**Faculty of Engineering and Computing**

**Department of Civil Engineering**

# The Reliability of Rock Mass Classification Systems as Underground Excavation Support Design Tools

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**Doctor of Philosophy**
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## DECLARATION

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university.

To the best of my knowledge and belief this thesis contains no material previously published by any other person except where due acknowledgment has been made.

The following publications have resulted from the work carried out for this degree.

**Peer Reviewed Conference Papers:**

1. **Ranasooriya, J.** and Nikraz, H. (2007) "Comparison of Rock Mass Classification Systems with an Analytical Method of Underground Excavation Design", *Sri Lankan Geotechnical Society's First International Conference on Soil & Rock Engineering*, Colombo, Sri Lanka, 5-11 August 2007, CD-ROM.

2. **Ranasooriya, J.** and Nikraz, H. (2007) "Comparison of Empirical Methods with Analytical Methods of Underground Excavation Design", *Common Ground, !0$^{th}$ Australia New Zealand Conference on Geomechanics*, Australian Geomechanics Society and the New Zealand Geotechnical Society, 21-24 October 2007, Brisbane, Vol. 2, pp. 436-441.

3. **Ranasooriya, J.** and Nikraz, H. (2008) "Tetrahedral Wedge Stability Under Empirically Derived Support", *First Southern Hemisphere International Rock Mechanics Symposium - SHIRMS 2008*, Australian Centre for Geomechanics and the International Society for Rock Mechanics, 16-19 September 2008, Perth, Vol. 1, pp. 633-440.

4. **Ranasooriya, J.** and Nikraz, H. (2008) "An Evaluation of Rock Mass Classification Methods Used for Tunnel Support Design", Proc ISRM Intl Symp, ARMS5, Tehran, Vol. pp. 819-826

5. **Ranasooriya, J.** and Nikraz, H. (2009) "An Assessment of Rock Mass Classification Methods Used for Tunnel Support Design", *International Conference on Rock Joints and Rock Masses*, Tucson, Jan 2009, CD-ROM.

6. **Ranasooriya, J.** and Nikraz, H. (2008b) "Evaluation of Empirically Derived Support for an Access Tunnel – a Case Study", *SINOROCK 2009 Rock Characterisation, Modelling and Engineering Design Methods, ISRM International Symposium on Rock Mechanics*, 19-22 May 2009, Hong Kong, Paper No. 264, CD-ROM.

**7.** Bahadorbeygi, D., **Ranasooriya, J.** and Nikraz, H. (2009) "Predicted and actual conditions of the Namroud project diversion tunnel". *Rock Engineering in Difficult Ground Conditions - Soft Rocks and Karst*, ISRM Regional Symposium EUROCK'2009, 29-31 October 2009, Dubrovnik-Cavtat, Croatia, Paper No. D001, CD-ROM (accepted for publication).

**Refereed Journal Papers:**

8. **Ranasooriya, J.** and Nikraz, H. (2009) "Reliability of the linear relationship between Rock Mass Rating (RMR) and Tunnelling Quality Index (Q)". *Australian Geomechanics*, Institution of Engineers Australia, June 2009.

9. **Ranasooriya, J.** and Nikraz, H. (2009) "On the linear relationship between Rock Mass Rating (RMR) and Tunnelling Quality Index (Q)". *International Journal of Mining and Mineral Processing*, Serials Publications, (to be published).

10. **Ranasooriya, J.,** Nikraz, H. and Myo Min Swe (2009) "Application of rock mass classification methods to the Klong Tha Dan project tunnels", *International Journal of Mining and Mineral Processing*, Serials Publications, (to be published).

Copies of these papers are presented in Appendix A.

**The Reliability of Rock Mass Classification Systems as Underground Excavation Support Design Tools**


## ABSTRACT

This thesis examines the reliability of rock mass classification systems available for underground excavation support design. These methods are sometimes preferred to rational methods of support design particularly if detailed information required for the latter mentioned methods is lacking. The classification approach requires no analysis of any specific failure mechanisms or the forces required to stabilise unstable rocks, yet, the support measures thus designed are considered to deal with all possible failure mechanisms in a rock mass.

Amongst the several rock mass classification methods developed for application in underground excavation engineering, two have stood out. These are known as rock mass rating (RMR) and tunnelling quality index (Q), introduced by Bieniawski (1973) and Barton et al. (1974), respectively. Over the years, the two methods have been revised and updated so as to improve their reliability as support design tools, yet the two methods are know to have limitations and their reliability has long been a subject of considerable debate. Nevertheless, attempts to assess their reliability in a systematic manner have been limited. Further, some practitioners in the field of rock engineering continue to use these methods as the sole methods of support design for underground rock excavations. The objective of thesis, therefore, is to contribute to a better understanding of the reliability of the two classification methods.

This study considered that the reliability of the RMR and Q methods can be assessed by comparing their support predictions with those derived by other applicable methods and also with the actual support installed. Such an assessment can best be carried out during excavation of an underground opening because representative data can be collected by direct observation of the as-excavated ground conditions and monitoring the performance of the support installed. In this context, the geotechnical data obtained during the construction of several case tunnels were reviewed and the two classification methods were applied. The effectiveness of their support

predictions was then evaluated against the potential failures that can be predicted by some of the applicable rational methods. Since the rock masses intersected in the case tunnels are jointed, mostly the structurally controlled failure modes were analysed. The support measures predicted by the two methods were compared with each other and with the actual support installed in the case tunnels. Further, the RMR and Q vales assigned to the case tunnels were correlated to observe any relationship between the two.

The study showed that the RMR and Q predicted support measures are not always compatible. In some circumstances, the two methods can either overestimate or under estimate support requirements.

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Thanks are also due to Mr D Bahadorbeygi for providing data from the Namroud project diversion tunnel in Iran reported as Case Study 8. For the Ramboda Pass tunnel in Sri Lanka (Case Study 10) the data were provided by Messrs L Gaspe and Weerakkody. The latter has also facilitated a site visit. Without their support the inclusion of this case study would not have been possible.

While employed full-time, I conducted this research mostly on evenings and weekends neglecting routine domestic matters. This might have caused some inconvenience to my son Moshan who was an undergraduate student at this engineering school at the time. He has been a great listener whenever I wanted to say anything about my work and a tremendous moral supporter throughout.

Finally, I wholeheartedly thank Curtin University for providing me the opportunity to achieve my goal at this late stage of my career. In fact it was my daughter Bonita who first entered Curtin from my family and completed Bachelors and Masters degrees. I simply followed in her footsteps back to the university.

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# CHAPTER 1

# INTRODUCTION

## 1.1 Background

Rock mass classification methods constitute an integral part of empirical design tools used in rock engineering, particularly in underground excavation support design and construction. They have always played an important role in predicting support requirements for excavations. The main reason for their application in underground excavation design can be directly related to the following:

- Rock, being a complex material with widely varying properties, presents unique design and construction problems that are not common in other engineering materials.
- Engineering excavations in rock, underground excavations in particular, often intersect more than one rock type, each with its own range of properties.
- Determination of the exact engineering properties of rock masses involved in an excavation project is virtually impractical, even with the sophisticated tools and techniques available at present.
- The stress field in a rock is governed by both the weight of the overlying rocks and several other factors including geological structure, the tectonic forces and geological history of the rock, and is often difficult to determine accurately.

Put simply, underground excavations are often planned and made in an environment with widely varying engineering properties and loading conditions that are not easily determinable with the currently available tools and techniques.

To undertake engineering design tasks in such complex and unknown environments, it is possible to apply theoretical solutions developed based on rock mechanics principles. However, this usually requires a considerable level of simplification of the environment by making simplifying assumptions. This means the theoretical solutions do not necessarily guarantee an accurate representation of the environment, unless a detailed and rigorous sensitivity analysis is undertaken taking into account its inherent variability, i.e. variation in rock mass properties, groundwater conditions and stresses in the rock mass. Such detailed assessments require access to accurate information on rock mass properties, groundwater and in situ stress conditions, together with high levels of experience and skills in the application of the theoretical methods. Further, these assessments are time consuming and costly.

In contrast, the empirical design methods based on rock mass classifications systems provide quick assessments of the support requirements for underground excavations at any stage of a project, even if the available geotechnical data are limited. Compared to the analytical methods the empirical methods do not require high levels of skills. Thus, unlike other disciplines of engineering such as structural or mechanical where engineering material properties and loading conditions are fairly well understood and can be controlled, the underground excavation industry tends to lean on empirical approaches such as rock mass classification methods, which provide a rapid means of assessing rock mass quality and support requirements.

To meet the industry's demand for rapid assessments of rock mass quality and support requirements for excavations, over the years, several classification methods have been developed. Two methods known as Rock Mass Rating (*RMR*) and Tunnelling Quality Index (*Q*), introduced by Bieniawski (1973) and Barton et al. (1974), respectively, have stood out. They are easy to use and can be applied from the preliminary planning stage through to the construction stage of a project. Applied within their limitations they serve as excellent means of communication between all parties involved in an underground excavation project. When used in conjunction with other applicable design methods, they are useful design tools, particularly in the early stages of a project.

Since their introduction some three and half decades ago, these two methods have been applied to various underground excavation projects throughout the world, particularly to rock masses and projects that are closely related to the conditions and circumstances for which these methods were originally developed, and a plethora of technical papers have been published on their successful application. The information provided in these publications is useful not only to users of the classification systems, but also to their creators. Much of this information has subsequently been added to the databases used in developing the classification systems so as to revise and improve them. Despite these, the two methods are known to have limitations, some of which have been reported by Einstein et al. (1983); Kaiser et al. (1986); Speers (1992); Palmstrom et al. (2000); Peck (2000); Stille & Palmstrom (2003); Palmstrom & Broch (2006): and Pells and Bertuzzi (2008).

In fact, the reliability of tunnel support designed using these methods has long been a subject of considerable debate. However, attempts to evaluate their reliability in a systematic manner have been few and far between and publications on case studies in which these methods are of limited use are at best rare. Hence the available literature on rock mass classification systems is somewhat biased towards their successful applications while case histories in which they are unsuccessful are not given much attention and are seldom reported. In this background, among some practitioners of rock engineering, there has been a tendency to overly rely on these methods without due regard for their limitations, and this could (and has) lead to potentially disastrous consequences in underground excavations. In recognition of the seriousness of this injudicious tendency, several eminent experts in the field of rock engineering have cautioned rock engineering practitioners by making the following comments:

> "*In essence, rock mass classifications are not to be taken as a substitute for engineering design. They should be applied intelligently and used in conjunction with observational and analytical studies to formulate an overall design rationale compatible with the design objectives and site geology.*"
>
> \- Bieniawski (1989) -

> "*The use of empirical design methods such as the RMR method and the Q method will lead to under designs, particularly with regard to the capacity of the bolts.*"
>
> \- Speers (1992) -

"*It is important to understand that the use of a rock mass classification scheme does not (and cannot) replace some of the more elaborate design procedures.*"

- Hoek, Kaiser and Bawden (1995) -

"*… none of the techniques has any solid scientific foundation and can quite clearly be dangerously misleading, if the potential failure mechanism is not identified within the classification system.*"

- Hudson and Harrison (1997) -

"*Neither the Q nor RMR systems apply to all rock masses. It is important that practitioners know what behaviour and what types of ground are covered by the classification system and that they become familiar with its database.*"

- Palmstrom, Milne and Peck (2000) -

"*The classification scheme approach does not always fully evaluate important aspects of a problem, so that if blindly applied without any supporting analysis of the problem, it can lead to disastrous results.*"

- Brady and Brown (2004) -

"*… Classification systems, and not least the Q system, may be useful tools for estimating the need for tunnel support at the planning stage, particularly for tunnels in hard rock and jointed rock masses without overstressing. There are, however, a number of restrictions that should be applied if and when the system is going to be used in other rock masses and in complicated ground conditions. So far such restrictions have not been much discussed in available literature.*"

- Palmstrom and Broch (2006) -

"*… the design correlations published in the various papers on the Q and RMR systems should be used with great caution in geological environments significantly different from those comprising the original case studies.*"

- Pells and Bertuzzi (2008) -

In spite of clear warnings, some practitioners continue to use rock mass classification methods as the sole method of support design for underground excavations. This may be partly attributed to the fact that, up to now, as pointed out by Palmstrom and Broch (2006), efforts to identify the limitations of the rock mass classification methods on a systematic basis have been limited. It is in this background that this research aims to contribute to a more detailed understanding of the limitations of the most commonly used rock mass classification systems at present. This understanding can lead to the development of improved guidelines for more reliable ways of applying the classification methods for design purposes and to form recommendations on where they have their best applications. The findings of this research provide insight for future research on this subject and are one step towards a more rationally based application of rock mass classification methods.

## 1.2 Objective and Scope of Work

The fundamental objective of this research was to assess the reliability of rock mass classification systems used for underground excavation support design. The study focused on the most widely used rock mass classification systems, *RMR* and *Q*, and their reliability when applied to tunnels excavated in jointed rock formations. More specifically, the objective of the research was to examine the reliability of the two classification systems under different jointed rock formations and project conditions, suggest improvements, if necessary, and highlight their limitations so that caution can be exercised when using them for underground excavation design.

The scope of the research was to fulfil the above mentioned objectives by applying the two classification methods to several case tunnels and compare their predictions with those of other relevant methods. Rock mass and project data for the application of the two methods were obtained from the published and unpublished literature and from study visits to underground excavations. More specifically, the scope of the research was to:

- Compile and review previous studies on the reliability of the two classification methods.
- Compare and evaluate the support requirements predicted by one classification method with those of the other.
- Compare the support requirements predicted by the two classification methods with those of the other methods applicable to jointed rocks.
- Evaluate the support predicted by the classification methods against the performance of the support installed in excavations.
- Compare the *RMR* and *Q* predicted support pressures with each other.
- Examine whether a reliable correlation exists between the two methods so as to confirm that the two methods lead to similar conclusions regarding the rock mass quality and the support predictions. If a reliable correlation exists the ratings of one classification method can be transformed to those of the other.
- Draw conclusions based on the findings of the research and highlight the limitations of the classifications and where they have their best applications.

The tasks undertaken for this research included a review of instability in underground excavations, an overview of the underground excavation support design methods, a literature review on the development, application and evaluation of the two rock mass classification methods, application of the two methods to several case tunnels, comparison of the support predicted by the two methods with those predicted by the analytical methods, followed by a comparison of the support predicted by the two methods with the support installed in the excavations.

In order to assess the adequacy of the support predicted by the classification methods, two analytical approaches were used to determine support requirements for the case tunnels. The two approaches were limit equilibrium analyses of rock block stability and numerical simulation of the rock mass behaviour around the tunnel.

The study covers the application of the *RMR* and *Q* methods to permanent excavations such as tunnels constructed for infrastructure development as well as access declines, drifts and drives excavated for mining mineral deposits. Applications of these methods to large mine openings such as stope voids, which are usually abandoned immediately after the extraction of ore, are not dealt with in this thesis.

## 1.3 Significance

The significance of this research is it highlights on a scientific basis some of the limitations of the two rock mass classification methods and provides a list of practical examples where these methods are of only limited use. On the basis of the findings of the research, rock engineering practitioners can avoid misuse and/or incorrect application of these methods for underground excavation support design.

## 1.4 Research Approach

The method of this research was case study driven. It was considered that, under a given set of conditions, the reliability of the classification methods derived support can be assessed by comparing them with those derived by other applicable methods and also with the actual support installed. It was also considered that such an assessment can be carried out more efficiently during excavation of an underground opening by close observation and monitoring of the intersected ground conditions. In this context, the geotechnical data obtained during the construction of several case tunnels were reviewed and the two classification methods were applied. The effectiveness of the support measures predicted by the two methods was then evaluated against the potential failures that can be predicted by some of the applicable theoretical methods. Both structurally controlled gravity driven failures and stress induced failures in jointed rocks were considered. The structurally controlled gravity driven failures were analysed using limit equilibrium methods of analysis and stress controlled failures were analysed using the numerical approach.

The structurally controlled failure modes considered in this study were tetrahedral rock wedges formed by three intersecting joints and the free surface of the excavation. Tetrahedral wedge analysis was undertaken using UNWEDGE software code (Rocscience, 2003), based on the block theory proposed by Goodman and Shi (1985). UNWEDGE provides an effective means of identifying all possible tetrahedral wedges in a rock mass, provided discontinuity orientations are known. The stability of kinematically unstable tetrahedral rock wedges identified in the rock mass is then assessed by limit equilibrium analysis, and the support measures required to stabilise potentially unstable rock wedges are then determined. The UNWEDGE analysis assumes that the geological discontinuities are ubiquitous. This is justifiable because the application of the two rock mass classification methods also assumed that the joints were ubiquitous in each sector (or structural domain) of the case tunnels. Beam failure was also analysed when horizontally bedded or laminated rocks were intersected in the case tunnels. Beam analysis was undertaken using suspended beam concept presented by Stilborg (1994) and Brady and Brown (2004).

For numerical simulation of the rock mass behaviour around the case tunnels, the two dimensional software package Universal Distinct Element Code (UDEC) (Itasca, 2004), based on the distinct element method, was used in which a rock mass is represented as an assembly of discrete blocks and discontinuities are viewed as interfaces between distinct bodies. UDEC simulates the response of jointed rock masses subjected to either static or dynamic loading and allows modelling of rock mass failure along discontinuities as well as through intact rock material.

The outcomes were developed by analysis of a case study database consisting of data representing approximately 7000 m of tunnels from ten projects in five different countries.

### 1.5 Structure of the Thesis

This thesis is divided into seven chapters as outlined below:

- Chapter 1 presents an introduction to the research highlighting the background, objectives, scope and methodology, and briefly outlines the structure of the thesis.

- Chapter 2 presents a discussion on the modes and mechanisms of instability in underground openings in rock.

- Chapter 3 outlines the support design methods available for preventing rock instability in underground excavations.

- Chapter 4 presents a brief review of the rock mass classification methods. Detailed discussions are presented on the structure of RMR and Q classification methods and their modifications.

- Chapter 5 reviews the previous studies on the reliability of the RMR and Q classification methods. These studies include previous applications, evaluations and correlation of the two methods.

- Chapter 6 presents the details of the application of the two classification methods and selected analytical methods to case studies.

- Chapter 7 presents the conclusions drawn from the present study and recommendations for future research.

# CHAPTER 2

# INSTABILITY IN UNDERGROUND EXCAVATIONS

## 2.1 Underground Excavations in Rock

Underground excavations in rock have played an important role in the development of human civilisations throughout the world. According to Bieniawski (1984) the oldest known underground mine in the world, Bomvu Ridge in Swaziland in South Africa, was in operation before 40 000 B.C. Here Neanderthal Man mined hematite, literally “blood stone”, which due to its colour was much prized for burial rites and personal decoration. Detailed accounts of historical developments in tunnelling provided by Sandstrom (1963), Beaver (1972) and Bieniawski (1984) indicate that underground excavations have been used for various purposes throughout recorded human history, and some have stood in serviceable conditions for several centuries. At present, the uses of underground space in rock are many, varying from simple stand-alone openings for exploration to large complexes of interconnected openings in three dimensions for commercial and public facilities and for mining underground ore bodies. Several different types of excavations are in use including tunnels, shafts, caverns, and stopes etc, the first being the most common.

Today, underground openings are made for both infrastructure development (civil applications) and the extraction of economically valuable mineral deposits (mining applications). In civil applications the most common excavations are tunnels constructed for highways, railways, water conveyance, sewer discharge, and utility corridors etc. Tunnels built for civil applications may be short and shallow beneath valley sides or urban areas (e.g. for roads, sewage etc), or very long and deep structures underneath major mountains or below the ocean floor. Caverns are also commonly used in civil applications for power stations, train and bus stations, shopping and sporting complexes, defence chambers, oil and gas storage facilities and nuclear waste repositories etc. Caverns of up to 62 m span are in use for public sporting facilities in Norway (Bhasin et al., 1993; Barton et al., 1994) and shafts are used for the access and ventilation of other subsurface openings and water supply to underground power stations etc throughout the world.

In mining applications, there are two main types of underground openings: (a) those that are intended to be stable while the ore is extracted from a particular area of a mine or during the operating life the mine, and (b) those that are created to produce broken-rock (ore) that is drawn off as the ground caves. The first type of excavations in mining includes tunnels and shafts to gain access to ore bodies and caverns for underground workshops, crib rooms, refuge chambers, temporary stockpiling of broken ore, and storage of explosives and other consumables for mining operations. The second type includes various forms of stopes created to produce broken-rock for subsequent removal from the mine. Some of these are backfilled with mine waste while others are left open for ever. In general, stopes can be abandoned once the extraction of ore is complete. In contrast, the tunnels, shafts and caverns are important infrastructure and must remain open for their expected operating life.

With such a vast range of underground excavations, many different kinds of construction and user requirements need to be addressed for successful design of a project. However, there are certain features common to all underground excavations in rock. One of the key features is that the rock within which an excavation is to be made is not usually accessible or directly observable until the construction is in progress. Often the designer will have to begin deliberations from information acquired from exploratory drill holes, shafts and galleries, which usually represent only a very small part of the conditions likely to be encountered in a project. Occasionally, when an existing installation is expanded it is possible to begin by direct observations at the site. There are other common features as well. Rock in which underground openings are constructed is initially stressed and openings cause changes in the stress field. Further, rocks often have pre-existing weakness planes or zones, along which rock block can move against each other after the creation of openings. Regardless of the purpose of the excavations, the effects of such features on them are generally similar, if not identical. The modes of rock instability and the mechanisms involved are common to all excavations depending on the condition of the rock and the excavation shape and dimensions.

In spite of the common features mentioned above, civil excavations generally differ from mining excavations. Often the location of mining excavations is fixed in and around the ore body to be mined. The location of a civil excavation on the other hand may be adjusted for convenience or economic reasons. Usually, the operating lives of civil excavations are much longer than those of mining excavations. Mining excavations are generally accessed only by a small number of individuals who are specifically trained to work in that environment and are familiar with the problems such as rock falls in underground excavations. In contrast, civil excavations are often used by the general public and others who are not skilled to deal with rock instability problems in underground excavations. The foremost difference is that the consequences of rock instability in civil excavations are vastly different to those of mining excavations. In a civil tunnel constructed for a major railway, for example, rigorous measures are required to prevent instability during its operating life, whereas in a mining excavation some instability may be tolerable providing that the condition of the excavation is inspected and assessed before entering. This is possible in mining projects because the regular users of these excavations are trained to do so.

## 2.2 Intact Rocks and Rock Masses

Rock is an assemblage of minerals formed by various geological processes in the earth's crust. The composition of mineral constituents in a rock can vary widely and their texture can be crystalline, fine grained or clastic. On the basis of mineralogy and texture, rocks can be divided into different groups but from a genesis point of view, they are divided into three main groups: igneous, metamorphic and sedimentary.

Rock usually contains geological structural features such as joints, beddings, foliations and faults etc which subdivide it into discrete blocks of different sizes and shapes. These structural features, also known as discontinuities or defects, possess the common characteristics of low shear strength and negligible tensile strength compared to the surrounding rock material. For engineering purposes the solid rock material bounded by various discontinuities is termed "intact rock" material. Intact rock pieces may range from several cubic millimetres to several cubic meters in size and their behaviour is generally elastic and isotropic. Rock when taken together with the discontinuities and their inherent characteristics is defined as the "rock mass". Figure 2.1, modified after Hoek and Brown (1980a), schematically illustrates the transition from intact rock to a heavily jointed rock mass.

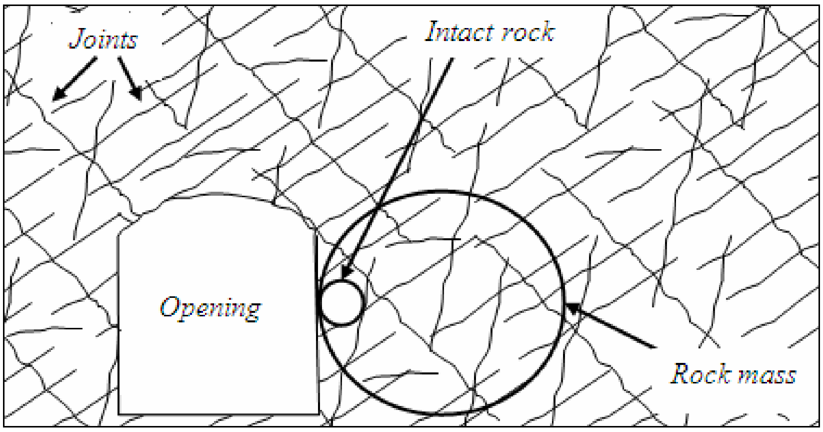

Figure 2.1 Intact rock and rock mass (modified after Hoek and Brown, 1980a)

## 2.3 Geological Structural Features

Rock masses invariably include numerous structural features or discontinuities formed from a wide range of geological processes, the most common of which present in a rock mass are joints, beddings, foliations and faults. Joint is a plane of weakness or break in the continuity of a body of rock along which there has been little or no displacement. Joints may be tightly healed or open with an aperture (opening) of a fraction of a millimetre to several centimetres. The joint openings may contain infill materials weaker than the parent rock. Faults are planes of shear failure with obvious signs of movement of the rock on either side of the plane. Fault surfaces are often striated and polished resulting from the shear displacement and the fault openings can be several millimetres to several meters wide. Faults usually contain crushed rock and clayey gouge. Bedding and foliation partings are planer features of significant surface extent in sedimentary and metamorphic rocks, respectively, and are created by a change in the size, orientation, shape or composition of the constituent mineral grains. These changes occur during the genesis of the sedimentary and metamorphic rocks. There are other structural features such as fractures, fissures, flow bands, shears and cleavage etc depending on the origin, mineralogy and geological history of a rock mass. Since joints are by far the most common geological structure, the term “joint” is sometimes used to include all types of discontinuities in a rock. From an engineering point of view the discontinuity orientation, persistence and spacing as well as their surface characteristics such as roughness, aperture, infilling material and waviness are of paramount importance, as these properties govern the strength and engineering behaviour of rock masses. Discontinuities, joints, bedding, cleavage and foliation in particular, occur in sets of more or less parallel members. Often, several sets can be present in a rock mass. An example of some of the discontinuities is presented in Figure 2.2.

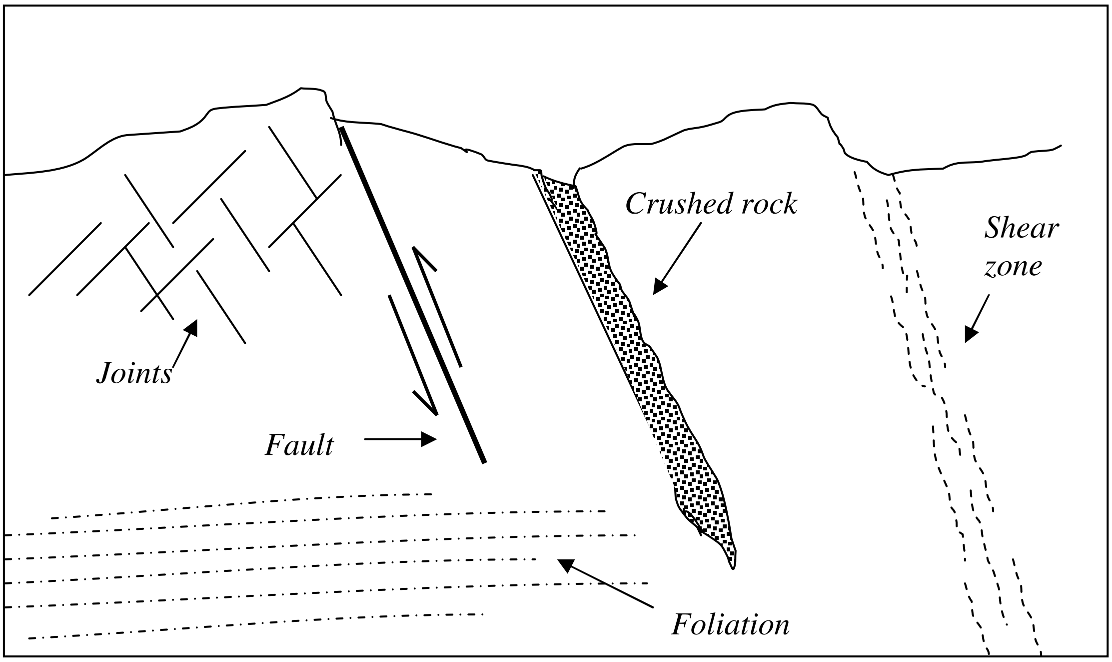


Figure 2.2 Example of geological structural features in rock

## 2.4 In Situ Stress

Rock masses in the earth’s crust are subjected to a stress field related to the weight of the currently overlying rocks, the present state of tectonic activity and the previous loading conditions caused by various geological processes throughout their history. By compiling and analysing the in situ stress measurement data from various different projects in several countries in the world, Hoek and Brown (1980a) observed that in general, the vertical stress component is related to the depth below surface. This is, however, not always the case due to the presence of structural features such as faults and folds, or local variations in rock material properties. It has also been observed that the ratio of the average of the horizontal stress components to the vertical stress varies with depth. At shallow depths the ratio is extremely variable and frequently greater than unity (Brady and Brown, 2004). With increasing depth, variability of the ratio decreases and approaches unity. Hudson and Harrison (1997) and Brady and Brown (2004) noted that several factors contribute to changes in the state of stress in a rock mass. These include surface topography, surface erosion, residual stresses, thermal effects, inclusions, tectonic stress and discontinuities. Often in discontinuous or jointed rocks, a highly variable stress distribution can be present. It should be mentioned here that as pointed out by Brady and Brown (2004), the virgin state of stress in a rock mass is not amenable to calculation by any known method, but must be determined experimentally.

## 2.5 Strength and Deformability of Rock Masses

As discussed in the preceding sections, a rock mass consists of intact rock material and naturally occurring discontinuities subjected to an initial state of stress. When an opening is created in a rock mass, the virgin state of stress is disturbed and a new stress field is induced. Further, the discrete blocks formed by the discontinuities, which were perfectly fitted and interlocked before the creation of the opening, may now be free to move or rotate. Depending on the induced stress magnitude and the degree of kinematical feasibility of discrete rock block movement, the behaviour of the rock mass surrounding the opening is governed by mechanical properties of the intact rock material or the individual discontinuities, or by the combination of the two. It is, therefore, important to understand the mechanical properties of intact rock materials, discontinuities and also of the jointed rock mass as a single composite system.

### 2.5.1 Strength and deformation of intact rock

Throughout the history of rock mechanics, samples of intact rock material have been tested under a variety of laboratory conditions and there is a vast amount of information on almost every aspect of intact rock behaviour. From the wealth of available information it has been known for several decades that the strength of intact rock increases with increasing confinement. Under unconfined or low confining stresses, strong intact rock suffers *brittle* failure, which may occur suddenly and catastrophically with little or no permanent deformation before failure. When confining pressures are high, the material can sustain permanent deformation without losing its ability to resist load. Failure of intact rock in this manner is said to be *ductile*. Ductility of competent rock increases with increasing confining pressures, but can also occur in weathered rocks, some weak rocks and heavily jointed rock masses under moderate stress. Between fully brittle and fully ductile behaviour, there is a transition known as *brittle-ductile transition*. Idealised stress-strain curves for *brittle*, *ductile* and *brittle-ductile transition* are shown in Figure 2.3.

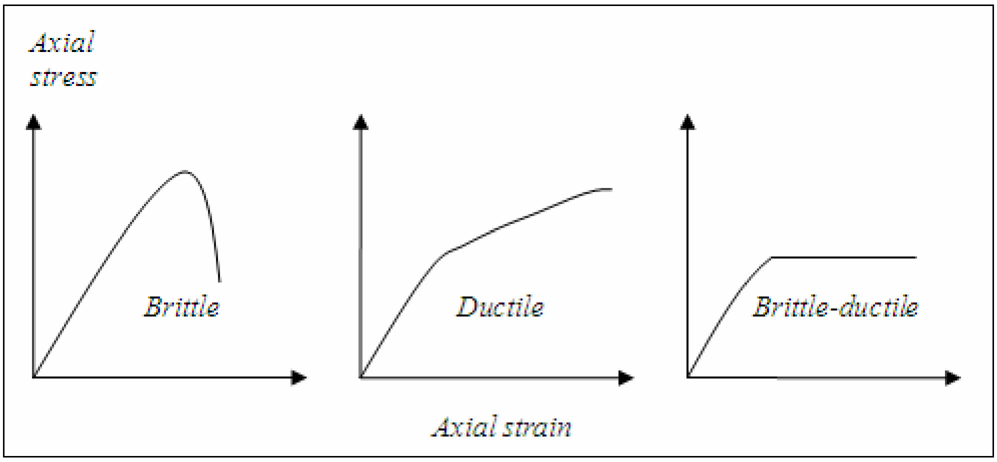


Figure 2.3 Stress-strain curves for *brittle*, *ductile* and *brittle-ductile transition*

Since intact rock may fail by crushing (compression), tension or shearing, its strength may be expressed in terms of compressive, tensile or shear strength, with the most commonly used indicator being the uniaxial compressive strength (UCS) determined by testing cylindrical rock core specimens under axial loading with zero confining stress. Tensile strength of intact rock can be determined by uniaxial extension of a similar cylindrical core sample or by indirect tests such as the Brazilian test and bending tests. Shear strength of rock material may be determined by triaxial testing of cylindrical rock core specimens (i.e. with an applied confining stress) or direct shear testing of weak rock samples. UCS is used in rock mass classification systems and as a basic parameter in the rock mass strength failure criterion to be discussed later in this section and in Section 2.5.3.

For analysing the stability of underground excavations that are subjected to medium to high stress regimes it is also important to understand and be able to predict the response of the rock under confining stresses. Two other important mechanical properties of rock material which influence the response of a rock under stress are elastic constants, i.e. Young's modulus (static modulus of elasticity) $E$, and Poisson's ratio $v$. These are measured during UCS tests and the methods used are identical to those used for concrete samples.

The strength of intact rock under different loading conditions can be described by the Coulomb criterion or Griffith criterion, discussed in detail by Jaeger (1972) and Jaeger and Cook (1976), with the Coulomb failure criterion being the most widely accepted of the two. The Coulomb shear strength criterion expresses the relation between the shear stress and the normal stress at failure. The shear strength $\tau$ that can be developed on a failure plane through the intact rock is given as

$$\tau = c + \sigma_n \tan \Phi \tag{2.1}$$

where, $c$ is cohesion, $\sigma_n$ is the normal stress acting on the failure plane, and $\Phi$ is the angle of internal friction. By stress transformation the uniaxial compressive strength $\sigma_c$ and tensile strength $\sigma_t$ of the rock material can also be related to the shear strength as given below:

$$\sigma_c = \frac{2c \cos\phi}{1 - \sin\phi} \tag{2.2}$$

$$\sigma_t = \frac{2c \cos\phi}{1 + \sin\phi} \tag{2.3}$$

The Griffith crack criterion expresses the uniaxial tensile strength, $\sigma_t$, in terms of the strain energy required to propagate micro-cracks, and expresses the UCS in terms of the tensile strength. The formula for tensile failure is

$$\sigma_t = (2E\alpha/\pi C)^{0.5} \tag{2.4}$$

where, $E$ is the Young's modulus of the uncracked material, $\alpha$ is the surface energy per unit area of the crack surface, $C$ is half the initial crack length. This criterion can also be expressed in terms of the shear stress and normal stress acting on the plane containing the major axis of the crack, but it does not lend itself to accurately predict the uniaxial compressive strength of rock material. The Griffith crack theory has been modified by several researchers to overcome its limitations and comprehensive discussions on these modifications are provided by Jaeger (1972) and Jaeger and Cook (1976). Despite its modifications the theory is not in wide use at present.

Following on from the Griffith theory, Hoek and Brown (1980a, 1980b) proposed an empirical failure criterion for rock, Hoek-Brown failure criterion, based on a review of published information on intact rock strength and a trial and error search for a relationship that best describes the response of a rock to the principal stresses acting on it. Based on experience using this criterion on a number of projects, an updated version was presented by Hoek and Brown (1988) and a modified criterion was published by Hoek et al. (1992). The criterion for intact rock expressed in effective stress terms (Hoek et al., 1995) is

$$\sigma_1^{'} = \sigma_3^{'} + \sigma_c \left( m_i \frac{\sigma_3^{'}}{\sigma_c} + 1 \right)^{\frac{1}{2}} \tag{2.4}$$

where, $\sigma_1^{'}$ is the major principal effective stress, $\sigma_3^{'}$ is the minor principal effective stress, $\sigma_c$ is the uniaxial compressive strength of intact rock, $m_i$ is a material constant for the intact rock. In the original version of the criterion, for intact rock $\sigma_c$ and $m_i$ are determined by linear regression analysis of the results of triaxial tests. For situations where triaxial testing programmes are prohibitive, $\sigma_c$ and $m_i$ are estimated from the tables, which relate $\sigma_c$ and $m_i$ to rock types based on the analysis of published strength data. The relevant tables are provided by Hoek et al. (1995); Marinos and Hoek (2001) and Hoek (2009).

### 2.5.2 Strength and stiffness of discontinuities

For most practical rock engineering work, it is generally assumed that discontinuities have zero or negligible tensile strength and that it is the shear strength of the discontinuities that is important. In discontinuous rock masses, conditions for rock blocks sliding along discontinuities are governed by the shear strength. The shear strength of planar discontinuities can be described by the Coulomb shear strength criterion given in equation 2.1. However, it is generally assumed that the shear strength of discontinuities is a function of the angle of friction only rather than both the friction and cohesion, and the Coulomb equation is reduced to that given in equation 2.5:

$$\tau = \sigma_n \tan \Phi \qquad (2.5)$$

Since discontinuities are not perfectly planar and are often wavy and have surface roughness (asperities), the Coulomb criterion has some limitations. When sheared at low normal loading conditions, the roughness causes the discontinuity surface to dilate (or ride up on the asperities) giving an additional frictional component. According to Patton (1966) this additional friction can be approximated to the asperity angle $i$ and the shear strength is then expressed in terms of total friction as given by

$$\tau = \sigma_n \tan (\Phi_b + i) \qquad (2.6)$$

where, $\Phi_b$ is basic friction angle of the surface and $i$ is the asperity angle measured from the horizontal. This equation is valid at low normal stresses, but at high normal stresses the discontinuity opening will be tightly closed and asperities tend to break off resulting in a shear strength behaviour which is more closely related to the intact material strength than to the frictional characteristics of the surface. The effects of discontinuity roughness and waviness were further studied and different failure criteria have been proposed by several workers including Ladanyi and Archambault (1970), Jaeger (1971), McMahon (1985) and Papaliangas et al. (1996). These criteria have their own merits and limitations.

