# Tunnels and Caverns Design in Rock

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doi: https://doi.org/10.21467/proceedings.171.3

**ABSTRACT**

In the field of underground space design, numerous publications, design theories and software are readily available and easily accessible in the market. But it can be unclear as to which software to use, or which design approach to adopt to complete a tunnel or cavern design. Factor in varying ground conditions and tunnels with different sizes, the experience will become overwhelming. As such, it is necessary to adopt a design approach that is acceptable by the industry and recognizes the appropriateness of the engineering application that was first developed (so called "first principal"). Such design approach shall not only be safe, but also understand the limit of the applied theories or software. This paper aims to present one general and complete design approach for drill and blast or mechanically excavated tunnels and caverns in rock of all qualities (using one completed project as example), that would eliminate such ambiguities. This paper will also be a useful reference for those pursuing a career in tunnelling. The design approach follows the three general stages below.

I. Derive initial support (rock bolt and shotcrete) from empirical approach.
II. Check the adequacy of support from Step 1 using theory of reinforced rock arch (RRA).
III. Verify and modify the support design by numerical method.

## 1 BACKGROUND

Numerous design guidelines and approaches for tunnels and caverns have been extensively discussed and published. While each of these publications are mostly supported by case studies to demonstrate validity, it is important to realise that there is no one single design approach, or factor of safety, that would fit all cases of tunnels or caverns support. In one of his most widely referenced publication "Practical Rock Engineering" (Hoek, 2023), Dr. Evert Hoek wrote the following:

"*…there are no simple universal rules for acceptability nor are there standard factors of safety which can be used to guarantee that a rock structure will be safe and that it will perform adequately. Each design is unique and the acceptability of the structures has to be considered in terms of the particularly set of circumstances, rock types, design loads and end users for which it is intended… Tables 1 to 4 summarise some of the typical problems, critical parameters, analysis methods and acceptability criteria which apply to a number of different rock engineering structures. These examples have been drawn from my own consulting experience and I make no claims that this is a complete list nor do I expect readers to agree with all of the items which I have included under the various headings.*"

Figure 1 shows one of the tables mentioned by Dr Hoek which is captured unedited in its entirety.

## 2 INTRODUCTION

The design approach presented in this paper is by no means a one-solution-fit-all approach. But this is one that is overall most suitable for the conditions and rock types in Hong Kong. Adaptation of this approach elsewhere should be done with appropriate consideration and modification according to local conditions and rock types.

In the subsequent sections, each of the three general design stages outlined in abstract will be elaborated in more details with figures and diagrams. The three general design stages can be expanded in the following seven sub-steps.



Proceedings DOI: 10.21467/proceedings.171; Series: AIJR Proceedings; ISSN: 2582-3922; ISBN: 978-81-970666-7-2

1. Assessed all available information (geotechnical investigation and laboratory testing result) to determine relevant design parameters. The outcome of this assessment included qualitative and quantitative assessment of the rock mass such as intact rock unconfined compressive strength (UCS), jointset data, joint strengths etc.

2. Carried out initial tunnel temporary lining design based on Q-System (NGI, 2015). The outcome of this is a temporary support table of tunnel such as thickness of sprayed concrete lining, length and spacing of rock bolts, reinforced ribs (this is Stage I outlined in abstract).

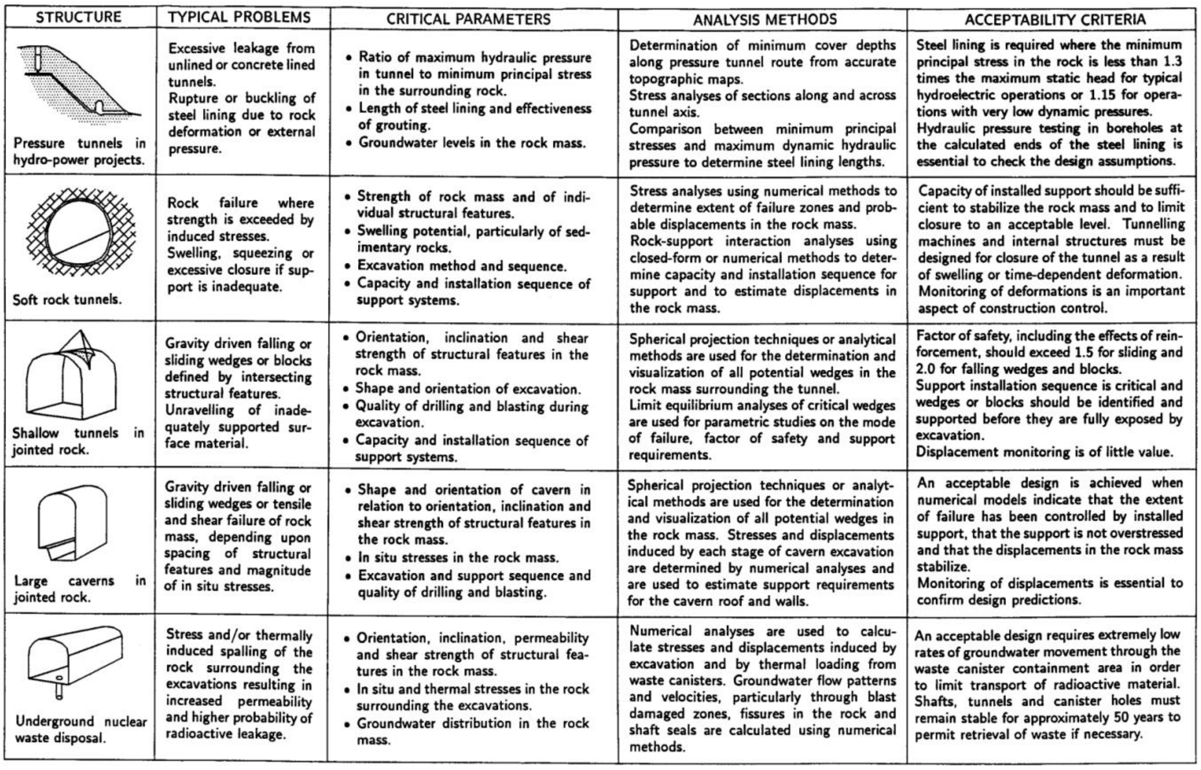
3. The next step involved kinematic analysis. At the end of this, the largest possible rock blocks can be determined. And the bolt length and spacing determined from previous step 2 above can be verified to be sufficient to stabilize these blocks.

4. With the initial support derived from empirical method, the next stage involved the checking of the support design using the reinforced rock arch (RRA) method (stage II outlined in abstract). At the end of this step, the pattern rock bolts, together with the rock mass holding these bolts, will be checked, and assessed as a collective structural beam.

5. The next step involved the use of finite element program with assumption of certain rock-structure interaction behaviors (such as arching effect of rock, relaxation factor etc.) and determine the state of stress of the sprayed concrete lining (bending moment and hoop force) (stage III outlined in abstract).

6. The next step is to determine if this state of stress is within the structural capacity of the tunnel lining. This is achieved by plotting the characteristic interaction diagram of the lining and plotting the data points (bending moment and hoop forces obtained from previous step 5) on this curve. If all the data points are located within this interaction curve, the lining is structurally sufficient. Otherwise the tunnel lining needs to be further strengthened.

