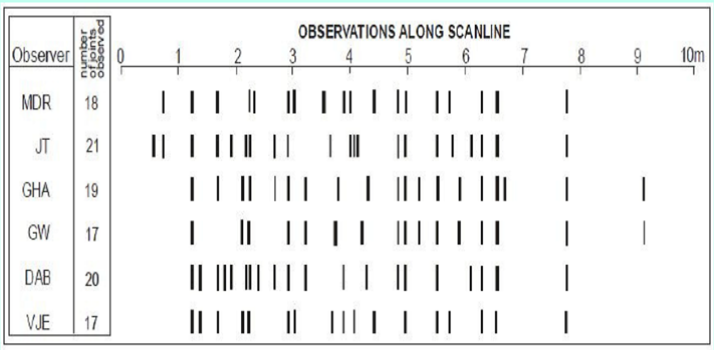

Figure 16: Scanline subjectivity

10. MITIGATING LIMITATIONS

Rock mass classifications used for sub-surface excavation support have imitations. Suggested measures to overcome these are summarized in **Table 8**.

Limitation	Mitigation
Uncertainties with the parameters	Use appropriate input parameter ranges carried out as part of a sensitivity analysis or risk mitigation process during design.
Limitations with the parameters used	Use additional classification systems and numerical analysis as a cross reference. Appropriate judgement is needed to assess whether additional support is needed.
The parameters do not account for adverse geology.	Prepare rock quality assessments (GEO, 2007), systematically recording all geological data, emphasising factors influencing stability.
Consistency between classification records.	Site specific manual used to clearly define the design assumptions related to the rock mass classification system and how each parameter is assessed.
Lack of experienced staff	Peer review from experienced staff and cross checks between site staff

Table 8: Limitations and Mitigation

The support should be assessed considering 3 dimensional effects; examples include persistent discontinuities running parallel to the tunnel alignment which may not be seen during mapping:

- Horizontal – if present within say 0.5m of the tunnel crown may allow slab release.
- Sub-vertical – potential shoulder wedge release

11. SUMMARY

Rock Mass Classifications, particularly the Q and RMR systems are established requirements for all tunnel and cavern design and construction works in Hong Kong. They are mainly used for excavation support estimation during design and construction. Given the variability and geological complications associated with rock masses, a thorough understanding of the support application is needed and should be carried out by engineering geologists with relevant experience and qualifications. The classifications should be prepared within a project management framework, which includes peer reviews, independent cross checks and systematic recording of the rock mass condition to assess rock mass quality.

(Special thanks to Mr Sandy Mackay, ALYSJ JV, Doha; and Miss Ada Chan, Leighton Asia Ltd, for preparation of this guideline)

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