Barton (1976) and Barton and Choubey (1977) proposed that the Coulomb equation modified by Patton (1966) can be further modified to include discontinuity surface roughness and compressive strength of the discontinuity wall rock and rewritten as given below:

$$\tau_p = \sigma_n \tan \{ \Phi_b + JRC \ \mathrm{Log}_{10}(JCS/\sigma_n)\} \tag{2.7}$$

where $\tau_p$ is joint shear strength; $\sigma_n$ is joint normal stress; $\Phi_b$ is basic (or residual) friction angle of intact rock material; *JRC* is joint roughness coefficient; and *JCS* is joint compressive strength. *JRC* is a number representing the discontinuity surface roughness and waviness. Barton and Choubey (1977) provided guidelines for determining *JRC*. Barton (1987) also published a table relating *Jr* (joint roughness term used in the Q system) to *JRC* as shown in Figure 2.4. Barton and Bandis (1990) suggested that *JRC* can also be estimated from a simple tilt test in which two matching discontinuity surfaces are tilted until one slides on the other. *JCS* is estimated by the methods suggested by ISRM (1978). Barton and Bandis (1982, 1990) showed that *JRC* and *JCS* are scale dependent and provided methods for scaling these parameters with increasing physical dimension. Although subject to conjecture, these methods are often used when expensive and complicated laboratory testing is prohibitive.

For discontinuities with infilling materials, the shear strength is a function of among other things, the strength and deformability of infill material and the strength and planarity of discontinuity surface. For major discontinuities with a thick layer of infilling and no rock wall contact during shearing the shear strength of infilling material should apply. For planar surfaces, a thin coating of clay could result in a significant reduction in shear strength compared to that of a clean discontinuity with wall-to-wall contact.

For most practical applications many of the stability analysis methods calculate factor of safety against sliding using the shear strength parameters expressed in terms of the Mohr-Coulomb cohesion, *c*, and friction, *Φ*, defined in Equation 2.1. It is therefore necessary to transform the shear strength determined by none linear relationships such as Equation 2.7 into cohesion and friction angle. This can be carried out using a spreadsheet program such as that proposed by Hoek et al. (1995). Such programs can be used to estimate cohesion and friction for a range of normal stress values.

| Description | Profile | $J_r$ | *JRC* 200mm | *JRC* 1 m |
|---|---|---|---|---|
| Rough | Stepped | 4 | 20 | 11 |
| Smooth | Stepped | 3 | 14 | 9 |
| Slickensided | Stepped | 2 | 11 | 8 |
| Rough | Undulating | 3 | 14 | 9 |
| Smooth | Undulating | 2 | 11 | 8 |
| Slickensided | Undulating | 1.5 | 7 | 6 |
| Rough | Planar | 1.5 | 2.5 | 2.3 |
| Smooth | Planar | 1.0 | 1.5 | 0.9 |
| Slickensided | Planar | 0.5 | 0.5 | 0.4 |

Figure 2.4 Relationship between *Jr* in the *Q* system and *JRC* for 200 mm and 1000 mm samples (after Barton, 1987)

*2.5.2.1 Stiffness of discontinuities*

Similar to the effect of elastic constants on the deformation of intact rock material the deformation of discontinuities is controlled by their normal and shear stiffness given in units of MPa/m. These are important parameters for numerical analysis of rock mass behaviour under stress. Both normal stiffness and shear stiffness are dependent on discontinuity roughness, wall strength and infilling materials. Determination of joint stiffness is difficult. However, results of experimental data and empirical relationships for estimating discontinuity stiffness have been reported by several authors including Bandis et al. (1983), Bandis (1990), Nelson and Kanji (1990), Yoshinaka and Yamabe (1986) and Rechitskii (1998).

### 2.5.3 Strength of rock masses

The strength of composite rock masses has not been investigated as frequently as the strength of intact rock and discontinuities. However, it is generally known that the strength of a rock mass is a function of the strength of intact rock and discontinuities and is significantly reduced with increasing sample size as more and more discontinuities will be included in the composite. This has been verified for laboratory scale samples by Hoek and Brown (1980a) and for field scale testing on coal pillars by Bieniawski (1968). It has been also established that rock masses with only a single discontinuity may fail preferentially along the surface of the discontinuity depending on the direction of loading with respect to its orientation. When the number of discontinuities increase, the rock blocks will have more than one direction to move or rotate and when the rock mass is heavily jointed, its behaviour can be approximated to an isotropic material, having no preferential direction of weakness. Since the determination of strength of a rock mass is difficult and theoretical models for describing the strength of a rock mass are very scarce, empirical approaches such as the correlations between rock mass quality and the Coulomb shear strength parameters (e.g. Bieniawski, 1976; Laubscher, 1990; Laubscher and Jakube, 2001; and Barton, 2002) may be used, but with significant limitations.

In the absence of reliable theoretical models for describing the strength of a rock mass, the most commonly used empirical failure criteria is that introduced by Hoek and Brown (1980a, b) and subsequently modified by Hoek (1983), Hoek and Brown (1988), and Hoek et al. (1992) in order to meet the needs of users who were applying it to problems that were not considered when the original criterion was developed. Based on the experience acquired in practical application, Hoek and Brown (1997) changed the criterion and introduced new elements. Further updates and clarifications of the criterion were provided by Marinos and Hoek (2000), Hoek et al. (2002) and Brown (2003). The current generalised version of the Hoek-Brown failure criterion for rock masses given by Hoek (2009) is:

$$\sigma_1' = \sigma_3' + \sigma_c \left( m_b \frac{\sigma_3'}{\sigma_c} + s \right)^a \tag{2.8}$$

where, $\sigma_1$ is the major principal stress at failure, $\sigma_3$ is the minor principal stress applied to the specimen, $\sigma_c$ is the UCS of intact rock material in the specimen, $m_b$ and $s$ are constants which depend upon the properties of the rock and upon the extent to which it has been broken before being subjected to the stresses $\sigma_1$ and $\sigma_3$, with $s = 1$ for intact rock materials. In the original version of the Hoek-Brown failure criterion the $m_b$ and $s$ were related to RMR and Q values and guidelines were provided on how to estimate them. However, in recognition of the problems associated with the use of RMR and Q for estimating these constants, a new classification called the Geological Strength Index (GSI) (Hoek, 1994; Hoek, Kaiser and Bawden, 1995; Hoek and Brown, 1997; Hoek, Marinos and Benissi, 1998; Marinos and Hoek, 2001) for estimating the $m_b$ and $s$ was developed. A major revision was carried out by Hoek et al. (2002) in order to smooth out the curves, necessary for the application of the criterion in numerical models and to update the methods for estimating Mohr Coulomb parameters.

In determining constants $m_b$, $s$ and $a$, Hoek et al. (2002) took into account the influence of blast damage on the near excavation surface rock properties as follows:

$$m_b = m_i \exp\left( \frac{GSI - 100}{28 - 14D} \right) \tag{2.9}$$

$$s = \exp\left( \frac{GSI - 100}{9 - 3D} \right) \tag{2.10}$$

$$a = \frac{1}{2} + \frac{1}{6}\left( e^{-GSI/15} - e^{-20/3} \right) \tag{2.11}$$

where, D is a factor depends on the degree of disturbance due to blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses. D applies only to the blast damaged zone and it should not be applied to the entire rock mass. For example, in tunnels the blast damage is generally limited to a 1 to 2 m thick zone around an excavation. Guidelines for the selection of D are presented by Hoek et al. (2002). The GSI values for different rock masses are selected using the charts (i.e. Figure 2.5) provided in references cited earlier. The Hoek-Brown failure criterion is generally recommended for intact rock and moderately to heavily jointed rock masses.

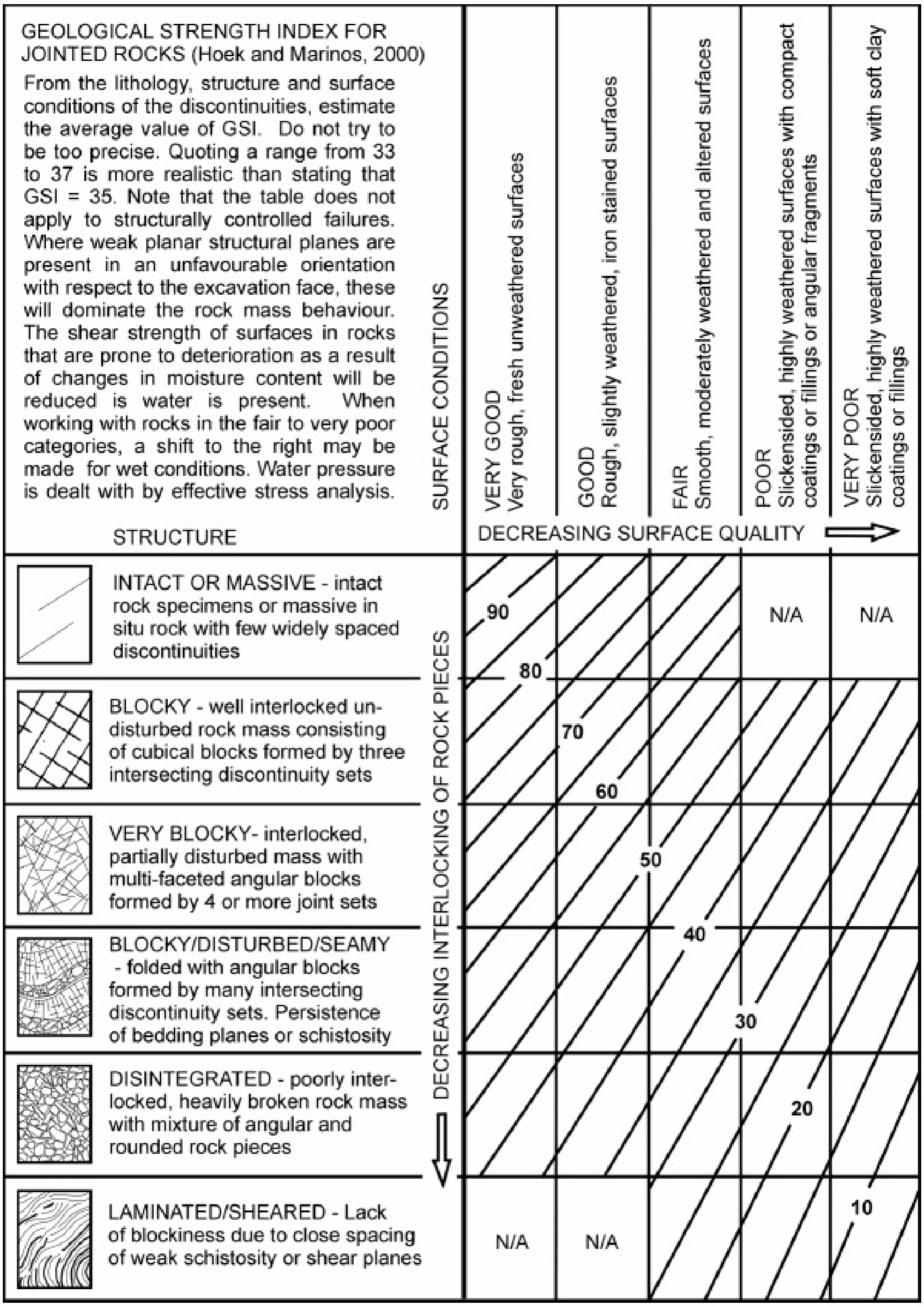


Figure 2.5 GSI chart for blocky rock masses

### 2.5.4 Deformability of rock masses

Deformability of a rock mass is a function of the elastic modulus of intact rock material and the stiffness of the discontinuities. For rock masses with a regularly spaced single set of parallel discontinuities, it is possible to calculate elastic constants for an equivalent continuous material representative of the rock mass. The formula given by Goodman (1989) for the modulus of elasticity $E_m$, when loading is normal to the discontinuities is

$$\frac{1}{E_m} = \frac{1}{E} + \frac{1}{KnS} \tag{2.11}$$

where, $E$ is elastic modulus of intact rock, $K_n$ is normal stiffness of discontinuities and $S$ is discontinuity spacing. The formula for shear modulus (or modulus of rigidity) $G_m$ when loading is parallel to the orientation of the discontinuities is

$$\frac{1}{G_m} = \frac{1}{G} + \frac{1}{KsS} \tag{2.12}$$

where, $G$ is shear modulus of intact rock material, $K_s$ is shear stiffness of discontinuities and $S$ is discontinuity spacing. Similar solutions for cases involving more than one set of discontinuities are given by Amadei and Goodman (1981) and by Gerrad (1982). If the rock mass is highly jointed, the mathematics associated with the derivation of the relevant formulas becomes complex and the data required to apply these solutions are not available in practice. In these cases, it is common to determine $E_M$ as the modulus of deformation from in situ compression tests. The term modulus of deformation signifies that the value $E_M$ is calculated from the data of the loading portion of the load-deformation curve using both elastic and permanent deformation.

The rock mass modulus of deformation can also be estimated from empirical relations. Bieniawski (1978) showed that $E_M$ in GPa can be estimated from $RMR$ value when the latter is greater than 50 using the following formula

$$E_M = 2\ RMR - 100 \tag{2.13}$$

For rock masses with $RMR$ < 50, based on the work of Serafim and Pereira (1983), the formula given by Bieniawski (1989) is

$$E_M = 10^{(RMR\text{-}10)/40} \tag{2.14}$$

Hoek and Brown (1997) proposed a correction for Equation 2.14 by introducing average UCS of intact rock material $q_c$ in MPa as given in Equation 2.14a, where $q_c \leq 100$ MPa.

$$E_M = (q_c/100)^{1/2}\ 10^{(RMR\text{-}10)/40} \tag{2.14a}$$

For dry and weak rock masses in underground excavations at depths exceeding 50 m where modulus of deformation dependent on the confining pressure due to overburden, Verman (1993) also proposed a correction as given in Equation 2.14b.

$$E_M = 0.3\ \mathrm{H}^{\alpha}\ 10^{(RMR\text{-}20)/38} \tag{2.14b}$$

where, α=0.16 to 0.30 (higher for poor rocks) and H=depth in metres (≥50m).

The Q system can also be used for estimating in situ modulus of rock mass deformation ($E_M$). Barton et al. (1980) and Barton (1995) provided the following formulas for estimating $E_M$ (in GPa) when $Q$ >1 and $Q$ <1, respectively

$$E_M = 25 \log Q\ \ (when\ Q > 1) \tag{2.15}$$

$$E_M = 10Q_c^{1/3}\ (when\ Q < 1) \tag{2.16}$$

where $Q_c = (Q\sigma_c)/100$ with $\sigma_c$ being the compressive strength of intact rock substance. Hoek et al. (2002) also proposed a somewhat complex empirical relationship between $E_M$ and $GSI$ as given in Equation 2.17:

$$E_M = \left(1 - \frac{D}{2}\right)\sqrt{\left(\frac{\sigma_c}{100}\right)} 10^{(GSI-10)/\ 40} \tag{2.17}$$

Hoek and Diederichs (2006) re-examined the existing empirical methods for estimating rock mass deformation modulus and found that the following equation gave the best fit to the reliable data available at present:

$$E_M = E_i\left(0.02 + \frac{1 - D/2}{1 + e^{((60+15D-GSI)/11}}\right) \tag{2.18}$$

where, D is a factor depends on the degree of disturbance due to blast damage and stress relaxation, as defined earlier. These formulas relate rock mass classification indices to measured static deformability values of rock masses that show considerable scatter. They cannot always be expected to provide estimates of $E_M$ for all rock masses.

## 2.6 Rock Mass Instability in Underground Excavations

When an excavation is created in a rock mass, the in situ stress field is disturbed and a new stress field is induced around the excavation. If the induced stresses are appreciably lower than the intact rock strength, rock mass instability is possible only through the geological structural features, which have low shear strength and negligible tensile strength compared to the intact rock material. In such situations the main stability concern is the dislodgement of rock blocks formed by the discontinuities, primarily under the influence of gravity with the in situ stress field playing only a secondary role.

In high in situ stress conditions, discontinuities are usually rare or tightly closed together the excavation induced stresses could be high enough to cause failure of the intact rock material. Thus, instability in underground excavations in rock can arise from two broad categories of rock mass failures:

- structurally controlled gravity driven failures
- strength controlled stress driven failures

While the above two categories of failure may be considered to represent most of the instability problems encountered in underground excavations, there are situations where rock mass instability is actually a combined effect of the geological structure, intact rock strength, stress field and gravity. Nevertheless, traditionally, instability in underground excavations has been dealt with under the two categories of failures mentioned above. There are also other categories of instability caused directly by the presence of large volumes of water. These situations are usually dealt with on a case by case basis as they are encountered during construction.

In the following sections the two categories are briefly described. This is followed by a brief discussion on the failures caused by the combined effect of geological structure, intact rock strength, stress field and gravity. The effects of water on the stability of underground excavations are also briefly outlined. In discussing the instability in underground excavations, emphasis is placed on the failure modes observed in the case studies used in this research.

### 2.6.1 Structurally controlled gravity driven failures

In low in situ stress conditions, as in near surface hard rock excavations, the stability of underground openings is significantly influenced by the geological structural features or discontinuities of the rock in which the opening is excavated. The significance of discontinuities is that they are planes of weakness in much stronger intact rock so failure tends to occur preferentially along these surfaces. The properties of discontinuities that may influence stability include orientation, persistence (continuity), spacing, aperture and their surface characteristics such as roughness, waviness, weathering and infilling.

The structurally controlled instability in underground openings can be defined as the dislodgement of rock blocks, prisms, wedges, beams and slabs etc from the roof and walls, primarily under the force of gravity. Other forces such as in situ stress, blast vibration, seismic activity, groundwater pressure and the presence of swelling or expansive infilling material in discontinuities can also contribute. The pieces of rock that dislodge from the excavation periphery are formed by interesting discontinuities in the rock mass and can be of any shape and size. Two modes of structurally controlled instability are common: tetrahedral wedge and beam (or slab) failures. Polyhedral wedge and triangular prism failures are also common depending on the arrangement of the discontinuities in the rock mass.

#### *2.6.1.1 Wedge failure*

In jointed rocks the most common mode of failure is the collapse of tetrahedral wedges formed by three intersecting discontinuities and the free surface of the excavation. The size and shape of the wedges depend on the size, shape and orientation of the opening and the orientation, spacing and persistence of the discontinuities. Two mechanisms are possible in tetrahedral wedge failure: falling and sliding. Falling is possible only from the roof of an excavation whereas sliding is possible from either the roof or the sidewalls. Falling occurs when a wedge detaches from the surrounding rock mass without sliding on any of the bounding discontinuities. Sliding can occur either on one of the bounding discontinuity planes or on a line of intersection of two discontinuity planes dipping into the excavation.

In order for sliding failure to occur the wedge must overcome the shear strength of the discontinuity plane(s) on which the wedge is sliding. In the case of gravity driven falling and sliding, rock wedges usually move as rigid blocks without any significant deformation of the wedge. A three dimensional illustration of potentially unstable tetrahedral wedges in the roof and wall of an excavation is presented in Figure 2.6. A wedge fall example is shown in Figure 2.7.

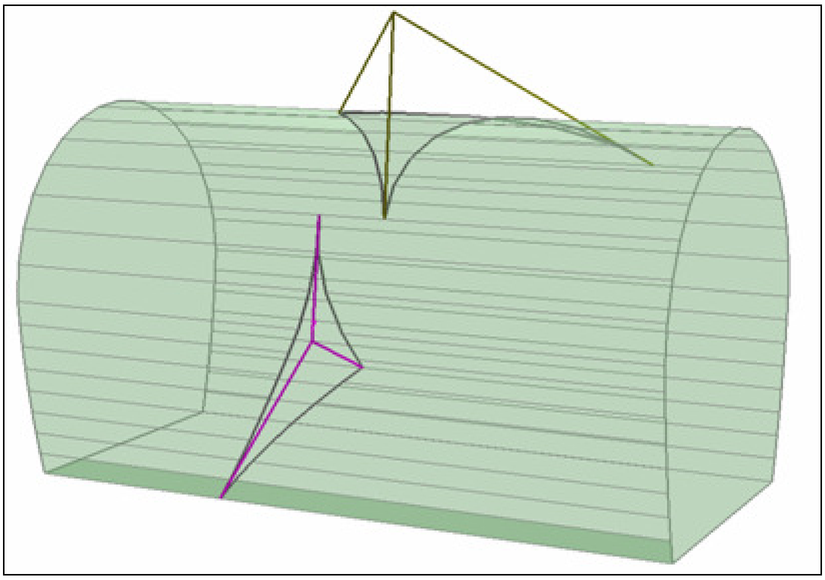

Figure 2.6 Potentially unstable tetrahedral rock wedges

Figure 2.7 Wedge fall from roof near the portal of an excavation

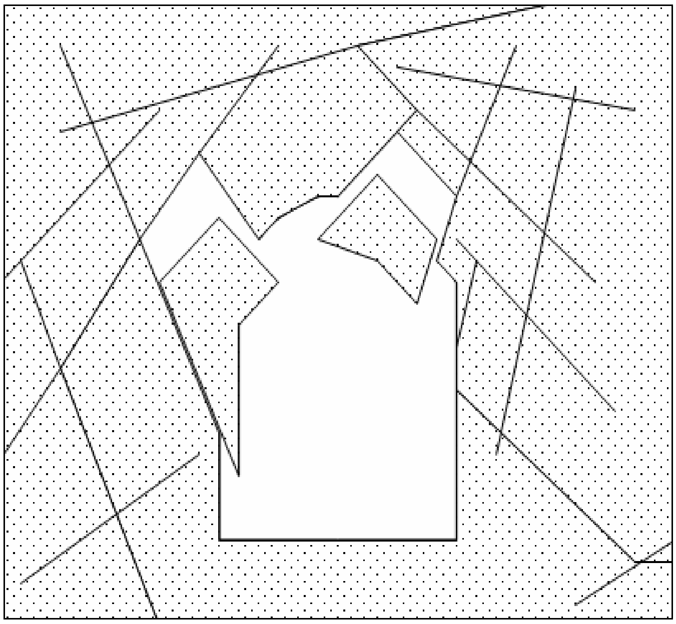

Figure 2.8 Structurally-controlled failure of polyhedral rock blocks

As mentioned earlier, in addition to the tetrahedral wedge failure, structurally controlled gravity driven polyhedral rock wedges of various shapes and sizes can also be dislodged from the periphery of an excavations as schematically illustrated in Figure 2.8. Such failures could occur when the movement of polyhedral blocks into the excavation is kinematically feasible as two-dimensionally shown in Figure 2.8, or as a result of the failure of critically located wedges (key blocks) as shown in Figure 2.9a. The key wedge that dislodges from the excavation periphery may undermine the adjoining wedges of any shape or size and the reduction in restraint of movement may results in progressive failure of the rock mass (Figure 2.9b).

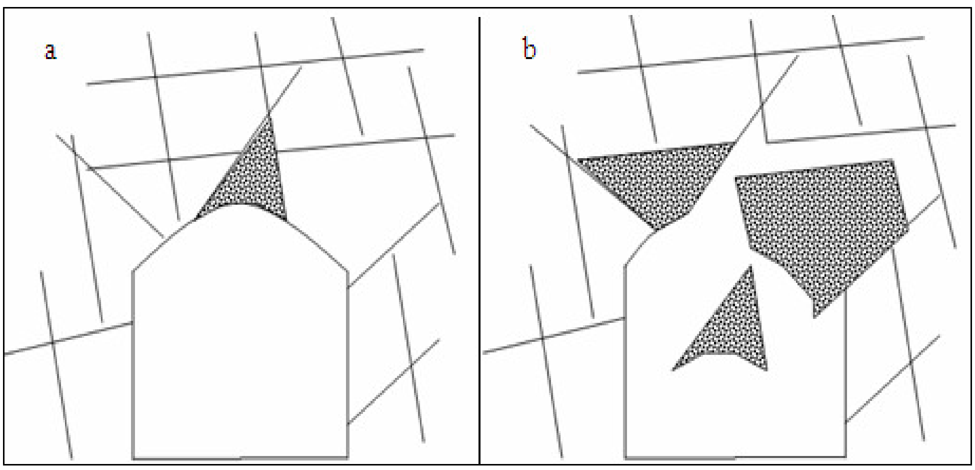


Figure 2.9 Wedge failure (a) key wedge (b) progressive failure

*2.6.1.2 Prism failure*

A fundamental requirement for the tetrahedral and polyhedral wedge failures mentioned in the preceding paragraphs is the presence of three or more intersecting sets of discontinuities in the rock mass. However, in some circumstances, the presence of only two discontinuity sets striking parallel to the axis of the excavation is sufficient to generate potentially unstable discrete rock blocks or prisms. The condition that arises from two moderately dipping discontinuities parallel to the excavation axis is schematically illustrated in Figure 2.10. The two intersecting discontinuities can form a long triangular prism of rock in the roof of the excavation. Depending on the spacing and persistence of the two discontinuity sets, the prism can be as wide as the width of the excavation and persist for a significant length along the excavation. In this situation the volume of rock that could potentially dislodge from the roof of the excavation is significant. Such failures may be avoided by changing the orientation of the excavation relative to the strike of the discontinuities. However, in some cases it may not be possible to change the axis direction of the excavation due to other constraints and requirements. An example of prisms failure is shown Figure 2.11.

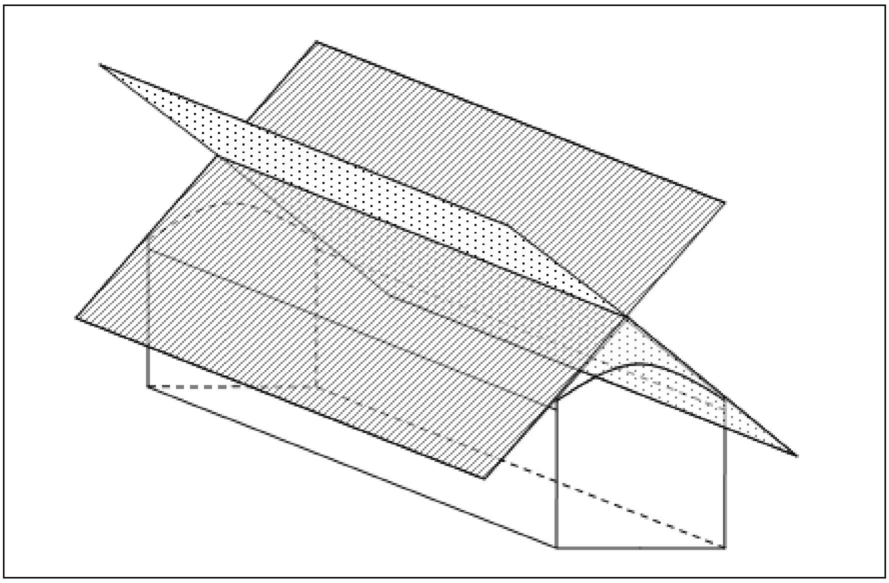

Figure 2.10 A discrete prism in the crown of an excavation

Figure 2.11 A prism failure from the roof of an excavation

*2.6.1.3 Beam or slab failure*

In openings created in horizontally laminated or bedded rocks, separation of strata is possible leading to beam or slab failure from the roof of the opening. Beam failure may transpire under three basic mechanisms: bending, free falling and compression (voussoir beam). When horizontal lamination or bedding partings are the only discontinuities present in a rock mass, flexural or bending failure of rock beams or slabs may occur, if the rock beds are thin compared to the width of the opening and no significant compressive stresses are induced in the rock mass. First the rock beds in the immediate vicinity of the roof will be subjected to bending under self weight and separate from the rock above and flex downwards as illustrated in Figure 2.12. If the tensile stresses induced by bending forces in the rock layers are greater than the tensile strength of the rock, the beam would crack. Cracks will develop on its upper surface at the ends and on the lower surface in the middle causing the beam to collapse under gravity. Collapse of the first beam leaves cantilevers as abutments for the next beam so each layer above the roof has, in effect, a smaller span.

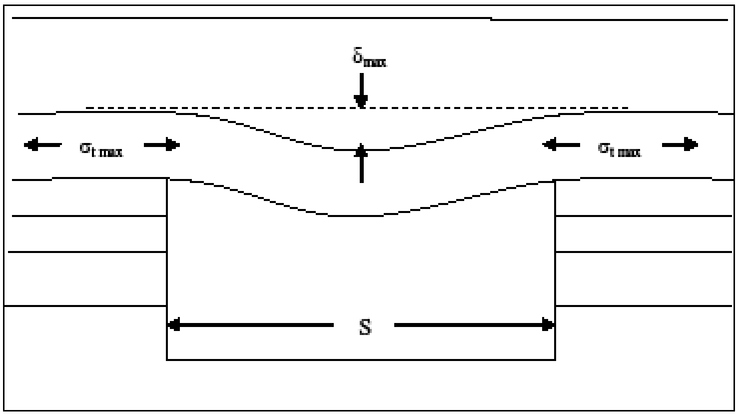


Figure 2.12 Deflection of roof beam

However, in some horizontally laminated or bedded rocks, apart from the lamination and bedding partings, other discontinuities such as cross cutting joints may also present (Figure 2.13). These cross cutting discontinuities reduce and, in the extreme, eliminate the ability of the rock mass to sustain tensile stresses such as those that would develop in elastically deformable beams illustrated in Figure 2.12. As observed by Ran et al. (1994), if the cross-cuttings joints dip at shallow angles or sufficient compressive stresses are not generated to mobilise the inter-block shear strength, slip along these joints and premature shear failure of the beam is likely. In such situations roof stratum immediately above the excavation would collapse under the influence of gravity without any significant deformation or bending. Depending on the degree of cross-cutting joints in the rock mass and the thickness of the rock beds, the failure may progress until a stable arch is formed as shown in Figure 2.13. An example of a gravity induced failure of a rock slab in which cross joints were also present is shown in Figure 2.14.

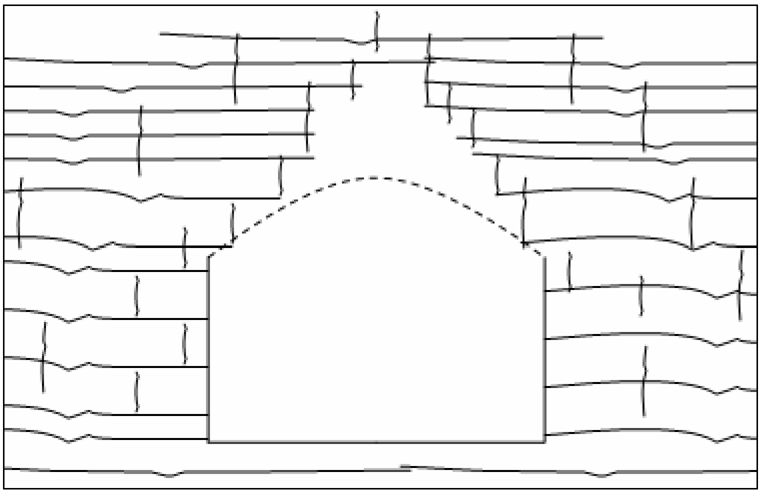

Figure 2.13 Cross-cutting joints and peaked roof formed by beam failure

Figure 2.14 Gravity induced rock slab failure

If the cross cutting joints in the laminated or bedded rocks are steep and sufficient compressive stresses are induced horizontally, it is possible to generate a compression arch within the beam which will transmit the beam loads to the excavation abutments (Figure 2.15). This is known as a self-supporting or voussoir beam. While a voussoir beam is generally considered to provide a stable excavation roof, it can also collapse under certain conditions. According to Sterling (1980) the primary modes of failure in the voussoir beam are buckling or snap-through failure, lateral compressive failure (crushing) at the mid span and abutments, abutment slip, and diagonal fracturing (Figure 2.16).

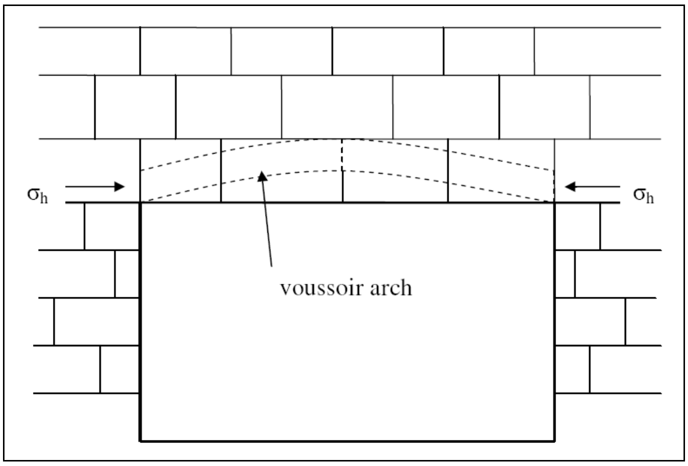


Figure 2.15 Jointed rock with compression arch or voussoir beam

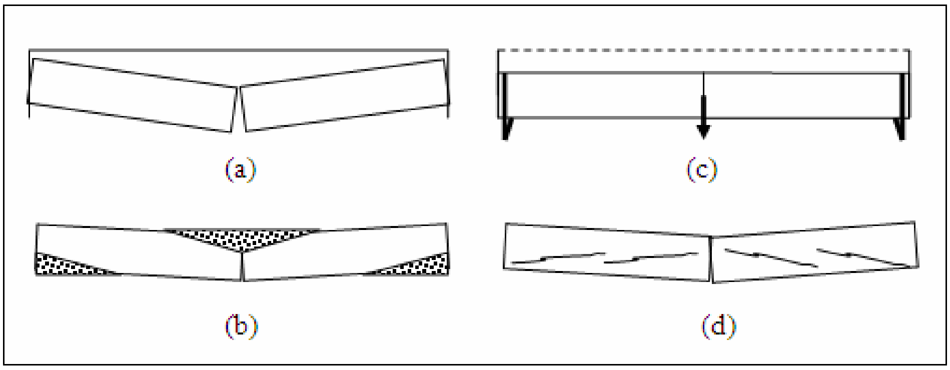


Figure 2.16 Failure mechanisms of voussoir beam (a) snap-through, (b) crushing, (c) sliding, (d) diagonal cracking

Continued failure of beams under the three basic mechanisms mentioned in the preceding paragraphs eventually produce a stable trapezoidal opening similar to that schematically illustrated in Figure 2.13. Obviously, this is significantly different to the required shape of the excavation and may often be unacceptable both from economic and serviceability points of view.

### 2.6.2 Strength controlled stress driven failures

Stress induced failures can be divided into two subclasses; brittle failure and plastic deformation, with the former being dominant over the latter. Brittle failures include bursting, spalling and crushing or rupturing. Plastic deformation includes shearing along the pre-existing discontinuities or through the intact rock leading to squeezing.

In brittle rocks with high in situ stress conditions, the new stresses induced by the creation of an excavation could be high enough to cause failure of the intact rock substance with or without any influence from the discontinuities in the rock mass. The nature of the failure and its exact location in the excavation periphery depend on the orientation and magnitude of the major stress components, shape and orientation of the excavation, and the strength and deformation properties of the rock mass.

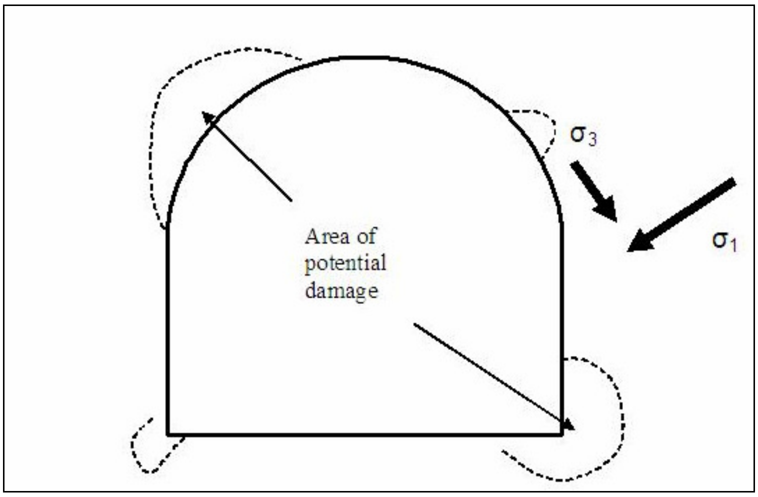


Figure 2.17 Spalling and crushing under high stress

The traditional approach to determine the stresses and displacements induced around underground openings is to employ the theory of elasticity mathematically presented by Love (1927) and Timoshenko and Goodier (1951). For highly stressed competent rock, in which discontinuities are widely spaced or tightly healed, it is generally acceptable to assume that the theory of elasticity is applicable. Early text books and technical papers dealing with underground excavation design were based, almost entirely on elastic theory and ignored the structural features in the rock masses. Although this is an over simplification of the complex rock mass, it provided an insight into the zones of stress concentration and potential areas of stress induced failure around underground openings. Most text books on rock mechanics (e.g. Obert and Duvall, 1966; Jaeger and Cook, 1976; Goodman, 1980) provide elastic solutions for predicting stresses and displacements around underground openings. Plots of principal stress contours surrounding underground excavations with regular shapes subjected to different stress fields are also provided by Hoek and Brown (1980a) and Brady and Brown (2004). These plots indicate the potential zones of stress induced instability around excavations. Figure 2.17 presents a simple example of the locations and the relative extent of brittle failure in an excavation under the stress directions shown.

Figure 2.18 Rock burst causing failure of the installed support in sidewall

*Rock burst*: Of various forms of stress induced brittle failure of rock in underground excavations, rock bursting is the most hazardous and difficult to manage. Rock bust is defined as a spontaneous, violent break of rock from the periphery of the excavation. This phenomenon involves the release of up to several tonnes of rock in excavations at great depths with high in situ stress conditions. The most explosive failures occur in rocks that have compressive strength and Young's Modulus values greater than 140 MPa and 34 GPa, respectively (Bell, 1980). The stronger the rock the more likely it is to burst. Figure 2.18 shows an example of rock bursting failure in a wall of an excavation, where installed support measures were inadequate to prevent failure. In extreme cases rock bursting can also occur in the advancing face or the floor of the excavation. Figure 2.19 shows an example of rock burst damage on the advancing face of a mine excavation.