7. The next and final step of tunnel support design involves determining the largest unsupported span and the longest unsupported time of the tunnel.



| STRUCTURE | TYPICAL PROBLEMS | CRITICAL PARAMETERS | ANALYSIS METHODS | ACCEPTABILITY CRITERIA |
|---|---|---|---|---|
| Pressure tunnels in hydro-power projects. | Excessive leakage from unlined or concrete lined tunnels. Rupture or buckling of steel lining due to rock deformation or external pressure. | • Ratio of maximum hydraulic pressure in tunnel to minimum principal stress in the surrounding rock. • Length of steel lining and effectiveness of grouting. • Groundwater levels in the rock mass. | Determination of minimum cover depths along pressure tunnel route from accurate topographic maps. Stress analyses of sections along and across tunnel axis. Comparison between minimum principal stresses and maximum dynamic hydraulic pressure to determine steel lining lengths. | Steel lining is required where the minimum principal stress in the rock is less than 1.3 times the maximum static head for typical hydroelectric operations or 1.15 for operations with very low dynamic pressures. Hydraulic pressure testing in boreholes at the calculated ends of the steel lining is essential to check the design assumptions. |
| Soft rock tunnels. | Rock failure where strength is exceeded by induced stresses. Swelling, squeezing or excessive closure if support is inadequate. | • Strength of rock mass and of individual structural features. • Swelling potential, particularly of sedimentary rocks. • Excavation method and sequence. • Capacity and installation sequence of support systems. | Stress analyses using numerical methods to determine extent of failure zones and probable displacements in the rock mass. Rock-support interaction analyses using closed-form or numerical methods to determine capacity and installation sequence for support and to estimate displacements in the rock mass. | Capacity of installed support should be sufficient to stabilize the rock mass and to limit closure to an acceptable level. Tunnelling machines and internal structures must be designed for closure of the tunnel as a result of swelling or time-dependent deformation. Monitoring of deformations is an important aspect of construction control. |
| Shallow tunnels in jointed rock. | Gravity driven falling or sliding wedges or blocks defined by intersecting structural features. Unravelling of inadequately supported surface material. | • Orientation, inclination and shear strength of structural features in the rock mass. • Shape and orientation of excavation. • Quality of drilling and blasting during excavation. • Capacity and installation sequence of support systems. | Spherical projection techniques or analytical methods are used for the determination and visualization of all potential wedges in the rock mass surrounding the tunnel. Limit equilibrium analyses of critical wedges are used for parametric studies on the mode of failure, factor of safety and support requirements. | Factor of safety, including the effects of reinforcement, should exceed 1.5 for sliding and 2.0 for falling wedges and blocks. Support installation sequence is critical and wedges or blocks should be identified and supported before they are fully exposed by excavation. Displacement monitoring is of little value. |
| Large caverns in jointed rock. | Gravity driven falling or sliding wedges or tensile and shear failure of rock mass, depending upon spacing of structural features and magnitude of in situ stresses. | • Shape and orientation of cavern in relation to orientation, inclination and shear strength of structural features in the rock mass. • In situ stresses in the rock mass. • Excavation and support sequence and quality of drilling and blasting. | Spherical projection techniques or analytical methods are used for the determination and visualization of all potential wedges in the rock mass. Stresses and displacements induced by each stage of cavern excavation are determined by numerical analyses and are used to estimate support requirements for the cavern roof and walls. | An acceptable design is achieved when numerical models indicate that the extent of failure has been controlled by installed support, that the support is not overstressed and that the displacements in the rock mass stabilize. Monitoring of displacements is essential to confirm design predictions. |
| Underground nuclear waste disposal. | Stress and/or thermally induced spalling of the rock surrounding the excavations resulting in increased permeability and higher probability of radioactive leakage. | • Orientation, inclination, permeability and shear strength of structural features in the rock mass. • In situ and thermal stresses in the rock surrounding the excavations. • Groundwater distribution in the rock mass. | Numerical analyses are used to calculate stresses and displacements induced by excavation and by thermal loading from waste canisters. Groundwater flow patterns and velocities, particularly through blast damaged zones, fissures in the rock and shaft seals are calculated using numerical methods. | An acceptable design requires extremely low rates of groundwater movement through the waste canister containment area in order to limit transport of radioactive material. Shafts, tunnels and canister holes must remain stable for approximately 50 years to permit retrieval of waste if necessary. |

Figure 1: Original Table 3 in Dr Hoek's publication "Practical Rock Engineering" (Hoek, 2023)


Proceedings DOI: 10.21467/proceedings.171
ISBN: 978-81-970666-7-2
Series: AIJR Proceedings
ISSN: 2582-3922

# 3 DESIGN STEPS FOR TUNNELS AND CAVERNS IN ROCK

The following sections will elaborate in detail each of the seven design steps outlined in Section 2 with supporting documents and examples. For purpose of avoiding repeated term, the term "tunnel" instead of "tunnel and cavern" will be used throughout this paper but both are equal for topics presented in this paper unless otherwise mentioned.

## *3.1 Geological Review*

The first task for any tunnel projects is to always collect and review all available geological and geotechnical data including aerial photo interpretation, whether project specific or non-project specific. This task is best to be carried out by a qualified engineering geologist, where key parameters such as rock types, weathering grades, uniaxial compressive stress (UCS) results along the depth of each log are summarized into a spreadsheet.

Concurrently, relevant borehole televiewer data must be reviewed to determine dip angle and dip direction of rock jointsets. This data is entered into the program DIPS which will be used by the program UNWEDGE explained later in Section 3.3.

After all the review and analyses were completed, the most representative and characteristic jointset orientation (total of 4 major, 1 minor jointset in the example used in this paper) and engineering parameters of rock mass (UCS of intact rock, joint shear strength) can be determined to be adopted in subsequent design of tunnel support.

Another geological information that will be extremely helpful in early stage of a project is the expected rock mass quality (the *Q*-value) along the tunnel complex. The success of this highly depends on few things: quality of horizontal directional coring (HDC) along the alignment, meticulous focus on HDC details, and unbiased application of Q-system parameters. One of the most common ways to estimate *Q*-value along the tunnel is by varying the parameters *RQD*, $J_n$, $J_a$ and $J_r$ in Equation (1) based on data obtained along the HDC. *RQD*, $J_a$ and $J_r$ can be estimated directly from HDC core samples, supplemented by boreholes and televiewer along the alignment. These three parameters are to be grouped along the alignment from direct observation. $J_n$, which relates to number of jointsets, can be estimated according to rock type. In Tuff, where rock mass is expected to be more fractured than Granite, the $J_n$ value will be higher. Jw is assumed to be 1.0 because any groundwater seepage observed through joints is expected to be grouted. *SRF* relates to in-situ stress condition or overburden and can be estimated from any UCS results along the alignment. In highly fractured or fault zones, *SRF* will be locally adjusted accordingly. When all these parameters are being properly assessed, the expected *Q*-value along the alignment can be estimated using Equation (1).

## *3.2 Initial Support System using Empirical Method*

One of the most widely used rock classification and support system is an empirical approach developed by the Norwegian Geotechnical Institute, the Q System (NGI, 2015), first developed in 1971 for use in metamorphic rock. This has been widely adopted due to its simplicity and reliability when used correctly on site. Over the years of application and improvement from experience, several factors were introduced for better representation on the scale and complexity of underground projects.

This method has been developed based on data collected through direct observations and historical failures from past projects and experience. The fundamental concept of classification method is to recognize the fact that the quality of rock mass as construction material in underground excavations cannot be measured by strength tests alone but have to be characterized by several controlling geological parameters. These parameters and correlations are derived for classification purpose, and grouped into different classes accordingly, to estimate rock mass behaviour and support requirements for prediction of tunnel supports.

The classification involves grouping areas of consistent geological structures based on the generality of observed features such as groundwater inflow condition, intact rock volumes, density and orientation of discontinuities, and infilling within discontinuities. By grouping the types of conditions likely to be encountered, it enables designers to prescribe a handful of generalized support types to cover those conditions. As result, the classification allows an engineer an invaluable understanding of the rock mass quality.

The general form of $Q$-value is determined by the estimation of the following six (6) rock mass parameters. The $Q$ values range from 0.001 to 1,000 and represents an empirical and quantitative classification system showing the rock mass quality with respect to tunnel stability and support. On site, the $Q$ values are mapped through direct observation of the exposed tunnel excavation face.

$$Q_{mapped} = \frac{RQD}{J_n} \times \frac{J_a}{J_r} \times \frac{J_w}{SRF} \tag{1}$$

Where $RQD$ = rock quality designation, $J_n$ = jointset number, $J_a$ = joint alteration number, $J_r$ = joint roughness number, $J_w$ = joint water pressure number, $SRF$ = stress reduction factor.

These six factors are used to classify rock mass quality on site, in different degrees of direct observation. Figure 2**Error! Reference source not found.** is a table directly extracted from NGI handbook as a guideline of rating each of these parameters.

NGI also developed a $Q$-chart intended for use to determine the appropriate support system within predefined ranges of $Q$-values (Figure 2). In order to use this $Q$-chart the tunnel effective span ($D_e$) is first calculated, which is the ratio of tunnel span to $ESR$ factor and used this across the chart to determine the required support for each class of rock mass quality. Each class of rock mass quality is represented by an upper and lower bound $Q$-values. The tunnel span (either the width or height of tunnel) is determined from the tunnel cross section and the $ESR$ factor is determined based on the definition from NGI. Figure 2**Error! Reference source not found.** is the o riginal $Q$-chart extracted from the NGI handbook.

There are other factors that need to be applied to the $Q$ ranges determined directly from the $Q$-chart. These factors are summarized in the following paragraphs.

The original $Q$ chart was developed as permanent support for tunnel. For temporary support, the design $Q$-value for roof ($Q_{roof}$) is adjusted to five times the mapped $Q$-value $Q_{mapped}$, i.e. this factor of 5 is either applied directly on-site on the $Q$ values mapped by the engineering geologist, or a factor of 1/5 is applied on the design values of $Q$ support table. It is recommended to apply factor of 1/5 on design values and clearly mention this on the drawing, so to avoid accidental omission on site.

$$Q_{roof} = 5 \times Q_{mapped} \tag{2}$$

At tunnel junction or intersection, e.g. cross passages connecting two main tunnels in parallel, the mapped joint set number ($J_n$) should be adjusted by factor of 3.0 with $ESR$ taken as 1.0. This factor is also recommended to apply on design drawing for the same reason.

$$Q_{roof} = \frac{Q_{mapped}}{3.0} \tag{3}$$

The $Q$ value determined from the $Q$-chart is intended for tunnel crown. For tunnel sidewall support, the design $Q$-value for wall ($Q_{wall}$) is related to design $Q_{roof}$ as shown in Table 1.

Table 1: Factors to be applied on $Q_{roof}$ to determine $Q_{wall}$

| $Q_{mapped} \geq Q_{roof}$ | | $Q_{mapped} < Q_{roof}$ | |
|---|---|---|---|
| $Q_{roof}$ | $Q_{wall}$ | $Q_{mapped}$ | $Q_{wall}$ |
| > 10 | $5 \times Q_{roof}$ | > 10 | $5 \times Q_{roof}$ |
| 0.1 – 10 | $2.5 \times Q_{roof}$ | 0.1 – 10 | $2.5 \times Q_{roof}$ |
| < 0.1 | $1 \times Q_{roof}$ | < 0.1 | $1 \times Q_{roof}$ |

For $Q$-value less than 0.1, the design rock dowel length of both the roof and wall should be the same even if the wall dowel lengths are shorter. This is to avoid the risk of potential rock bursting from wall. An $ESR$ value of

1.0 has been adopted for calculating the dowel lengths due to the large span of tunnel.

In general, the relationship between the design *Q*-value for roof support and the mapped *Q*-value can be summarized as follow:

$$Q_{roof} = f^* \times f_{Jn} \times Q_{mapped} \tag{4}$$

Where $f^*$ = the factor on support condition (temporary or permanent), $f_{Jn}$ = the factor on joint set number ($J_n$) for the support location

These two coefficients account for all the factors given above, and the $Q_{mapped}$ value determined on site can be used directly to determine the support classes and related requirements according to the drawings.

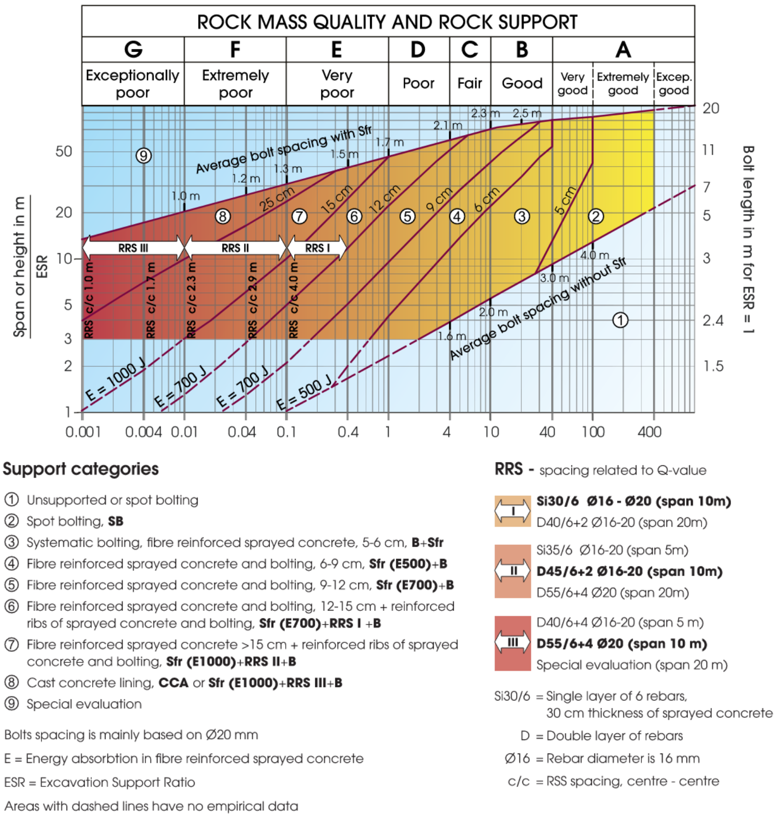



Figure 2: The *Q*-chart and support categories from the NGI *Q*-System (NGI, 2015) handbook

The dowel length $L_d$ can be determined directly from the *Q* chart above or using the following equation (Barton, et al., 1974). It is recommended to use the following equation because it is slightly more accurate than directly reading off from chart.

$$L_d = 2 + 0.15D/ESR \tag{5}$$

### *3.3 Kinematic Analysis of Wedge Failure*

A stiff support system needs to be provided after tunnel excavation to limit any rock wedge movement. Only a very small movement is needed to trigger the failure of a wedge. Sliding of a few millimeters along one plane or the line of intersection of two planes is generally sufficient to overcome the peak strength of the joint surfaces. Tensioned mechanically anchored rock bolts, untensioned-fully-grouted rock dowels and other continuously coupled devices can be used to support the wedges.

For roof wedges, the rock bolts should be capable of supporting the entire self-weight of the rock wedge. And the bolts should be placed uniformly about the wedge centroid to prevent rotation that reduces the factor safety. For sidewall wedges, the bolts can be placed in the sliding plane to increase the shear resistance of the joints.

The program UNWEDGE is used next to verify the preliminary support determined from Section 3.2. This requires the size and orientation of the tunnel, combined with the jointset data (dip and dip direction of each set), as input into the program. Based on all these input parameters, the program determined the largest possible size and weight of rock wedges at the exposed tunnel wall and crown. These wedges are formed by physically intersecting the planes from jointsets and tunnel.

After the input into UNWEDGE the material property (capacity), spacing and length of rock dowels determined from the *Q*-chart, the program will calculate the factor of safety (FOS) of the provided support system (pattern dowels) against the failing wedges. The dowels must physically penetrate beyond the wedges in order for an FOS to be determined. Otherwise, the dowels will fall off together with the wedges yielding an FOS of zero.

The FOS for each wedge is calculated as the ratio of total support pressure provided by all intersecting dowels to the weight of the wedges. There are two other key parameters that would affect the FOS, namely joint water pressure and joint cohesion. The higher the joint water pressure, the more unstable the block, hence yielding a lower FOS. This is because joint water pressure directly contributes to the causal effect for wedge to fall off. Joint cohesion, on the other hand, has the opposite effect of joint water pressure. A small presence of joint cohesion (say 5 to 10 kPa) would yield very stable wedges that requires low support pressure, hence higher FOS.

In Hong Kong, pressurized joints from full groundwater head are seldom observed but minimal joint water pressure is often assumed to account for unforeseen groundwater inflow. For joint cohesion, friction does exist regardless of joint infilling. However, as a conservative approach, joint friction cohesion is assumed to be zero.

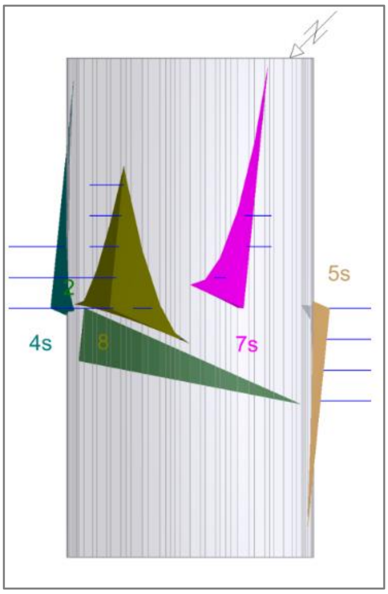
An example of UNWEDGE output findings is shown in Figure 3. In this example joint water pressure equivalent to 1.0 m of water head is adopted. This is to account for any transient groundwater through open joints that are not being effectively grouted by fissure grouting when measurable inflow is observed from probe holes. Joint cohesion is ignored due to difficulty in justifying its presence and the magnitude.

From the findings, the most critical rock wedges are found to be the ones at the crown (wedge no. 7s and 8), formed by prescribed jointsets at the project site.