*Spalling*: This is a similar to but less violent than rock bursts. It is a sudden ejection of thin rock slabs from the roof and sides of an excavation. It initiates in the region of maximum tangential stresses. The slabs can vary in thickness from a few millimetres to a few centimetres. Despite the relatively small size of the slabs, the progressive nature of the spalling process results in the detachment of large volumes of rock if not controlled or prevented by an adequate support system. Figure 2.20 shows an example of progressive spalling behind the welded wire mesh support measures installed in an excavation.

Figure 2.19 Strain burst damage on the excavation face

*Rupturing*: rupturing is defined as gradual breaking up of rock into pieces, flakes or fragments in the excavation surface. In this case cracking and bulging of the surface of the excavation indicating that the rock mass has been subjected to considerable stress may be observed.

Figure 2.20 Spalling rock in the roof held by mesh

*Buckling*: is another form of failure that can occur in highly stressed rock masses around excavations. It is defined as deflection of rock slabs in hard brittle steeply dipping layered rocks under high stresses, and is schematically illustrated in Figure 2.21. Since buckling requires the presence of both closely spaced discontinuities and high stresses, it can be described as a structurally controlled stress driven failure.

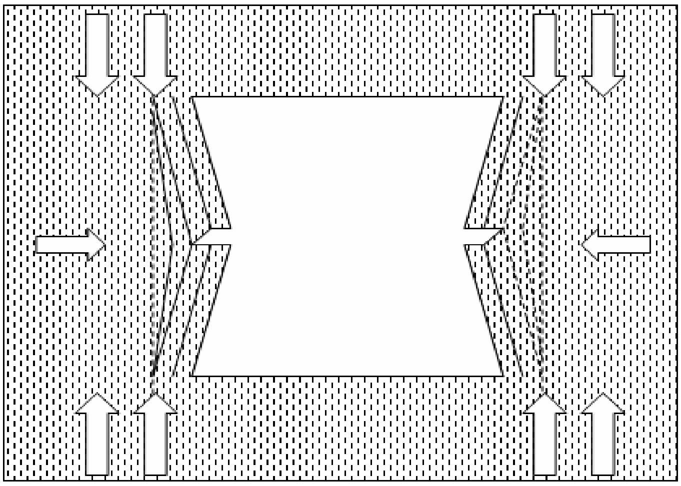

Figure 2.21 Buckling failure

*Squeezing*: Relatively weaker rocks, particularly those which suffer plastic deformation, undergo varying degrees of transient and steady-state creep into the opening under high stress conditions induced by the creation of the opening. This phenomenon is called squeezing. The initial deformation of such rocks may be by shear failure along discontinuities or through intact rock or both. Usually squeezing rocks show no signs of fracturing of the rock, but the rock would advance slowly and imperceptibly into the excavation. The failure or the slow convergence in such rocks may continue for a period of time ranging from days to many years from the date of excavation. Squeezing is synonymous with over-stressing and does not comprise deformation caused by loosening as might occur at the roof or at the walls of tunnels in jointed rock masses. Rock busting phenomena do not belong to squeezing (Singh and Goel, 1999). Figure 2.22 shows an example of squeezing.

Figure 2.22 Squeezing rock on left wall

### 2.6.3 Combined effect of stress and discontinuities on rock instability

As discussed in the preceding sections, instability in underground excavations in rock is classically divided into structurally controlled and stress controlled failures. There are, however, situations in which rock instability is governed by both structural features and the stress field in the rock mass. The dominant behaviour is a function of the in situ stress level relative to the intact rock strength and the degree of jointing or discontinuities in the rock mass. Figure 2.23, adopted from Kaiser et al. (2000) presents a generalised relationship between the degree of naturally occurring discontinuities in a rock mass, the level of applied stress with respect to the compressive strength of the intact rock, and the likely modes of failure around an excavation.

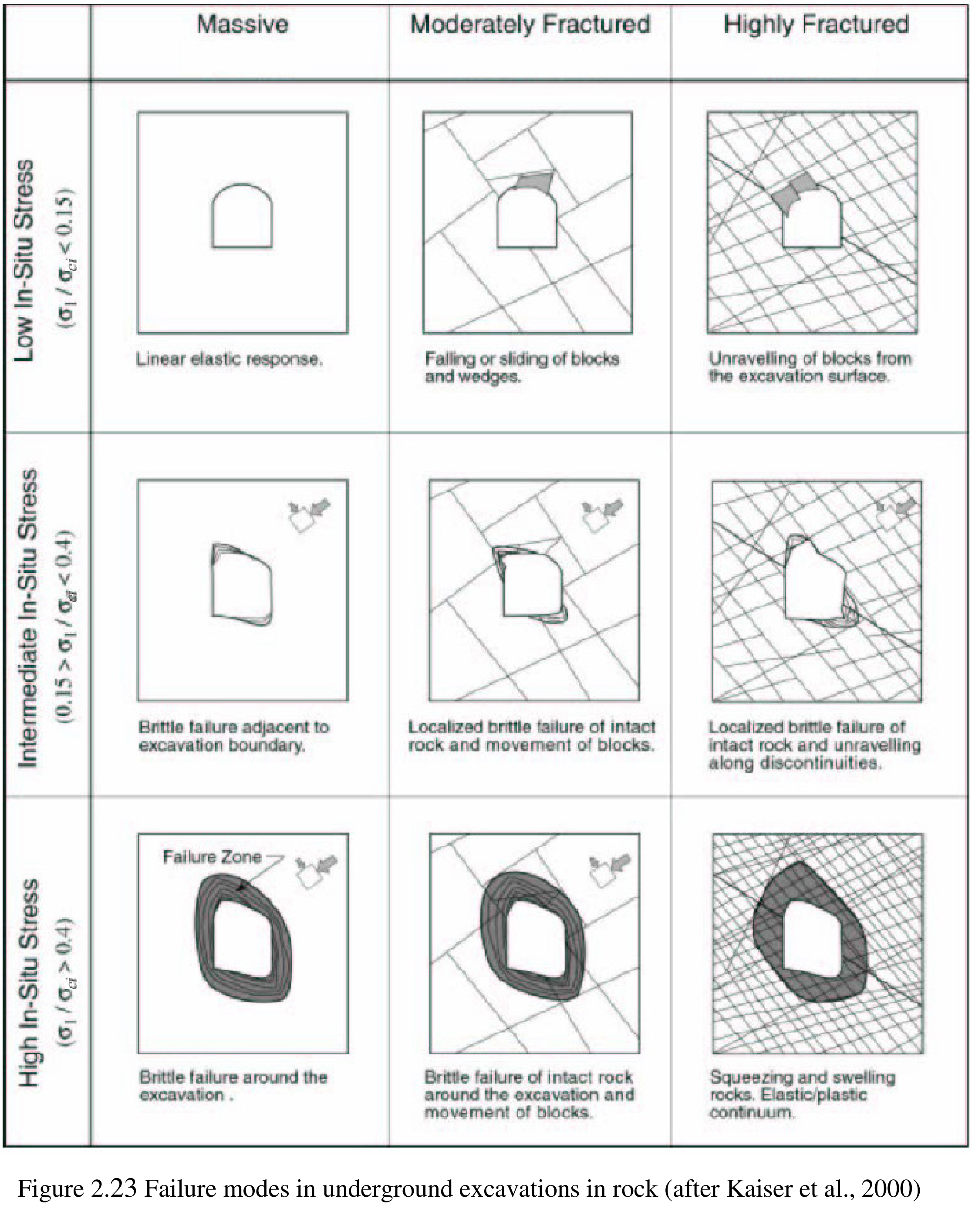


Figure 2.23 Failure modes in underground excavations in rock (after Kaiser et al., 2000)

As schematically shown in element 11 (top left element) of the failure modes matrix presented in Figure 2.23, massive rock under low stress should usually be free of instability issues. In low in situ stress conditions, in both moderately to highly fractured rock masses, instability is essentially structurally controlled as shown by elements 12 and 13 in Figure 2.23. These two cases were discussed under the wedge failure in Section 2.6.1. In the latter case with highly fractured rock, unravelling occurs as each piece of rock that fails causes a reduction in the restraint and interlocking of adjacent rock mass and allows other blocks and wedges to follow.

Under high stress, massive strong rocks respond to excavation by brittle failure (element 31 in Figure 2.23). Examples of brittle failure include spalling and bursting discussed in Section 2.6.2. If the rock mass is moderately jointed, shear movement can also occur along the discontinuities as indicated in element 32, additionally to the brittle failure of intact rock around the excavation. In heavily jointed rock masses under high stress, squeezing and swelling could give rise to partial closure of the excavation (element 33 in Figure 2.23). In this case elastic-plastic deformation caused by the discontinuities and the weakness of the intact rock material in comparison to the induced stress levels could result in gradual closure of the opening.

Figure 2.24 Unravelling in heavily jointed rock in a intermediate stress conditions

In massive rock under intermediate stress conditions (element 21 in Figure 2.23), localised brittle failure occurs depending on the strength of the rock material. With increasing fracture intensity (elements 22 and 23), in addition to the localised brittle failure of the intact rock material, the discontinuities also play a role by allowing block movement into the excavation. In heavily jointed rock masses with intermediate stress conditions, if rock blocks or wedges are allowed to fall, failure may progress into unravelling as in the example shown in Figure 2.24.

### 2.6.4 The Effects of Groundwater in Excavations

The underground excavation rock mass failures discussed earlier are either structurally controlled-gravity driven or strength controlled-stress induced or a combined result of the two. In some rocks containing water sensitive minerals, a third group of failure is possible if groundwater is present in the rock mass surrounding an underground excavation. The most common water induced failures that do not fall into the failure categories discussed earlier are slaking and swelling. Slaking is defined as a gradual breaking up of rock into pieces, flakes or fragments when in contact with water. Slaking occurs in some moderately coherent and friable rocks such as mudstone. Swelling is defined as gradual advancement of surrounding rock into the excavation due to expansion caused by water absorption into the constituent minerals. Typical examples of rocks that may suffer swelling are anhydrite, halite and rocks that comprise swelling clay minerals such as montmorillonite.

Apart from the above, as briefly mentioned in Section 2.6.1, water pressures in discontinuities could contribute to structurally controlled failure by forcing the rock blocks out of their sockets and also by reducing the shear strength of the discontinuity surfaces, especially those with soft fillings. Large quantities of water may also cause flowing ground in highly fractured or crushed rock masses with little or no coherence.

Groundwater could also play a role in strength controlled-stress driven failures in massive rock with no discontinuities discussed in Section 2.6.2. Here the water pressure acting within the micro-cracks in a solid rock could reduce its strength and influence failure. This phenomenon is similar to the effective stress concept in soil mechanics but at much greater depths with significantly high stress conditions.

# CHAPTER 3

# METHODS OF UNDERGROUND SUPPORT DESIGN

## 3.1 Introduction

The design of support for underground excavations is often complicated due to the lack of control over the rock mass conditions and the in situ stress field. For instance, it is seldom possible to make an accurate measurement of either the mechanical properties of a rock mass or the forces acting on it (Palmstrom and Stille, 2007). In almost all major underground openings constructed in rock, it is not uncommon to intersect several different rock mass conditions. The differences could be due to the presence of several rock types and discontinuity systems at the excavation site or simply due to spatial variations in weathering, alteration and fracturing of the same rock. An example of rock mass conditions that may be encountered along a tunnel route is graphically presented in Figure 3.1. Even if a comprehensive site investigation program is implemented, all variations in ground conditions along an excavation route may not be detectable. In addition, the in situ stresses acting on a rock mass can also vary due to several factors including surface topography, depth and geological structure etc, and accurate determination of stresses is difficult. As Brady and Brown (2004) succinctly put it, "*The virgin state of stress in a rock mass is not amenable to calculation by any known method, but must be determined experimentally*". Thus, for major projects involving several excavations, site specific stress fields may be determined during geotechnical investigation or in the detailed design stage. However, for small to medium size projects site specific in situ stress measurements may be cost prohibitive.

The complexity of the operating geological environment in some projects is such that a precise understanding of its engineering behaviour can be obtained only after the completion of the construction. Thus the underground excavation support design process often has to be formalised for a largely unknown environment. Further, the inherent variability of the engineering characteristics of rock masses and the variability in the in situ stress field mean that no single design method would be versatile enough to deal with all possible rock mass conditions that may be encountered in an excavation project, even if conditions are known in advance.

Despite these complexities, during the past four decades, there have been significant developments in support design methods. These are based on different approaches, but they can supplement each other when applied to the conditions for which they were developed. They can be broadly categorised into three different approaches:

- Observational,
- Empirical, and
- Rational.

These approaches are outlined in the following sections.

### 3.2 The Observational Approach

This approach calls for the instrumentation and monitoring of the excavation and the development and implementation of the support design as excavation is progressed. The aim is to determine the ground response to installed support, allowing the early identification of possible problems and areas for improvements in the implemented design. A well known observational method is the New Austrian Tunnelling Method (NATM) (Rabcewicz, 1964a, 1964b, 1965). It is generally accepted that NATM has evolved as a result of experience and innovations achieved in Austrian Alpine tunnelling conditions (Whitaker and Frith, 1990). It involves the installation of immediate temporary support (usually shotcrete; wire mesh, rock bolts and steel sets, if necessary) to preserve the rock mass strength by minimising deformations and is particularly suited to squeezing ground conditions. A major aim of NATM is to learn from experience and continually update the support system by monitoring the rock mass from within the tunnel (Whittaker and Frith, 1990). NATM relies on performance monitoring for prediction and classification of ground conditions. It is adapted to each new project based on previous experience and is also adapted during a single project based on performance monitoring. A particular (NATM) classification is, therefore, only applicable to the case for which it was developed and modified so its use by others on other projects may be difficult (Barton, 1988).

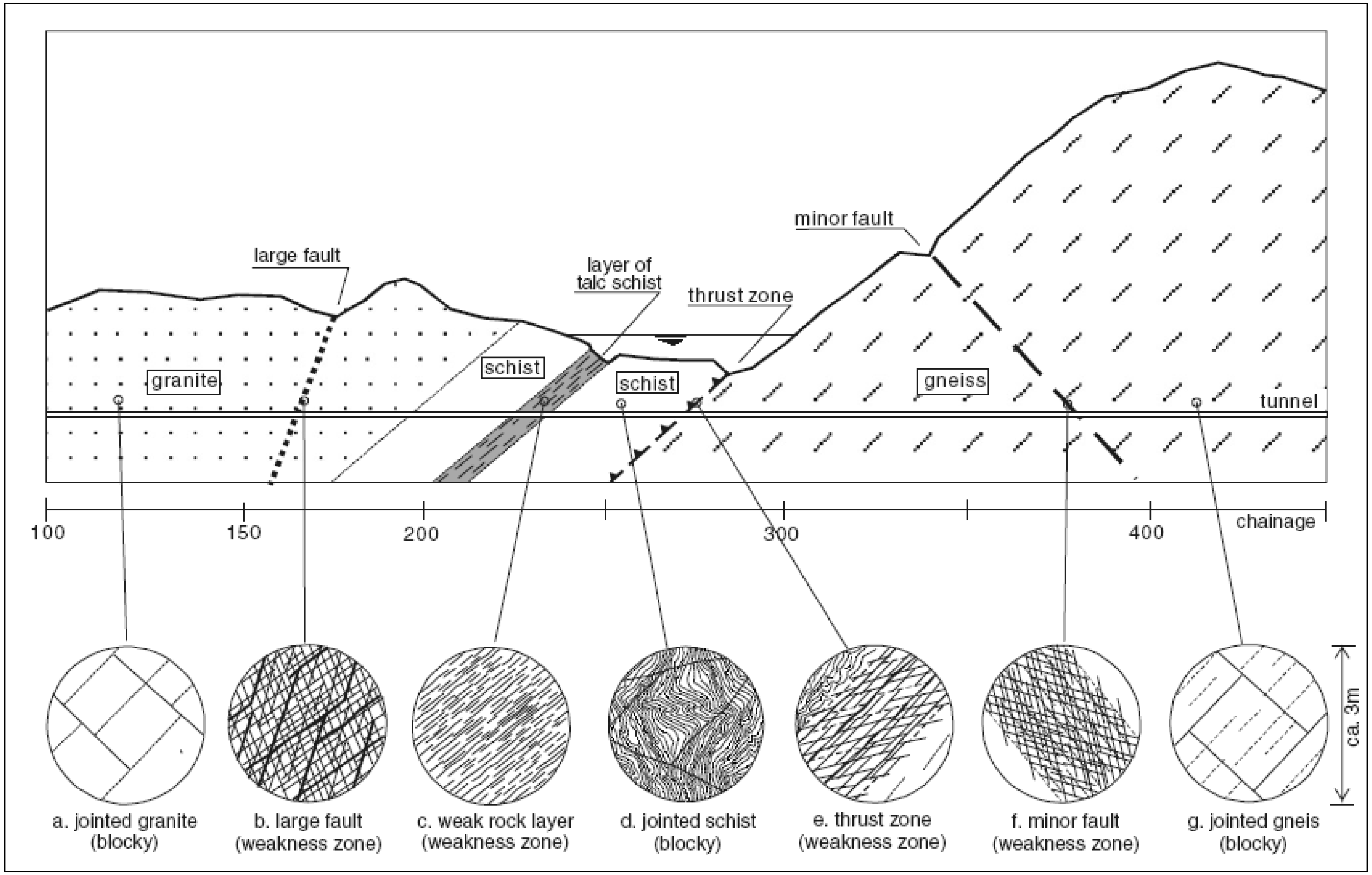


Figure 3.1 Examples of rock mass compositions along a tunnel route (after Stille and Palmstrom, 2008)

## 3.3 The Empirical Approach

A wide range of empirical methods has been developed over the years for the design of support for underground excavations. These methods are based on precedent practice and often involve some form of ground characterisation and a set of design rules or guidelines. The rock mass classification systems such as RMR and Q, the reliability of which is the subject of this study, constitute an integral part of the empirical design approach of support design. Detailed discussions on the development and relative merits of these methods are covered in Chapter 4. The empirical design rules which are not based on rock mass classification are also in use, such as that proposed by Lang (1961) for pattern bolting of underground excavation. These guidelines developed based on a range of laboratory, field and theoretical studies were subsequently adopted by the U.S. Corps of Engineers (1980). Lang's design rules are for the minimum bolt length $L$, with respect to bolt spacing $s$, excavation span $B$, and the width of the critical and potentially unstable rock blocks defined by the average discontinuity spacing $b$. Detailed discussion of Lang's guidelines can be found in Stillborg (1994), Brown (1999) and Brady and Brown (2004). These guidelines can be used to determine the minimum rock bolts to be included in the preliminary design.

## 3.4 The Rational Approach

Rational design methods are primarily aimed at predicting rock mass behaviour or failure after the creation of underground excavations and the influence of different support systems. They are founded on theoretical solutions involving the analysis of (a) forces and stresses acting on the rock mass, (b) strains and deformations in the rock mass, and (c) modes and mechanisms of its eventual collapse. In this approach the problem at hand is expressed mathematically using simple equations in analytical form such that a solution can be found by direct calculations, or using complex differential equations which are solved by numerical methods. Depending on the problem and the method used, design calculations may be simple limit equilibrium type or may use comprehensive computations involving rock-support interaction calculations taking account of the deformation and strength properties of the support system and the complete stress-strain response of the rock mass.

Analytical methods include solutions based on elastic-plastic analysis of rock-support interaction and rigid block stability analysis based on statics. As mentioned in Section 2.6.2, for competent massive rock masses that may be assumed to behave elastically, the solutions developed using the theory of elasticity are used. Such solutions are readily available in numerous rock mechanics text books including those by Obert and Duvall (1966), Jaeger (1972), Jaeger and Cook (1976), Goodman (1989), Hoek and Brown (1980a) and Brady and Brown (2004). When the rock is layered where bending and separation of strata are possible and the beds are elastically deformable, the theory of elastic beam and plates can be invoked. For weak rocks that undergo slow convergence or squeezing under stress, a solution for stress and displacement derived from the theory of plasticity can be used. For rocks that present time dependent properties, such as rock salt, the theory of linear viscoelasticity provides useful concepts (Goodman, 1989). A main disadvantage of these solutions is that they exist for simple or regular excavation geometries and idealised rock mass conditions only. Nevertheless, they allow a better understanding of rock-support interaction when the behaviour of rock mass is governed by its elastic and/or plastic deformation properties and the stress field acting on it.

Analytical methods based on statics on the other hand specifically deal with the stability issues arising from the presence of discontinuities in the rock mass. They are aimed at predicting the stability of pre-identified kinematically unstable rock blocks by limit equilibrium analysis. These methods assume that the potentially unstable rock blocks are rigid and their movement is induced by the force of gravity. They often ignore the in situ stress field and elastic deformation of the unstable rock blocks and the surrounding rock mass. However, theoretical solutions such as those developed by Elsworth (1986), Sofianos (1986), Sofianos et al. (1999), Nomikos et al. (2002) can incorporate the in situ stresses and deformation of the rock mass.

Numerical modelling involves the derivation and solving of complex differential equations representing the conditions and behaviour of rock masses. Basic pre-requisites are the idealisation of the actual excavation within the rock mass and, based on the available information, the division of the rock mass into different sectors. Material property models are established for each of the sectors and also for support elements. Complex differential equations are then derived to represent these models and are solved by numerical methods. Most numerical modelling methods apply discretisation of the rock mass into a large number of individual elements and achieve an iterative solution by repetitive calculation in a computer. This technique is used mainly for predicting induced stresses and elastic or plastic deformation in massive rock masses, but can also be used for predicting discrete block movement in jointed rock.

### 3.4.1 Support Design by the Rational Approach

Traditionally, the rational approach to underground excavation support design is divided into two procedural domains based on the analysis of structurally controlled gravity driven modes of instability and strength controlled stress driven instability discussed in Chapter 2.

Structurally controlled gravity driven falls are common in underground excavations in jointed rocks. The kinematically potential collapse modes such as slabs, beams, prisms and wedges etc in the roof and walls of an excavation are identified by detailed examination of the configuration of blocks defined by the known geometric properties of the discontinuities and the size, shape and the orientation of the excavation. Several techniques can be applied to assess the potential for such ground falls provided that an appropriate failure mode is assumed. Typical failure modes that can be analysed include prism, wedge and slab or beam failures. Models taking these failure modes into account are generally utilized for stability assessments in low-stress or near-surface excavations in jointed rock masses. For deep excavations with high stress conditions or for shallower excavations in weaker rock masses, models based on linear elastic stress and strain analysis, or variations thereof, can be used to determine the location and extent of problematic stress concentrations around the openings.

However, as outlined in Section 2.6.3 and illustrated in Figure 2.22, there are many situations in which instability is a result of both structurally controlled and stress induced failure of the rock mass. Recent developments in sophisticated numerical modelling tools have made it possible to analyse some of the potential failures caused by the combined effects of the geological structure and the high in situ stress field in the rock mass surrounding excavations. Some of these tools are based on discrete block models and can model the discontinuous nature of rock masses.

At present, depending on the rock mass conditions, several different rational methods are used for underground excavation support design including: beam analysis, tetrahedral wedge analysis, symmetric triangular prism analysis, natural rock arch concept, rock-support interaction analysis, convergence control method and numerical modelling based on both continuum and discrete block methods. In this study three rational methods were used as and when applicable to the case studies considered, namely, (a) suspended beam concept, (b) tetrahedral wedge analysis, and (c) numerical modelling based on distinct element method. They are briefly discussed in the following sections.

*3.4.1.1 Design against beam failure*

In horizontally stratified or layered rocks, the horizontal discontinuities create a structural setup for rock beams or slabs to be formed in the excavation roof. In such rocks, if the lamination partings are the only discontinuity set present, roof deflection and stability can be assessed using conventional elastic beam deflection and lateral stress calculation methods presented by Obert and Duvall (1966) and Hoek and Brown (1980a). However, it is not uncommon to encounter other joint sets cutting through the laminations of the stratified rock. These cross-cutting joints reduce and, in the extreme, eliminate the ability of the rock mass to sustain boundary parallel tensile stresses such as those assumed in conventional elastic beam analysis. In laminated rocks with cross cutting joints, design against beam or slab failure can be undertaken by two approaches: (a) the self-supporting (voussoir) beam concept, and (b) the suspended beam concept.

*The self-supporting beam concept*

Several researchers have made significant contributions based on both experimental and computational investigation to develop theoretical solutions for self supporting roof beam (voussoir beam) design. Notable among these include the work of Adler and Sun (1968), Barker and Hatt (1972), Wright (1974), Sterling (1980), Beer and Meek (1982), Lorig and Brady (1983), Brady and Brown (1985), Sofianos (1996, 1999) and Diederichs and Kaiser (1999a, 1999b). Brady and Brown (2004) captured the salient features of the solutions proposed by the above mentioned researchers and others and provided a comprehensive set of solutions for the analysis and design against known failure mechanisms of voussoir beams. This concept (illustrated in Figure 2.15 in Chapter 2) was not necessarily applicable to the rock mass conditions in the case tunnels considered in this study.

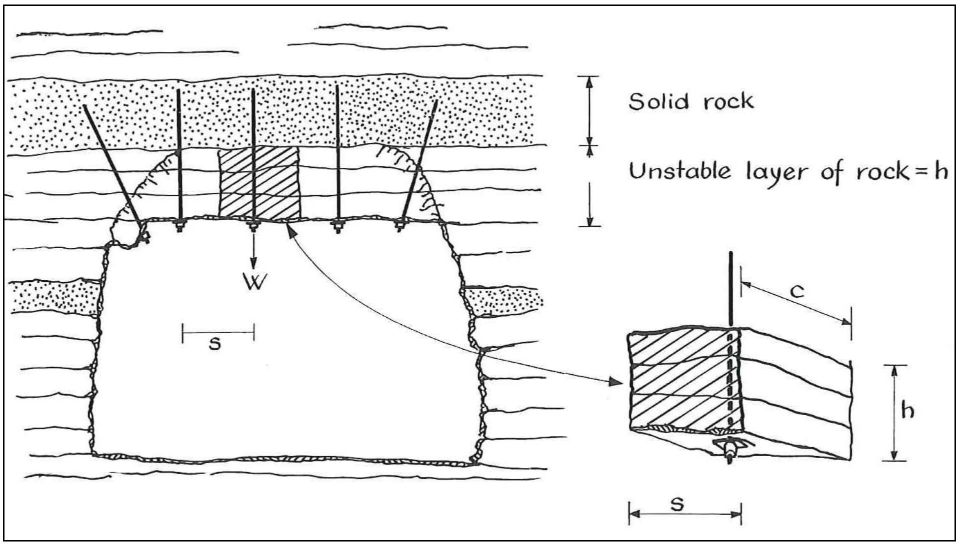


Figure 3.2 Suspended beam support for horizontally layered rock

*The suspended beam concept*

In some practical situations, the horizontally laminated rocks may contain cross-cutting discontinuities and the horizontal stresses could be low where accurate determination of the parameters required for a detailed analysis using the theoretical voussoir beam solutions may be difficult and unwarranted. In this situation, if the horizontal stress component is tensile or sufficient compressive stress is not induced to mobilize frictional shear force required to prevent block movement along the cross-cutting discontinuities, the suspended beam concept may be adopted. This concept illustrated in Figure 3.2 is a conservative analysis which ignores the effect of shear and flexural strengths of the rock strata and the in situ stress field around the excavation, and assumes that the weight of the rock in the unstable zone is supported entirely by the force developed in the rock bolts anchored in the overlying solid rock.

From Figure 3.2, for an unstable rock beam of thickness *h,* the weight of the rock, *W*, to be supported by a single bolt in a rectangular bolting pattern is given by

$$W = \gamma\, h\, s\, c \tag{3.1}$$

where, $\gamma$ is unit weight of the rock, *s* is rock bolt spacing perpendicular to the excavation axis and *c* is rock bolt spacing along the excavation axis. If the ultimate load capacity per rock bolt is *T*, the allowable maximum bolt spacing can be determined by

$$T = WF = \gamma\, h\, s\, c\, F \tag{3.2}$$

where *F* is the required factor of safety against failure for long term roof stability. If bolt spacing in both longitudinal and transverse directions is taken to be the same, *s*, the required bolt spacing is determined by

$$s = (T/\gamma\, h\, \mathrm{F})^{1/2} \tag{3.3}$$

In horizontally layered rocks, tensioned bolts can also be used to make the rock layers interact and thereby increase the stability of the excavation roof. Here a self supporting roof beam (voussoir beam) is formed by the use of tensioned bolts, as opposed to the support provided by the bolts anchored in the overlying solid rock. Tensioned bolts design guidelines in the form of a monogram were formulated by Panek (1964) and were discussed in detail by Stillborg (1994).

The design can be undertaken using the solution proposed by Lang and Bischoff (1982) which is an extension of the suspended beam analysis discusses earlier and incorporates the shear strength developed by the rock mass on the vertical boundaries of the rock unit reinforced by a single rock bolt. Their solution assumes that the rock mass is destressed to a height *h* (shown in Figure 3.2), but variable vertical stresses, $\sigma_v$, and horizontal stresses, $\sigma_h = k\sigma_v$, are induced within the destressed zone (Brady and Brown, 2004). Typically k may be taken as 0.5. The underlying assumption in this analysis is that sufficient inter-block compressive stress is generated in the horizontal direction to mobilize the frictional shear resistance to prevent loosening and failure of rock blocks from the roof beam. In other words, a compression arch is generated within the beam which will transmit the beam loads to the abutments

(Figure 3.3). The formula given by Lang and Bischoff for determining the required bolt tension is

$$\frac{T}{AR}=\frac{\alpha}{\mu k}\left(1-\frac{c}{\gamma R}\right)\left(\frac{1-\exp(-\mu kD/R)}{1-\exp(-\mu kL/R)}\right) \tag{3.4}$$

where, $T$ is rock bolt tension, $A$ is area of roof carrying one bolt (= $s^2$ for $s$ x $s$ bolt spacing), $R$ is shear radius of the reinforced rock unit, = $A/P$, where $P$ is the shear perimeter (=4$s$ for a $s$ x $s$ bolt spacing), $\alpha$ is a factor depending on the time of installation of the rock bolts ($\alpha$ = 0.5 for active support, and 1 for passive support), and $L$ is bolt length which will often be less than $h$, the height of the destressed zone.

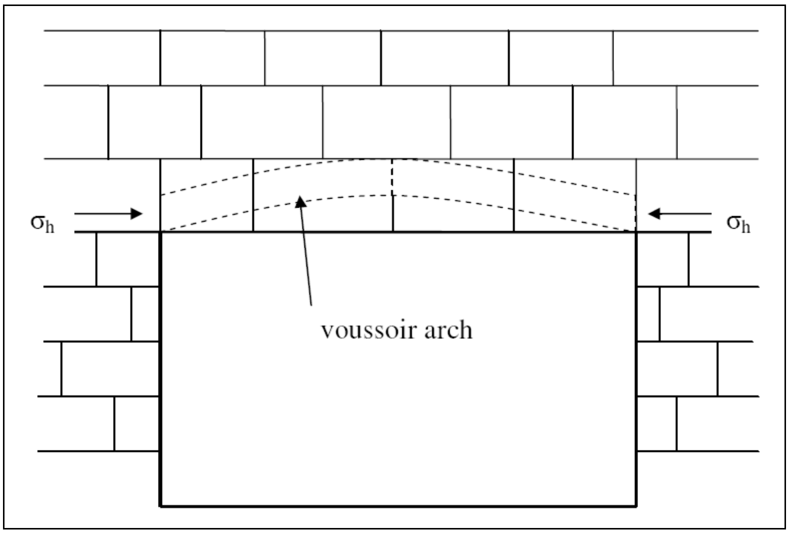


Figure 3.3 Compression arch or voussoir beam in jointed rock

### 3.4.2 Triangular roof prism

As discussed in Section 2.6.1, the basic requirement for the formation of a prismatic wedge is the presence of moderately dipping two discontinuity sets parallel to the axis of the excavation. Identification of triangular roof prism can be undertaken by detailed examination of the discontinuity orientations. This may be carried out by manually or by stereographic projection techniques. For a symmetric triangular roof prism Hudson and Harrison (1997), Sofianos et al. (1999), Nomikos et al. (2002), and Brady and Brown (2004) provided comprehensive analytical solutions considering gravitational forces, discontinuity shear strength and stress field in the rock mass surrounding the excavation. However, for asymmetric roof prisms, the analytical solution is somewhat more complex than that for the symmetric prism. For the general (asymmetric) case, the solution can be simplified if the stress field around the excavation is ignored. For a falling roof wedge illustrated in Figure 3.4 which ignores the effect of stress field, the number of bolts required is given by

$$N = \frac{W \times F}{B} \tag{3.5}$$

where, *N*=number of rock bolts, *W*=weight of wedge, *F*=factor of safety, *B*=load bearing capacity of each bolt.

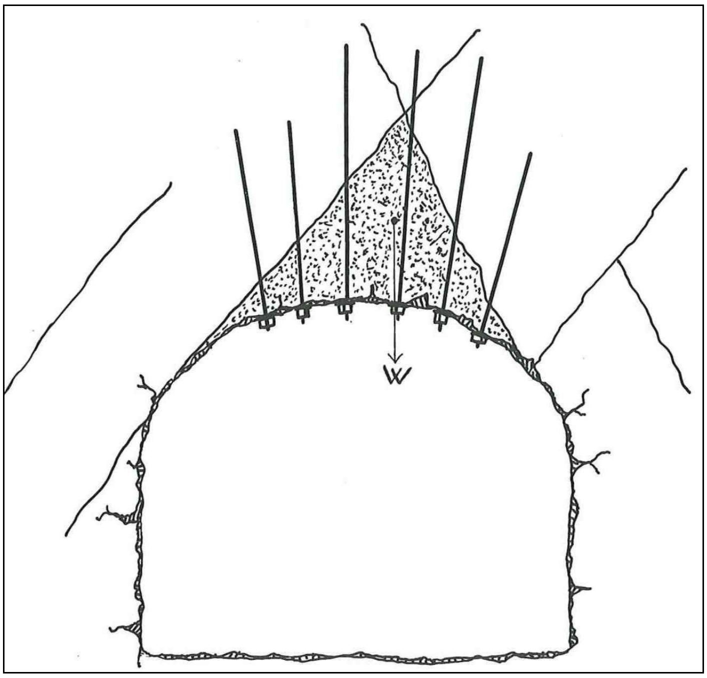


Figure 3.4 Rock bolt support for a falling roof wedge (after Stillborg, 1994)

For a sliding wedge in sidewalls as illustrated in Figure 3.5, the number of bolts, N, required is

$$N = \frac{W\,(F \sin\beta - \cos\beta \tan\phi) - cA}{B\,(\cos\alpha \tan\phi + F \sin\alpha)} \tag{3.6}$$

where, $W$=weight of wedge, $F$=factor of safety, $\beta$=dip of the sliding surface, $\phi$=friction angle of the sliding surface, $c$=cohesive strength of the sliding surface, $A$=base area of the sliding surface, $B$=load bearing capacity of each bolt, $\alpha$=angle between the plunge of the bolt and the normal to the sliding surface.

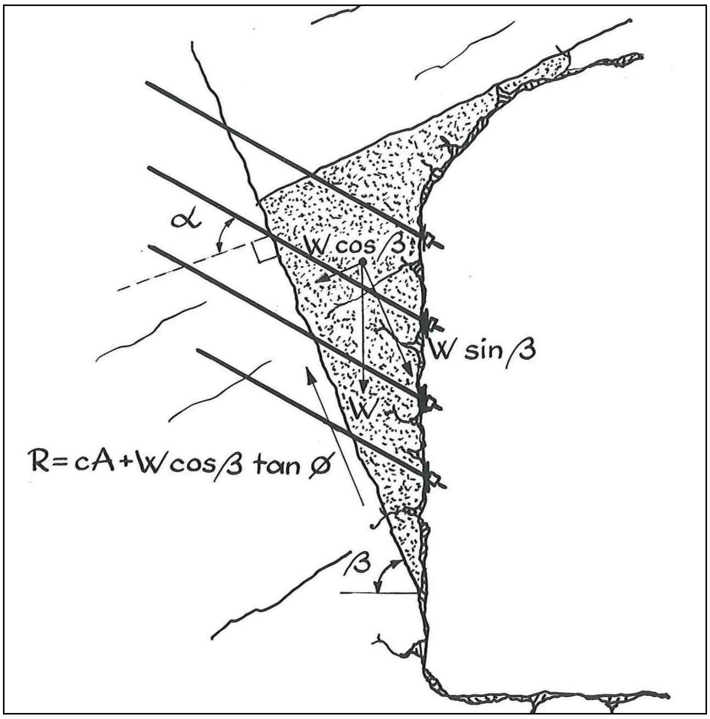


Figure 3.5 Rock bolt support for a sliding wedge (after Stillborg, 1994)

### 3.4.3 Tetrahedral wedge analysis

The problem of tetrahedral wedge instability in jointed rock masses at relatively shallow depths can be dealt with in three distinct analytical steps. The first is to identify the kinematically possible modes of potential collapse in the block assemblage. The second is to determine the state of equilibrium of the kinematically unstable rock wedges identified in the rock mass. The third is to determine the support required to stabilise the potentially unstable rock wedges.

The kinematically possible tetrahedral wedges in the roof and walls of an excavation can be identified by stereographic projection of known discontinuity orientations. The stereographic projection technique is well documented in numerous text books on structural geology and its applications in rock engineering is discussed in detail by Goodman (1976, 1980), Hoek and Bray (1977), and Priest (1985, 1993). Elaborated examples of its application in underground excavation design are presented by Cartney (1977), Hoek and Brown (1980a) and Hoek et al. (1995). Before the advent of computerised tools for stereographic projection of discontinuity orientations, the technique was used manually to identify the kinematically unstable rock wedges in underground excavations. The manual use of the stereographic projection techniques is tedious and inefficient, particularly when more than three discontinuity sets, each having a range of orientations, are present in a rock mass.

However, during the last two decades, this difficultly has been overcome by using the Shi's Block Theory (Goodman and Shi, 1985; Goodman, 1989). It is a method to identify the types of blocks that can be formed in a jointed rock mass and to separate those that are kinematically moveable into the excavations. It can handle an unlimited number of discontinuities and identify the shape and location of the movable blocks anywhere in the periphery of an excavation. The technique and its application to underground excavation design is discussed by Goodman (1989), Hudson and Harrison (1997) and Brady and Brown (2004). By using the block theory, computer software packages were developed for the identification and analysis of the potentially unstable rock wedges in underground excavations. One such package developed specifically for tetrahedral rock wedge analysis for underground excavation design is UNWEDGE (Rocscience, 2003). The following discussion utilises the figures generated using UNWEDGE version 3.0.