In the case where the dowels capacity is found to be insufficient (i.e., FOS < 1.5 from UNWEDGE), one of the following can be done to increase the FOS: bolt spacing needs to be reduced, length of bolts to be increased or use bolts with higher capacity. Several iterations are needed in this step to determine the most optimised bolt spacing and length that yielded the desired FOS.

Proceedings DOI: 10.21467/proceedings.171
ISBN: 978-81-970666-7-2

Series: AIJR Proceedings
ISSN: 2582-3922



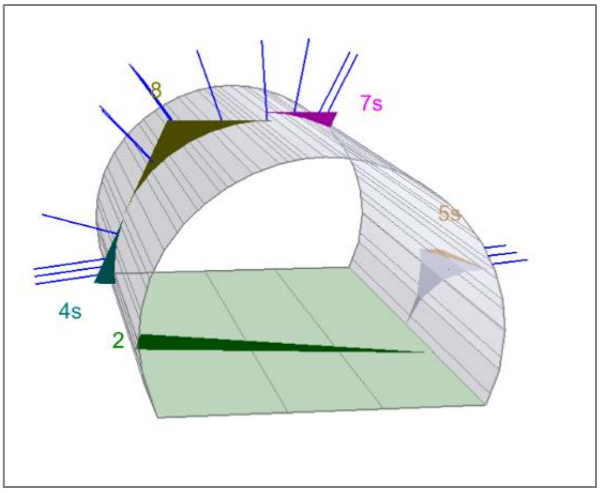




Figure 3: The Q-chart and support categories from the NGI Q-System *(NGI, 2015)* handbook

Due to its ease of use, UNWEDGE is a program that is most often being misused for purpose other than FOS of failing wedges. Below are some of the main points of misuse that the author has noticed that must be reminded to young engineers:

- UNWEDGE is intended for use in situation where the in-situ stresses are low or not critical, where their influence can be largely neglected without the introduction of significantly errors. Where high in-situ stress levels occur in blocky rock masses, the FOS predicted by the program can be incorrect.
- The wedge analysis is considered part of the tunnel support system verification. However, the findings from UNWEDGE should not be used to replace typical tunnel design from empirical approach or numerical analysis.
- As sliding of a few millimeters is generally sufficient to overcome the peak strength of the joint surfaces, rock wedge will fail when only small displacement occurs mobilizing its effective strength. As such, peak strength of discontinuities should not be used because this may overestimate the joint shear strength. And residual strength should be adopted instead.
- At deeper rock mass, the rate of increase of joint shear strength decreases. Therefore, using a constant joint friction angle for all depth of rock mass may overestimate the FOS leading to unconservative design especially for deep rock tunnels.

### *3.4 Support System as Reinforced Rock Arch (RRA)*

After an initial support system has been developed and verified with kinematic approach, the next step in the design process involves checking the collective effect of pattern bolts. When pattern rock bolts are installed around tunnels, they are providing reinforcement effect to the rock mass, rather than as support to the rock mass. The reinforcement concept treats the rock mass with pattern bolts collectively as an arch under compression; whereas the old fashion concept of “support” would treat the rock mass as loading supported by the bolts. This shift in tunnel design concept resulted in a different analysis approach called reinforced rock arch (RRA) approach. This concept of tunnel rock mass reinforcement has been demonstrated excellently by Dr Evert Hoek’s educational experiment shown in Figure 4. The small pieces of rocks in the inverted bucket did not fall out because they were being held firmly by the equally spaced bolts forming a collective slab under compression.

In order for the bolts to hold the slab of rock pieces together, the spacing, *s*, of the bolts must not be too large compared to the average size of rock pieces, *a*. However, there is common misconception about how rock bolts contribute to creating a linear zone of compression that keeps the rock pieces from falling. The arching effect is not due to the clamping action of the end plates but rather due to its reinforcement effect allowing stresses to arch linearly to the adjacent rocks thus creating a confinement effect. As such, this arched beam of reinforced rock mass can be considered as a structural element supporting the rock mass above it.

The most common analytical approach that allows the reinforced rock arch strength to be calculated is the design approach proposed by Bischoff and Smart (Bischoff & Smart, 1977). In this computational method, the increased in the capacity of thrust of the reinforced arch is being compared with the rock pressure acting on the arch. If the acting pressure is less or equal to the thrust capacity of the reinforced arch, then the tunnel is considered stable. The equations to calculate the increased thrust capacity of reinforced arch is as follow:

$$\Delta T_A = q \frac{\sigma_b A_b}{s^2} (L - s) \tag{6}$$

$\Delta T_A$ represents the increase in unit thrust of the reinforced arch zone; $\sigma_b$ is the yield strength of bolts; $A_b$ is the cross-sectional area of the bolts; $s$ is the spacing of the reinforcing pattern bolts both transverse and longitudinally to the tunnel axis. Parameter $q$ is the function of internal friction angle, $\varphi$, of the rock mass that correlates between effective increase in the allowable stress of the rock mass and rock reinforcement confining pressure ($\Delta\sigma_1/\Delta\sigma_3$) and can be calculated using the following equation:

$$q = \tan^2\left(45^\circ + \frac{\varphi}{2}\right) \tag{7}$$

The rock mass pressure acting on the reinforced arch ($P_{arch}$) can be estimated using calculation method proposed by Grimstad and Barton (Grimstad & Barton, 1993), which is also the calculation method recommended by GeoGuide 4 (GEO, 2018). The equation to estimate rock pressure of tunnel is as follow, where $J_n$, $J_r$, and $Q$ are parameters determined in step 2 (page 18) using Q-System (NGI, 2015).

$$P_{arch} = \frac{2 J_n^{\frac{1}{2}} (Q)^{-\frac{1}{3}}}{3 J_r} \tag{8}$$

The tunnel is considered sufficient as reinforced rock arch when the condition $\Delta T_A \geq P_{arch}$ is fulfilled.

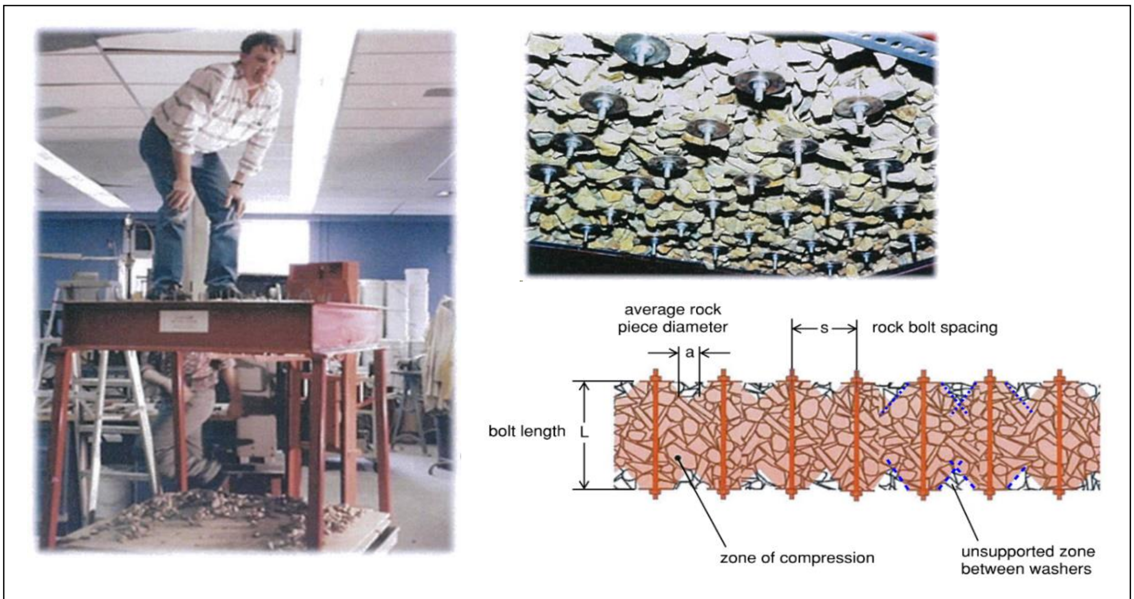



Figure 4: Demonstration on the concept of reinforced rock arch with pattern bolts *(Hoek, 2023)*

One of the most common misconception when calculating $P_{arch}$ is whether factor $f_{Jn}$ from Equation (4) needs to be applied to Equation (8). It should be clarified that $f_{Jn}$, which is a factor to account for additional exposed jointsets at junction and portal, is only intended for mapping purpose and not to be applied in calculation of $P_{arch}$ at junction or portal.

### 3.5 Finite Element Analysis (FEA)

This last step in the tunnel design perhaps is one of the more complicated steps throughout the entire design and involves several sub-steps to yield any representative results. In general, numerical analysis program *RS2* (previously *Phase2*) was used to further verify and optimise tunnel support in highly jointed ground developed from all previous steps.

In the project example for this paper, continuum numerical modelling has been adopted for highly jointed ground condition for two main reasons: (1) in highly jointed rock mass condition, the rock bolts and rock mass surrounding the tunnel opening behave collectively as reinforced medium and form part of the support system. Where other empirical approaches consider surrounding rock as a load that requires support, the concept of rock reinforcement is best represented as continuum model in RS2 program; and (2) it is recommended by GeoGuide 4 (GEO, 2018) that in ground condition with mapped $Q$ value of less than 0.1, the support system must be verified by numerical modelling.

In order to build a representative numerical model, there are several key parameters that need to be determined: (1) horizontal to vertical stress ratio at design depth of rock formation, and (2) relaxation factor of surrounding rock mass of the tunnel opening. The following are detailed explanation and determination of these parameters. There are other key parameters such as shear strength of the rock joint (joint roughness coefficient JRC, and joint compressive strength, JCS) however they are not applicable in continuum model because discontinuities are not being modelled in continuum model, where failure criteria is stress related rather than rock blocks related.

The rock head cover of the project tunnel is greater than 100m, and in such deep and highly fractured ground condition it is anticipated that the ground would behave as isotropic and homogeneous material. In the published paper by Kwong and Wong (Kwong & Wong, 2013), the results from 124 hydraulic fracturing test carried out in Hong Kong have been analysed and summarized to show that at this depth, the lower bound in-situ stress ratio is about 1.0. Therefore, the FEA ground model and in situ stress field is assumed to be isotropic and horizontal to vertical stress ratio of 1.0 is adopted for the numerical analysis.

#### 3.5.1 Degree of Relaxation

The second critical rock mass parameter is the degree of relaxation of the surrounding rock mass when a tunnel opening is formed. The degree of relaxation of a tunnel directly contributes to the amount of stress being transferred to the tunnel lining. When the rock mass surrounding a tunnel opening relaxes, some of the energy stored in the rock mass is being released and dissipated through deformation of the tunnel. The higher the deformation of a tunnel, the less stress is being transferred from the ground to the lining. However, if the deformation gets excessive, the deformation could become plastic resulting in potential caving or collapse. On the other hand, if the tunnel is only allowed to deform very slightly (e.g., in the case of immediate support installation), the support pressure required to stabilize the tunnel will be very high and unnecessarily costly to do so. As such, a balance between cost and safety must be reached.

In order to achieve this, convergence-confinement analysis is carried out to determine the degree of ground relaxation and to determine the resultant support pressure that applies to the lining. In general, the degree of relaxation can be determined from the following two equations published in the 2008 Kersten Lecture by Hoek et al. (Hoek, et al., 2008). These two equations combined to determine the level of convergence of excavated tunnel behind the tunnel face.

$$\frac{u}{u_{max}} = 1 - \left(1 - \frac{u_0}{u_{max}}\right) \cdot e^{-\frac{3d_t}{2P_r}} \tag{9}$$

$$d_t = -\frac{2}{3}P_r \cdot ln\left(\frac{u_{max}-u}{u_{max}-u_o}\right) \tag{10}$$

where $u$ = convergence behind tunnel excavation face, $u_{max}$ = maximum short-term tunnel convergence (determined from FEA model), $u_0$ = convergence at tunnel excavation face, $d_t$ = ratio of $x$ / tunnel radius, $x$ is distance behind tunnel face, $P_r$ = ratio of plastic radius/tunnel radius (measured from FEA model)

The above two equations can be represented by Figure 5 to determine $u$ / $u_{max}$. The following steps explain how to use numerical modelling to determine tunnel convergence and degree of relaxation.

1. Set up an *RS2* model with an unsupported, unlined tunnel opening. Right after excavation of the tunnel, apply 100% of support pressure to the tunnel opening then allow the support pressure to gradually reduce to zero. The result of this is shown in Figure 6 where tension and shear failure points formed around the deformed tunnel opening. The extend of this plastic zone is measured directly from the model, which is 13.1 m on average.

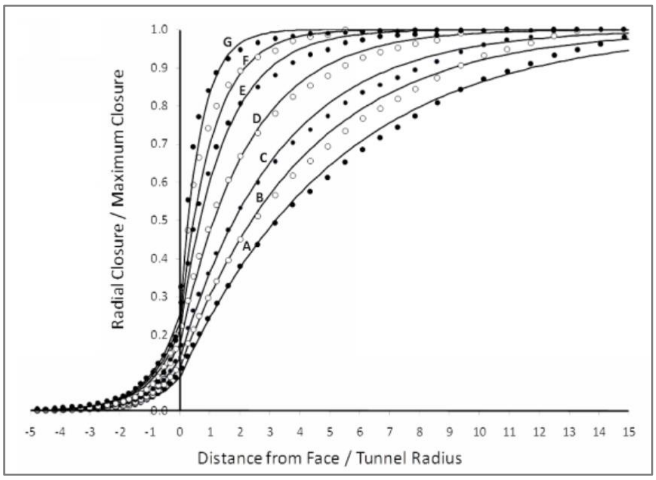



Figure 5: Normalised tunnel convergence behind tunnel excavation face *(Hoek & Marinos, 2000)*

2. Determine the maximum displacement from the same model, which is approximately 95.2 mm at tunnel crown from Figure 6.

3. Assuming a 1.0 m excavation advance length of tunnel, the values determined from steps 1 and 2 are used to calculate the tunnel convergence, $u$, either from the two equations above or the curve in Figure 5. The curves representing Equation (9) and (10) can be re-created using spreadsheet. Figure 7 shows one of these plots at day 0 when excavation length is 1.0 m. The convergence value determined from the curve is approximately 28.1 mm. The longitudinal displacement profile of the tunnel at day 1, 3, and 7, which corresponds to advance length of 2 m, 4 m, and 8 m respectively, is 35.4 mm, 47.6 mm, and 65.2 mm, respectively. In this calculation, the tunnel radius, plastic radius, and maximum displacement of the tunnel are all obtained from the *RS2* model, which is shown in *Figure 6*. The tunnel advance length in the calculation of convergence is at the mid-point of the excavated length, therefore at day 0 when tunnel excavated 1.0 m, advance length in the calculation is 0.5 m.

4. Next, a ground reaction curve (GRC) is created from the stress and deformation values obtained from the FEA model. Right after the tunnel is excavated, 100% of ground stress is applied radially outward on the tunnel perimeter. This ground stress is gradually reduced to zero. At each step of the decrement the displacement value at crown is recorded and plotted against the corresponding ground stress. For this example, the blue curve in Figure 8 shows the specific GRC for the tunnel.

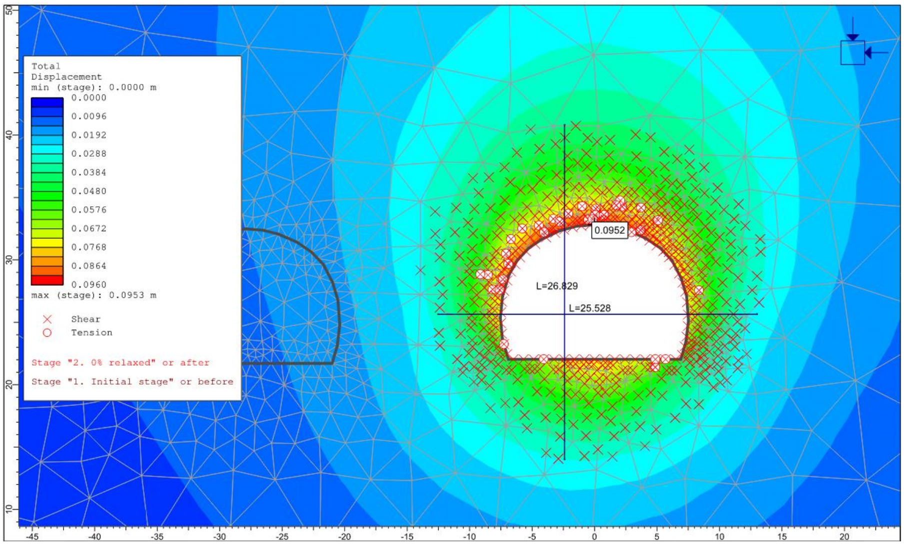



Figure 6: FEA model output with plastic shear points around tunnel opening to determine the radius of plastic zone.