Figure 3.6 illustrates stereographic projection of three joints present in a tunnel. The joints are denoted as 1, 2 and 3 and their dip and dip directions are 72/271, 68/170 and 57/031, respectively. Figure 3.6 also shows the tunnel axis direction, 140$^{o}$ to the north, as a chain dotted line and the tunnel plunge, 5$^{o}$ to the horizontal, is marked as a cross in the southeast quadrant. In Figure 3.6, the three great circles represent the three joints and the triangular area formed by them defines the base of the tetrahedral rock wedge. Figure 3.7 shows the three-dimensional view of the tetrahedral wedge formed by the three joints in the tunnel roof.

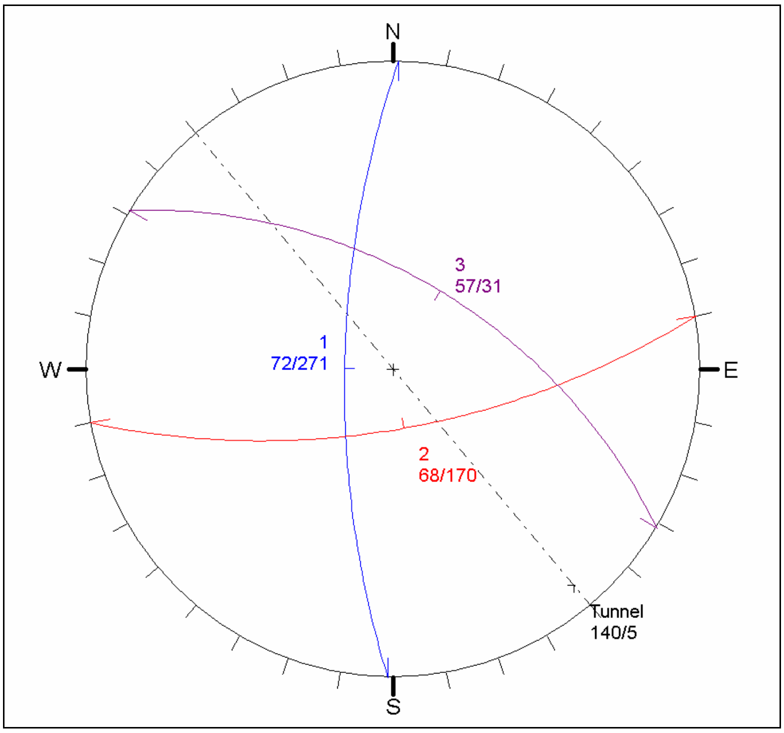


Figure 3.6 An equal area lower hemisphere stereographic plot of three joints, which form a tetrahedral wedge in the tunnel roof

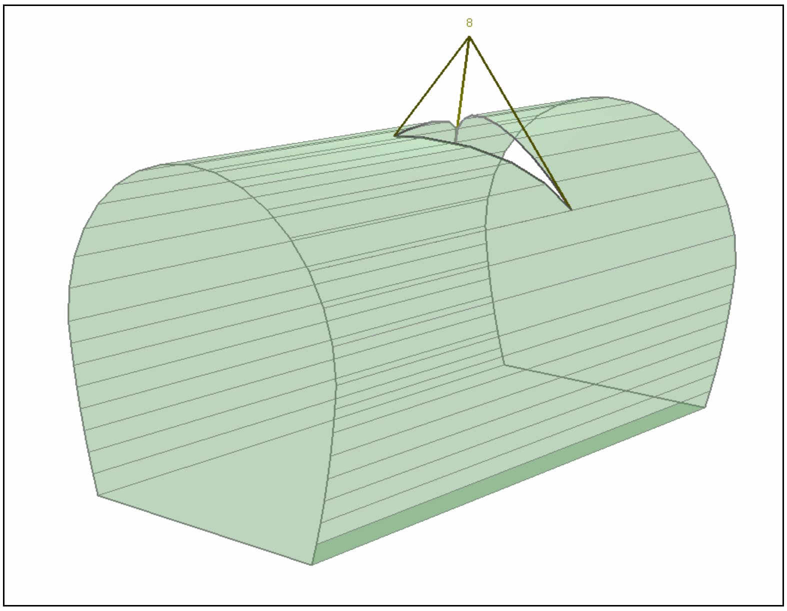


Figure 3.7 A tetrahedral roof wedge formed by three joints

In the lower hemisphere stereographic projection shown in Figure 3.6, any vertical line is represented by the cross in the centre. If a vertical line drawn through the apex of a tetrahedral wedge falls within its base area as in Figure 3.6, the wedge has the kinematic feasibility to fall without sliding. If the vertical line falls outside the base area, failure can occur only by sliding on a joint dipping into the tunnel or along the line of intersection of two joints plunging towards the tunnel. Figure 3.8 illustrates stereographic projection of three joints with the dip and dip directions of 78/271, 57/170 and 30/307. Since the vertical line drawn through the apex of the wedge falls outside its base area (Figure 3.8), failure of this wedge from the roof is possible only by sliding. These discontinuities can form tetrahedral rock wedges anywhere in the perimeter of the excavation, but they may not necessarily be kinematically unstable. Figure 3.9 shows the three dimensional view of the kinematically unstable rock wedge in the left wall of the tunnel; the wedge is formed by the three discontinuities given in Figure 3.8.

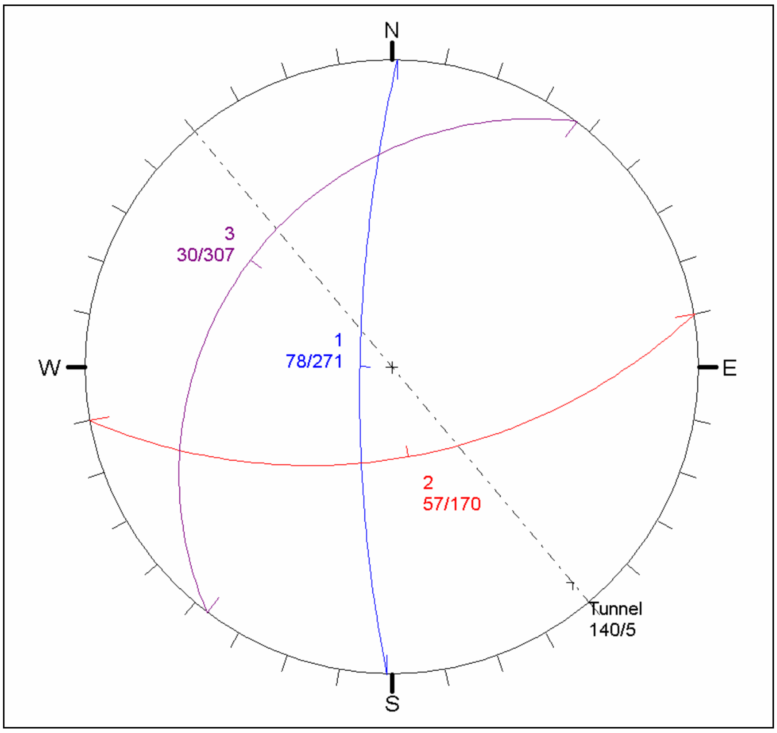


Figure 3.8 An equal area lower hemisphere stereographic plot of three joints which form a tetrahedral wedge in the left wall of the tunnel

The size and shape of the potentially unstable wedges in the periphery of an opening depends on the size, shape and orientation of the opening and orientation and spacing of the discontinuities in the rock mass. For an excavation of known dimensions, the maximum size of the potential wedges can be determined by plotting the relevant discontinuities on an excavation plan as described in Hoek and Brown (1980a).

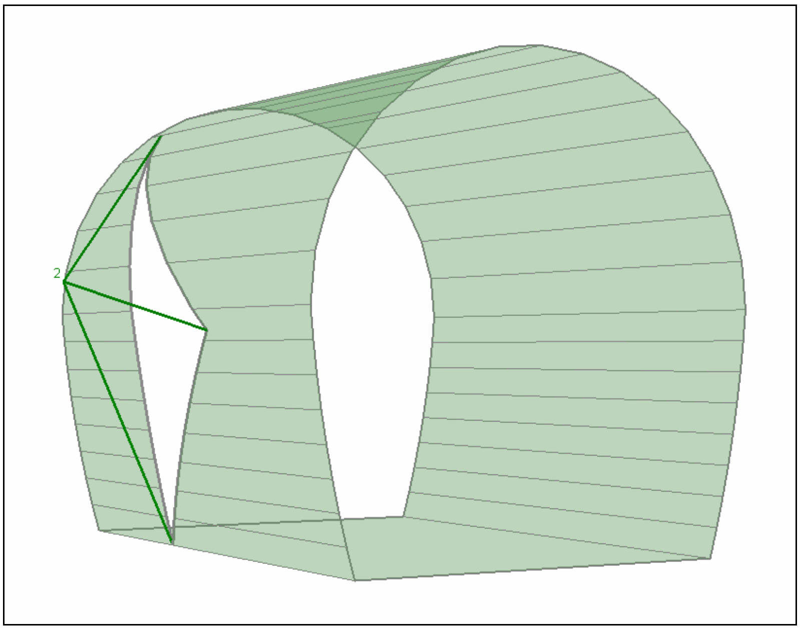


Figure 3.9 The tetrahedral wedge in the left wall formed by three joints

A comprehensive analysis of tetrahedral rock wedge stability requires the consideration of forces, deformations and displacement on each surface of the wedge. The problem becomes even more complicated when the wedge is non-regular or asymmetric. For example, on any face of the tetrahedron it is necessary to consider two components of mutually orthogonal shear displacement as well as a normal displacement component (Brady and Brown, 2004). If it is assumed that the stability of a wedge is controlled by the gravitational force and the shear resistance offered by the wedge surfaces, the analysis can be simplified to a simple limit equilibrium analysis as proposed by Hoek and Brown (1980a). This may be further reduced to a two dimensional problem as proposed by Hoek and Brown (1980a) and Stillborg (1994), and the methods used for asymmetric prism analysis (Figures 3.4 and 3.5) may be adopted. However, the two dimensional analysis is an over simplification because in neglecting the third dimension, the savings achievable through geometry are abandoned.

Further, rock wedge stability is not only a function of discontinuities but also of the in situ stress field in the rock mass. The induced stresses around the excavation can have a stabilising influence, particularly if the rock wedges are narrow and deep, but its effect is reduced by loosening. In some cases the stress field can actually have destabilising influence on rock wedges by forcing them out of their sockets, particularly if the wedges are broad and shallow. A considerable amount of work was undertaken by various researchers (Sofianos, 1986; Elsworth, 1986; Hudson and Harrison, 1997; Sofianos et al., 1999 and Nomikos et al., 2002) to facilitate a more comprehensive understanding of the effect of in situ stress on the stability of rock wedges in underground excavations. The current version of the UNWEDGE package (version 3), with some limitations, can incorporate the in situ stress field in the wedge stability calculations, and has been used in the present study. The effect of stress field on the wedge stability as a function of the wedge apical angle is illustrated in Figure 3.10.

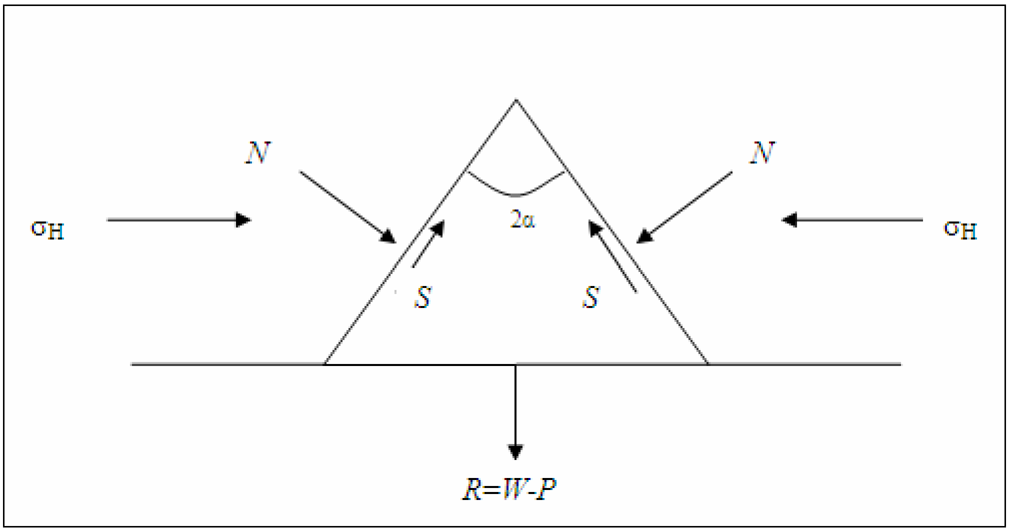


Figure 3.10 The effect of stress on symmetric roof wedge

In Figure 3.10 a symmetric triangular roof prism (wedge) of weight *W* with an apical angle of 2α subjected to a horizontal stress field of $\sigma_H$ is considered. The wedge has the potential to fall with no sliding on any of the bounding discontinuity surfaces. The influence of horizontal stress on the wedge is a function of the wedge apical angle, the shear stress (friction and cohesion) along the discontinuity surfaces and the elastic properties of the intact rock. For illustration purpose wedge is assumed to be rigid and the shear strength is only provided by angle of friction ($\Phi$) with no cohesion component. Resolving forces vertically, the resultant force R is given by

$$R = W - P \qquad (3.5)$$

where, $W$ is the weight of the wedge, and $P$ is the resisting force along the discontinuity surfaces. If $P>W$, the wedge will be restrained. Assuming $S=N \tan \Phi$, $P$ is given by

$$P = \frac{2N \sin(\phi - \alpha)}{\cos \phi} \tag{3.6}$$

If $N>0$, then $P>0$ only if $\alpha < \Phi$. If $\alpha>\Phi$, then $P<0$ and contributes to squeeze the wedge out of the rock mass. Similar solutions can be developed for asymmetric wedges as well as wedges that are sliding on one or more discontinuity surfaces. As mentioned previously the influence of the stress field is a function of not only the wedge apical angle, but also of the elastic properties of the intact rock and discontinuity surfaces. Detailed treatment of this aspect may be found in the references cited earlier.

### 3.4.3 Numerical modelling of stress controlled failure

The primary function of numerical modelling in the underground excavation design process is to simulate the stress distribution and the rock mass behaviour around the excavation and the influence of different support systems on the excavation stability. The main benefits of numerical modelling are that (a) both stress and displacements around the excavation can be computed, and (b) different constitutive relations for the rock mass can be employed. The numerical models used in rock engineering assume that rock masses can be mathematically represented either as a continuous media with elastic properties or as an assemblage of discrete blocks formed by pre-existing weakness planes; the blocks may be rigid or elastically deformable. As mentioned earlier, most numerical modelling methods discretise the rock mass into a large number of individual elements and achieve an iterative solution by repetitive calculation in a computer.

These methods are based on two different techniques: boundary and domain techniques. In the first only the boundary of the excavation is divided into elements and the interior of the rock mass is represented mathematically as an infinite continuum. The boundary element method (BEM) of analysis falls into the first class. Examples of commercially available BEM codes that are widely used in underground excavation design are EXAMINE-2D and EXAMINE-3D (Rocscience, 2009a, 2009b).

The domain technique divides the interior of the rock mass into geometrically simple zones each with assumed properties. The collective behaviour and interaction of these simplified zones model the more complex behaviour of the rock mass. Both continuous and discontinuous media can be modelled using domain technique. The finite element method (FEM) and finite difference method (FDM) are domain methods that treat the rock mass as a continuum, and the distinct element method (DEM) and discontinuous deformation analysis (DDA) are domain methods that model each individual block as a unique element in a discontinuous medium. PHASE2 (Rocscience, 1999) is a two dimensional, plane strain, FEM code suitable for modelling continuous media. FLAC and FLAC3D (Itasca, 2003a, 2003b) are FDM codes intended for rock masses that may behave as a continuum media, or sparsely jointed media. These software codes are useful for the analysis of rock instability caused by stresses with little or no influence from the discontinuities in the rock mass. The problem with the continuum media codes is that the assumption of a continuum may not be realistic for moderately jointed rock masses with intermediate stress conditions.

In contrast, UDEC and 3DEC (Itasca, 2004a, 2004b) are DEM codes that treat rock mass as an assemblage of discrete polygonal blocks and are well suited for modelling the effects of discontinuities in the rock mass and the behaviour of rock blocks formed by intersecting discontinuities. Since the rock masses encountered in the case tunnels studied in this research were mostly moderately jointed with intermediate stress conditions, UDEC was suitable and used.

*The Universal Distinct Element Code (UDEC)*

UDEC is a two dimensional stress and deformation analysis program that simulates the response of discontinuous media such as jointed rock masses subjected to either static or dynamic loading; the media is represented as an assemblage of discrete blocks. This approach requires that the location and, more importantly, the orientation of the discontinuities are known before the analysis is begun. The program can best be used when the geologic structure of the rock mass is fairly well understood from observations or mapping. Both manual and automatic joint generators are built into the program to create individual and sets of discontinuities which represent in two dimensions the jointed structure in a rock mass. Figure 3.11 shows a two dimensional presentation of four sets of discontinuities in a rock mass. Another advantage of UDEC is its ability to model excavations with irregular shapes resulting from geological over breaks, as can be seen from Figure 3.11.

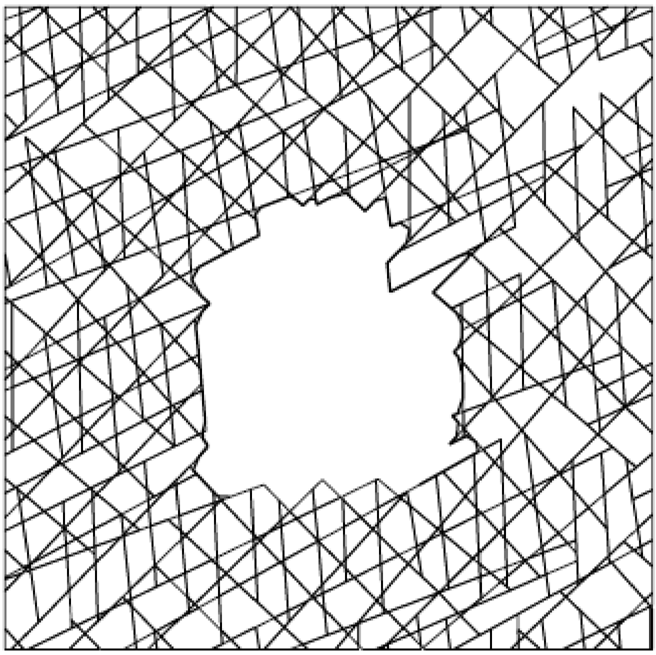

Figure 3.11 UDEC model for four joint sets in a rock mass

In UDEC the discontinuities are treated as boundary conditions between blocks; large displacements along discontinuities and rotation of blocks are allowed. Individual blocks behave as either rigid or deformable material. Deformable blocks are subdivided into a mesh of finite difference elements, and each element responds according to a prescribed linear or nonlinear stress-strain law. The relative motion of the discontinuities is also governed by linear or nonlinear force-displacement relations for movements in both the normal and shear directions. UDEC is ideally suited to study potential modes of rock failure directly related to the presence of discontinuities. Failure through the intact rock material can be judged from the results of the modelling in the same manner as for the continuum models, but it is not possible to simulate the formation of a fracture through intact rock material.

The program allows the simulation of the effects of support elements such as rock bolts, cable anchors ands shotcrete etc installed in an excavation as well as several other features including water seepage into and out of an excavation.

# CHAPTER 4

# ROCK MASS CLASSIFICATION METHODS

## 4.1 Introduction

In underground excavation engineering, rock mass classification methods have always played an important role, particularly in predicting support requirements for excavations in rock. Based on experience in broadly similar ground conditions elsewhere in previous projects, these methods relate rock mass conditions to support requirements and construction procedures in new projects. In other words, the classification methods are derived from a collection of prototype observations and they avoid the analysis of the potential failure mechanisms and the forces required to stabilise unstable rocks.

In contrast the rational or theoretical approach to underground excavation design uses explicit models representing the behaviour of rock masses developed based on the principles of the mechanics of materials. The application of this approach requires access to accurate information on the rock mass properties, groundwater conditions and in situ stress condition, and is often time consuming and costly.

While both approaches serve the same purpose, the classification methods are used when there is insufficient information to establish an explicit model or when time and cost limitations prevent the use of other models. This means in underground excavation engineering these are primarily found in two applications:

1. Before the commencement of construction when geological, geotechnical and construction data are limited, but time is not strictly limited. At this stage the main applications are for detailed planning and the design of initial support, determination of construction procedure and preliminary design of final support.

2. During construction when detailed information on the rock mass can be readily obtained by observations or simple tests, but time is limited due to contractual obligations and project completion deadlines. The main applications at this stage are for the determination and adaptation of initial support details, determination or confirmation of construction procedure and detailed design of the final support.

In order to be efficient and reliable at both stages of these applications, as noted by Steiner and Einstein (1980), a rock mass classification method should:

- be easily applicable and robust
- use easily determinable input parameters
- accurately represent rock mass behaviour
- avoid subjectivity
- ensure safety and economy

These are briefly discussed.

### 4.1.1 Applicability and robustness

A classification method should be applicable to a wide range of ground conditions, opening sizes and shapes, different construction procedures and support types. Although some experience in underground excavation design and construction may be a prerequisite, the application process of classification methods should not require a high level of skills. After a few applications, a user should be able to easily and confidently make required judgemental decisions. Simplicity of form and clarity and un-ambiguity of the terminology used are important features. In particular, the method should be relatively insensitive to vagaries in judgement either by the same user or by different users (Steiner and Einstein, 1980). It should be robust and repeatable without subjectivity and user bias, and also be applicable to a wide range of projects and project specific requirements. If this is not the case, the range of applicability should be explicitly described. For instance, road and railway tunnels have different requirements compared to water conveyance tunnels. Diverse requirements may also be applicable to different water conveyance tunnels. For example, water seepage from a water tunnel may be acceptable in some situations but not in others due to the economic value of water and potential problems in the surrounding environment that may not be acceptable.

### 4.1.2 Easily determinable parameters

While incorporation of the most significant geological and geotechnical parameters is of paramount importance, these should be easily determinable from standard investigation methods, which generally include core drilling and logging, rock outcrop mapping and geophysical techniques, and also from direct observations in excavations. Some simple physical testing may also be used. Only a limited number of parameters can be determined from the data sources available at the pre-construction (or exploration) stage, but time is not a limiting factor. In contrast, observations during construction in the excavation detect many details, but time is often limited. Parameters that can be easily obtained from outcrops and boreholes or quickly observed or measured in the excavation are desirable. Ideally, the parameters obtained at any stage of a project should lead to the same conclusions regarding the rock mass conditions and support requirements for excavations.

### 4.1.3 Accuracy of the method

Ground behaviour around an excavation is generally influenced by several factors such as lithology, the geological structure, groundwater and the in situ stress field. Some of these factors have several sub-factors with widely varying properties. For instance, geological structure includes joints, bedding shears and faults etc as well as their orientation, spacing, continuity, surface characteristics and filling materials etc. Lithology on the other hand may represent intact rock characteristics such as strength, elastic or plastic deformability, swelling and slaking etc. In strong discontinuous rock masses, the physical characteristics of geological structural features, i.e. orientation, persistence, spacing, aperture etc can vary in a wide range and their effect on the rock mass can be significant. However, in weak or weathered rock, the intact rock strength may become more important compared to geological discontinuities. The effect of geological structure may also depend on the size and orientation of the excavation. Similarly in deep excavations in situ or induced stress field may be the governing factor of stability while geological structure plays only a secondary role. Ideally the classification method should assess the relative importance of these factors and represent as exactly as possible their relative influence on different excavations. All relevant parameters should be accounted for by giving adequate ratings in the system, but no parameter should be counted more than once as this may reduce the weight of the other relevant parameters. In other words, the ground should be characterised by parameters that are exactly congruent with the true factors and their relative influence, and precautions be taken to avoid double counting of the same parameter.

### 4.1.4 Subjectivity

Naturally occurring rocks are anisotropic and non-homogeneous with widely varying properties. For instance, as already indicated, the characteristics of geological structural features, i.e. joint spacing, orientation, persistence etc that govern the behaviour of the rock mass can vary in a wide range. Similarly, other factors such as groundwater and lithology etc can also vary significantly. The determination of representative conditions or values of these factors should not involve uncertainty and subjectivity.

For a classification system to be devoid of subjectivity, the rock mass parameters should be identified quantitatively and their variations within the zone of interest accounted for in the assessment of a rock mass. The assessment or the allocation of numerical values for the parameters should not be user dependant. Where possible use of lump-sum ratings should be avoided as this leads to subjective adjustments when allocating rating values. Continuous rating systems, instead of lump-ratings, as proposed by Sen and Sadagah (2003) could reduce subjectivity. When the condition of a parameter varies, meaning the allocation of more than one value is possible, the system should provide guidance on the selection of the most critical value or a range of values for the assessment of the rock mass.

### 4.1.5 Safety and economy

Support and construction procedures should neither be overly conservative nor be optimistic and compromise safety. Ideally, the degree of safety provided should be known and be congruent with that required for different projects and different stages and/or sections of the same project. This means a pre-determinable factor of safety, which may change from case to case, should be included in the method. Built-in safety factors of unknown magnitude are not desirable.

### 4.1.6 The Available Rock Mass Classification Methods

Rock mass classification methods are known to have been used as a tool in rock engineering for over 100 years. According to Hoek et al. (1995), the earliest recorded attempt to formalise a rock mass classification method for predicting tunnel support requirements was by Ritter in 1879. Since then several methods have been developed and used in tunnel support design in different parts of the world. A comprehensive review of the classification methods was undertaken by Steiner and Einstein (1980). Subsequent to that review, some of the methods have been revised and updated and new methods have also been introduced. Methods that have been used in designing support for underground excavations in rock known to be available in the public domain include:

- Ritter's Method (Ritter, 1879)
- Bierbaumer's Method (Bierbaumer (1913)
- Kommerell's Method (Kommerell, 1940)
- Terzaghi's Rock Load Method (Terzaghi, 1946)
- Lauffer's Method (Lauffer, 1958, 1960)
- Stini's Method (Stini, 1960)
- Rock Quality Designation - RQD (Deere, 1963, 1968; Deere et al., 1967)
- Rock Structure Rating – RSR (Wickham et al., 1972, 1974)
- Rock Mass Rating – RMR (Bieniawski, 1973); and modifications thereof
- Tunnelling Quality Index – Q (Barton et al., 1974)
- Louis Method (Louis, 1974a, 1974b)
- Franklin's Method (Franklin, 1975, 1976)

- Rock Mass Index – RMi (Palmstrom, 1995, 1996a, 199b)

Up until the mid 1970s, Terzagi's Method, which is based on a descriptive classification of rock masses to estimate the rock load carried by steel sets, received the greatest attention of the tunnelling industry. Since its introduction, this method was extensively used for some 40 years, particularly in the USA, however, it is not directly applicable to the modern techniques of tunnel support comprising shotcrete and rock bolts, and is not widely used at present.

In general, of all the methods listed, Rock Quality Designation (RQD) can be singled out as the only method accepted universally, albeit not necessarily for tunnel support design. While RQD may be used for tunnel support design, it was originally developed as a simple and inexpensive indicator of general rock mass quality for various rock engineering projects. Subsequently, its use was extended to several different applications including the estimation of support for underground excavations. However, its application as a support design tool did not receive much acceptance, because a single parameter index system such as this cannot accurately predict the behaviour of a rock mass.

At present, the most widely used rock mass classification methods for underground excavation support design are Rock Mass Rating (RMR) and Tunnelling Quality Index (Q). In some situations some of the other methods can be of relevance and of considerable use, but unlike RMR and Q, they are not widely used in underground excavation design applications at present.

A relatively recent addition to the list of rock mass classifications is the RMi system (Palmstrom, 1996, 2000 and 2005). Compared to the other systems, this method is considered to better represent the size of rock blocks that form the rock mass. However, it is yet to gain wide acceptance as a support design tool for underground excavations. Among the other rock mass classification methods used in rock engineering applications, the Geological Strength Index (GSI) discussed in Section 2.5.2 deserves a mention. Developed by Hoek and co-workers (Hoek, 1994; Hoek, Kaiser and Bawden, 1995; Hoek and Brown, 1997; Hoek, Marinos and Benissi, 1998; Marinos and Hoek, 2001) it is a tool for estimating two of the constants in the

Hoek-Brown failure criterion of intact rock and rock masses. Although GSI is useful as a platform for providing input values to numerical methods of underground excavation design, on its own right, it is not a method of support design.

During the past few decades, rock engineering researchers in China have also proposed several rock mass classification methods for the application in tunnelling. These methods were published in Chinese and up to now are not available in English to rock engineering practitioners in other countries. Chinese researchers Lee et al. (1996), however, presented a classification system intended for water resources and hydroelectric power tunnels in China. This system is based on six rock mass parameters similar to the RMR and Q systems and may be considered an improvement of the RMR and Q systems as it takes into account almost all the parameters covered by the two.

A method known as the CRIEPI system also deserves mention as it was used in some of the case tunnels discussed in this research. The CRIEPI system was initially developed for dam engineering works by Tanaka (1964) and Kikuchi et al. (1982) of the Central Research Institute of Electric Power Industry (CRIEPI) of Japan. The Electric Power Development Co Ltd (EPDC) of Japan modified the CRIEPI system and used a simplified version for underground excavations in sedimentary rocks in the Northeastern Province of Thailand and the modified system was found to give reasonable results in underground excavation design (Phienwej and Anwar, 2005). The system uses three rock mass parameters: weathering, hardness (expressed in terms of UCS of intact rock substance) and joint spacing. However, up to now, the use of this method is limited to projects designed by EPDC.

One other method widely used (not included in the list) is the New Austrian Tunnelling Method (NATM) (Rabcewicz, 1964a, 1964b, 1965). Although NATM is sometimes considered as an empirical design tool based on the classification of rock mass, it can be more accurately described as an observational approach. A brief discussion of NATM is provided in Section 3.2.

Singh and Goel (1999) provided a comprehensive discussion on the application of some of the rock mass classification methods to both underground and surface excavations as well as to predict engineering properties of rock masses. They dealt with the application of Terzaghi's rock load concept, RQD, RMR, Q, RMi, GSI, NATM and one truncated version each of RMR and Q. They proposed some improvements, particularly to the Q system, based on the research undertaken in India in collaboration with the Norwegian Geotechnical Institute where the Q system was developed. Their research has mostly being in the tunnel constructed under high in situ stress conditions in the Himalayas. Their finding and recommendations will be discussed in Chapter 5.

Despite the availability of several rock mass classification methods, only the RMR and Q methods have been universally accepted as underground excavation design tools. Unlike the single parameter RQD, these two methods are multi-parameter classifications. Both use RQD as one of the key parameters to classify a rock mass. The RQD, RMR and Q methods are discussed in the following sections.

**4.2 Rock Quality Designation (RQD)**

Rock Quality Designation (RQD) is a measure of the quality of rock core recovered by core drilling. It was introduced by Deere (1963) as an index of rock quality and was defined as the proportion of borehole core that consists of intact lengths that are 100 mm or longer. To calculate the RQD value, these intact lengths are summed and expressed as a percentage of the total length as given below:

$$RQD = 100\sum_{i=1}^{n}\frac{X_i}{L} \tag{4.1}$$

where $X_i$ is the length of the i$^{th}$ length $\geq$ 100 mm, $n$ is the number of intact lengths $\geq$ 100 mm, $L$ is the length of borehole section along which the RQD value is required. Though RQD can be calculated for various sections or the complete length of the borehole, usually $L$ is taken as the length of each core run (governed by the length of core barrel) in the determination of the RQD value. The correct procedure for measuring RQD is illustrated in Figure 4.1.

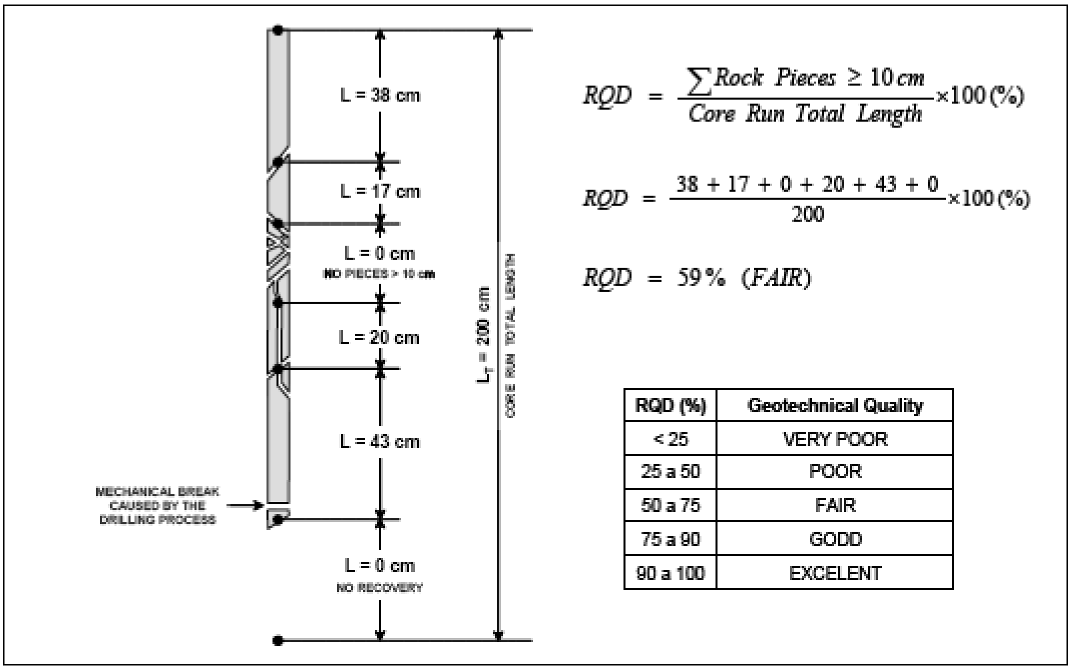


Figure 4.1 Definition and example application of RQD (after Deere and Deere, 1988)

The RQD percentage includes only the pieces of sound core over 100 mm long and pieces of highly weathered or disintegrated rock are not counted even though they possess the requisite 100 mm length.

Deere et al. (1967) suggested that a scanline, along which discontinuity intensity and spacing on a rock face can be measured, may be regarded as directly analogous to a borehole core since the RQD can be found in both cases. When a drill core is unavailable, RQD may also be estimated from discontinuity frequency measured along a sampling line (scanline) as suggested by Priest and Hudson (1976) or from volumetric joint (discontinuity) count proposed by Palmstrom (1982). The approximate relation between RQD and discontinuity frequency suggested by Priest and Hudson (1976) is

$$RQD = 110.4 - 3.68\lambda \tag{4.2}$$

where $\lambda$ is discontinuity frequency along a sampling line. The formula for volumetric joint count in clay free rock masses, proposed by Palmstrom (1982) is

$$RQD = 115 - 3.3J_v \quad (4.3)$$

where $J_v$ is the total number of discontinuity per cubic meter.

RQD is a simple and inexpensive general indicator of rock mass quality. The recording of RQD is virtually standard practice in drill core logging for a wide variety of rock engineering investigations. This index has been frequently used to indicate the general quality of rock and is a good indicator of the presence of fractured or weak zones which usually receive a low RQD value. Since RQD is relatively easy to calculate and provides an unambiguous numerical value, it has become widely accepted as a measure of discontinuity spacing (Priest, 1993). The RQD values provide a basis for (a) making preliminary design decisions involving estimation of required depths of excavation for foundations of structures, (b) identifying potential problems related to bearing capacity, settlement, erosion, or sliding in rock foundations, and (c) indicating rock quality in quarries for concrete aggregate, rock fill, or large riprap.

Based on RQD, Deere and co-workers (Deere et al., 1967, Deere, 1968, Cording and Deere, 1972 and Merritt, 1972) developed guidelines for the selection of support for 6 to 12 m wide rock tunnels. Different support quantities are given for conventional drill and blast excavated tunnels and bored tunnels. The support types are steel sets, rock bolts and shotcrete or a combination of the three.

RQD has some limitations. Although it is considered to represent the “jointing”, and to some extent the “strength of intact rock substance” in a rock mass, it does not fully represent the true nature of the joints or the intact rock strength. For instance, the same RQD may represent different rock structures with respect to joint spacing, roughness, tightness, persistence and orientation. RQD is sensitive to the orientation of joint sets with respect to the orientation of the core, that is, a joint set parallel to the core axis will not intersect the core unless the drill hole happens to run along the joint. A joint set perpendicular to the core axis will intersect the core axis at intervals equal to the joint spacing. For intermediate orientations, the spacing of joint intersections with the core will be a cosine function of angle between joints and the core axis. If zones of low joint shear strength are in a rock with high RQD, more support may be necessary than predicted only with RQD. Further, the pieces of core that are 100 mm or larger may have different strengths. In other words, the intact rock strength is also not well represented by the RQD value. It is universally accepted that no single parameter or index such as RQD can describe completely and quantitatively rock mass behaviour for engineering purpose (Sen and Sadagah, 2003). Used alone, RQD is not sufficient to provide an adequate description of rock mass quality. It must be used in combination with other geological and geotechnical input. While the determination of RQD is an integral part of a wide variety of geotechnical investigations, it alone is no longer used as a tool to predict tunnelling conditions and support requirements. However, as already mentioned, it forms an important input parameter to the two most widely used classification systems available at present which are discussed in the following sections of this chapter.