5. Using the radial convergence determined from step 3 and plotting this on the X-axis of the GRC, the percentage of support pressure required for this particular tunnel is found to be approximately 19% of in-situ stress. This means that for this particular tunnel when 1 m is excavated, the tunnel crown will converge approximately 28.1 mm, and consequently approximately 19% of ground stress is transferred to the lining. The summary of the GRC and relaxation at day 1, 3, and 7, which corresponds to advance length of 2 m, 4 m, and 8 m respectively, is being plotted on the same curve on Figure *8*.

6. The degree of relaxation for 1.0 m advance is therefore 1 – 0.19 = 81% (say 80% approximately). This relaxation factor will then be adopted on a second FEA model, applied after excavation and before tunnel lining installed. The bending moment and hoop force extracted directly from the second FEA model will be used to check against the flexural capacity of tunnel lining as shown in Section 3.6.

The phenomenon of ground relaxation is a key observation and concept to represent the behavior of ground-structure interaction for tunnel, especially in rock. This can be *loosely* thought of the percentage of in-situ rock mass loading being transferred to and supported by the lining. In the above example when tunnel is excavated 1.0 m ahead, it will converge an average of 28.1 mm radially, consequently only approximately 19% of the in-situ rock stress will be transferred to the tunnel lining. Hence the lining is subject to approximately 19% of the in-situ rock mass loading; the other 81% has been "relaxed" and dissipated by stress redistribution around the opening and the "softening" of rock stiffness due to deformation.

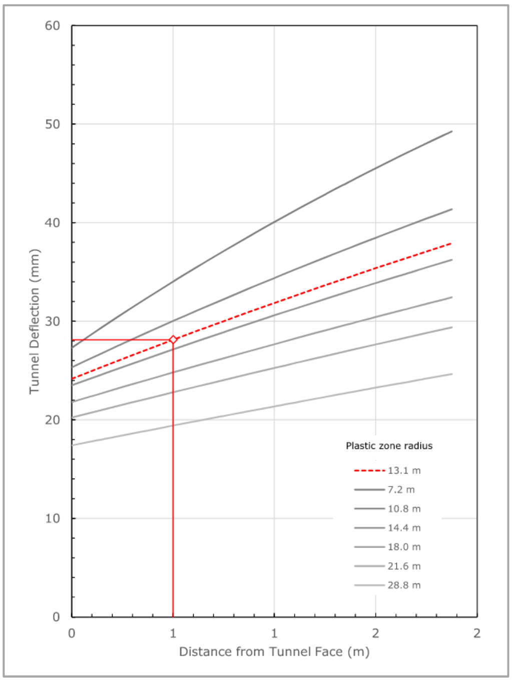



Figure 7: Longitudinal displacement profile at day 0 with excavation length of 1.0 m (recreated based on original curve (Figure 5) and Hoek et al. *(Hoek, et al., 2008)*)

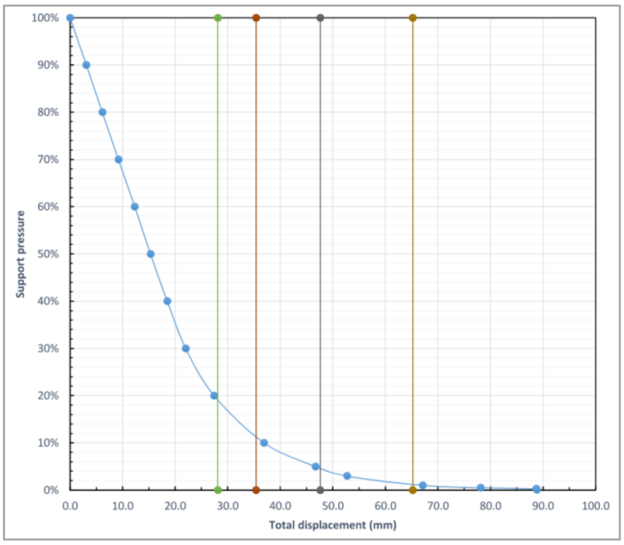



Figure 8: Ground reaction curve (GRC) of tunnel with maximum displacement of tunnel as input to determine the percentage of support pressure.

The degree of relaxation to be adopted in the design of tunnel lining has always been the subject of heavy discussion and shall be verified. Despite having a healthy amount of experience in similar ground and size of tunnel excavation in Hong Kong, and supported by solid scientific theory and data, the perception of relaxation is often based on anything but. The main reason for this is relaxation is often being misunderstood as under-design of the lining simply because the relaxed support pressure is lower in value compared to full unrelaxed load. And designer is often being requested to adopt a lower relaxation factor or just simply ignore it. This cannot be any farther than the truth. Ground stress redistribution and relaxation factor around underground openings is a physical phenomenon that must be properly considered in design and construction of underground space. Failing to do so would result in overdesign with unnecessary additional support requiring additional effort to install. The potential exposure to unnecessary safety risk from ignoring the relaxation of underground space

Proceedings DOI: 10.21467/proceedings.171
ISBN: 978-81-970666-7-2

Series: AIJR Proceedings
ISSN: 2582-3922

must be avoided by properly considering and including relaxation in the design.

#### *3.5.1 Engineering Model and Rock Mass Parameters for FEA*

In setting up the numerical model for the analysis of the tunnel support, an important consideration is the selection of appropriate computer program that suits the subject and purpose of the analysis. The most commonly used geotechnical FE program in Hong Kong is PLAXIS, applicable for design of excavation and lateral support (ELS) and tunnels in soft ground. However, for tunnelling in weak or jointed rock mass, the program *RS2* (previously *Phase2*) is generally more preferable.

The geotechnical parameters for the analysis and design for tunnel in jointed rock mass are heavily depending on the presence of discontinuities and orientation of jointsets, where joint spacings, widths, infillings are some of the critical factors that will alter the behavior of tunnel. The most widely adopted engineering model for jointed rock mass is the failure criterion developed by Dr Evert Hoek and Professor E.T. Brown (Hoek & Brown, 1980) for the design of underground excavation. This criterion was derived from the research findings on brittle failure of intact hard rock by Hoek, combined with model studies of jointed rock mass by Brown. In 2002, there was a major update where the correlation between the model criterion and the geological strength index (GSI) was further improved (Hoek, et al., 2002). There was another major update in 2018 where, among others, the paper discussed the use and application of GSI; expanded guidelines for the selection of disturbance factor *D*; and set out a recommended sequence of calculations for use in applying the Hoek-Brown criterion. The detail derivation of these equations will not be discussed, but in the following sections, the equations for Hoek-Brown failure criterion based on the 2002 version (Hoek, et al., 2002) will be presented, and the numerical values of the model parameters adopted for the FEA will be presented.

The generalized Hoek-Brown criterion can be expressed in the following equation:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \tag{11}$$

Where $\sigma_1'$ and $\sigma_3'$ are major and minor principal effective stresses at failure; and $\sigma_{ci}$ is the uniaxial compressive strength (UCS) of intact rock; $a$ and $s$ are material constant given by the following equations, and $m_b$ is a reduced value of the material constant $m_i$ determined by the following equation:

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \tag{12}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \tag{13}$$

$$m_b = m_i\, e^{\left(\frac{GSI-100}{28-14D}\right)} \tag{14}$$

The material constants $m_i$ and $\sigma_{ci}$ can be determined from triaxial tests. The recommended values of $m_i$ for various rock type is summarized in the following table. In Hong Kong, the recommended values of $m_i$ for granitic and tuff rock are 32 and 13 respectively.

The UCS of the jointed rock mass can be obtained by assuming $\sigma_3' = 0$ in the generalized H-B equation, which resulted in the following relationship:

$$\sigma_c = \sigma_{ci} s^a \tag{15}$$

The tensile strength of the jointed rock mass is obtained by assuming $\sigma_1' = \sigma_3' = \sigma_t$ in the generalized H-B equation, which resulted in the following relationship:

$$\sigma_t = -\frac{s\sigma_{ci}}{m_b} \tag{16}$$

| Rock type | Class | Group | Texture: Coarse | Medium | Fine | Very fine |
|---|---|---|---|---|---|---|
| SEDIMENTARY | Clastic | | Conglomerates* (21 ± 3)<br>Breccias (19 ± 5) | Sandstones 17 ± 4 | Siltstones 7 ± 2<br>Greywackes (18 ± 3) | Claystones 4 ± 2<br>Shales (6 ± 2)<br>Marls (7 ± 2) |
| | Non-Clastic | Carbonates | Crystalline Limestone (12 ± 3) | Sparitic Limestones (10 ± 2) | Micritic Limestones (9 ± 2) | Dolomites (9 ± 3) |
| | | Evaporites | | Gypsum 8 ± 2 | Anhydrite 12 ± 2 | |
| | | Organic | | | | Chalk 7 ± 2 |
| METAMORPHIC | Non Foliated | | Marble 9 ± 3 | Hornfels (19 ± 4)<br>Metasandstone (19 ± 3) | Quartzites 20 ± 3 | |
| | Slightly foliated | | Migmatite (29 ± 3) | Amphibolites 26 ± 6 | | |
| | Foliated** | | Gneiss 28 ± 5 | Schists 12 ± 3 | Phyllites (7 ± 3) | Slates 7 ± 4 |
| IGNEOUS | Plutonic | Light | Granite 32 ± 3<br>Granodiorite (29 ± 3) | Diorite 25 ± 5 | | |
| | | Dark | Gabbro 27 ± 3<br>Norite 20 ± 5 | Dolerite (16 ± 5) | | |
| | Hypabyssal | | Porphyries (20 ± 5) | | Diabase (15 ± 5) | Peridotite (25 ± 5) |
| | Volcanic | Lava | | Rhyolite (25 ± 5)<br>Andesite 25 ± 5 | Dacite (25 ± 3)<br>Basalt (25 ± 5) | Obsidian (19 ± 3) |
| | | Pyroclastic | Agglomerate (19 ± 3) | Breccia (19 ± 5) | Tuff (13 ± 5) | |

Figure 9: Recommended values of material constant $m_i$ from Hoek *(Hoek, 2023)*.

The modulus of deformation for the rock mass is given by the following equation with unit of *GPa*.

$$E_m = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \cdot 10^{((GSI-10)/40)} \tag{17}$$

The two parameters in the H-B criterion equations above, *GSI* and *D*, were introduced to the H-B failure criterion to estimate the reduced rock mass strength in different geological conditions. The *GSI* is more of a descriptive and qualitative representation of the rock mass, which can be estimated from the chart in Figure 10. The parameter *D* is disturbance factor that depends on the degree of damage at the perimeter of the tunnel from blasting works. It varies from 0 for undisturbed in-situ rock mass to 1.0 for very disturbed rock mass.

*GSI* has been introduced to consider site specific condition, and GSI values for any specific project must be adjusted accordingly. The shaded area in Figure 10 represents the most common range of rock mass quality and *GSI* in Hong Kong.

Due to the fact that ground condition is not discrete and different degrees of weathering is always observed, *GSI* for each project shall always be a range of values to represent any geological variation. The wider the range of *GSI*, the higher the variation of ground condition, and hence the higher the likelihood to encounter worse than expected ground condition. This has been in good agreement among practicing tunnel engineers and engineering geologist. To cover all possible ground condition, the support system for tunnel will be split into different classes (according to the $Q$ ranges determined from Section 3.2, and each support classes will be associated with a single *GSI* value for purpose of numerical analysis. And the *GSI* values are calculated from the following equation, first proposed by Hoek et al. (Hoek, et al., 1995), to be used as input to *RS2* FEA.

$$GSI = 9 \ln Q' + 44 \tag{18}$$

Proceedings DOI: 10.21467/proceedings.171
ISBN: 978-81-970666-7-2

Series: AIJR Proceedings
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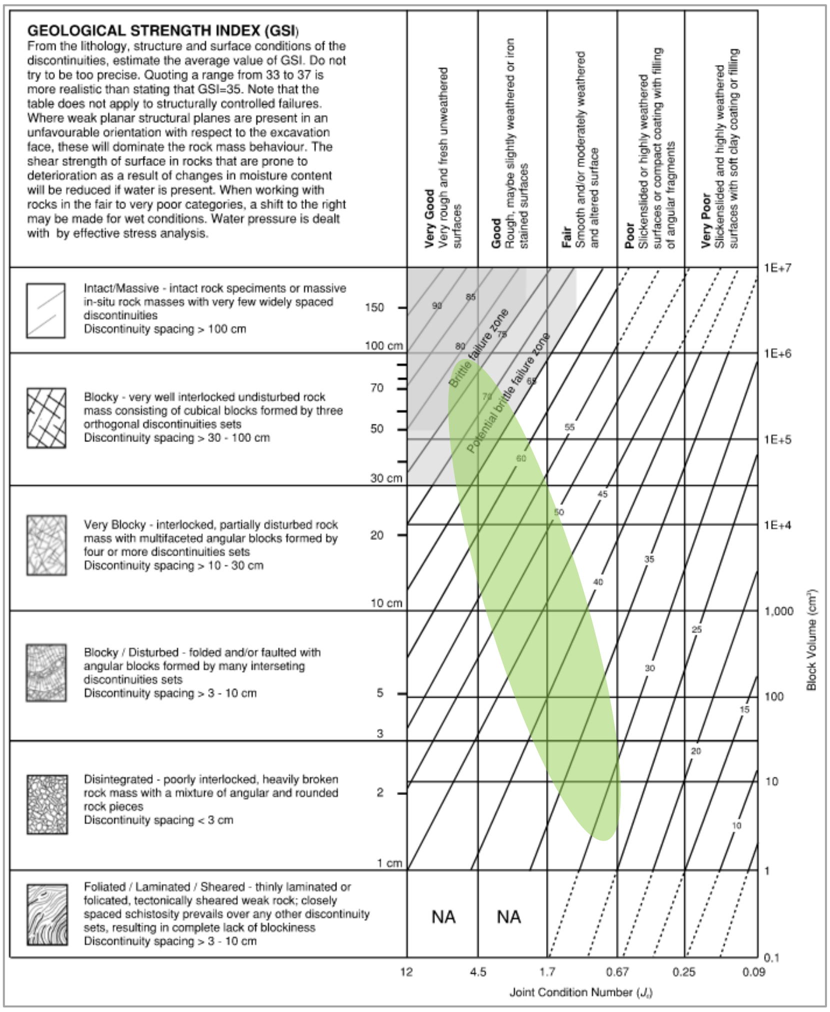


Figure 10: Reproduced geological strength index (GSI) by Cai et al. *(Cai, et al., 2004)* showing common range of GSI for rock in Hong Kong in green shade.

## *3.6 Structural Capacity of Tunnel Lining*

In the last step of tunnel lining design, the structural capacity of tunnel lining is being checked using a separate numerical model (step 6 in Section 3.5). The setting up of the model and stages of analyses are largely the same as the steps described in Section 3.5, except in this FE model, liner structure and rock bolts have been added to the excavation for purpose of design check and optimisation.

Due to the fact that sprayed concrete lining takes time to build up its strength, it is necessary to check intermediate stages or early strength of the sprayed concrete capacity. While concrete is gaining strength, ground load to be supported by the lining also increases gradually. Therefore, in the numerical model, early sprayed concrete strength of 1, 3, and 7-day (with corresponding relaxation values at 1, 3, and 7-day) have been checked, in addition to the final 28-day full strength. The following table summarized the material properties of sprayed concrete in accordance with EC2 (BSI, 2004).

The steps to determine the relaxation factors corresponding to the early strength of the sprayed concrete is the same as the six steps outlined in the previous section, except in step 3, the x-axis value of the load displacement profile chart (correspond to distance from excavated face) are adjusted to reflect the number of days, assuming that daily excavation round length is 1.0 m. The result of the structural check where the state of stresses is plotted against the flexural capacity of the lining (axial force-bending moment diagram) is attached in Figure 11 for each stage.

Table 2: Material properties of sprayed concrete lining

| Age (day) | Characteristic compressive strength $f_{ck}$ (MPa) | Elastic Modulus $E$ (MPa) | Poisson's Ratio | Corresponding relaxation factor of ground stress (%) |
|---|---|---|---|---|
| 0 | 0 | 3 | 0.2 | 80 |
| 1 | 9 | 13 | 0.2 | 90 |
| 3 | 15 | 16 | 0.2 | 96 |
| 7 | 21 | 18 | 0.2 | 99 |
| 28 | 30 | 22 | 0.2 | 100 |

It can be observed from the four charts that maximum axial force of the lining increases from approximately 900 kPa at day 1 to slightly less than 4,000 kPa at day 28. This increasing trend in the hoop force of the lining correspond well with the increase of stiffness of the lining; whereas the bending moment of the lining remains at or below approximately 50 kNm due to deformation of the lining, which is relatively flexible at 300 mm thick.