## 4.3 Rock Mass Rating (RMR)

The Rock Mass Rating (RMR) system was developed by Bieniawski (1973) based on experience gained from civil engineering tunnels constructed primarily in South Africa. The method uses six parameters considered to represent the engineering behaviour of rock masses. Each parameter is divided into five separate ranges of values (ratings), and is rated based on the observed or measured condition in a rock mass. The sum of the ratings assigned to the six parameters is defined as the RMR value. The six parameters are not given equal importance in the overall classification of a rock mass and different ranges of ratings are allocated to different parameters. Nevertheless, a higher rating indicates a better rock mass condition. The six parameters used in the RMR method are:

Intact rock strength (*IRS*): is the uniaxial compressive strength (UCS) of intact rock substance determined by laboratory testing of cylindrical core samples. The UCS may be indirectly determined by point load strength (PLS) index test.

| | |
|---|---|
| Drill core quality (*RQD*): | is the rock quality designation. This may be determined by drill core logging or scan line surveys of the exposed rock surfaces (see section 4.2). |
| Joint spacing (*JS*): | is the spacing of the joints (which included joints, bedding, shears and faults etc) present in the rock mass. |
| Joint surface condition (*JC*): | represents the joint surface characteristics which include joint persistence, aperture, roughness, infilling material and weathering. |
| Groundwater condition (*GW*): | represents the amount of water inflow in 10 m intervals of a tunnel or the groundwater pressure in joints. |
| Rating adjustment (*RA*): | represents the orientation of the most significant joint set with respect to the direction of the tunnel. |

Since its introduction in 1973, revised versions of the RMR method have also been published (Bieniawski 1974, 1975, 1976, 1979 and 1989).

From 1973 to 1989 the ratings scales and some of the parameters used in the RMR system have changed as listed in Table 4.1. In the 1973 version, eight parameters were used and from 1974 this was reduced to six by combining joint separation, continuity and weathering parameters of the first version to create the joint condition parameter, *JC*. From 1974 to 1975 the maximum ratings given to *JC* and *IRS* were increased by 10 and 5 points, respectively.

In the 1973 and 1974 versions, the *RA* parameter was given a positive rating ranging from 0 for the most unfavourable orientation to 15 for the most favourable orientation. Since 1975 this parameter was given a negative rating from 0 for the most favourable orientation to -12 for the most unfavourable orientation. From 1975 to 1976 the rating scales were not changed but the rock mass class boundaries for support selection were. In the 1979 version, the maximum rating for *JS* term was reduced by 10 points and the influence of both *JC* and *GW* was increased by 5 rating points each. In the 1989 version the assessment of sub-horizontal discontinuities (joints) was changed from unfavourable to fair for the stability of tunnels. This results in a difference of 5 rating points in the overall assessment of the RMR value.

Table 4.1 Rating allocations in different versions of the RMR system

| Parameter | 1973 | 1974 | 1975 | 1976 | 1979 | 1989 |
|---|---|---|---|---|---|---|
| Intact rock strength (*IRS*) | 0 - 10 | 0 - 10 | 0 - 15 | 0 - 15 | 0 - 15 | 0 - 15 |
| *RQD* | 3 - 16 | 3 - 20 | 3 - 20 | 3 - 20 | 3 - 20 | 3 - 20 |
| Joint spacing (*JS*) | 5 - 30 | 5 - 30 | 5 - 30 | 5 - 30 | 5 - 20 | 5 - 20 |
| Separation of joints | 1 - 5 | | | | | |
| Continuity of joints | 0 - 5 | | | | | |
| Weathering | 1 - 9 | | | | | |
| Condition of joints (*JC*) | - | 0 - 15 | 0 - 25 | 0 - 25 | 0 - 30 | 0 - 30 |
| Groundwater (*GW*) | 2 - 10 | 2 - 10 | 0 - 10 | 0 - 10 | 0 -15 | 0 - 15 |
| Rating adjustment (*RA*) | 3 - 15 | 3 - 15 | 0 - (-12) | 0 - (-12) | 0 - (-12) | 0 - (-12) |

As a result of the changes in the ratings scales and some of the parameters used, different *RMR* values may be given to the same rock mass by different versions. Detailed reasons for the changes in different versions are not spelt out. It is apparent that with experience gained by applying the method to more cases, modifications became possible and necessary. The 1989 version provided Rating Charts for the *IRS*, *RQD* and *JS* parameters additionally to the ranges of ratings presented in tabulated form. The charts are helpful for borderline cases and also remove an impression that abrupt changes in ratings occur between categories (Bieniawski, 1989). The 1989 version also provided detailed guidelines on the selection of ratings for the *JC* parameter. The classification parameters described earlier represent the 1989 version. The recommended ratings for the six parameters as per Bieniawski (1989) are presented in Table 4.2.

Table 4.2 RMR classification parameters and their ratings (after Bieniawski, 1989)

| | Parameter | | Ranges of values | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 1 | *IRS* | PLSI (MPa) | >10 | 4 - 10 | 2 - 4 | 1 - 2 | For this low range UCS is preferred | | |
| | | UCS (MPa) | >250 | 100 - 250 | 50 - 100 | 25 - 50 | 5-25 | 1-5 | <1 |
| | 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 | *JS*: joint spacing (mm) | | >2000 | 600 - 2000 | 200 - 600 | 60 - 200 | <50 | | |
| | Rating | | 20 | 15 | 10 | 8 | 5 | | |
| 4 | *JC*: joint condition | | Very rough surfaces. Not continuous. No separation. Unweathered wall rock | Slightly rough surfaces. Separation < 1 mm. Slightly weathered walls. | Slightly rough surfaces. Separation < 1 mm. Highly weathered walls. | Slickensided surfaces OR Gouge < 5 mm thick OR Separation 1-5 mm. Continuous | Soft gouge > 5 mm thick OR separation > 5 mm. Continuous | | |
| | Rating | | 30 | 25 | 20 | 10 | 0 | | |
| 5 | *GW*: groundwater condition | Inflow per 10 m tunnel length | None | <10 l/min | 10 – 25 l/min | 25 – 125 l/min | >125 l/min | | |
| | | Ratio (Joint water pressure:Major principle stress) | 0 | 0.0 - 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 | | |
| 6 | Strike and dip of joints | | Very favourable | Favourable | Fair | Unfavourable | Very unfavourable | | |
| | *RA*: Rating adjustment | Tunnels | 0 | -2 | -5 | -10 | -12 | | |
| | | Foundations | 0 | -2 | -7 | -15 | -25 | | |
| | | Slopes | 0 | -5 | -25 | -50 | -60 | | |

For the first five parameters, the average typical conditions are evaluated and the ratings are interpolated using the table and charts provided. The charts in Figures 4.2, 4.3 and 4.4 are used in combination with Table 4.2 for determining the ratings for *IRS, RQD* and *JS* parameters, and the chart in Figure 4.5 is used if either *RQD* or discontinuity spacing data are lacking. Bieniawski (1989) states that, the importance ratings of discontinuity spacing apply to rock masses having three or more sets of discontinuities. When less than three sets of discontinuities are present, the rating for discontinuity spacing parameter may be increased by 30%. The ratings for *JC* parameter are obtained by considering five discontinuity properties, namely persistence, aperture, roughness, infilling and weathering, as given in Table 4.3. Each descriptive parameter is rated using Table 4.3 and the sum of the five ratings is the rating value for the *JC* parameter. Ratings for *GW* parameter are obtained directly from Table 4.2. After the importance ratings for the first five parameters (*IRS, RQD, JS, JC* and *GW*) are established, the five rating are summed to yield the basic RMR (unadjusted for discontinuity orientation) value for the rock mass region under consideration. The sixth parameter (rating adjustment) is treated separately because the influence of strike and dip orientation of discontinuities depends on the engineering application, such as a tunnel, slope or foundation. To obtain ratings for the sixth parameter, Table 4.2 is used in conjunction with Table 4.4 which explains the effect of discontinuity strike and dip orientation on the stability of tunnels, foundations and slopes. The *RA* parameter reflects on the significance of the orientation of various discontinuity sets present in a rock mass. The RMR method considers that the discontinuity set whose strike is parallel to the tunnel axis controls the stability. In situations where no one discontinuity set is dominant and or of critical importance, ratings from each discontinuity set are averaged for the appropriate individual classification parameter. The sum of the ratings assigned to all the six parameters yields the overall *RMR* value for a given rock mass as follows:

$$RMR = IRS + RQD + JS + JC + GW + RA \tag{4.4}$$

where *IRS, RQD, JS, JC, GW* and *RA* are as defined earlier.

The *RMR* value, which varies from 0 to 100 on a linear scale, is then related to five rock mass classes (Table 4.5) and each class in turn is related to support measures.

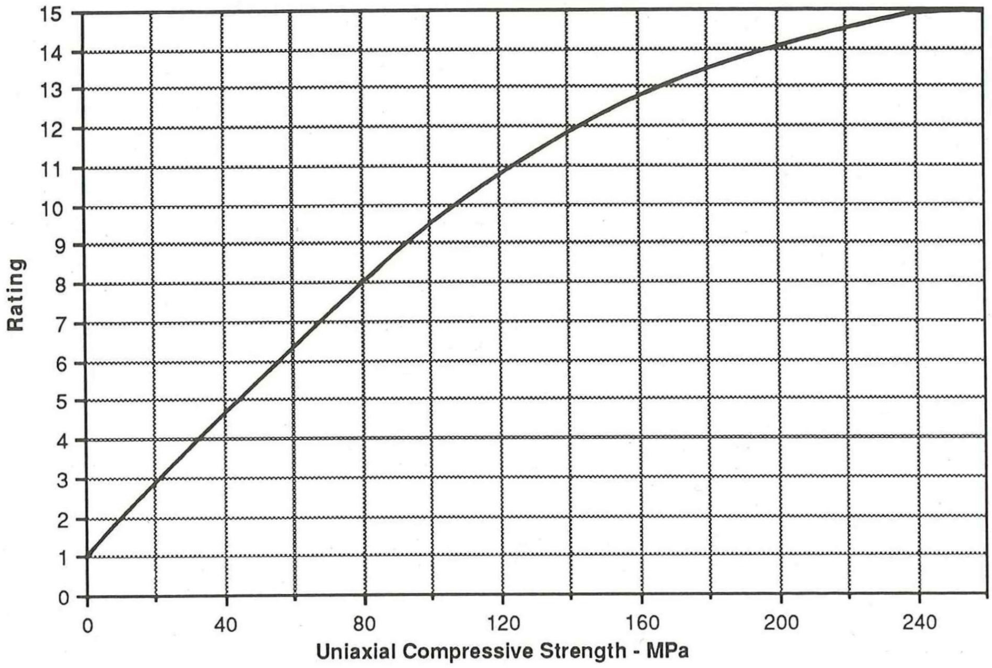


Figure 4.2 Ratings chart for intact rock strength (IRS) (after Bieniawski, 1989)

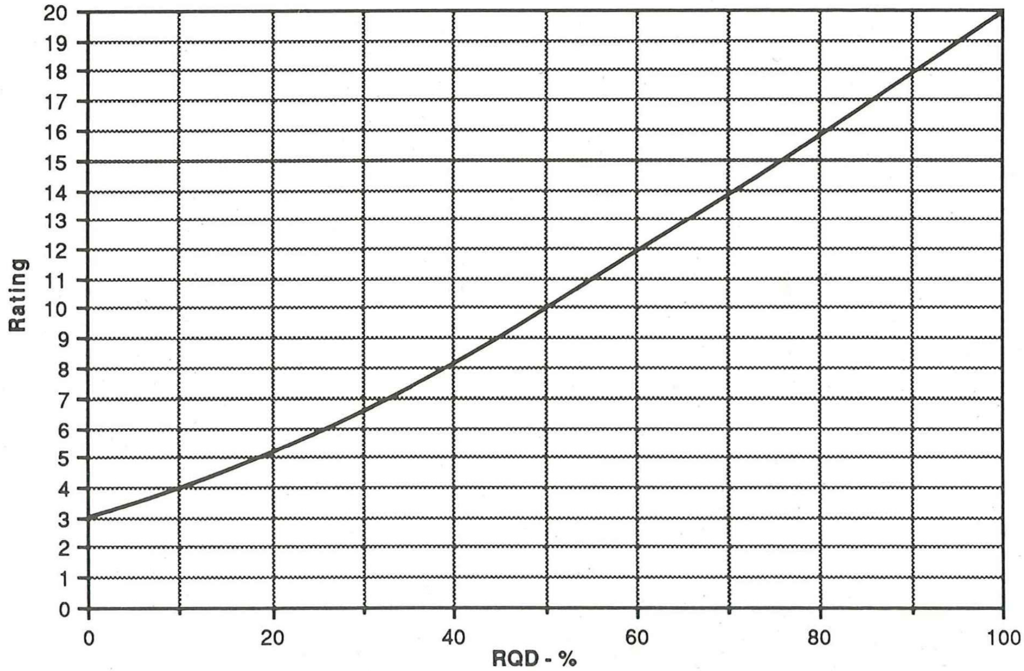


Figure 4.3 Ratings chart for RQD (after Bieniawski, 1989)

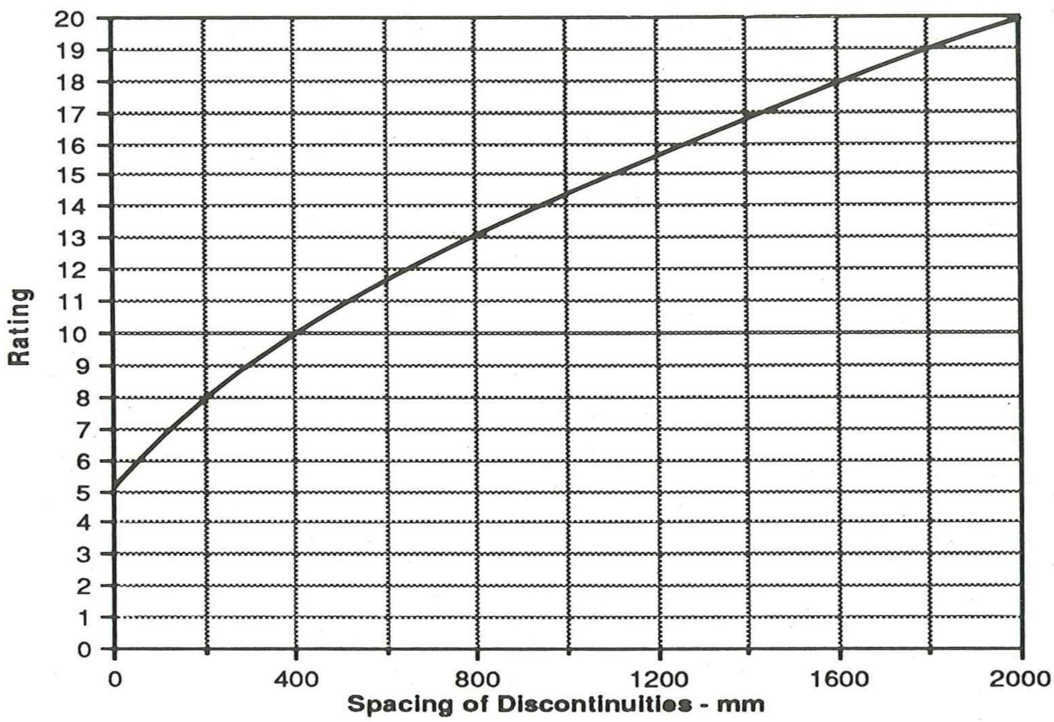


Figure 4.4 Ratings chart for discontinuity spacing (after Bieniawski, 1989)

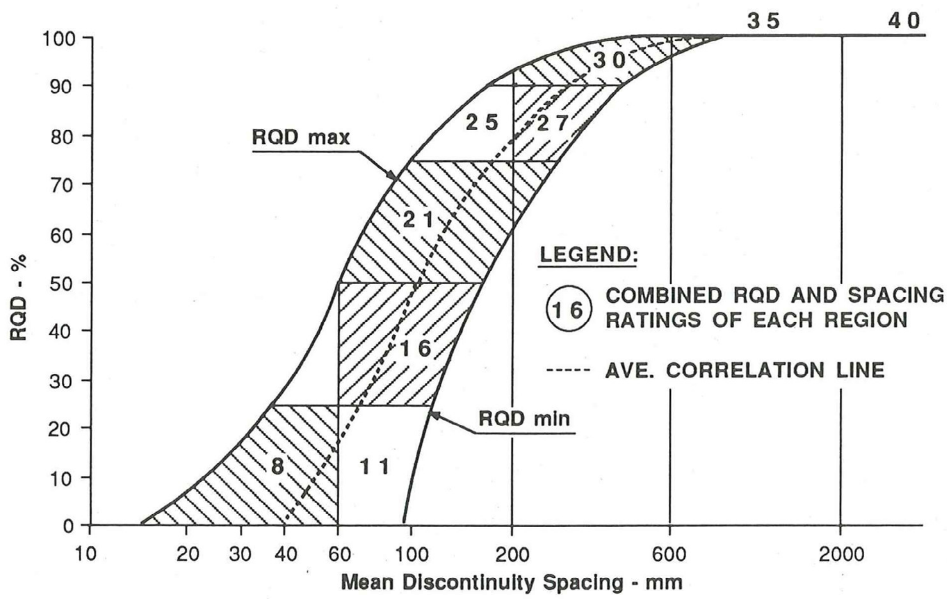


Figure 4.5 Chart for correlation of *RQD* and discontinuity spacing (*JS*)
(after Bieniawski, 1989)

With changing versions the RMR class boundaries have also changed somewhat. From 1973 to 1975 rock mass class V was assigned for *RMR*<25, class IV for *RMR*=25-50, class III for *RMR*=50-70, class II for *RMR*=70-90 and class I for *RMR*=90-100. From 1976 the classes were separated evenly on the RMR scale with the boundaries at multiples of 20. The class boundaries given in Table 4.5 represent the current version. Table 4.5 also provides rock mass shear strength in terms of cohesion and friction angle. From 1976 to 1979 strength properties of some of the RMR classes were also changed. In class I cohesion was increased, the friction angle remained the same. In class V cohesion remained the same but the friction angle was reduced by a factor of two.

The RMR system provides guidelines detailed in Table 4.6 for the selection of support and construction procedures for 10 m wide tunnels with a vertical stress magnitude of less than 25 MPa excavated by drill and blast methods. The support measures recommended are for permanent and not primary support. It should be noted that between different versions some changes were also made to the RMR recommended support and excavation procedures. The recommendations of 1973 and 1974 versions had three separate support systems a user can select: the first being rock bolts with shotcrete and wire mesh as additional support; the second is shotcrete with rock bolts, wire mesh and steel sets as additional support; the third is steel sets with shotcrete as additional support. In 1975 the support comprised a combination of rock bolts, shotcrete, wire mesh and steel sets depending on the rock class. From 1975 to 1989 the support recommendations virtually remained the same apart from minor adjustments. Recommendations up to the 1975 version were for 5 to 12 m wide tunnels with a maximum vertical stress of 30 MPa (the maximum stress was given only in 1975). From 1976 the support measures are only for 10 m wide tunnels with a maximum vertical stress of 25 MPa. Ironically, the support recommended from 1976 onwards is identical to that recommended for 5 to 12 m wide tunnels with a maximum vertical stress of 30 MPa.

Table 4.3 Guidelines for classification of discontinuity conditions (*JC*) (after Bieniawski, 1989)

| Parameter | Ranges of values | | | | |
|---|---|---|---|---|---|
| Persistence (m) | < 1 | 1-3 | 3-10 | 10-20 | >20 |
| Rating | 6 | 4 | 2 | 1 | 0 |
| Aperture (mm) | None | < 0.1 | 0.1-1.0 | 1-5 | > 5 |
| Rating | 6 | 5 | 4 | 1 | 0 |
| Surface roughness | Very rough | Rough | Slightly rough | Smooth | Slickensided |
| Rating | 6 | 5 | 3 | 1 | 0 |
| | | Hard filling | | Soft filling | |
| Infilling (mm) | None | < 5 | > 5 | < 5 | > 5 |
| Rating | 6 | 4 | 2 | 2 | 0 |
| Weathering | Unweathered | Slightly weathered | Moderately weathered | Highly weathered | Decomposed |
| Rating | 6 | 5 | 3 | 1 | 0 |

Table 4.4 Effect of discontinuity strike and dip in tunnelling (after Bieniawski, 1989)

| Strike perpendicular to tunnel axis | | | | Strike parallel to tunnel axis | | Dip 0-20$^o$ irrespective of strike |
|---|---|---|---|---|---|---|
| Drive with dip | | Drive against dip | | | | |
| Dip 45-90$^o$ | Dip 20-45$^o$ | Dip 45-90$^o$ | Dip 20-45$^o$ | Dip 45-90$^o$ | Dip 20-45$^o$ | |
| Very favourable | Favourable | Fair | Unfavourable | Very favourable | Fair | Unfavourable |

Table 4.5 Rock mass classes determined from total ratings (after Bieniawski, 1989)

| Final RMR value | Rock mass class and description | Average stand up time | Rock mass cohesion (kPa) | Rock mass friction angle |
|---|---|---|---|---|
| 100 - 81 | I - Very good rock | 10 years for 15 m span | >400 | >45$^o$ |
| 80 - 61 | II – Good rock | 6 months for 8 m span | 300 - 400 | 35 - 45$^o$ |
| 60 - 41 | II - Fair rock | 1 week for 5 m span | 200 - 300 | 25 - 35$^o$ |
| 40 - 21 | IV – Poor rock | 10 hrs for 2.5 m span | 100 - 200 | 15 - 25$^o$ |
| ≤20 | V – Very poor rock | 30 min for 1 m span | <100 | <15$^o$ |

Table 4.6 Excavation and support in horseshoe shaped 10 m wide drill and blast excavated rock tunnels with vertical stress < 25 MPa (after Bieniawski, 1989)

| Rock Mass Class | Excavation | Support | | |
|---|---|---|---|---|
| | | Bolts: 20mm diameter fully grouted | Shotcrete | Steel sets |
| I - Very good<br>RMR: 100-81 | Full face 3 m advance. | Generally no support required except for occasional spot bolting | | |
| II – Good<br>RMR: 80-61 | Full face 1.0-1.5 m advance. Complete support 20 m from face. | Locally bolts in crown, 3 m long, spaced 2.5 m, with occasional mesh | 50 mm in crown where required | None |
| III – Fair<br>RMR: 60-41 | Top heading & bench, 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face. | Systematic bolts 4 m long, spaced 1.5-2 m in crown & walls with wire mesh in crown | 50-100 mm in crown & 30 mm in sides | None |
| IV - Poor<br>RMR: 40-21 | Top heading & bench, 1.0-1.5 m advance in top heading. Install support concurrently with excavation 10 m from face. | Systematic bolts 4-5 m long, spaced 1-1.5m in crown & walls with mesh | 100-150 mm in crown & 100 mm in sides | Light to medium ribs spaced 1.5 m where required. |
| V - Very poor<br>RMR: 20-0 | Multiple drifts. 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting. | Systematic bolts 5-6 m long, spaced 1-1.5 m in crown & walls with wire mesh. Bolt invert | 150-200 mm in crown & 150 mm in sides & 50 mm in face | Medium to heavy ribs spaced 0.75 m; lagging & forepoling if required. Close invert |

The other main output of the RMR method is the stand-up time and the maximum stable unsupported span of underground excavations such as tunnels, chambers and mines. The method provides an unsupported span versus stand-up time chart (Figure 4.6) for the five RMR rock classes. In combination with the RMR value, the chart may be used to estimate the stand-up time before the installation of initial support.

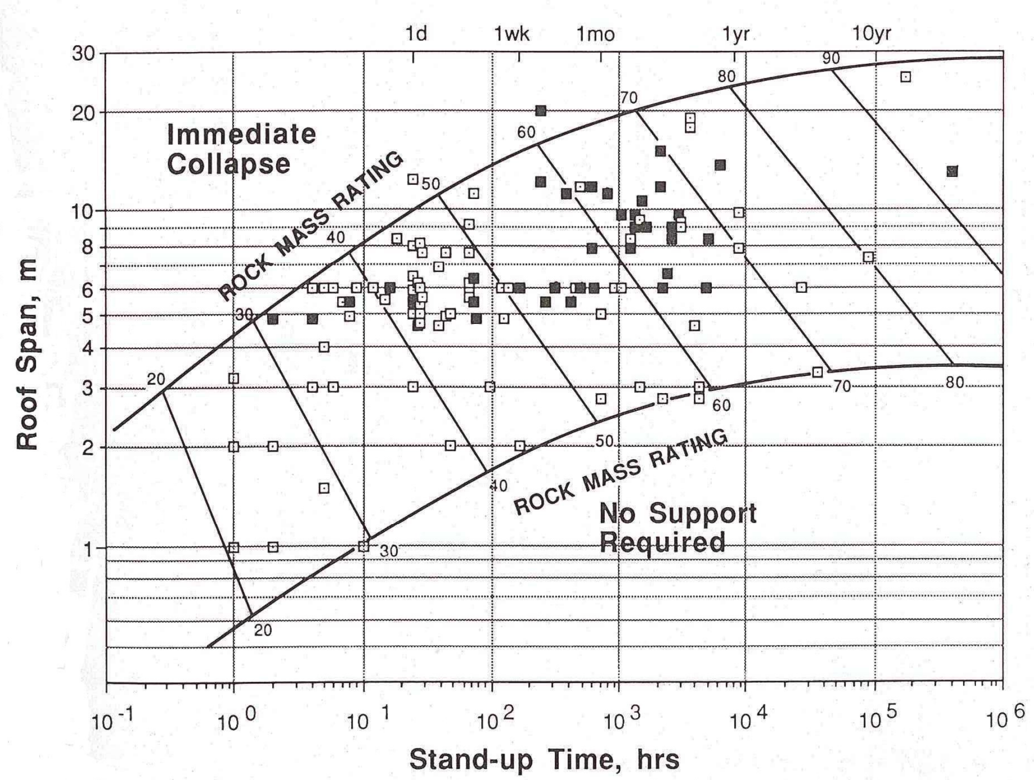


Figure 4.6 Stand-up time and span relationship for various rock classes (after Bieniawski, 1989)

The stand-up times and unsupported spans in the RMR systems have changed from version to version. For instance, in the 1974 version the maximum unsupported span was 10 m and the maximum stand-up time was 10 years. The maximum self supporting span (for indefinite time) was 2 m. In the 1975 version the maximum unsupported span was increased to 20 m. Bieniawski (1976) included the Scandinavian and Austrian cases in the 1976 version and claimed that the maximum unsupported span in rock classes I and II could reach 20 to 70 m and for class III it could be between 7.5 and 20 m. In that version the maximum self supporting span was increased up to 9 m. In 1979, stand-up time for a given rock class was increased and the maximum unsupported span was reduced to 30 m. The unsupported span versus stand-up time chart was amended once more in 1989 to that given as Figure 4.6, in which the maximum stand-up time is 100 years as opposed to 10 years in the previous versions. A point that should be mentioned here is that while the aforementioned changes are being made to the unsupported span versus stand-up time chart, the support recommendations remained the same as in the 1976 version.

Bieniawski (1989) also provided a RMR versus support load relationship that may be used to determine the support pressure required to stabilise the rock mass surrounding an excavation. The relationship proposed on the basis of the work by Unal (1983) in flat roofed coal mine openings is

$$P = \frac{100 - RMR}{100} \gamma B \tag{4.5}$$

where, $P$ is the support load in kN, $B$ is the tunnel width in meters and $\gamma$ is the rock density in kg/m$^3$. Goel and Jethwa (1991) have evaluated the applicability of Equation 4.5 to rock tunnels with arched roof and found that the estimated support pressures were unsafe for all sizes of tunnels under squeezing ground conditions. Further the estimates for non-squeezing ground conditions were unsafe for small diameter tunnels (diameter up to 6 m) and over-safe for large diameter tunnels (diameter>9 m), which implied the size effect was over-emphasised for large diameter arched openings. Subsequently, based on measured support pressure values from 30 Indian tunnels, Goel and Jethwa (1991) proposed Equation 4.6 for estimating short term support pressures for tunnels in both squeezing and non-squeezing ground conditions in the case of excavation by drill and blast methods with steel arch support:

$$P_s = \frac{0.75B^{0.1}\ H^{0.5} - RMR}{2RMR} \tag{4.6}$$

where, $P_s$ is short-term support pressure in MPa, $B$ is tunnel span in metres, $H$ is overburden or tunnel depth in metres.

As can be seen from the above equations, the RMR system considers that the support pressure not only depends on the rock mass quality (or RMR value), but also on the width of the opening. This means that although the RMR recommended support quantities are primarily aimed at 10 m span tunnels, the *RMR* versus support pressure relationship can be applied to any tunnel regardless of its span. Naturally, as would be expected, according to Equation 4.5 and 4.6 different support pressures will be required for different tunnel spans in the same rock mass.

The RMR system can also be used for estimating the modulus of rock mass deformation ($E_M$). Bieniawski (1978) provided the following formula for estimating $E_M$ (in GPa) when $RMR > 50$ is

$$E_M = 2RMR - 100 \qquad (4.7)$$

For rock masses with $RMR < 50$, based on the work of Serafim and Pereira (1983), the formula given by Bieniawski (1989) is

$$E_M = 10^{(RMR\text{-}10)40} \qquad (4.7a)$$

Hoek and Brown (1997) suggested that a correction in terms of intact rock material strength, $\sigma_c$, should be included in Equation 4.7a to better represent poor rocks with $\sigma_c$<100 MPa. The suggested correction is given below in Equation 4.7b:

$$E_M = \{(\sigma_c)^{1/2}/10\}\ 10^{(RMR\text{-}10)40} \qquad (4.7b)$$

The RMR system has also found applications in Hoek-Brown failure criterion for rocks (Hoek and Brown, 1980a), which is widely used for estimating rock mass strength for various rock engineering applications. The RMR method is easy to use, with relatively less expertise requirements.

### 4.3.1 Modification of the RMR System

In spite of the fact that the RMR system was primarily aimed at civil engineering tunnel construction, the method was not robust enough to apply to all possible rock conditions in civil projects. In order to broaden its application to a wider range of ground conditions, several researchers have modified the RMR system. Notable among these is the work of Gonzalez de Vallejo (1983, 1985) who modified it primarily to take into account the in situ stress field in a rock mass. The modified version denoted as Surface Rock Classification (SRC) is intended to be used in civil engineering tunnels under moderately to very highly stressed rock mass conditions and allows for the consideration of data obtained from rock outcrops, rock durability, the effect of adjacent excavations and portals on stability and tunnel construction method.

Similar to the RMR system, the modified system also first determines the basic *SRC* value based on five parameters. In the SRC system, the RMR parameters *IRS, JC* and *GW* have not been changed and the *RQD* and *JS* parameters have been linked to eliminate double counting of joint spacing and renamed as *RQD/JS*. A new parameter called State of Stress (*SS*) which comprised four sub-parameters, namely, competence factor, tectonic structure, stress relief factor and neo-tectonic activity has been added. The sum of the ratings allocated to the first five parameters in the basic *SRC* value. The basic *SRC* value is then adjusted to take into account the construction conditions using construction adjustment (*CA*), which is a function of method of excavation, stand-up time (based on RMR), distance to the adjacent excavation and/or portal, rock resistance to weathering (durability) and discontinuity orientation (as per RMR). In the SRC system, the rating scales and the manner in which the final *SRC* value is computed is very much similar to that of the RMR system and therefore the same rock mass classes and support measures are used. The SRC system takes into account some of the missing parameters in the RMR system, namely, in situ stress, rock durability, construction method and the effect of adjacent excavations. The system has been applied in civil engineering tunnels in Spain and Northern Italy. However, it has not received much attention in other parts of the world.

### 4.3.2 Modification of RMR for Mining

The RMR method, originally developed based on case records mainly drawn from civil engineering projects, was modified to make it more relevant to mining applications. One such modification introduced by Laubscher in the mid 1970s is designated as Mining Rock Mass Rating (MRMR) (Laubscher, 1975, 1977; Laubscher and Taylor, 1976). This MRMR system 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. A set of support recommendations is also provided with the MRMR. Application of the MRMR to mining projects has been presented by Laubscher (1984, 1990, 1993, 1994, 2000), Laubscher and Page (1990), Jakubec and Laubscher (2000) and Laubscher and Jakubec (2001). In MRMR, the form of computation and the values of the ratings associated with the different parameters used have changed several times since the introduction of the method in 1975 to the latest version presented by Laubscher and Jakubec (2001). MRMR was initially proposed for block caving mines in Africa, but other cases were subsequently added to its database. Nevertheless, up to now MRMR has not been widely used in other mining operations.

Cummings et al. (1982) and Kendorski et al. (1983) have also modified the RMR method for mining applications in the USA and introduced Modified Basic Rock Mass Rating (MBR). This system was also proposed for block caving operations. It involves the use of different ratings for the original parameters used to determine the value of RMR and subsequent adjustment of the resulting MBR value to allow for blast damage, induced stresses, structural features, distance from the cave front and size of the caving block (Hoek et al., 1995). It provides support recommendations for isolated or development drifts as well as the final support of intersections and drifts. This method also has not been widely used in other mining applications.

Brook and Dharmaratne (1985) proposed a simplified version of RMR (SRMR) for mining applications. From the available literature it is apparent that this version also did not receive much attention.

### 4.3.3 Limitations of the RMR method

In addition to the six rock mass parameters considered in the RMR method, the support requirements for an excavation depend on the in situ stress field, excavation size and shape, and the method of excavation. In the RMR method, the in situ stress field is not considered specifically though it is used indirectly when recommending support. The 25 MPa maximum vertical stress limit means that the support measures recommended are applicable to an approximate tunnel depth or overburden thickness range of 0 (near surface) to 1000 m. Within this depth range, depending on the horizontal to vertical stress ratio k, a range of horizontal stress magnitudes is possible and the horizontal stresses could be more critical to the stability of an excavation compared to the vertical stress. This aspect does not appear to have been considered in developing the RMR method.

For different *IRS* and *JS* values of the RMR system, similar *RMR* values could be achieved for different rock masses if the other four parameters are identical. This may be explained using a hypothetical example. In a rock mass with no discontinuities and an *IRS* range of 5 to 25 MPa, the *JS* and *IRS* parameters would contribute 22 points to the total *RMR* value. Similarly, a rock mass with *JS* of 200 to 600 mm with an *IRS* range of 100 to 250 MPa would also contribute 22 points to the total *RMR* value. In the first case, as there are no discontinuities the *RQD* is likely to be 100%. This means the combined contribution of *IRS*, *JS* and *RQD* to the total *RMR* value is 42. With a joint spacing range of 200 to 600 mm, the *RQD* in the second case is also likely to be high and a rating of 20 can be assigned assuming *RQD*≥90%, which is possible. The contribution of *IRS*, *JS* and *RQD* to the total *RMR* is again 42. If the other parameters of the two rock masses are similar, the overall *RMR* values of the two rock masses will be the same. However, the behaviour of the two rock masses can be significantly different, even if the stress fields in the two rock masses are the same.

Kirsten (1988a) observed that the structure of the RMR system is not sufficiently sensitive to the individual parameters. As a result, the functional dependence of the *RMR* value on any one of the parameters is not strongly represented in the system. Kirsten illustrated this lack of sensitivity of the *RMR* value to changes in the parameters by considering a value of 79 which is based on a rating of 26 for the *JC* parameter. This RMR value corresponds to good rock in which the joints are very rough, tight and discontinuous with unweathered wall rock. If, instead, the joints are slickensided or contain gouge up to 5 mm thick or are separated up to 5 mm, the corresponding *JC* rating could be 9 and the *RMR* value would be reduced to 62. This would still represent good rock but the behaviour of the rock mass in a tunnel, foundation, or slope would be very different from the first case because of the major differences in the joint strength as represented by the difference of 17 points in the *JC* rating.

Further, in the earlier versions of RMR, the ratings allocated to classification parameters are lumped and a wide gap existed between two rating values. For instance, the joint spacing (*JS*) parameter is allocated ratings of 20, 15, 10, 8 and 5. As pointed out by Sen and Sadagah (2003), the lump ratings in this system lead to quite subjective adjustments. This may be overcome by using the charts provided by Bieniawski (1989) for *IRS*, *RQD* and *JS* parameters and those provided by Sen and Sadagah (2003) for *IRS*, *RQD*, *JS* and *GW* parameters. These charts allow allocation of continuous ratings based on observed or measured conditions of the relevant parameters. The subjectivity of the RMR method can then be confined to *JC* and *RA* parameters. Nevertheless, the rock mass class boundaries in RMR are abrupt and a difference of ±5 rating points, which is considered possible between two different users, could push the rock mass up or down into a different class. Common criticisms of the RMR method are that the system is relatively insensitive to minor variations in rock quality and that the support recommendations appear conservative and have not been revised to reflect new reinforcement tools (Milne et al., 1998).

The current version of RMR (Bieniawski, 1989) provides support recommendations only for a 10 m diameter horseshoe shaped tunnel with a vertical stress magnitude of less than 25 MPa. No guidelines are provided for the selection of support quantities for tunnels with dimensions other than 10 m span. Users of this method have to rely on other empirical guidelines or rules of thumb to determine rock bolt lengths and spacing for tunnel diameters other than 10 m. For instance a 10 m span tunnel in a RMR rock mass class III may require systematic rock bolting plus mesh reinforced shotcrete, whereas a 3.5 m span tunnel in the same rock mass class may not require systematic bolting, shotcrete alone may be sufficient to stabilise the potentially unstable rocks. While the support recommendations provided in the literature may be overly conservative for smaller diameter tunnels, they may not be adequate for excavations with more than 10 m spans.

Singh and Goel (1999) cautioned that the RMR system is found to be unreliable in very poor rock masses and that care should, therefore, be exercised in applying in such rock masses. They also noted that in the case of wider tunnels and caverns, the RMR value obtained may be somewhat less than that obtained from drifts because in drifts, one may miss intrusions of weaker rocks and joint sets having lower condition ratings.

## 4.4 Tunnelling Quality Index (Q)

The tunnelling quality index (Q) was proposed by Barton and co-workers of the Norwegian Geotechnical Institute (Barton et al., 1974, 1975, 1977, and Barton, 1976). The Q system can be used for classifying rock masses and estimation of support requirements for underground excavations. Developed primarily based on the data collected from civil engineering tunnels and caverns, the Q system uses six parameters considered to represent the behaviour of rock masses:

| | |
|---|---|
| Drill core quality (*RQD*): | is the rock quality designation. This is determined by drill core logging or scan line surveys of the exposed rock surfaces as discussed in Section 4.2. |
| Joint set number (*Jn*): | is the number of geological discontinuity sets (joints, shears, bedding etc) in the rock mass. |
| Joint roughness number (*Jr*): | is the surface roughness of the most unfavourable joint (discontinuity) set in the rock mass. |
| Joint alteration number (*Ja*): | is the extent of alteration of the discontinuity surfaces and/or extent and the nature of any filling material in discontinuities. |
| Water reduction factor (*Jw*): | is a parameter related to the groundwater condition in the discontinuities in the rock mass. |
| Stress reduction factor (*SRF*): | is a parameter that takes into account the possible effects of in situ stresses and major weakness zones intersecting or adjacent to the excavation. It is a measure of stress in competent rock, squeezing loads in plastic incompetent rocks and loosening loads in excavations in shear zones or clay bearing rocks. |

In the Q system, the *RQD* value (the first parameter) is used as determined by bore core logging or other applicable methods discussed in section 4.2. *RQD* intervals of 5 (e.g. 100, 95, 90 … etc) are considered to be sufficiently accurate and if it is measured as less than 10 (including 0), a nominal value of 10 is used. Numerical rating values for the other five parameters are determined using the guidelines provided by the creators of the system. The recommended ratings are presented in Tables 4.7 to 4.11.