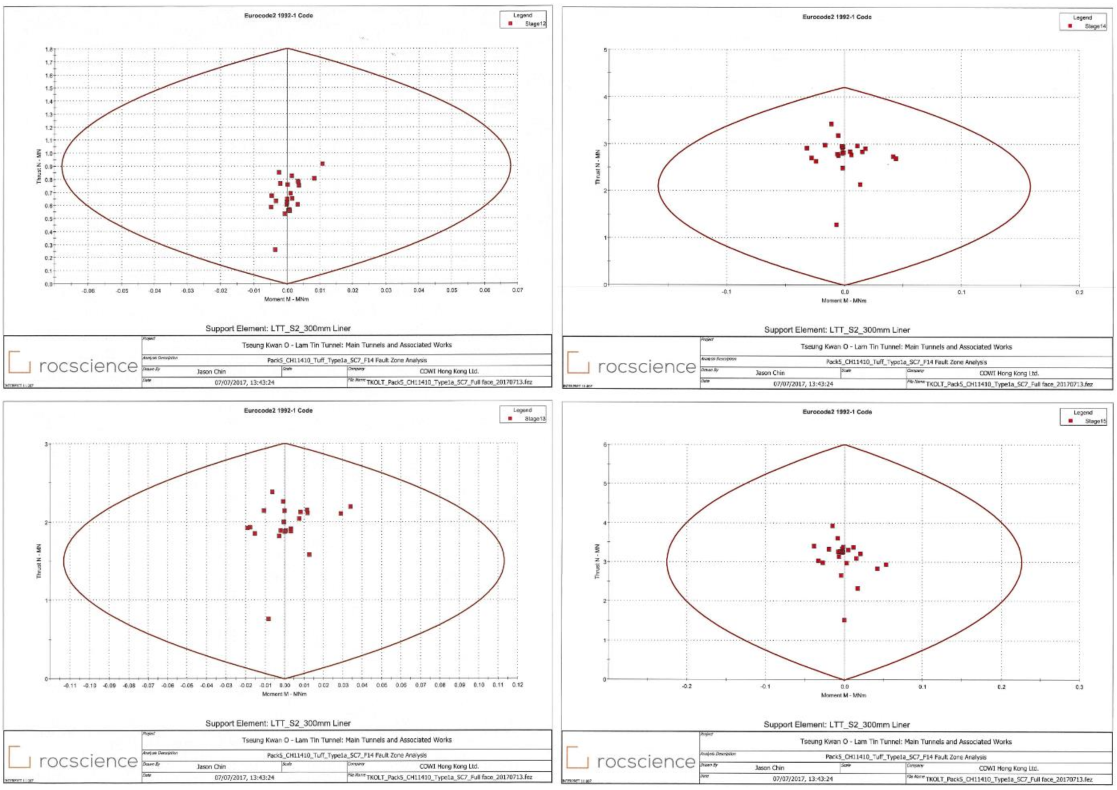


Figure 11: Structural interaction diagram for tunnel lining. Top left: 1-day; bottom left: 3-day; top right: 7-day; bottom right: 28-day strength.

The example and results presented are the final product after numerous rounds of discussion with the contractor in attempt to optimise the design. To provide a safe while economical tunnel support system, it is always more preferable to reduce the thickness of the lining because reduced lining means smaller excavation profile and lower mucking volume. This would have much more impact to the project program compared with adopting lower concrete strength or different excavation staging.

### *3.7 Unsupported Span and Stand-up Time*

The last step of tunnel support design is to determine the corresponding stand-up time of a tunnel with a certain span. In theory this stand-up time can be viewed as the time within which the tunnel will remain self-standing, after which the tunnel will collapse. However, I would advise young engineers that this overly simplified and seemingly easy to use chart is more than just drawing vertical (or horizontal) line to determine the corresponding value but should instead be applied cautiously and not blindly. This is being explained in subsequent paragraphs.

Proceedings DOI: 10.21467/proceedings.171
ISBN: 978-81-970666-7-2

Series: AIJR Proceedings
ISSN: 2582-3922

Figure 12 is a typical chart I replicated from the original chart from Bieniawski (Bieniawski, 1989) applicable to the example for this paper.

The term "stand-up time" was first used in the late 1950s and further developed and modified by Bieniawski (Bieniawski, 1993). To use this chart, the RMR value is first determined from the equation below proposed by Bieniawski (Bieniawski, 1989). For the example used in this paper, the adopted $Q$ value is 0.05 and $RMR = 17$ calculated from the following equation:

$$RMR = 9\ln Q + 44 \tag{19}$$

Along the line of $RMR = 17$ there are different combination of unsupported roof span and stand-up time, and different combinations can directly change the decision on method of construction, excavation cycle, tunnel reinforcement system, and duration of the reinforcement system installation. Short unsupported span (short tunnel excavation round length) and long stand-up time would favor construction method that could take longer to excavate (e.g., roadheader, mechanical breaking); while long unsupported span and short stand-up time could favor shorter than allowed excavation round length limited by the duration of sprayed concrete and rock bolts installation. Therefore, unsupported span and stand-up time must be discussed with the contractor to take into consideration the cycle time and available equipment for the works.

In this project, the unsupported length of 1.0 m (light blue circle in Figure 12) is simply too short for the excavation program, while the unsupported span of 2.5 m (light pink circle) is right at the upper bound line with very short stand-up time, which is impractical to carry out any physical works. It should be cautioned not to adopt the unsupported span right at the upper bound especially in ground with low rock mass quality. This is to avoid potential collapse due to unexpected adverse ground condition, overexcavation of tunnel face, and/or bad workmanship – all of which may result in larger span causing the actual point falling beyond the upper bound of the chart. As such, it is recommended to adopt a boundary line which is slightly lower than the upper bound line (blue line in Figure 12) to minimize the aforementioned risks and fixed the unsupported span at 2.0 m (red circle) with stand-up time of less than 1.0 hour specified.

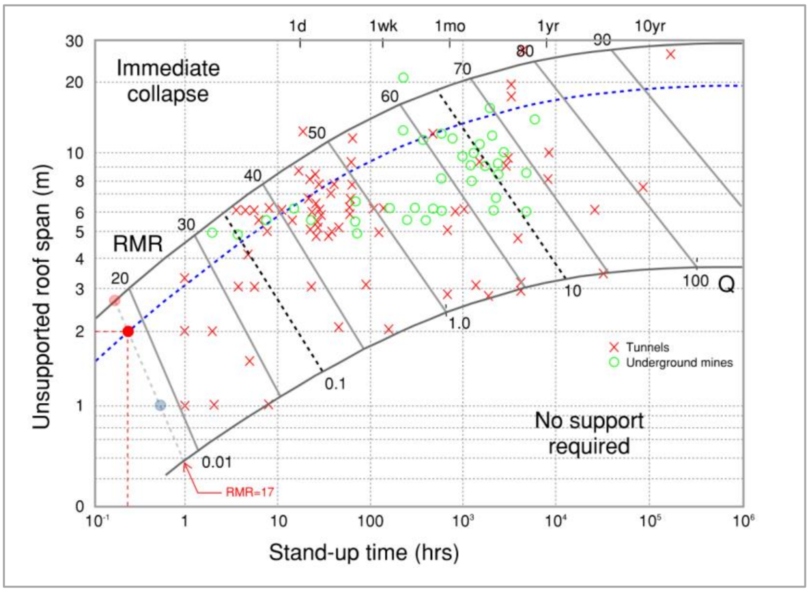


Figure 12: Reproduced unsupported roof span and stand-up time chart from Bieniawski *(Bieniawski, 1989)* showing the adopted excavation round length.

## 4 CONCLUSIONS

There are many published literatures on tunnel and cavern design in rock, the design process presented in this paper is by no mean the only one, but it is one that has been adopted and accepted in Hong Kong. The design process presented in this paper is intended to be a guideline for new or experienced professionals, and it is supplemented with background theories, detailed examples, practical experience and clarification on limitation and common misunderstandings.

### PUBLISHER'S NOTE

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## HOW TO CITE

Jason T.Y. Chin (2024). Tunnels and Caverns Design in Rock. *AIJR Proceedings*, 17-34. https://doi.org/10.21467/proceedings.171.3

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