Table 4.7 Ratings for *RQD* and joint set number (*Jn*)

<table>
<tr><th colspan="2">Description</th><th>RQD%</th><th colspan="2">Number of joint sets</th><th>Jn</th></tr>
<tr><td>A.</td><td>Very poor</td><td>0-25</td><td>A.</td><td>Massive, no or few joints</td><td>0.5-1</td></tr>
<tr><td>B</td><td>Poor</td><td>25-50</td><td>B.</td><td>One joint set</td><td>2</td></tr>
<tr><td>C.</td><td>Fair</td><td>50-75</td><td>C.</td><td>One joint set plus random joints</td><td>3</td></tr>
<tr><td>D.</td><td>Good</td><td>75-90</td><td>D.</td><td>Two joint sets</td><td>4</td></tr>
<tr><td>E.</td><td>Excellent</td><td>90-100</td><td>E.</td><td>Two joint sets plus random joints</td><td>6</td></tr>
<tr><td colspan="3" rowspan="6">Notes:<br>(i) Where RQD is reported or measured as <10 (including 0), a nominal value of 10 is used to evaluate Q,<br>(ii) RQD intervals of 5, i.e. 100, 95, 90 etc, are sufficiently accurate.</td><td>F.</td><td>Three joint sets</td><td>9</td></tr>
<tr><td>G.</td><td>Three joint sets plus random joints</td><td>12</td></tr>
<tr><td>H.</td><td>Four or more joint sets, random, heavily jointed, “sugar cube”, etc.</td><td>15</td></tr>
<tr><td>J.</td><td>Crushed rock, earth like.</td><td>20</td></tr>
<tr><td colspan="3">Notes: (i) For tunnel intersections use (3.0 x Jn),<br>(ii) For portals use (2 x Jn).</td></tr>
</table>

Once the ratings are assigned to the six parameters, the *Q* value is calculated using the equation:

$$Q = \left(\frac{RQD}{Jn}\right) \times \left(\frac{Jr}{Ja}\right) \times \left(\frac{Jw}{SRF}\right) \tag{4.8}$$

where the first quotient corresponds to an estimation of the relative size of rock blocks that form the rock mass, the second corresponds to an estimation of the inter block shear strength, and the third represents the active stress. The physical meanings assigned to the three quotients are rough guides only and are unlikely to be exactly congruent with the true factors.

Table 4.8 Ratings for joint roughness number (*Jr*)

| Joint roughness description | | Jr |
|---|---|---|
| (a) Rock-wall contact, and (b) rock-wall contact before 10 cm shear | | |
| A. | Discontinuous joints | 4 |
| B. | Rough or irregular, undulating | 3 |
| C. | Smooth, undulating | 2 |
| D. | Slickenside, undulating | 1.5 |
| E. | Rough or irregular, planar | 1.5 |
| F. | Smooth, planar | 1.0 |
| G. | Slickensided, planar | 0.5 |
| (c) No rock-wall contact when sheared | | |
| H. | Zone containing clay minerals thick enough to prevent rock-wall contact | 1.0 |
| J. | Sandy, gravely or crushed zone thick enough to prevent rock-wall contact | 1.0 |

Notes: (i) Descriptions refer to small-scale and intermediate scale features, in that order. (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m. (iii) *Jr* = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are oriented for minimum strength. (iv) *Jr* and *Ja* classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance, $\tau$ [where $\tau = \sigma_n$ ta1n$^{-1}$ (Jr/Ja)].

Table 4.9 Ratings for joint alteration number (*Ja*)

| Condition of joint alteration/filling | | Approx $\Phi_r$ (deg) | *Ja* |
|---|---|---|---|
| (a) Rock-wall contact (no mineral fillings, only coatings) | | | |
| A. or | Tightly healed, hard, non-softening, impermeable filling, i.e. quartz epidote. | - | 0.75 |
| B. | Unaltered joint walls, surface staining only. | 25-35 | 1.0 |
| C. | Slightly altered joint walls, non-softening mineral coatings , sandy particles, clay-free disintegrated rock etc. | 25-30 | 2.0 |
| D. | Silty or sandy clay coatings, small clay fractions (non-softening). | 20-25 | 3.0 |
| E. | Softening or low friction clay mineral coatings, i.e. kaolinite or mica. Also chlorite, talc, gypsum, graphite etc, and small quantities of swelling clays. | 8-16 | 4.0 |
| (b) Rock-wall contact before 10 cm shear (thin mineral fillings) | | | |
| F. | Sandy particles, clay-free disintegrated rock etc. | 25-30 | 4.0 |
| G. | Strongly over-consolidated non-softening clay mineral fillings (continuous, but < 5 mm thickness) | 16-24 | 6.0 |
| H. | Medium or low over-consolidation, softening clay mineral fillings (continuous, but < 5 mm thickness) | 12-16 | 8.0 |
| J. | Swelling-clay fillings, i.e. montmorillonite (continuous, but < 5 mm thickness). Values of Ja depends on percent of swelling clay-size particles, and access to water etc. | 6-12 | 8-12 |
| (c) No rock-wall contact when sheared (thick mineral fillings). | | | |
| KLM. | Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition). | 6-24 | 6, 8 or 8-12 |
| N. | Zones of bands of silty or sandy-clay fraction (non-softening). | - | 5.0 |
| OPR. | Thick, continuous zones or bands of clay (see G, H, J for description of clay condition). | 6-24 | 10, 13 or 13-20 |

Table 4.10 Ratings for joint water reduction factor (*Jw*)

| Joint water condition | | Water pressure ($kg/cm^2$) | *Jw* |
|---|---|---|---|
| A. | Dry excavation or minor inflow, i.e. < 5 l/min locally. | <1 | 1.0 |
| B. | Medium inflow or pressure, occasional outwash of joint fillings. | 1-2.5 | 0.66 |
| C. | Large inflow or high pressure in competent rock with unfilled joints. | 2.5-10 | 0.5 |
| D. | Large inflow or high pressure, considerable outwash of joint fillings. | 2.5-10 | 0.33 |
| E. | Exceptionally high inflow or water pressure at blasting, decaying with time. | >10 | 0.2-0.1 |
| F. | Exceptionally high inflow or water pressure continuing without noticeable decay. | 25-30 | 4.0 |

Notes: (i) Factors C to F are crude estimates. Increase *Jw* if drainage measures are installed. (ii) Special problems caused by ice formation are not considered. (iii) For general characterisation of rock masses distant from excavation influences, the use of *Jw* = 1.0, 0.66, 0.5, 0.33 etc as depth increase from say 0-5, 5-25, 25-250 to >250 m is recommended, assuming that *RQD/Jn* is low enough (e.g. 0.5-25) for good hydraulic conductivity. This will help to adjust Q for some of the effective stress and water softening effects in combination with appropriate characterisation values of *SRF*. Correlations with depth-dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed.

Table 4.11 Ratings for stress reduction factor (SRF)

| Description of weakness zones or stress level | | $\sigma_c/\sigma_1$ | $\sigma_\theta/\sigma_c$ | 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 |
| B. | Single weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (depth of excavation ≤ 50 m) | | | 5 |
| C. | Single weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (depth of excavation > 50 m) | | | 2.5 |
| D. | Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth) | | | 7.5 |
| E. | Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation ≤ 50 m) | | | 5.0 |
| F. | Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation > 50 m) | | | 2.5 |
| G. | Loose, open joints, heavily jointed or "sugar cubes", etc (any depth) | | | 5.0 |
| *(b) Competent rock, rock stress problems* | | | | |
| H. | Low stress, near surface, open joints | >200 | <0.01 | 2.5 |
| J. | Medium stress, favourable stress condition | 200-10 | 0.01-0.3 | 1 |
| K. | High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability | 10-5 | 0.3-0.4 | 0.5-2 |
| L. | Moderate slabbing after > 1 h in massive rock | 5-3 | 0.5-0.65 | 5-50 |
| M. | Slabbing and rock burst after a few minutes in massive rock | 3-2 | 0.65-1 | 50-200 |
| N. | Heavy rock burst (strain-burst) and immediate dynamic deformation in massive rock | <2 | >1 | 200-400 |
| *(c) Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure* | | | | |
| O. | Mild squeezing rock pressure | | 1-5 | 5-10 |
| P. | Heavy squeezing rock pressure | | >5 | 10-20 |
| *(d) Swelling rock: chemical swelling activity depending on presence of water* | | | | |
| R. | Mild squeezing rock pressure | | | 5-10 |
| S. | Heavy squeezing rock pressure | | | 10-15 |

Notes: (i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation. This will also be relevant for characterisation. (ii) For strongly anisotropic virgin stress field (if measured); when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce $\sigma_c$ to $0.75\sigma_c$; when $\sigma_1/\sigma_3 > 10$, reduce $\sigma_c$ to $0.5\sigma_c$, where $\sigma_c$ is unconfined compression strength, $\sigma_1$ and $\sigma_3$ are the major and minor principal stresses, and $\sigma_\theta$ is the maximum tangential stress (estimated from elastic theory). (iii) Few case records available where the depth of crown below the surface is less than span width, suggest an SRF increase from 2.5 to 5 for such cases (see H). (iv) Cases L, M and N are usually most relevant for support design of deep tunnel excavations in hard massive rock masses with RQD/Jn ratio from about 50-200. (v) For general characterisation of rock masses distant from excavation influences, the use of SRF = 5, 2.5, 1.0 and 0.5 is recommended as depth increases from say 0-5, 5-25, 25-250 to >250 m. This will help to adjust Q for some of the effective stress effects, in combination with appropriate characterisation values of Jw. Correlations with depth-dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed. (vi) Cases of squeezing rock may occur for depth $H > 350Q^{1/3}$. Rock mass compression strength can be estimated from $SIGMA_{cm} \approx 5\gamma Q_c^{1/3}$ (MPa) where $\gamma$ is the rock density in t/m$^3$, and $Q_c = Q \times \sigma_c/100$.

The Q value varies on a logarithmic scale from 0.001 to 1000 and the system has nine rock mass classes as given in Table 4.12.

Table 4.12 Q index rock mass classes

| Rock mass class | Q value | Rock mass class | Q value |
|---|---|---|---|
| Exceptionally poor | 0.001-0.01 | Good | 10-40 |
| Extremely poor | 0.01-0.1 | Very good | 40-100 |
| Very poor | 0.1-1 | Exceptionally good | 100-400 |
| Poor | 1-4 | Extremely good | 400-1000 |
| Fair | 4-10 | | |

The *Q* value is related to support requirements through an "equivalent dimension", *De*, which is a function of the size and purpose of the excavation and defined as:

$$De = \frac{Span, diameter\ or\ height}{ESR} \tag{4.9}$$

where *ESR*, excavation support ratio, is analogous to an inverse factor of safety and is a dimensionless function of the purpose of the opening. A large permanent excavation requiring a high level of safety is given a small *ESR*, while a temporary excavation needing only short term stability can be assigned a high *ESR*. The recommended values for *ESR* are presented in Table 4.13. The relationship between the *Q* value and equivalent dimension, *De*, of an excavation determines the appropriate support measures, as depicted in Figure 4.7. Depending on the "zone" into which the *Q-De* pair falls on the support chart (Figure 4.7), one of the nine support categories is assigned to the excavation. Category 1 requires no support while categories 2 to 9 inclusive, require varying levels of support from spot bolting to cast concrete lining.

Table 4.13 Recommended *ESR* for selecting safety level

| | Type of excavation | *ESR* |
|---|---|---|
| A. | Temporary mine openings | ca. 2-5 |
| B. | Permanent mine openings, water tunnels for hydropower (excluding high pressure penstocks), pilot tunnels, drifts and headings for large openings, surge chambers. | 1.6-2.0 |
| C. | Storage caverns, water treatment plants, minor road and railway tunnels, access tunnels. | 1.2-1.3 |
| D. | Power stations, major road and railway tunnels, civil defence chambers, portals, intersections. | 0.9-1.1 |
| E. | Underground nuclear power stations, railway stations, sports and public facilities, factories, major gas pipeline tunnels. | 0.5-0.8 |

The Q system was first proposed on the basis of the analysis of 212 case histories from Scandinavian countries. For nearly 20 years the system remained unchanged from its original version proposed in 1974 which consisted of 38 tunnel support categories plus a no support "zone" in the support chart. In 1993, the system was revised and updated (Grimstad and Barton, 1993; Barton and Grimstad, 1994) to incorporate the experience gained by applying the method to a large number of projects and the technological advances since its introduction. In the updated version the original classification parameters have not changed and their rating ranges also remain largely unchanged. The only changes that have been made are in the *SRF* term. One *SRF* case was added to the "Competent rock, rock stress problems" (Table 4.11 part b) and two of the existing cases were renamed. These additions and revisions were made so that slabbing and rock bursting cases can also be accommodated in the support recommendations. The 1993 version, however, provided a revised support chart and reduced the total number of categories to 9 (see Figure 4.7). Considering the developments in shotcrete technology, the revised version recommends the use of fibre reinforced shotcrete instead of mesh reinforced shotcrete recommended earlier. The revised support chart has considerably simplified the support selection process and is more user-friendly compared to the earlier version. Barton (2002) further expanded the possible applications of the Q system without changing the classification parameters and support recommendations provided in the 1993 version.

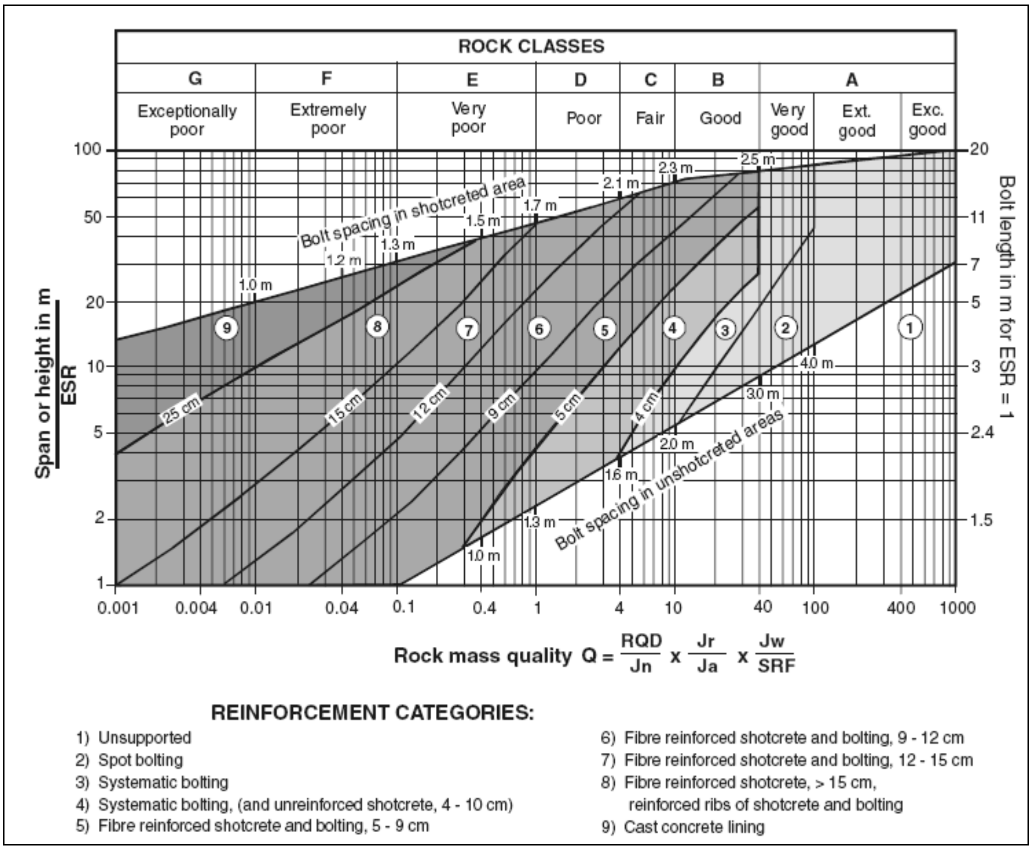


Figure 4.7 Q-System support chart (after Barton and Grimstad, 1994)

The recommendations provided in the Q support chart are for permanent arch (roof) support for excavations. The chart can also be used to determine temporary support and wall support by making appropriate adjustments given in Table 4.14.

The Q index also provides empirical correlations for estimating permanent radial support pressures required to stabilise the roof and walls of an excavation. Barton et al. (1974) provided the following formulas for estimating permanent roof and wall support:

$$P_{roof} = \left(\frac{200}{Jr}\right)Q^{-1/3} \tag{4.10}$$

$$P_{wall} = \left(\frac{200}{Jr}\right) Q_w^{-1/3} \tag{4.11}$$

where $P_{roof}$ and $P_{wall}$ are in kPa, $Jr$ is as defined earlier, and $Q_w$ is the modified $Q$ value for walls as per Table 4.14. From the above equations, temporary support pressures may also be estimated by making appropriate adjustments to the $Q$ values as suggested in Table 4.14.

Table 4.14 Guidelines for temporary and wall support using observed Q values

| | |
|---|---|
| Temporary support | (a) increase *ESR* to 1.5*ESR*<br>(b) increase $Q$ to $5Q$ (arch)<br>(c) increase $Q_w$ to $5Q_w$ (wall) |
| Wall support<br>(based on modified quality $Q_w$ for walls; $Q_w$ is not the observed value of $Q$ in a cavern wall) | (a) select $Q_w = 5Q$ (when $Q > 10$)<br>(b) select $Q_w = 2.5Q$ (when $Q < 10$)<br>(c) select $Q_w = 1.0Q$ (when $Q < 0.1$) |
| Notes: Use total excavation height (H) for wall support.<br>$Q$ is the general rock quality observed when inspecting the arch or walls of a tunnel.<br>For local variations of rock quality (arch or walls) map locally and change support as appropriate. | |

In subsequent publications (Barton et al., 1977; Grimstad and Barton, 1993) the above equations were modified by incorporating separate weighting for the number of joint sets (*Jn*). When more than three joint sets are present in the rock mass, the empirical formula for estimating the expected roof support loads (Barton et al., 1977; Grimstad and Barton, 1993) is as follows:

$$P_{roof} = \frac{200}{3Jr} Jn^{\frac{1}{2}} Q^{-\frac{1}{3}} \tag{4.12}$$

The empirical formula given by Barton et al. (1977) for estimating the expected wall support loads is

$$P_{wall} = \frac{200}{3Jr} Jn^{\frac{1}{2}} Q_w^{-\frac{1}{3}} \tag{4.13}$$

where $P_{roof}$ and $P_{wall}$ are in kPa, $Jn$ and $Jr$ are as defined earlier, and $Q_w$ is the modified $Q$ value for walls as per Table 4.14. From these two equations it is clear that the Q system assumes that the support pressure is a function of only the rock mass quality ($Q$ value, $Jn$ and $Jr$). The width or span of the excavation is ignored. This is somewhat surprising because in determining the support requirements, the Q system explicitly takes into consideration the span (or diameter) of the excavation. For instance, for a 2 m wide tunnel driven in a rock mass with a $Q$ value of 0.2 the recommended bolt length is 1.5 m, if $ESR$=1, whereas for a 5 m wide tunnel in the same rock mass with the same $ESR$ the recommended bolt length is 2.4 m. This implies that increasing tunnel span increases the thickness of the potentially unstable rock zone, and is rightly so. It follows that with an increase in span, in jointed or crushed rock in a low stress environment the rock load for a unit surface area (hence the required support pressure) also increases. The corollary is that when deriving support requirements directly using the support chart and indirectly by estimating support pressures, seemingly, the Q system contradicts itself.

Bhasin and Grimstad (1996) considered that in crushed and brecciated rock masses, the amount of loosened rock at the roof increases with the width of a tunnel and proposed that in poorer quality rock masses, the dimension of the excavation can be taken into consideration while evaluating support pressures. They proposed an amended correlation which includes tunnel width, $B$, (in meters) as given in Equation 4.14, in which $P_{roof}$ is given in kPa.

$$P_{roof} = \left(\frac{40B}{Jr}\right)Q^{-1/3} \tag{4.14}$$

Based on Indian tunnelling experience Singh et al. (1992, 1997), Goel et al. (1995, 1996) and Singh and Goel (1999) proposed further amendments to the support pressure correlations of Barton et al. (1974), which are discussed in Chapter 5. These amendments were based on support pressure measurements made in tunnels supported with steel ribs, a support type not recommended in the Q system.

By analysing the records of both man made and natural openings in rock, Barton (1976) presented an equation relating permanently unsupported safe span $B$ (in meters) to $ESR$ and $Q$ as given below:

$$B = 2\ ESR\ Q^{0.4} \tag{4.15}$$

While the relationship given in Equation 4.15 may represent the case studies in Barton's database, it should be noted that the term *ESR* is not a rock mass property but a value chosen depending on the level of safety required for excavations of predetermined purposes and operating lives. A permanently unsupported safe span of an excavation should not be function of user dependent terms such as *ESR* and should only be a function of the rock mass quality which in this case is the $Q$ value.

In spite of the inclusion of the term *ESR* in Equation 4.15, Barton (1988) maintains that the general (or preferable) requirements for permanently unsupported openings are $Jn \leq 9$, $Jr \geq 1.0$, $Ja \leq 1.0$, $Jw=1.0$ and $SRF \leq 2.5$. He also presented a list of conditional requirements as follows:

- If $RQD \leq 40$, need $Jn \leq 2$
- If $Jn=9$, need $Jr \geq 1.5$ and $RQD \geq 90$
- If $Jr=1.0$, need $Jn<4$
- If $SRF>1$, need $Jr \geq 1.5$
- If Span>10 m, need $Jn<9$
- If Span>20 m, need $Jn \leq 4$ and $SRF \leq 1$

The above requirements are apparently based on the permanently unsupported case records in the Q database. However, if the $Q$ value falls into the "no support" zone of the Q support chart and the above requirements are not met, no guidance is given for the selection of support.

The Q system can also be used for estimating in situ modulus of rock mass deformation ($E_M$). Barton et al. (1980) and Barton (1995) provided the following formulas for estimating $E_M$ (in GPa) when $Q>1$ and $Q<1$, respectively:

$$E_M = 25 \log Q \ (when\ Q > 1) \qquad (4.16)$$

$$E_M = 10Q_c^{1/3} \ (when\ Q < 1) \qquad (4.17)$$

where $Q_c=(Q\sigma_c)/100$ with $\sigma_c$ being the compressive strength of intact rock substance.

The Q system also found applications in Hoek-Brown failure criterion (Hoek and Brown, 1980) used for estimating rock mass strength. The main advantage of the Q system is its sensitivity to minor variations in rock properties. The descriptions used to assess the joint conditions are relatively rigorous and leave less room for subjectivity, compared to other classification systems (Milne et al., 1998). Further, the Q system recommended support reflects the span and the purpose of the excavation. Different support measures are recommended for the roof and walls of an excavation and also for temporary excavations. The use of the Q system for the design of support has evolved over time and the revised version has introduced a support chart that accounts for the use of fibre reinforced shotcrete.

### 4.4.1 Limitations of the Q system

A commonly held view regarding the Q system is that it is relatively difficult for inexperienced users to apply. One of its limitations is its failure to directly account for the discontinuity orientation for the assessment procedure. Barton (1988) states that, "Orientation is included implicitly in the Q system by classifying the joint roughness and alteration of only the most unfavourably oriented joint sets or discontinuities". However, no guidelines are provided on how to decide on the critical or most unfavourable orientation of discontinuities. The selection of the most unfavourably oriented joint sets will, therefore, be user dependent.

Regarding joint orientation, Barton et al. (1974, 1977) stated that it was not found to be sufficiently important to include it directly in the classification. Part of the reason for this was due to the fact that some of the case studies in the database used in developing the Q system were already oriented to avoid the adverse effects of weakness zones. Barton et al. added that it is certainly necessary to orientate the axes of important excavations favourably with respect to both stress anisotropy and weakness zones, as usually attempted. The author of this thesis concurs with Barton et al. that it is possible to avoid the adverse effects of known weakness zones and major sets of discontinuities on most major infrastructure caverns such as train stations, power stations etc, but this is not always the case for longer tunnels and mining excavations. Barton et al. (1974, 1977) asserted that the parameters *Jn, Jr* and *Ja* appear to play more important roles than joint orientation because the number of joint sets determines the degree of freedom of block movement (if any), and the frictional and dilatational characteristics (*Jr*) can vary more than the down dip gravitational component of unfavourably oriented joints. While this is true for shear movement of rock blocks along the discontinuity surfaces, this is not the case in jointed rock formations with low in situ stress conditions in which joint orientations play a very significant role on the tetrahedral (or polyhedral) wedge falls. Contrary to Barton et al.'s assertion, the joint orientation is the governing factor for wedge falls and the parameters *Jr* and *Ja* play a secondary role, only if sufficient compressive stresses are induced to provide clamping effects on the potentially falling rock wedges in an excavation. This too becomes irrelevant if the joint orientations are such that the induced stresses contribute to the wedge falls by driving them out of their bounding joints rather than restraining them in place.

Discontinuity spacing is another area of concern. Consider two hypothetical rock masses which are identical if not for the different spacing of the three joint sets present. In the first, the joint spacing is 200 to 600 mm with an average of 300 mm. The second has and average spacing of around 2 m. With the assumed joint spacing the *RQD* values of both rock masses are likely to be the same. Accordingly, both rock masses would receive the same Q rating and as a result the same support system will be recommended. If two shallow tunnels with identical dimensions and depths are excavated in the two rock masses in which potential instability is structurally controlled, two different support systems would be applicable. The first would require shotcrete, perhaps in addition to rock bolts, to prevent small rock falls, while the second may only require rock bolts

Also there is considerable room for subjectivity and personal bias in the *SRF* parameter. For example, no direct guidance is given on the selection of a rating for *SRF* if gravity induced rock block movements are the main modes of instability in a jointed rock mass in which no major weakness or shear zones (described in Section "a" of Table 4.11) intersect the excavation. Further, for weakness zones intersecting excavation (Table 4.11 Section "a"), the rating boundaries are abrupt. For some of the possible conditions listed under *SRF* parameters, wide ranges of ratings are recommended (see Table 4.11 Section "b"). For instance, "moderately slabbing" condition is given a rating range of 5 to 50, which could result in a vastly different *Q* values depending on the rating chosen by the user and this could lead to the recommendation of distinctly different support types and quantities for the same rock mass by different users of the Q system.

Kirsten (1988b) considered that selection of ratings for *Ja* parameter in the Q system is quite complex and relatively open to interpretation. He provided a revised table based on the original table (Table 4.9) of ratings for the *Ja* parameter provided by Barton et al. (1974). Kirsten's revised table included changes to the original rating values and also additional rating values for the *Ja* parameter. However, Barton and co-workers did not include Kirsten's suggestions in the revised Q system published in 1993 or in subsequent publications.

Kirsten (1988b) maintains that the determination of *SRF* is also open to interpretation because of the qualitative nature of the criteria given in the Q system and the difficulties that often arise in the case of homogeneous rock masses with regard to assessing the degree of stressing, bursting, squeezing and swelling. He observed that in the case of non-homogeneous rock, *SRF* is related to the overall quality of the rock, Q, and, in the case of homogeneous rock, to the field stress state relative to the rock mass strength. The perspective relationships for *SRF* of non-homogeneous and homogeneous rock masses were given by the expressions

$$SRF_{\mathrm{n}} = 1.809\, Q^{-0.329} \tag{4.18}$$

$$SRF_{\mathrm{h}} = 0.244\, K^{0.346}\, (H/\mathrm{UCS})^{1.322} + 0.176(\mathrm{UCS}/H)^{1.413} \tag{4.19}$$

where, $Q = (RQD/Jn)(Jr/Ja)(Jw/SRF)$; $K$ = maximum to minimum principal stress ratio; $H$ = head of rock corresponding to maximum principal stress field; UCS is unconfined compressive strength of rock in MPa. As per the first expression, before determining $SRF_n$ the $Q$ value should be determined for the rock mass in question. However, it is uncertain as to which *SRF* value should be used in the determination of the $Q$ value to be used in the first expression. Although these expressions may remove some of the uncertainties in the *SRF* term, they may complicate an otherwise relatively straightforward assessment of a rock mass using the Q system. Barton and co-workers did not include these suggestions in the updated version of the Q system or in subsequent publications dealing with the Q system. Nevertheless, Kirsten's suggestions for estimating the *SRF* term have been adopted by some users of the Q system.

For cases L, M and N (moderate slabbing, slabbing and rock burst, and heavy rock burst, respectively) the *SRF* values selected based on $\sigma_c/\sigma_1$ and $\sigma_\theta/\sigma_c$ ratios pertain to massive rocks only, which should really be the case since slabbing, rock burst etc are associated with competent massive rocks only. As pointed out by Kumar et al. (2004) there are, however, situations in which ratios $\sigma_c/\sigma_1$ and $\sigma_\theta/\sigma_c$ lie in ranges corresponding to conditions L, M and N, but the rock is moderately jointed and not massive. For these situations Table 4.11 does not provide *SRF* values. If *SRF* values are selected for "rock stress problems" purely on the basis of $\sigma_c/\sigma_1$ and $\sigma_\theta/\sigma_c$ ratios, the results might be correct for massive rocks but are bound to be incorrect for jointed rock masses.

## 4.5 The Main Differences between the RMR and Q Systems

As mentioned previously, both methods are based on six parameters considered to represent the behaviour of rock masses. When viewed from a broader perspective, the basic concepts of both schemes are similar and allocate ratings to the properties that influence the rock mass behaviour. However, there are differences in the parameter ratings and the manner in which the final RMR and Q values are computed. For instance, the RMR value is computed by adding the ratings assigned to the constituent parameters, while the Q value is obtained by division and multiplication. In addition, the assessment of some of the key rock mass parameters is significantly different in the two methods as discussed below:

- Intact rock strength (*IRS*) is a factor in the *SRF* term of the Q system, only if the excavation stability is affected by the in situ stress field. In contrast *IRS* is always included in the *RMR* value. If *IRS* changes while all the other parameters remain virtually the same, several *RMR* values are possible for a single Q value.

- The in situ stress field is not accounted for in the RMR system in classifying a rock mass. In the Q system it is a factor in the *SRF* term if excavation instability is stress driven. Thus for a rock mass with a given *RMR* value, several different *Q* values are possible depending on the SRF value used.

- Joint spacing (*JS*) is a key parameter in the RMR system; the closer the *JS* the lower the *RMR* value and the wider the *JS* the higher the *RMR* value. This is not so in the Q system. As pointed out by Milne et al. (1998), if three or more joint sets are present and the joints are widely spaced, it is difficult to get the Q system to reflect the competent nature of a rock mass. For widely spaced jointing, the joint set parameter *Jn* in the Q system appears to unduly reduce the resulting *Q* value (Milne et al., 1998). Thus for a single *Q* value several *RMR* values are possible depending on *JS*.

- *RQD* is used in both methods, and is a function of joint spacing, albeit it does not fully represent the true nature of joint spacing. In addition to *RQD*, as already mentioned, *JS* is also a key parameter in the RMR method. In the Q system, although the number of joint sets is taken into account, spacing is not considered directly. This means joint spacing is counted twice in the RMR method, while the Q system uses it indirectly only once.

- Joint orientation (*JA*) is accounted for directly in the RMR method by allocating a rating between 0 and -12. In the Q system this is considered implicitly, but no guidelines are provided to identify the adversely oriented discontinuities. Thus the selection of the most critical discontinuity set is user-dependent. In any case no rating is given to the *JA* in the Q system. Thus for a given *Q* value, different *RMR* values are possible depending on the orientation of the excavation relative to the discontinuity set orientation.

- Rating scale: The rating scales in the RMR method have been changed several times as shown in Table 4.1, while Q remained unchanged for some 20 years until 1993. For a rock mass with a given *Q* value, different *RMR* values can be obtained depending on the RMR version used. Since 1993, the *SRF* parameter of the Q system is given a rating scale of 1 to 400 for competent rock with rock stress problems. As mentioned earlier, depending on the *SRF* valued used, different *Q* values can be obtained for a rock mass with a given *RMR* value. With the 1 to 400 range of *SRF* values, the difference in the *Q* value can be more than two orders of magnitude. By setting the *SRF* value to 1 in deriving the *Q* values this problem may be overcome if the *SRF* term represents only the stress. It is not, however, strictly a stress factor as it also represents weakness zones, which are rock mass parameters.

From the foregoing, it is clear that the predictions made by the two systems are unlikely to match perfectly for all rock mass conditions in underground excavations. A universally applicable single formula for linking *RMR* and *Q* value is also unlikely to be obtained.

# CHAPTER 5

# PREVIOUS STUDIES ON RMR AND Q

## 5.1 Introduction

Research on the literature undertaken for the present study has shown that a plethora of technical papers have been published on the RMR and Q indices since their advent as underground excavation support design tools. The vast majority of these papers covered the general application of the two indices to new underground excavation projects without attempting to evaluate their reliability and therefore has no direct relevance to the present study. Several studies dealing with the limitations and reliability of the RMR and Q classification indices and possible areas for their improvements were, however, found and this chapter presents a brief overview of these.

## 5.2 Studies on the Reliability of RMR and Q

Studies on the reliability of the two classifications methods can be divided into two broad groups. The first comprises experience-based reviews with no detailed application of the two methods to any particular excavation project. The second group, which represents the majority, is based on the direct application of the two indices to underground excavation support design.

The first group reviews the two classification methods from the points of view of the ease of application, the accuracy and subjectivity of ratings allocation, and the accuracy of their predictions for assumed rock mass conditions. These studies are very limited in number and are largely desk studies carried out by researchers fully conversant with the classification methods. They do not directly apply the two methods to any particular case study, however, where necessary, in order to clarify issues of concern, the classification methods are applied to examples taken from excavation projects. Notable amongst these are the studies by Bieniawski (1976), Speers (1992), Stille and Plamstom (2003), and Palmstrom and Broch (2006).

The second group, which represent the majority, is based on the direct application of the two indices, as a design tool, to underground excavations during either the initial design stage or the construction stage or both stages. This group maintains that despite some differences in the RMR and Q methods, both are based on six parameters considered to represent the behaviour of rock masses. It is, therefore, reasonable to expect the two methods to deliver comparable conclusions with respect to support requirements of excavations. If the two methods are reliable and applicable to any project, their predictions should also be comparable to the design predictions made by other applicable methods. Taking these factors into consideration the second group was mostly based on the comparison of:

- the applicability of different classification methods,
- the support measures and support pressures (rock loads) predicted by one classification method with those of the other,
- the support requirements predicted by classification methods with those of the other applicable methods,
- the support requirements predicted by classification methods with the support installed in excavations, and
- the support pressures predicted by classification methods with those measured using instrumentation.

While no single study has attempted to cover all these five items, most have dealt with more than one. A summary of the second group of studies is presented in Table 5.1. A brief overview of each previous study on the reliability of the two methods and a summary of their conclusions are presented in Section 5.2.

The literature also revealed that several researchers have attempted to establish a correlation between the rating values produced by the RMR and Q systems, some researchers specifically focussing on this aspect alone. A discussion on the correlations between the two methods is presented in Section 5.3.

Table 5.1 Summary of previous studies on the reliability of the RMR and Q indices

| Geographic location | Excavation type (& No.) | Width (m) | Length | Material type(s) | References |
|---|---|---|---|---|---|
| England | Experimental tunnels (4) | 3.3 | >400 m | Limestone, mudstone, sandstone | Houghton (1976) |
| New Zealand | Hydroelectric tunnels (9) & power station cavern (1) | 2.9-14.5 & 20 | Several km | Mostly sedimentary, one igneous | Rutledge & Preston (1978) |
| Scandinavia & Austria | Road, rail & hydroelectric (16) | 6.5-12.5 | Several km | Various | Steiner & Einstein (1980) |
| South Africa | Infrastructure tunnels (3) | 3 | Several km | Various | Cemeron-Clarke & Budavari (1981) |
| Spain | Road tunnels (3) | 11.6 | >6.5 km | Shale, limestone, sandstone, quartzite | Moreno-Tallon (1982) |
| Japan | Infrastructure tunnels (152) | Various | Not given | Faulted/crushed/heavily jointed | Nakao (1982, 1983, 1984) |
| Australia | Mine stopes (several) | 30 | Varies | Dolomitic shales | Baczynski (1983) |
| USA | Train station cavern (1) | 21 | 150 m | Argillite | Einstein et al. (1983) |
| Canada | Mine excavations (several) | Varies | Varies | Meta-anorthosites | Udd & Wang (1985) |
| Sri Lanka | Hydro tunnel (1), mine tunnels (2) | 2.8-7.2 | Several km | Gneisses, quartzite | Brook & Dharmaratne (1985) |
| Canada | Rail tunnels (4) | 5-5.5 | >15 km | Sedimentary | Kaiser et al. (1986) |
| Canada | Mine drifts (57) | 2.8-7.5 | Varies | Various | Choquet & Charette (1988) |
| India | Road, rail & hydroelectric (24) | Various | Tens of km | Squeezing & elastic | Singh et al. (1992, 1997); Goel et al. (1995) |
| India | Coal mine roadways (44) | 3-5.2 | Several km | Sandstone, shale, coal, siltstone etc. | Sheorey (1993) |
| UK | Infrastructure (1) | Not given | Bore core data | Volcanic | Rawlings (1995) |
| Israel | Infrastructure (7) | 3-10 | Not given | Chalk | Polishook & Flexer (1998) |
| Australia | Rail tunnel (1) | 11 | 2.3 km | Sandstone | Asche and Quigley (1999) |
| Italy & Spain | Rail tunnels (20), hydro tunnels (5) | Varies | Several km | Shale, schists, argillite, meta-basalt | Gonzalez de Vallejo (2002) |
| India | Hydroelectric tunnel (1) | 10.15 | 27.4 km | Gneisses, schists, amphibolites | Kumar et al. (2004) |
| Australia | Roads, railways & car park (8) | 4-24 | Several km | Sandstone, | Pells & Bertuzzi (2008) |

### 5.2.1 Application to the Kielder Experimental Tunnel

In a comprehensive study on the role of rock quality classification indices in the assessment of rock masses, Houghton (1976) applied the $RMR_{73}$, $Q_{74}$ and RSR methods to the Kielder experimental tunnel system in England. It comprised an access decline (adit) and three horizontal tunnels, all with a diameter of approximately 3.3 m. A sequence of more or less horizontally stratified four rock types, Great Limestone, Four Fathom Mudstone, Four Fathom Limestone and Natrass Gill Sandstone, in that order from top to bottom, were present on the site. No major faults were present in the rocks although joints were well developed. Drill and blast and machine excavation methods were employed for tunnel construction as part of the experiment. The adit was driven through all four rock types and one tunnel each was driven in Four Fathom Mudstone, Four Fathom Limestone and Natrass Gill Sandstone. Houghton produced a series of analyses using the three classification indices in the access adit and three horizontal tunnels. Different support systems were tested and classification indices were related to the actual support used. The main conclusions drawn from the study were:

- The three classification systems gave consistent predictions and variations reflect the different parameters used in the three systems rather than functional differences in the interpretation of rock mass behaviour.

- Each classification system gives a range of values reflecting the variation in the constituent parameters. Thus for any rigorous analysis a series of values should be obtained rather than one value.

- Classification systems need to be interpreted in the context of local geological environment.

- For practical work RMR is easier to apply. The degree of sophistication in the Q system requires considerable expertise and did not produce any significant improvement in the final predictions.

- A classification index is only an average value. Zones of lower quality rock will exist, defined often by a single major discontinuity controlling tunnel stability. Thus in making any assessment one needs not only to define the average condition, but also to indicate potentially poorer quality zones.

- Machine excavated rock has a higher overall quality score compared to that excavated by conventional drill and blast techniques.

Houghton also found that the quantification of the effect of groundwater is the least understood parameter in the classification systems. Since both groundwater pressure and flow quantity exert a considerable influence on the rock mass, further improvements should be made in the measurement and assessment of this parameter in a more meaningful way.

### 5.2.2 Review and Comparison by Bieniawski

Bieniawski (1976) presented a review of the rock mass classification methods available up to that time and found that RMR and Q methods received particular attention both from researchers and underground excavation industry personnel. Bieniawski emphasised that for a rock mass classification method to be of practical value, it should be possible to select ratings for input parameters from bore core data alone. This is important because they are the only available sources of information at early stages of most new projects. Based on his review, Bieniawski made the following observations with regard to the RMR and Q indices:

- The RMR system places emphasis on, amongst other things, the orientation of structural features and material strength which are not direct parameters in the Q system. On the other hand the Q system takes account of the rock stress which is only indirectly considered in the RMR system.

- A classification approach should not be taken too far as a substitute for rock engineering design. While very powerful when correctly used, in the case of complex structures created in rock such as large multiple caverns, the classification approaches are just not sufficient.

- One should not necessarily rely on any one classification system and aim at its standardisation but cross check the findings of one classification with those of others.

Bieniawski (1976) also presented a correlation based on a linear regression analysis of 111 sets of *RMR* and *Q* values obtained from several Scandinavian, South African, North American, European and Australian case histories as given below:

$$RMR = 9 \ln Q + 44 \tag{5.1}$$

Since the data used in deriving Equation 5.1 were widely scattered about the linear regression line, Bieniawski also provided the 90% confidence limits (given as Equation 5.1a) within which 90% of the data used would fall.

$$RMR = 9 \ln Q + 44 \pm 18 \tag{5.1a}$$

Subsequently in 1989, by adding data from Indian case histories compiled by Jethwa et al. (1982), Bieniawski (1989) supplemented the database used for Equation 5.1.

### 5.2.3 New Zealand Tunnelling Experience

Rutledge and Preston (1978) applied $RMR_{76}$ and $Q_{74}$, together with the RSR method, to six tunnelling projects in New Zealand. They consisted of nine tunnel headings, with horseshoe, inverted U and circular shapes, and one powerhouse cavern. The excavation techniques employed in these projects were drill and blast, tunnel boring machine (TBM), and road-header with and without partial shield. The tunnel spans varied from 2.9 to 14.5 m. The powerhouse cavern span is 20.5 m with a height of 36 m. Five of the projects were constructed in sedimentary rock formations and the sixth was in igneous rocks. The tunnels were mostly supported with steel sets. The study included comparisons of support pressures predicted by Q (using Equation

4.10) and RSR with those measured using strain gauges installed in steel sets. The main conclusions drawn from the study of relevance to RMR and Q were:

- In general, the classification methods were easy to apply.

- The systems place strong emphasis on geological/rock mechanics parameters and not enough emphasis on the method of excavation. The most important parameter affecting the loading (support pressure) is the method of excavation.

- In general, one should use all the methods available without relying on any one classification method.

- Rock durability is an important parameter not included in any of the systems.

- For a given rock class, the support pressure does not increase linearly from zero at zero tunnel width, but shows a less than linear increase with increasing tunnel width.

- Many of the rock support pressures predicted by the Q system were greater than the measured rock loads by an excessively conservative margin.

- An equation can be developed for the estimation of support pressure using *RMR* values (note that the 1976 version of RMR used in this study did not provide a formula for estimating support pressures).

The most important observation in this study is that many of the support pressures predicted by the Q system (using Equation 4.10) were greater than the measured rock loads by an excessively conservative margin. A possible argument against this observation is that the equation intended to predict the pressures on shotcrete and rock bolts rather than rock loads on steel sets. However, Rutledge and Preston commented that because of rock loosening, the loads on steel sets would be higher than loads on rock bolts and shotcrete, and therefore the observed conservatism in the Q predicted support pressures is realistic.

Rutledge and Preston also presented linear correlations between *RMR* and *Q*, *RSR* and *Q*, and *RMR* and *RSR* for the data collected from the six tunnelling projects. The correlation between *RMR* and *Q* is presented as Equation 5.2 below.

$$RMR = 5.9\ ln\ Q + 43 = 13.5\ log\ Q + 43 \qquad (5.2)$$

The data used for deriving the above formula are not presented by the two authors. However, a plot of data for *RSR* and *Q* presented in their paper shows wide scattering of the data. Regarding the *RMR*-*Q* correlation the two authors commented that, "*It was expected that there would have been a better correlation than that obtained and it is obvious that more development of the systems is required, especially in the weighting given to each parameter making up the classification*". This implies that *RMR* and *Q* data points are also scattered about the regression line defined by Equation 5.2.

### 5.2.4 Review by Steiner and Einstein

In a comprehensive study of empirical design methods in rock tunnelling, Steiner and Einstein (1980) reviewed several methods including the RMR and Q indices used for tunnel support design. The study included application of five empirical methods, RMR, Q, RSR, RQD and Terzaghi's method, to some of the Scandinavian case studies reported by Cecil (1970, 1975) and also to the Arlberg and Tauern tunnels in Austria. At the time, Cecil's cases formed the basis of the Q system. The Arlberg and Tauern tunnels were driven and supported using NATM. Steiner and Einstein (1980) provided the results of example application of the five methods to eight unsupported cases and six supported cases reported by Cecil. The widths of the eight unsupported cases are as follows: one tunnel each at 6.5 m, 8 m and 11.25 m; two tunnels at 12.5 m; and three tunnels 9 m. The widths of the six supported cases are: three tunnels at 5.9 m; two tunnels 12.5 m; one tunnel 6.5 m. The conclusions relevant to the RMR and Q systems drawn from the review were:

- The RMR system does not require much user experience. The six parameters can be easily determined either during preliminary investigations or during construction.

- The limit of applicability of RMR is not fully known. The changes, however, suggests that the method is not generally applicable.

- During construction, time constraints may limit the use of the Q system, not so much regarding measurements of the parameters but ground assessment with the relatively complex set of tables and notes.

- *Jr* and *Ja* parameters of the Q system require experience and may be difficult to determine if the tunnel is outside the range of base cases. The selection may also be problematic if multiple joint sets are encountered.

- Discontinuity orientation is only indirectly considered in the Q system and no guidelines are given on how to decide on the critical discontinuities and combination of *Jr* and *Ja* parameters.

- Unsurprisingly, the Q method predicted no support for the Cecil's unsupported cases as these formed part of its database. Q predictions agreed well with the Cecil's supported cases, except for one for which Q predicted no support.

- The RMR system predicted considerable support for all Cecil's unsupported cases. In the supported cases the RMR predictions were conservative compared to the actually placed support.

- Support pressure relations of the Q system are not reliable.

- For the Arlberg and Tauern tunnels the predictions using the Q method showed large variations for both ground support and support pressures. RMR predicted support requirements came very close to the actually placed ones.

- Overall none of the methods could be judged to be better than another.

According to Steiner and Einstein (1980), Cecil's cases were known to the creator of the RMR system when its first version was published. The RMR method's conservatism with respect to the Cecil's cases is therefore astonishing.

### 5.2.5 South African Tunnelling Experience

Cameron-Clarke and Budavari (1981) used both bore core data and in situ observations to evaluate the reliability of $RMR_{76}$ and $Q_{74}$. The reliability of the use of bore core data alone for classifying rock masses was also assessed by comparing the ratings obtained for the classification parameters from the same locations by both bore core and in situ observations. The in situ rock mass data were accepted as the basis of comparison. Data were obtained from three tunnels (Bushkoppies, Delvers and Du Toitskloof) in South Africa, excavated in widely differing geological environments. All three tunnels have an inverted U shape with equal horizontal and vertical dimensions of about 3 m. Bushkoppies is a sewer tunnel with a total length of 6.5 km and an overburden thickness of 5 to 65 m, Delvers is an electric cable tunnel of 1.2 km with an overburden of between 2.5 and 11 m, and Du Toitskloof is a pilot tunnel of 4 km long with an overburden range of 10 to 20 m for investigating the route of a major highway tunnel.

The Bushkoppies tunnel was driven through volcanic lava which varies from soft, completely weathered and closely jointed material at the eastern portal, to very hard almost massive rock in other sections. Most of it was excavated in hard, widely to closely jointed rock with three or more joint sets and occasional clay filled faults and fault zones. The Delvers tunnel passes through an alternating sequence of arenaceous and argillaceous strata, with its northern end passing through a sheared fault zone. The Du Toitskloof tunnel passes through granites and sandstone which are separated by the Du Toitskloof Fault. The study covered virtually the full lengths of the first two tunnels and approximately 60% of the third.

The results derived from the bore core and in situ observations in the three tunnels were compared and correlated. The support measures installed in the Bushkoppies and Du Toitskloof tunnels were also compared with those recommended by the two methods when bore core data and in situ observations were used separately as the basis of classifying the rock masses. The main conclusions drawn from the study were:

- For both classification systems, bore core can be used to classify rock masses for engineering purposes. Bore core measurements, however, tend to indicate poorer rock mass conditions than in situ measurements.

- With regard to site investigation for underground excavation in rock, both systems can be useful. Their values must be seen in perspective and their limitations always recognised. Their best applications would seem to be towards providing a general picture of the anticipated rock conditions and an initial assessment of the likely support requirements in a planned underground excavation. However, the data should never be regarded as the final results in this respect.

- Since bore core results are generally expected to be directly applicable to the rock mass in the immediate vicinity of the borehole only, great care is necessary when attempting to extrapolate between boreholes.

- At the Bushkoppies tunnel, the support predictions of the Q system using in situ observations compared favourably with the support actually installed. The agreement was not good for the prediction by Q using bore core observations. The RMR predicted support using both bore core and in situ observations did not compare well with the installed support.

- At the Du Toitskloof tunnel, the support correlations were again better for Q than for RMR. For the latter the predicted support measures were generally more conservative than those installed.

- With a few exceptions, the correlation between the bore core and in situ stand-up times and unsupported spans for both classification systems is very poor. The differences are so marked in some cases that although RMR stand-up time is less than Q stand-up time, the unsupported span predicted by the former is greater than the limiting span predicted by the latter.

- The support predictions from each system appear to correlate slightly better, although when examined in detail they are very often quite different.

Cameron-Clarke and Budavari also noted that the tendency for bore core data to produce lower classification values than those of in situ data is opposite to that indicated by the results presented by Barton (1976) from a similar type of investigation. Barton's bore core Q values were about twice his in situ values. This discrepancy appears to be related to the different rock conditions examined in each case. The study by Cameron-Clarke and Budavari was based on measurements taken from a variety of geological environments which included predominantly jointed rocks, although massive rocks were also examined. Barton's data were obtained from "quite massive biotite gneiss" only. This suggests that the relationship between the bore core and in situ classification values may be linked to the rock mass conditions with lower bore core data based classification values than in situ observations based values being associated with jointed rocks, and the opposite being the case in massive varieties.

They also obtained linear correlations for RMR and Q values determined by bore core observations (Equation 5.3) and in situ observations (Equation 5.4) in the three tunnels.

$$RMR = 4.6 \ln Q + 55.5 \text{ (from bore core data)} \quad (5.3)$$

$$RMR = 5.0 \ln Q + 60.8 \text{ (from in situ data)} \quad (5.4)$$

The relationships obtained for bore core and in situ observations were similar, however, the scattering of data points about the regression line is greater for the in situ values than for the bore core values. In general, for both cases the scatter around the regression lines is too great to suggest that a good correlation exists between the two systems. Further, the two relationships are different to Equation 5.1 obtained by Bieniawski (1976). Although the two correlations are based on a small data set compared to that used for Equation 5.1, Cameron-Clarke and Budavari considered that the two correlations are representative of the types of rock mass conditions investigated.

### 5.2.6 Spanish Tunnelling Experience

Moreno Tallon (1982) applied RMR, RSR and Q indices to three road tunnels in the northern part of Spain and the results compared. In this study the $RMR_{73}$ and $Q_{74}$ versions were used. (Note that $RMR_{73}$ used eight parameters.) The three tunnels are Pando, Negron and Barrios with lengths of 1227 m, 4106 m and 1576 m, respectively. Rock mass data were collected from Pando North, Negron North and South, and Barrios South headings. The tunnels were driven through sedimentary rocks consisting of shale, limestone, sandstone and quartzite, and are circular with a diameter of 11.6 m. The shale is sometimes in isolated layers and sometimes interstratified within other rocks. The whole area was subjected to severe tectonic disturbances which resulted in a high degree of fracturing with faults and folds. The direction of discontinuities varies, as do their characteristics. They vary from closed and non-filled joints to shear zones more than 1 m wide. The data were obtained by applying the three methods directly to 150 advancing tunnel faces. The ratings are therefore representative of the state of the rock mass at the excavation face. The number of data sets obtained from Pando, Negron North, Negron South and Barrios were 37, 31, 65 and 17, respectively.

In addition to the comparison of the rock mass quality ratings obtained directly from the advancing tunnel faces, Moreno Tallon analysed the behaviour of rock bolts in relation to the RMR value by pull testing a proportion of the bolts installed in the tunnels. For this purpose each tunnel was divided into 25 m lengths and percentages of satisfactory bolts were determined for each length. Of the eight parameters used in $RMR_{73}$ those which might directly affect the performance of the rock bolt anchorage (resin type) were selected as IRS, JS, joint separation and weathering. The sum of the ratings assigned to these four parameters was referred to as “A”.

Then a new parameter X, defined as X = (A/RMR)100% was considered to reflect the behaviour of rock bolt anchorage. In theory, the higher the X value, the better should be the anchorage. As for the RMR, this value was assigned to each of the 25 m lengths in the tunnels, and to simplify the analysis, a comparison between the average values for different lengths was carried out.

Moreno Tallon observed that the rock masses intersected in the case tunnels behaved differently depending on whether the RMR value was higher or lower than 60, hence the rock masses were divided into two classes based on whether the RMR value was greater or smaller than 60. The main conclusions drawn from the study were:

- Obtaining the quality index is made more difficult by increased heterogeneity of the rock mass exposed in the excavation face from which the data were obtained.

- Suitable criteria must be established among the members of the working group to reduce judgemental errors in deciding the most unfavourable combination of rock mass parameters.

- It is necessary to check the results obtained for the classification when rating the rock mass and adjust them for the working areas, based on experience.

- RMR usually makes a better distinction between average and good rock, but not as good a distinction for rocks of lower quality.

- The Q index establishes the best definition for average, good and bad quality rocks mass conditions.

- There is no clear correlation between bolt behaviour and RMR or X when the RMR value is greater than 60. It was inferred that the behaviour of installed rock bolts does not depend on the rock mass quality when RMR > 60.

- When the RMR value < 60, there is a logarithmic type correlation between X and the behaviour of rock bolts. It was inferred that bolt failures in this type of rock are related to the X value rather than to the RMR value itself.

Moreno Tallon correlated rock mass rating values obtained by the RMR, Q and RSR classification methods. By linear regression analysis of the data obtained from the four tunnel headings as well as the combined set of data from all headings, he obtained semi logarithmic type correlations between the three classification methods. The correlations between RMR and Q for the four tunnel headings and for the combined data are similar, but not identical, to those obtained by Bieniawski (1976) and Rutledge and Preston (1978). The RMR and Q correlation obtained for the combined data from the four headings is given as Equation 5.5.

$$RMR = 5.4\ ln\ Q + 55.2 = 12.5 \log Q + 55.2 \tag{5.5}$$

Equations 5.5a, 5.5b, 5.5c and 5.5d are the RMR and Q correlations for Pando North, Negron North, Negron South and Barrios South, respectively.

$$RMR = 5.9\ ln\ Q + 58.5 = 13.5\ log\ Q + 58.5 \tag{5.5a}$$

$$RMR = 7\ ln\ Q + 47.96 = 16\ log\ Q + 47.96 \tag{5.5b}$$

$$RMR = 5.3\ ln\ Q + 54.6 = 12.3\ log\ Q + 54.6 \tag{5.5c}$$

$$RMR = 6.5\ ln\ Q + 54 = 14.97\ log\ Q + 54 \tag{5.5d}$$

Moreno Tallon emphasised that the correlation derived from all the data from four tunnel headings is similar to those derived separately for each heading, although the data for the correlations were obtained at different places, from rocks of little similarity in many cases and between indices obtained by different people. Since the data used in deriving the correlations have not been provided, it is uncertain to what extent the data points scattered about the regressions lines of the above relationships. However, it can be shown that despite the apparent similarity of the five equations, they return different *RMR* values for a given *Q* value. While in most cases these differences may be insignificant, in some instances the difference could be high enough to cause erroneous results if these correlations were used for estimating the ratings of one system from the ratings of another. For instance, for a given *Q* value of 2, the relationship for Pando North (Equation 5.5a) returns a *RMR* value of 63, whereas that for Negro North (Equation 5.5b) returns a *RMR* value of 53. As mentioned previously, Moreno Tallon observed differing rock mass behaviour depending on whether the *RMR* value is greater or less than 60 and proposed a method for rock bolting design based on whether the *RMR* value was greater or smaller than 60. As can be seen from the foregoing, Equation 5.5a would transform a rock mass with a *Q* value of 2 to a good rock with a *RMR* value > 60 whereas, Equation 5.5b would transform the same rock mass into the poor quality in the *RMR* scale, with a *RMR* value < 60.

### 5.2.7 Application to Japanese Ground Conditions

Nakao and co-workers (1982, 1983, 1984) undertook a brief review of the databases used in developing the RMR, RSR and Q systems and examined whether the three systems are applicable to the soft and crushed rock conditions with complicated geological structures encountered in the tunnels constructed in Japan. They compiled a tunnelling conditions database comprising data concerning geology, excavation techniques and tunnel deformation. By statistical processing of the data from 152 case tunnels, they attempted to prepare a rock classification system suitable for the geological conditions in Japan. They examined the relative importance of the various parameters i.e. rock type, intact rock strength, joint orientation, joint spacing, joint alteration (opening), water inflow, overburden thickness and tunnel section and the degree of interdependence between different parameters. They then established the relevance and influence of the parameters used in the RMR, RSR and Q systems to the tunnel deformation under the Japanese tunnelling conditions. With respect to the RMR and Q indices, the study found that when they are to be applied to the Japanese conditions:

- Various parameters used in the RMR and Q systems seem to be effectively applicable to the geological classification of Japanese tunnelling conditions.

- The only difference lies in the great weight placed on *RQD* and joint condition in the two classifications whereas in the Japanese classification, the tunnel overburden thickness constitutes the greatest factor.

- Most of the geology of Japan belongs to the so called "fault zone", that is, the rock body has turned into debris with many joints, thereby making the rock mass classification by joint spacing difficult.

- It is, therefore, necessary to consider the strength ratio which is determined by the strength of rock specimen and the overburden thickness. This ratio was found to be the factor governing the stability of tunnels.

- The size of the tunnel section to be excavated should also be given full consideration in evaluating the geology.

In summary, Nakao et al. found that while input parameters of the RMR and Q indices may be applicable to the geological conditions of Japanese tunnels, it may be necessary to correct the weighting given to the parameters to better reflect the true nature of soft and crushed rock with complicated geological structures.

### 5.2.8 Application at the Mt Isa Mine, Queensland

Baczynski (1983) applied five rock mass classification methods including RMR, MRMR and Q to unsupported mine openings (open stopes) at the Mount Isa Mine in Queensland, Australia. The aim of his study was to assess the usefulness and limitations of the classification methods for unsupported open stope design in tabular lead-zinc ore bodies trending parallel to the bedding planes that dip $65^{o}$ to the horizontal. The hanging wall and foot wall of the stopes were defined by moderately to highly jointed and bedded dolomitic shales. He investigated three ore bodies within dolomitic shales and apparently used the $RMR_{76}$ and $Q_{74}$ versions.

Baczynski used the "Monte Carlo" simulation method (Hamersley and Handscomb, 1964) for statistical analysis of the RMR, MRMR and Q index parameters within the hanging wall shales of the three orebodies. The aim was to evaluate the local variability in ground conditions within single stopes as well as the overall variability in mean ground conditions between stopes within particular orebodies. The main conclusions drawn from the study were:

- There are certain difficulties associated with the application of the three systems, and none of the systems appeared to be completely satisfactory. However, they are considered to be potentially useful subject to modifications to suit local mining requirements.

- For the determination of stable unsupported stope spans the Q index appeared to be the most promising classification. However, the system needs modifications to suit the structural environment at the mine.

- The RMR system yields conservative estimates for unsupported stope spans. This appeared to be due to three factors: (a) the system does not provide for the incorporation of stress effects on stability; (b) although the system attempts to determine stand-up time which could then be indirectly related to stability in mining situations, the proposed times appear to be conservative with respect to past experience at the mine; (c) there is an upper limit of 20 m for the maximum permissible span dimension. This value is extremely conservative, especially since stopes with hanging wall spans in excess of 30 m have been mined.

Baczynski found a linear correlation between *RMR* and *Q*, based on the analysis of 2000 statistically generated rock mass blocks for two dolomitic shale ore bodies at the Mt Isa mine. With the Q system he used a *SRF* value of 2 in determining *Q* values. The correlation obtained by Baczynski (Equation 5.6) is in close agreement with Equation 5.1 proposed by Bieniawski (1976).

$$RMR = 7.5 \ln Q + 42 \tag{5.6}$$

Despite the apparent similarity of the two equations, Baczynski observed that the correlation would have been different if the Q system recommended *SRF* values were used in deriving the *Q* values, and stated that, "*However, it must be strongly emphasised that the correlations are stress dependent. The relationship will be significantly altered, if, for example, different SRF values are assumed in the determination of the Barton's Q rating. It is therefore important that any relationship for the transformation from one classification rating to another is not assumed to have universal application*".

### 5.2.9 Experience from the Porter Square Station Cavern, Massachusetts

As part of a study on empirical methods of underground excavation support design, Einstein et al. (1983) applied RMR and Q with three other methods, to a 21 m span, 14 m high and 150 m long subway station cavern and a 3.6 m x 3.6 m pilot tunnel driven along the crown of the cavern, and compared them with regard to the subjectivity and the influence of available information through four phases of the project (i.e. three phases of exploration and during excavation). Specifically the four phases were: Phase I (data from 19 borehole logs), Phase II (data from an exploration shaft and Phase I), Phase III (data from the pilot tunnel, the rock mass was classified every 3 m along the tunnel axis), and Phase IV (data from the station cavern, the rock mass was classified every 3 m along the cavern concentrating on the vertical face of a permanent bench, and supplementary mapping of the cavern face after each round of excavation).

The cavern located in Cambridge, Massachusetts, USA, was excavated in a sedimentary rock (argillite). The study conducted using several investigators also included a comparison of empirically derived support measures with those installed (derived by analytical and numerical methods). It was based on the assessment of three possible rock mass scenarios: best, worst and most probable. The main conclusions drawn from the study were:

- The boring detected only two joint sets. It did not reveal the presence of a third joint set, while the exploration shaft and the pilot tunnel did. This unfavourably oriented third joint set did not affect the predictions because other conditions had already led to classify the rock mass into lower classes. This may be different in other applications and the three dimensional exposure will have a greater effect.

- The additional data obtained from the exploration shaft and pilot tunnel did not change the most probable and extreme support requirements predicted from the (extensive) boring program.

- Increased support requirements that resulted from the cavern survey were not predicted in any of the exploration phases.

- The two methods are not affected by subjectivity of the user. This is mostly due to the dampening of parameter differences when relating rock classes to support. However, the effect of subjectivity did not disappear in the spatial fluctuation of rock class predictions by different investigators. These fluctuations, which indicate the length over which a construction procedure and a support system would be used, are important in longer tunnels.

- The support predicted for most probable ground conditions falls into a narrow range below the actually placed support. Two methods are thus roughly equivalent for this purpose.

- The predictions for the worst conditions in the cavern seem to be possible only with Q which covers the particular combination of large span and low quality rock mass conditions.

- The encountered conditions in the cavern deviated from those predicted in an extensive exploration program. In such situations a conservative design or an adaptable design and construction approach is required. For an adaptable approach the empirical methods can be used only if the methods cover the entire range of conditions.

### 5.2.10 Canadian Mining Experience

Two separate studies were conducted in Canadian hard rock mines: the first by Udd and Wang (1985) and the second by Choquet and Charette (1988).

Udd and Wang (1985) assessed the advantages and disadvantages of four classification systems by applying them to the rock masses in an underground mine in Canada. The four systems were RMR, Q, RQD and Rock Consolidation Coefficient (RCC). The RCC method, which is now obsolete, is defined as the ratio *IRS*/100. The main rock types in the mine are meta-anorthosite and its various stages of alterations consisting sericite, chlorite, chloritoid and talcose minerals. The rock masses in the mine are extensively fractured. Udd and Wang applied the RMR and Q systems to ten rock types and obtained 153 *RMR* and *Q* data pairs and then compared the results of the application of the two methods. The main conclusions drawn from the study were:

- Both RMR and Q methods were found to be easy to use in the field, with RMR being the easiest.

- There was not always good agreement between the RMR and Q methods. The rocks which were described as being good or better quality by RMR were described as of lesser quality on the Q scale. Rock masses rated as good or better by the Q system would sometimes be less than good on the RMR scale.

- The Q system was the most appropriate for the site conditions.

By linear regression analysis of the 153 data pairs Udd and Wang obtained a correlation between the RMR and Q values as give in Equation 5.7.

$$RMR = 5.3 \ln Q + 50.81 = 12.11 \log Q + 50.81 \qquad (5.7)$$

The range of *Q* values used in the study was 0.01 to 1000 and that of *RMR* was 20 to 100. These ranges do not include the lowest quality rock mass classes in the RMR and Q rating scales, i.e. exceptionally poor class in the Q system and very poor class in the RMR system. The *RMR-Q* data plot provided by Udd and Wang shows wide scattering about the regression line given by Equation 5.7 which confirms the commentary by the two authors: "*There is not always good agreement between the two methods*".

Choquet and Charette (1988), conducted their study in ten underground mines in Quebec, Canada, and assessed the applicability of six rock mass classification systems to mine drifts. The six methods were RMR, MRMR, Q, SRMR and one modified version each of RMR and MRMR. These were applied to 57 drifts in the ten underground mines representing different rock mass conditions. The drift spans varied between 2.8 and 7.5 m with the majority between 3.5 and 5.5 m. Their depths ranged from 50 to 1000 m with the majority between 100 and 500 m. All of them were stable.

The two authors have correlated the *RMR* and *Q* ratings assigned to the 57 drifts and obtained the relationship given by Equation 5.29 with a correlation coefficient of 0.86.

$$RMR = 10 \, ln \, Q + 39 \tag{5.8}$$

Their *RMR-Q* data plot shows data scattering. Comparing the above equation with that of Bieniawski (1976), Equation 5.1, the authors commented that the main differences were observed for the low values of the Q ratings.

Choquet and Charette also compared the predicted rock bolt densities with the actual bolt densities in the 57 drifts. They found that, in general, the installed support density is inversely proportional to the rock mass quality but the scatter of actual support densities about the predicted bolt density line is very large. They observed that seven drifts were unsupported and such a situation should correspond to better rock mass conditions with higher ratings compared to supported drifts. However, four of the classification systems assigned equal or higher ratings for some of the supported drifts. The four systems are RMR, MRMR, SRMR and $RMR_{mod}$. They concluded that these four methods cannot be used on their own to determine if a drift needs support. Preferably $MRMR_{mod}$ and Q should be used for this purpose. They found that the installed support measures in the drifts are significantly higher than those predicted by the classification systems.

### 5.2.11 Application to Civil and Mining Tunnels in Sri Lanka

Brook and Dharmaratne (1985) critically reviewed three rock mass classification systems: RMR, Q and the modified or adjusted RMR system proposed by Laubscher (1975). They applied these three systems to mine and hydroelectric tunnels in Sri Lanka and compared the support requirements predicted by them with the actual support installed. Three projects were considered: the Bogala graphite mine (BGM), the Kahatagaha-Kolonggaha graphite mine (KKGM) and the Victoria hydropower project (VHP) headrace tunnel.

At KKGM the rock types are garnet biotite gneiss and amphibolite biotite gneiss, inter-banded with quartzite. The tunnels are about 2.8 m in width and height, and the rock mass has only two joint sets. The mine is almost dry and no rock instability problems were present. The 700 m deep mine has been in operation for more than 100 years without any form of significant support apart from occasional spot bolting where joint intersections create potentially dangerous rock blocks.

At BGM the main rock types are gneisses, charnockite and quartzite. Tunnels are smaller in cross section to those of KKGM but the number of joint sets varies from one to three in different parts of the mine. The rock mass is weathered in many places. The tunnels and stopes encounter large water inflows in some areas giving rise to instability problems. The main support type is timber sets but rock bolting and concreting are also used occasionally. The mine is operated at a depth of 400 m.

The VHP headrace tunnel was excavated by full face drill and blast methods to a horseshoe shape with a 7.2 m span. The main rock types intersected along the tunnel were garnetiferrous quartz gneiss, granulite, quartz granulite and crystalline limestone (marble). The rock mass is fresh to highly weathered (HW) and the tunnel was dry to very wet during construction. Faults and foliation planes give rise to over-breaks and uneven tunnel periphery. Rock bolts, shotcrete and concrete were adopted as temporary support. The tunnel was fully concrete lined to give a 6.2 m internal diameter circular shape for hydraulic considerations.

Brook and Dharmaratne assessed eleven sites from the three projects: one, four and six sites from KKGM, BGM and VHP, respectively. They provided a representative sample of rock mass characteristics from each of the eleven sites with the final RMR and Q ratings assigned, support predicted by the two methods and the actual support installed at each site. A summary of the support predicted using the RMR and Q methods and the actual support installed is given in Table 5.2.

It should be emphasised that the RMR support recommendations are for 10 m span horseshoe shape permanent tunnels and are not necessarily applicable to 2.8 m wide mine tunnels at KKGM and BGM, A particular problem arises in the application of RMR deduced support because the recommended bolt lengths and spacing are too great for small span tunnels such as those of KKGM and BGM. For the VHP headrace tunnel with an as-excavated span of 7.2 m, the RMR recommended support may be considered applicable providing that bolts lengths are adjusted to suit the size of the excavation. Nonetheless, in Table 5.2 Brook and Dharmaratne listed the RMR recommended support for all three projects. It should also be noted that the VHP tunnel was fully concrete lined mainly for hydraulic reasons and partly for stability. Therefore the actual support installed, and listed in Table 5.2, was mainly for short term safety and can be considered the absolute minimum required to stabilise the potentially unstable rock masses observed during construction.

In their study, Brook and Dharmaratne (1985) used $RMR_{76}$ and $Q_{74}$. A check against $RMR_{89}$ showed no difference in the support recommendations from those presented by them except for what might be considered as minor typographical errors which were corrected in Table 5.2. A check against $Q_{94}$ showed a reduction in support quantities compared to those recommended by the earlier version. This means $Q_{94}$ support recommendations would be much less than the actual support installed.

Table 5.2 Predicted and actual support installed in the KKGM, BGM and VHP tunnels (after Brook and Dharmaratne, 1985)

| Site and rock type | RMR prediction | Q prediction | Actual support |
|---|---|---|---|
| *KKGM:* | | | |
| Garnet biotite gneiss *RMR*=80, *Q*=30 | Generally no support, except spot bolting | No support | None |
| *BGM:* | | | |
| Quartzite *RMR*=70, *Q*=20 | 3 m long, 2.5 m spaced bolts locally in crown, occasional wire mesh, 50 mm shotcrete as required | No support | None |
| Fresh garnet biotite gneiss *RMR*=65, *Q*=9 | 3 m long, 2.5 m spaced bolts locally in crown, occasional wire mesh, 50 mm shotcrete as required | No support | None (block failures occurred & may have been prevented by spot bolting) |
| SW[a] garnet biotite gneiss *RMR*=51, *Q*=4 | 4 m long, 1.5-2 m spaced systematic bolts, shotcrete 50-100 mm with mesh in crown, 30 mm in sides | No support | 600 mm of concrete in walls & roof (shotcrete may be adequate, but was not available) |
| HW[a] garnet biotite gneiss *RMR*=35, *Q*=1 | 4-5 m long, 1-1.5 m spaced systematic bolts, shotcrete 100-150 mm in crown & 100 mm in sides with mesh; light-medium ribs at 1.5 m spacing[b] | No support | Timber sets at 1 m spacing with lagging |
| *VHP:* | | | |
| Quartzite *RMR*=72, *Q*=25 | 3 m long, 2.5 m spaced bolts locally in crown, occasional wire mesh, 50 mm shotcrete as required | No support | Pattern grouted bolts at 1-1.5 m spacing |
| Fresh garnet quartz gneiss *RMR*=97, *Q*=84 | Generally no support, except spot bolting | No support | Pattern grouted bolts at 1-1.5 m spacing |
| SW[a] garnet quartz gneiss *RMR*=62, *Q*=21 | 3 m long, 2.5 m spaced bolts locally in crown, occasional wire mesh, 50 mm shotcrete as required | No support | Pattern grouted bolts at 1-1.5 m spacing |
| MW[a] garnet quartz gneiss *RMR*=53, *Q*=15 | 4 m long, 1.5-2 m spaced systematic bolts, shotcrete 50-100 mm with mesh in crown, 50 mm on sides | Bolts 1.5-2 m spacing, shotcrete 20-30 mm | Pattern grouted bolts at 1-1.5 m spacing, occasional wire mesh |
| HW[a] garnet quartz gneiss *RMR*=33, *Q*=2 | 4-5 m long, 1-1.5 m spaced systematic bolts, shotcrete 100-150 m in crown & 100 mm on sides with mesh; light-medium ribs at 1.5 m spacing[b] | Bolts 1-1.5 m spacing, shotcrete 150 mm with wire mesh | Steel arches at 1 m spacing with lagging |
| Crystalline limestone (marble) *RMR*=92, *Q*=711 | Generally no support, except spot bolting | No support | Pattern grouted bolts at 1-1.5 m spacing |

[a] SW=slightly weathered, MW=moderately weathered, HW=highly weathered; [b] where required

From the study carried out by Brook and Dharmaratne the following conclusions can be drawn regarding the RMR and Q classification systems.

- There is not always good agreement between RMR and Q predicted support measures. The two systems predicted comparable support for only four out of the eleven sites studied, three of these sites are in the VHP tunnel and the other is the KKGM tunnel.

- The RMR support predictions were generally conservative for the five mine tunnels. This is primarily due to the small span of the mine excavations.

- The RMR recommended bolt spacing was often greater than those used in the VHP tunnel.

- For HW garnet biotite gneiss in the VHP tunnel, the RMR recommended steel sets were generally comparable to those installed, yet the predicted rock bolts and shotcrete exceeded the actual support.

- The Q predicted support requirements matched only for two of the eleven sites considered, both being small span mine tunnels that required no support.

- For MW garnet quartz gneiss in the VHP tunnel, Q recommended support agreed with the installed support.

- For nine out of the eleven sites studied the Q system underestimated the support requirements.

Since the RMR recommendations are for permanent support, the comparison of the RMR predicted support with the temporary support installed in the VHP tunnel may seem an unfair test for this particular classification system. Yet, as can be seen from Table 5.1, smaller rock bolt spacing (higher bolt densities) was adopted for construction safety than those deduced using the RMR system. The Q system recommendations given in Table 5.1 are also for permanent support. Q predicted no support requirements for four out of the six sites in the VHP tunnel, yet all four sites were temporarily supported with rock bolts for construction safety. From the foregoing, it can be concluded that both RMR and Q systems underestimated support requirements for the VHP headrace tunnel.

### 5.2.12 Application to BC Rail Tunnels, Canada

In a detailed study to evaluate the reliability of the empirical classification approach to tunnel support design, Kaiser et al. (1986) applied RMR and Q along with RSR and RQD methods to four single track rail tunnels driven through sedimentary rocks in central British Columbia, Canada. The four tunnels were 271, 367, 9050 and 5936 m long with excavated widths 5 to 5.5 m and heights 8 to 8.5 m. All four are horseshoe shaped and were excavated by conventional drill and blast techniques. The study was based on detailed mapping of 85 sectors representing the seven different rock formations and the full spectrum of conditions encountered in the four tunnels. The study included comparisons of support predicted by different methods with each other and with the support installed, a correlation of final *RMR* and *Q* values, and an assessment of no-support limit and opening size effects. Some of the key conclusions drawn from the study were:

- Verbal descriptions given by RMR and Q systems do not correspond well. Too many ground class boundaries are specified by the Q system.

- The no support limit given in RMR was found to be too conservative. Based on RMR, stand-up times of as little as one day to several months were predicted for sections that were permanently unsupported.

- Poor agreement between RMR predicted and actually installed support was observed even if it is considered RMR charts were developed for permanent support. While no support (except for occasional bolting) was recommended only for *RMR*>80, unsupported sections were found at *RMR*=30 to 100. Heavy support with bolts at less than 1.5 m spacing and more than 100 mm of shotcrete at the crown and tunnel walls was recommended for *RMR*<60. In reality, pattern bolting was applied for a *RMR* value range of 20 to 80 and shotcrete was used when *RMR*=30 to 80. This clearly indicated that the RMR system is far too conservative for the rock types encountered and the size of openings.

- The significant overlap of support types for very similar ranges of RMR also suggests that the RMR method is not sensitive enough for the grouping of rock conditions to select tunnel support types.

- Despite established correlations, these two classification systems do not lead to similar conclusions concerning tunnel support requirements. Both are conservative but RMR is more so. This discrepancy must be attributed to the influence of opening size on tunnel performance and cannot be eliminated by simultaneous application of both classification systems to assess the factors that have been neglected.

- Bolts were consistently spaced at 2 m in areas where 1.0 to 1.5 m spacing was recommended by Q.

- In areas prone to rock popping and bursting, it may be more appropriate to neglect the *SRF* factor of Q during rock mass classification and assess the detrimental effects of high stresses separately.

- Overall, the recommendations for tunnel support based on Q were found to correspond better with the conditions encountered at the BC-Rail tunnels.

The study by Kaiser et al. (1986) also included a detailed analysis of *RMR* and *Q* values to obtain reliable correlations between the two ratings. They pointed out that the correlations developed using least square linear regression analysis should be viewed with caution because the results depend on the choice of the dependent variable. To overcome this weakness, they used a probabilistic approach to determine a unique relationship between RMR and Q assuming that *RMR* and *ln Q* are normal variates and satisfy the central limit theorem of probability theory. The correlations obtained by the least square linear regression analysis and the probabilistic approach are presented as Equations 5.9 and 5.10, respectively.

$$RMR = 6.3 \ln Q + 41.6 \qquad (5.9)$$

$$RMR = 8.7 \ln Q + 38 \pm 18 \qquad (5.10)$$

Despite the use of a probabilistic approach, Kaiser et al. observed wide scattering of the data and therefore the correlation was presented as two equations representing 90% confidence limits within which 90% of the data used for their study fall. However, they noted that the range of values represented by the two equations is of little practical value as it covers almost two RMR ground classes, as in the case of 90% confidence limits given by Bieniawski (1976).

### 5.2.13 Problems in Changing Loading Conditions

Speers (1992) investigated the suitability of the RMR and Q systems as support design tools for underground excavations with changing loading conditions. He cited three cases where, subsequent to the tunnel development, the field stress or rock loads changed with time. One such case, a tunnel located adjacent to a road cutting which was widened thereby causing unloading of rock, is illustrated in Figure 5.1.

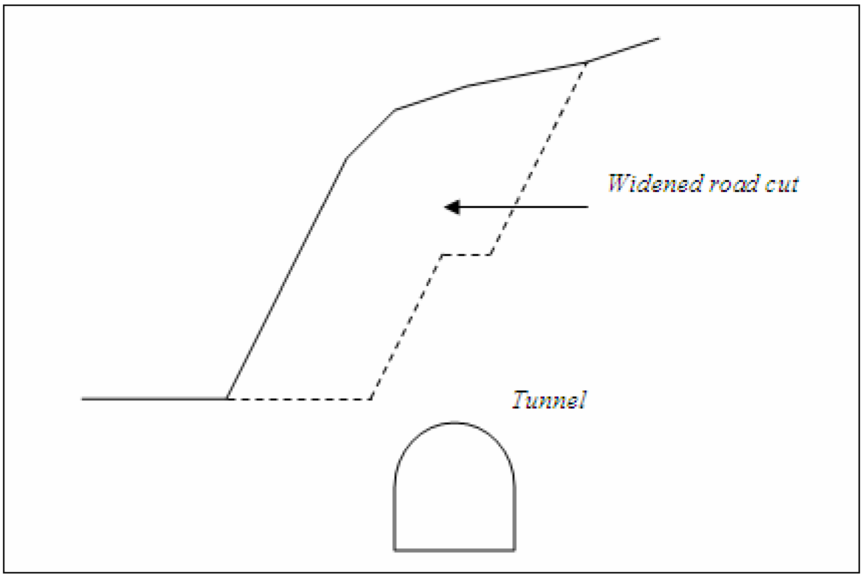


Figure 5.1 Tunnel beneath a road cutting which was subsequently widened

As part of his investigation, using a simplified numerical analysis, Speers illustrated that the support pressure can be constant for different size tunnels only if the length of rock bolts is equal to the tunnel radius. If the bolt length is not equal to the tunnel radius, the support pressure will depend on the tunnel size and the bolt length. This is important because the empirical formula given in the Q system for support pressure estimation does not take into account the size of the excavation.

Speers compared the RMR and Q derived support measures with those designed by numerical stress analysis using FLAC. The support designs were for a tunnel situated in a block cave mine with a constant horizontal load and three different vertical loads at various stages of its operation: (a) an initial load of 265 m during construction, (b) a 1325 m load while drawing caved ore, and (c) a 63 m load after exhaustion of the caved ore. Two tunnel sizes were examined: the first 3 m high and 3.5 m wide; the second 6 m high and 6.5 m wide. With the Q method, different *SRF* values were used as proposed by Kirsten (1988b) to calculate the support pressures. The main conclusions drawn from the study were:

- RMR does not take into account variable rock loads. Hence the same support measures are recommended for all stages of loading.

- For the initial stage with intermediate loading, the support pressures are independent of tunnel size for a given bolt length. If the bolt length is increased, however, the pressure increases. The pressures are similar in order of magnitude to those predicted by the Q method.

- For the maximum and minimum vertical loading stages, the pressures calculated using FLAC are considerably higher than those predicted by Q.

- The support pressures are not independent of tunnel size: the bigger the tunnel, the greater the pressure. Further, the pressure for a given tunnel size increases with increasing bolt length because the tunnel deformations are reduced with an increase in bolt length and, therefore, the equivalent pressure is higher.

- The highest pressures occur at minimum loading stage and not at maximum loading, as was predicted by the Q method. The result demonstrated the severity of unloading on an initially developed and supported tunnel.

- The use of empirical approaches such as the RMR and Q methods will lead to under designs, particularly with regard to the capacity of the bolts.

- RMR provides a reasonable estimate of rock mass strengths, but not the rock mass modulus.

- Although bolt lengths can be estimated using the Q method, the predicted bolt lengths will be too short. It appears that this is because the Q method inherently provides a means of stability control, but not a deformation means of control. Also, the Q method applies to tunnels that do not undergo changing rock loads after the initial excavation and support.

### 5.2.14 Indian Tunnelling Experience

Indian rock engineering researchers attached to the University of Roorkee, the Nagpur University and the Central Mining Research Station, over several years, have undertaken a detailed study on support pressures in tunnels excavated in both squeezing and non-squeezing ground conditions in India. Much of this work was undertaken in poor quality rocks with difficult tunnelling conditions in the Himalayas. The vast majority of the tunnels studied were for hydroelectric schemes and the rest were for roads and railways as well as mining. Squeezing conditions are usually encountered in the hydroelectric tunnels excavated in the lower Himalayas where the rocks are weak, highly jointed, faulted, folded and tectonically disturbed, and the overburden is very high. The combination of weak rock mass and the high in situ stress causes squeezing. A summary of the Indian research is presented below.

Singh et al. (1992) compared the observed support pressure and rock mass quality based on systematically collected field data from 24 tunnel sections in India. The field data collected were tunnel radius and depth, unit weight of overburden material, *Q* and *RMR* values of tunnel sections, hoop loads and radial pressures on steel ribs, tunnel closure and deep-seated radial displacement measurements. The study focussed on several important aspects including the prediction of squeezing ground conditions, comparison of predicted and measured support pressures, influence of overburden (tunnel depth) on support pressure, relationship between support pressure and tunnel closure in squeezing ground conditions, variation of support pressure with time, and the effect of excavation size on support pressure. For predicting roof support pressures, Singh et al. used six classification methods including RMR and Q and found that the predictions of all the six methods are unreliable except for the Q predictions for non-squeezing ground conditions. The wall support pressures were excluded from the comparison because of the insufficient number of measurements. The main conclusions drawn were:

- Q predicted roof support pressures were reliable for non-squeezing ground conditions, but the predictions tuned out to be unreliable for squeezing ground.

- Squeezing is likely to occur in a tunnel section where the height of overburden in meters exceeds $350Q^{1/3}$.

- The short term roof support pressure, $P_{roof}$, is given by

  $$P_{roof} = \frac{2}{Jr}(5Q)^{-1/3}\, f\, f^{'} \qquad (5.11)$$

  in which $f$ is the correction factor for overburden thickness ($H$) in meters and $f'$ is the correction factor for tunnel closure. The correction factor $f$ is defined as

  $$f = 1 + (H - 320)800 \geq 1$$

  The correction factor $f'$ for tunnel closure varies from 0.7 to 1.8 in the case of a single tunnel in squeezing ground conditions in which support pressure is significantly influenced by tunnel closure. For non-squeezing ground $f^{'}=1$.

- Minimum support pressure occurs when the tunnel closure is about 5% of the tunnel diameter and pressure increases rapidly beyond this limiting closure.

- Short term wall support pressure may be obtained from the above correlation by substituting $Q_{wall}$ for $Q$ (see Table 4.14). In general, the actual wall support pressure for non-squeezing rock conditions is likely to be negligible.

- The ultimate support pressure may be 1.75 times the short term support pressure for tunnel sections under non-squeezing ground conditions, except for cases of soluble and erodible joint filling with seepage. For estimating ultimate roof support pressure, a third correction factor, $f''$, which is a function of time $t$ in months since support installation, may be included in Equation 5.11, so that

$$P_{roof\,(ult)} = \frac{2}{Jr}(5Q)^{-1/3}\, f\, f'\, f'' \tag{5.11a}$$

where,

$$f'' = \log\,(9.5\, t^{0.25})$$

- The support pressure is independent of tunnel size provided that $Q$ is obtained from a full-sized opening.

Goel et al. (1995), based on more than 20 years experience, presented an evaluation of the support pressures predicted by RMR and Q for 25 tunnel sections of several projects in India. The tunnel widths varied from 2 to 14 m, and covered both squeezing and non-squeezing ground conditions. They compared the predicted support pressures with those measured, and proposed correction factors to RMR and Q for reliable estimation of support pressures. A new correlation was also proposed between RMR and Q values.

As can be seen from Equation 4.5, the RMR method advocates that the support pressure increases directly with the tunnel size, whereas the Q system suggests that the support pressure is independent of the tunnel size (see Equations 4.10, 4.11, 4.12 and 4.13). Goel et al. examined these issues covering both civil and mining sectors in India. Their study was based on the measurement of support pressures by inserting load cells into steel arch support joints and installing contact pressure cells at steel arch and backfill interfaces at the 25 tunnel sections. Some of the results of this study were previously presented by Jethwa et al. (1981, 1982) and Sing et al. (1992). Goel et al. noted that RMR and Q are not truly equivalent. For example, RMR does not take into account the stress condition of the rock mass, while Q does not consider joint orientation and intact rock strength as independent parameters. In order to correlate the two systems rationally, Goel et al. introduced a rock condition rating RCR defined as RMR without *JA* and *IRS* parameters and rock mass number *N* defined as *Q* with *SRF*=1. Note that *RCR* and *N* are identical to $RMR_{mod}$ and $Q_{mod}$ defined by Sheorey (1993) and discussed in Section 5.2.15.

The study undertaken by Goel et al. drew several important conclusions on the prediction of support pressures using Q and RMR methods under both squeezing and non-squeezing ground conditions and the effect of tunnel size on support pressures and prediction of squeezing conditions. Further, the study presented conclusions regarding correlations between Q and RMR. The conclusions that are directly relevant to Q, RMR, tunnel size and prediction of squeezing conditions are presented below. The conclusions regarding correlations between Q and RMR are presented in Section 5.3.

The conclusions directly relevant to Q were:

- The estimated support pressures for squeezing ground conditions were found to be unsafe in at least two 9 m diameter tunnel sections. The limited data showed that Q tends to be unsafe for large tunnels in squeezing ground conditions.

- The stress reduction factor, *SRF*, does not adequately represent the squeezing effect, due to the lack of sufficient number of case histories in the original database used in developing the Q system.

- For estimating roof support pressures in rock tunnels, *N* (i.e. *Q* with *SRF*=1) should be used. For non-squeezing conditions, correction factors based on tunnel depth and radius proposed by Goel et al (1995) given below should be used.

$$P_{roof\,(ult-nsq)}(N) = \left(0.12H^{0.1}r^{0.1} / N^{0.33}\right) - 0.038 \qquad (5.11b)$$

where, $P_{roof(ult\text{-}nsq)}$ is ultimate support pressure in non-squeezing ground in MPa, $H$ is tunnel depth in metres, $N$ is $Q$ with $SRF$=1, $r$ is tunnel radius in metres.

- For squeezing conditions, in addition to the above mentioned correction factor, a further correction factor reflecting the tunnel closure should be used.

$$P_{roof\,(ult-sq)}(N) = \left[f(N)/30\right]\left[10^{(H^{0.6}\,r^{0.1}/50N^{0.33}}\right] \qquad (5.11c)$$

where, $P_{roof(ult\text{-}sq)}$ is ultimate support pressure in non-squeezing ground in MPa, *f(N)* is dimensionless correction factor for tunnel closure (five values are given by Goel et al. for five different rock mass conditions), $H$, $N$ and $r$ are as defined above.

- For application to horizontally stratified coal measures, $Q$ has to be replaced with $Q_{mod}$ by replacing $Jn$ by $Jn^{2/3}$ and removing extra weightage given to $Jr$.

The conclusions directly relevant to RMR were:

- In rock tunnels in squeezing rock conditions, the estimated support pressures are unsafe for all sizes of tunnels investigated.

- The estimates for non-squeezing rock conditions are unsafe for small tunnels and overly safe for large tunnels.

- When applied to coalmine roadways, the estimated support pressures are unrealistically low in poor rock masses.

- For estimating roof support pressures in rock tunnels in non-squeezing conditions, the following correction factors proposed by Goel et al. based on tunnel depth and radius should be used:

$$P_{roof\,(ult-nsq)}(RMR) = (2.32 - 0.035RMR + 0.001H + 0.03r) \quad (5.12)$$

  where, $P_{roof(ult\text{-}nsq)}(RMR)$ is ultimate support pressure in MPa in non-squeezing ground, $H$ is tunnel depth in metres, $r$ is tunnel radius in metres.

- For squeezing conditions, in addition to the $H$ and $r$ corrections factors a further correction factor $f(RMR)$ reflecting the tunnel closure should be used as give below:

$$P_{roof\,(ult-sq)}(RMR) = [f(RMR)/12]10^{[(1.8H^{0.4}r^{0.1}/RMR^{1.2})]} \quad (5.12a)$$

  where, $P_{roof(ult\text{-}sq)}(RMR)$ is ultimate support pressure in MPa in squeezing ground, $f(RMR)$ is correction factor for tunnel closure (Goel et al. provide six values for different degrees of squeezing), $H$ is tunnel depth in metres, $r$ is tunnel radius in metres.

- Even with modifications for stress, the RMR system remains unsafe for mine roadways in poor coal measures.

The conclusions regarding the effect of tunnel size were:

- The support pressures for rock tunnels in non-squeezing ground conditions can be taken as independent of the tunnel size, whereas in squeezing ground conditions, support pressure increases significantly with tunnel size. Furthermore, the size effect increases with tunnel depth. In addition, poorer rock masses experience a higher size effect. For example, the support pressure increases by 40 percent when the tunnel width is increased from 3 to 12 m at a depth of 450 m, compared to an increase of 60 percent at a depth of 700 m for poor rock masses where $N$=0.5. The corresponding increases are only 14 to 19 percent for $N$=10. For very poor rock masses described by $N$=0.1, these increase would be between 75 and 100 percent.

- In the case of flat-roofed mine roadways through coal measures, the support pressure increases directly with the roadway width.

Goel et al. also found the tunnel depth $H$, tunnel radius $r$ and $N$ can be used to predict whether a tunnel section is likely to experience squeezing ground conditions. Such a situation can be avoided by realigning the tunnel through a reduced $H$ or improved $N$. A larger tunnel may be replaced by two or three smaller tunnels when these two alternatives are not possible.

Subsequently, Goel et al. (1996) assessed the effect of tunnel size on support pressure using Equations 5.11b and 5.11c. The results of this assessment are presented in Table 5.3.

Table 5.3 Increase in support pressure due to increase in tunnel width from 3 to 12 m

| Tunnel shape and rock mass condition | Increase in support pressure |
|---|---|
| (a) Tunnel with arched roof | |
| Non-squeezing ground | Up to 20% only |
| Poor rock/squeezing ground ($N$=0.5-10) | 20-60% |
| Soft plastic clays, running and flowing ground, clay filled moist fault gouges, slickensided shear zones ($N$=0.1-0.5) | 100% |
| (b) Tunnel with flat roof (irrespective of ground conditions) | Up to 100% |

Further, Goel et al. (1996) cautioned that the support pressure is likely to increase significantly with the tunnel size for tunnel sections excavated through the following situations:

- Slickensided zones
- Thick fault gouge
- Week clay and shales
- Soft plastic clays
- Crushed brecciated and sheared rock masses

- Clay filled joints, and
- Extremely delayed support in poor rock masses.

Singh et al. (1997) presented the results of a study on the assessment of support pressure in arched underground openings through poor rock masses. The assessment was based on Indian tunnelling experience of over 20 years in more than 60 instrumented tunnel sections in the Himalayas and in other parts of the country and also on the experience of the Norwegian Geotechnical Institute. The study considered Q, RMR, RSR, RQD and Terzaghi's rock load concept, and presented an assessment of predicted and measured support pressures under squeezing ground conditions in the lower Himalayas in India, where the rock masses are weak, highly jointed, faulted and the overburden is high. The study highlighted the difficulties in estimating the *SRF* parameter of Q and a criterion for predicting the ground conditions was proposed using the rock mass number *N* (defined as *Q* with *SRF*=1). Regarding the prediction of support pressures, Singh et al. reiterated the findings of Goel et al. that, "*while none of the approaches are applicable under squeezing ground conditions, Q provides a reasonable estimate of support pressure in non-squeezing conditions and for smaller tunnels under squeezing ground conditions*". The main conclusions drawn from the study were:

- Experience in Himalayan tunnels suggest that the correction factor *f* for tunnel depth proposed by Singh et al. (1992) adequately accounts for the stress conditions for both non-squeezing and squeezing ground conditions. Therefore, there is no need to use increased *SRF* values suggested by Grimstad and Barton (1993).

- The correction factor *f'* for tunnel closure proposed by Singh et al. (1992) is almost equal in magnitude for both roof and walls.

- In non-squeezing ground conditions the support pressure is independent of tunnel size between 2 and 22 m.

- The *Q* values estimated from a larger tunnel would be smaller than those obtained from small drifts in a similar rock mass. This is due to the possibility of intersecting more geological discontinuities and intrusions in a large opening.

- In poor quality brecciated rock masses experiencing squeezing conditions, the support pressure increases with tunnel span.

- The support pressure in rock tunnels and caverns does not increase directly with excavation size due mainly to the dilatant behaviour of rock masses, joint roughness and prevention of loosening of rock mass by modern tunnelling technology. However, the support pressure is likely to increase directly with the excavation width for tunnels through slickensided shear zones, thick clay filled fault gouges, weak clay shales and running or flowing ground conditions where interlocking of blocks is likely to be missing.

- The support pressures in squeezing ground conditions decrease with tunnel closure significantly and increase rapidly beyond a 6% closure. Assessment of support pressures in shear zones and walls of caverns should be carried out cautiously. Shear zones treatment should be done properly.

- Under swelling ground conditions, the reliability of any of the approaches is yet to be established. Therefore, laboratory tests and field instrumentation are suggested.

Singh et al. noted that there are problems in obtaining correct values of *SRF* near weakness zones intersecting an excavation. For example, two tunnels at depths of 100 and 300 m from the surface, indicating different magnitudes of cover pressure, are being excavated through the same rock mass with a single weakness zone containing clay or chemically disintegrated rock. For both tunnels, the *SRF* value will be 2.5, because the depth of excavation is more than 50 m. This clearly shows that precise weightage to stress condition is missing from *SRF*, thereby indicating inadequacy in the Q system in the complex Himalayan region.

Further, a large range of *SRF* values are suggested when a shear zone only influences but does not intersect the excavation. For "competent rock masses" the determination of *SRF* is based on $\sigma_1$, $\sigma_3$, $\sigma_c$ and $\sigma_\theta$ values (where $\sigma_1$ and $\sigma_3$ are major and minor principal stresses, $\sigma_c$ and $\sigma_\theta$ are UCS and tensile strength of rock material) and the suggested values of *SRF* have wide ranges.

It should be noted that the empirical formulas for predicting support pressures using Q values (or *N* values), discussed in this section, are based on the experience gained from measurements in tunnels driven in the Himalayas and supported with steel ribs. When using these equations for practical applications it is advisable to take cognisance of the following:

- The Q system does not recommend steel ribs as a method of tunnel support.
- The empirical formulas for support pressure given in the Q system intend to predict pressures on shotcrete and rock bolts. Mechanics of their behaviour are different to those of steel ribs.
- In the Indian case tunnels $\sigma_1$ is assumed to be due to the overburden and vertical. The horizontal stress magnitudes are not provided.
- In high horizontal stress conditions the reliability of the suggested formulas is doubtful.

### 5.2.15 Indian Coal Mining Experience

Sheorey (1993) applied three rock mass classification methods, namely RMR, Q and CMRS (Central Mining Research Station) method, to underground coal mine roadways in India and provided the results from 44 case studies. The rock types covered in the study were more or less horizontally bedded sandstone, shale, siltstone, coal, shaly coal, mudstone, shaly sandstone and sandy shale. The roadway widths varied between 3.0 and 5.2 m with the majority being 3.5 to 4.5 m.

The study included estimation of support pressures and the determination of safe unsupported spans using the empirical formulas given in the RMR and Q methods. For simplicity, Sheorey used Equation 4.10 (see Chapter 4) provided by Barton et al. (1974) instead of Equation 4.12 presented in subsequent publications on the Q system. Despite the fact that all the case studies presented by Sheorey had operating lives of more than five years, long-term excavations for coalmining, the permanent support pressures estimated using Equation 4.10 were found to be very high and were clearly impracticable from the point of view of mining. Hence, Sheorey estimated temporary support pressure by using 5*Q* (recommended in the system for temporary support) in place of *Q* in Equation 4.10 as given below with the expectation of obtaining more viable support pressures.

$$P_{roof} = (2/Jr)\ Q^{-1/3} \tag{4.10}$$

$$P_{roof} = (2/Jr)\ 5Q^{-1/3} \tag{4.10a}$$

Sheorey also determined the safe unsupported spans for roadways using Equation 4.14, but for roadway intersections, which are wider, he used an amended version (using *Q*/3 instead of *Q*) as given in Equation 4.14a.

$$B = 2\ ESR\ Q^{0.4} \tag{4.14}$$

$$B_j = 2\ ESR\ (Q/3)^{0.4} \tag{4.14a}$$

where, junction span $B_j$ is the distance between two adjacent corners in an intersection. The main findings of the Sheorey's study relevant to the Q system were:

- The support pressures estimated using the equation recommended by the Q system for permanent excavations were very high and were clearly impracticable from the point of view of mining.

- The modified Q Equation 4.14a also overestimated support pressures for 43 cases out of 44, sometimes excessively.

- When *ESR* was taken as 3 to 5 as recommended in the Q system for temporary mine openings, Equation 4.14 and the modified version (Equation 4.14a) predicted unrealistic unsupported spans for roadways.

- When *ESR* was taken as 1.6 for the 44 roadway cases, seven “unstable cases” were predicted as “stable”. In the case of junctions, three unstable cases were predicted as stable while five stable cases were predicted as unstable.

- The Q system is inadequate for certain geological features not covered by it and also when joint orientation is unfavourable.

Based on the results of the study Sheorey recommended the reduction of the rating given to *Jn* by one third for horizontally stratified rocks (i.e. change *Jn* to $Jn^{2/3}$) when computing the *Q* value and reduce (divide) the *Q* value by a factor of 3 to 5 depending on the rock mass conditions in coal measures before using it for predicting stable unsupported spans. Sheorey also made the following observation with respect to the RMR system:

- Support pressures were underestimated in poorer rock masses. This happened for two reasons: firstly, this system does not consider stress; and secondly, the load factor in support pressure equation of RMR system has a maximum value of 1.0.

In considering these two points, Sheorey attempted to include the influence of stress in the RMR system and to alter Equation 4.5 for better estimation of support pressures.

Sheorey presented a plot of RMR and Q values obtained from the 44 case studies representing coal mine roadways. Since the data points were widely scattered, he did not perform a regression analysis of the data to obtain a correlation between the two systems. As part of the present research, a regression analysis of the data presented by Sheorey (1993) was undertaken and the following equation was obtained:

$$RMR = 6.8 \ ln \ Q + 42 \tag{5.13}$$

### 5.2.16 Review and Application to Bore Core Data from the UK

Rawlings et al. (1995) reviewed the RMR and Q classification methods and applied the two methods to a geological formation comprising volcanic rocks. Rock mass data were obtained by logging approximately 1700 m of rock core recovered from three bore holes. The following conclusions were drawn:

- The RMR method has the advantage that its six parameters are relatively easy to estimate and the RMR value is formed by simply adding the ratings for the various parameters. However, this limits the range of materials over which the system can be applied.

- The RMR system considers *RQD* and joint spacing separately, both of which are measures of block size which is therefore overemphasised (receiving up to 50% of the total rating) at the expense of other parameters which may have greater influence on the engineering properties. Bieniawski (1989) recommends that where less than three joint sets are present, the joint spacing rating should be increased by 30% which puts even more emphasis on block size.

- The output of the RMR system tends to be rather conservative which can lead to over-design of support.

- In comparison, very detailed treatment of joint roughness and alteration are the strongest features of the Q system. In addition, the numeric *Q* value range, 0.001 to 1000, encompasses the whole spectrum of rock mass qualities from heavily squeezing ground up to sound unjointed rock.

- Of the six parameters in the Q system, *Ja* and *SRF* are probably the two most subjective. A correct assessment of *Ja* requires proper training in the use of the system and the *SRF* rating in rock affected by high stress is difficult to estimate by visual observations.

Rawlings et al. analysed *RMR* and *Q* values assigned to the bore core data to find a correlation between the two systems. Two sets of *Q* values were considered: the first assumed *SRF*=1 and the second used the *SRF* values recommended in the Q system. By correlating the two sets of *Q* values with the relevant *RMR* values, Rawlings et al. first obtained the following two correlations:

$$RMR = 6.5\ ln\ Q + 48.6 \text{ (with } SRF = 1) \quad (5.14)$$

$$RMR = 6.1\ ln\ Q + 53.4 \text{ (with relevant } SRF \text{ values)} \quad (5.15)$$

They observed that the addition of the relevant *SRF* values in the *Q* indices mostly affected the lower end of the *Q* scale leading to many more values of $Q<1$. They also observed the poorest fit to the linear relationship occurs when $Q<1$. In order to obtain a better fit for the correlations between *RMR* and *Q* values, bilinear relationships were applied. The bilinear relationship for the *RMR* and un-factored (meaning *SRF*=1) *Q* values were:

$$RMR = 10.3\ ln\ Q + 49.3 \text{ (when } Q \leq 1) \quad (5.16)$$

$$RMR = 6.2\ ln\ Q + 49.2 \text{ (when } Q > 1) \quad (5.17)$$

The bilinear relationship for the *RMR* and regular *Q* values obtained using the relevant *SRF* values were:

$$RMR = 6.6\ ln\ Q + 53.0 \text{ (when } Q \leq 0.65) \quad (5.18)$$

$$RMR = 5.7\ ln\ Q + 54.1 \text{ (when } Q > 0.85) \quad (5.19)$$

Rawlings et al. found that the bilinear relationship fitted well for the *RMR* and un-factored *Q* values. However, no such commentary was provided on the bilinear relationship derived for the *RMR* and regular *Q* values. Although Rawlings et al. (1995) did not provide a plot of the data used in the analysis, the possibility of obtaining two different formulas from each of the two data sets indicates that data may be widely scattered.

### 5.2.17 Experience from Israel

Polishook and Flexer (1998) assessed the applicability of the RMR and Q systems to chalk rock masses in excavations in Israel. They examined the chalk rock in seven unsupported excavations and from each site four to six measurements were taken by applying the two systems. The rock mass is characterised by non-continuous joints and bedding planes. The excavation spans varied from 3 to 10 m and their age varied from 10 to 1500 years. The purposes of the excavations were wide ranging and included contemporary infrastructure such as a railway tunnel, an access gallery and petroleum storage caverns etc and ancient tunnel complexes for residence, ritual, shelter and burial etc. The conclusions drawn from the study were:

- As a result of the presence of non-continuous joints and bedding planes, RMR and Q classification methods are too conservative and therefore unsuitable for chalk rock mass.

- The RMR and Q methods indicate that the tunnels studied needed support, yet they can in fact stand up independently (some of them for nearly 1500 years) with minimal or no support.

- Tunnels are unsupported and there were no signs of collapse.

### 5.2.18 Sydney Experience

Asche and Quigley (1999) present the results of a review of the application of the Q system to the New Southern Railway (NSR) tunnels in Sydney. The review was undertaken as part of a study undertaken to formalise a design approach for another project in the Sydney region. The NSR involved the construction of 11 m wide 10 km long tunnel, but only about 2.3 km of which was driven in rock and the review covered approximately 1.1 km representing Hawkesbury sandstone. Typically this rock has two joint sets and occasionally up to three plus random joints.

Asche and Quigley examined the support predicted based on the *Q* ratings obtained from 12 preconstruction boreholes with the installed support and observed that,

- There were noticeable differences between the Q ratings carried out by various parties using the bore core information.

- Support predicted using these ratings varied from random bolting to pattern bolting and fibre reinforced shotcrete. However, the installed support was limited to pattern bolting to pattern bolting and mesh.

- The differences in *Q* ratings and therefore the support predictions can be linked to those geological parameters that are not directly derived from bore core observation. Different assumptions were made for each set of *Q* values calculations.

- In Hawkesbury sandstone which is known to have two joints sets, there are areas where the spacing of one of the sets is so wide that it should not be taken into consideration for Q calculations. It is however very difficult to assess these variations from bore cores, especially if the joints and the boreholes are vertical.

- To improve on the accuracy of design parameter values, it is necessary to understand the geological characteristics of the rock mass before commencing the Q ratings. Where the geology varies locally, there will be a lower bound and upper bound Q rating resulting in a range of predicted support for each location.

Subsequent to the tunnel construction, Asche and Quigley, with the benefit of the observations made during excavation, reassigned three *Q* values for the 12 boreholes. For each borehole minimum, maximum and median *Q* values were computed and support requirements were determined for each of the three cases. From this Asche and Quigley concluded that support deduced using *Q* median value best reflects the support installed in the tunnel. However, the graph showing the predicted and installed support classes presented by Asche and Quigley (1999) shows that only seven out of the 12 cases of *Q* median support predictions agreed with the installed support. The correlation of both $Q$ maximum and minimum support predictions with the installed support was poorer than that of the Q median value.

In their study Asche and Quigley also compared the support predicted using the tunnel mapping data with the installed support. Minimum and maximum $Q$ values were calculated for each geological unit from the geological maps and support requirements were determined for both cases. Generally, compared to the support predicted based on borehole data, a good correlation existed between the installed support and that predicted using tunnel mapping data. In some areas of the tunnel length studied, however, the installed support was heavier than the Q predicted support, whereas in other areas it was less.

### 5.2.19 Experience from Italy and Spain

Gonzalez de Vallejo (2002) applied RMR and SRC (see Section 4.3.1) systems to 25 tunnels in Spain and Northern Italy, and evaluated the two systems in terms of their suitability for tunnels in weak rocks affected by high horizontal tectonic stresses. The evaluation was undertaken by comparing support estimated by RMR and SRC methods with those actually installed. The Q system was also applied occasionally and only partial results were obtained for this index.

Out of the 25 tunnels, 20 are high speed railway tunnels and the remaining five are hydroelectric power tunnels. These tunnels have cross sections up to 120 m$^2$ in highly variable conditions both in geological and construction aspects. However, they also have some common features such as predominance of low strength rocks (shales, schists and argillites etc), significant folding and deformation structures (folds, faults, thrusts etc) and overburden thicknesses under 700 m. In 22 tunnels the main rocks were low strength shales, schists and argilites with typical UCS values of 10 to 15 MPa and highly anisotropic behaviour. The other three tunnels intersected meta-basalt and gneiss. In the 25 tunnels, the state of stresses was evaluated by considering the tectonic history, the presence of deformation structures and current tectonic regime, measurements of in situ stress in the regions where the tunnels were excavated and instability problems occurred during tunnel excavation and their relation to tectonic structure. Based on this information, the in situ stress fields around the tunnels were classified into four classes using the horizontal to vertical stress ratio $k$ ($=\sigma_h/\sigma_v$). The four are Low ($k \leq 0.5$), Moderate ($0.5 < k \leq 1.0$), High ($1.0 < k \leq 2.0$), and Very high ($k > 2.0$). The relevant information showed that the stress fields around six of the tunnels were very high to high, around 13 of the tunnels were high and around the remaining six were low to moderate. In this study the RMR and SRC indices were applied to the 25 tunnels and support requirements were determined during the project design stage as well as during construction. In the latter case RMR and SRC data were collected from several excavation fronts in each tunnel. Additionally, tunnel section convergence measurements, problems related to ground instability and the actual support installed were also recorded. Q values were also determined for four of the tunnels. In order to analyse the data collected the tunnels were divided to three groups based on in situ stress levels:

- Group I: tunnels located in zones of high horizontal tectonic stresses with low overburden thicknesses (generally less than 150 m).

- Group II: tunnels located in zones of high horizontal tectonic stresses with high overburden thicknesses (higher than 150 m, mostly more than 250 m).

- Group III: tunnels located in zones of low to moderate horizontal stresses irrespective overburden thicknesses.

For the purpose of comparing the installed support with the RMR and SRC predicted support, the former was also classified into one of the five RMR classes. While the RMR and SRC were applied to all 25 tunnels, the Q system was applied only to a sample of Group II and III tunnels. The result of the analysis showed that

- For Group I tunnels the actual support installed significantly exceeded those predicted by the RMR system. In 64% of the cases examined, the difference between the predicted and installed was two RMR classes and for the remaining 36% the difference was one RMR class indicating in 100% of the cases, installed support exceeded the RMR predictions.

- In Group I tunnels the installed support also exceeded the SRC predictions, but the difference was less compared to the RMR predictions. In 9% of the cases examined, the difference between the predicted and installed was two RMR classes and for the remaining 55% the difference was one RMR class which means in 64% of the cases, the installed support exceeded the SRC predictions.

- The predictions for Group II tunnels showed the same tendency but the difference was less. In 22% of the cases, the difference between the RMR predicted and installed was two RMR classes and for 67% the difference was one RMR class which means in 89% the installed support exceeded the RMR predictions. In the remaining 11% the predicted and installed were comparable.

- For Group II tunnels, in 78% of the cases examined, the installed support and the SRC predictions were comparable but in the remaining 22%, the difference between the predicted and installed was two RMR classes.

- In Group III tunnels the installed support and those predicted by RMR and SRC were comparable in 100% of the cases examined.

- For Groups I and II, the ratio of the mean of the installed support classes to the mean of RMR predicted support classes was 0.5, and the corresponding ratio for SRC predictions was 0.8.

- In Group II tunnels the difference between the installed and the Q predicted support was one Q support class, i.e. predicted poor class, but actual was very poor class. In Group III tunnels the actual support installed was same as the Q predicted support.

In his study Gonzalez de Vallejo also attempted to establish correlations between the tunnel convergence or deformation and the RMR and SRC indices. He observed that in general, neither index could adequately predict convergence nor establish a reliable correlation between rock classification indices and deformation. This is interpreted to be due to (a) high horizontal stresses, (b) low intact rock strength, (c) thin overburden and (d) unfavourable structural anisotropy with respect to tunnel axis. The other possible contributing factors which are not accounted for in the classification systems are shape and size of tunnel sections, the excavation methods and type of support installed. In tunnels with low overburden the effect of structural anisotropy caused by the presence of bedding planes and schistocity was marked, while this effect was much reduced when overburden is high. The orientation of schistocity is also a governing factor. For instance, for a tunnel with low overburden and schistocity parallel to its axis the correlation between convergence and RMR and SRC classification indices was poor, but for a tunnel with high overburden and schistocity perpendicular to its axis the correlation was improved.

### 5.2.20 General Review Comments by Stille and Palmstrom

Stille and Palmstom (2003) reviewed the role of rock mass classification systems in rock engineering and design. They considered four classification systems, GSi, RMi, RMR and Q, as well as the well known observational method, NATM. The conclusions arising from their review were:

- The existing quantitative rock mass classification systems can be applied as a useful tool to establish a preliminary design. At least two systems should be applied. They are not recommended for use in detailed and final design, especially for complex underground openings.

- Classification systems are unreliable for rock support determination during construction, as local geometric and geological features may override the rock mass quality defined by the classification system.

- RMR cannot be used as the only indicator, especially when rock stresses or time dependent rock properties are of importance for the rock engineering issue.

- The accuracy of the estimation of rock support using the Q system is very difficult to evaluate. Especially in the poorer rock class ($Q$<1), the system may give erroneous design. The true nature of the rock mass (i.e. popping squeezing, swelling, etc) that is essential for the determination of the support measures is not explicitly considered in the Q system.

Regarding the Q system, the two authors also commented that in fractured ground the orientation of joint is an important parameter. In such cases it is very important to follow the guidelines given by Barton et al. (1974) that the parameters *Jr* and *Ja* should be related to joint surfaces most likely to allow failure to initiate. From the rock mechanics point of view, it is obvious that even such a simple case as block instability is much more complicated than can be given by a single number like a *Q* value.

### 5.2.21 Experience from the Higher Himalayas

Kumar et al. (2004) applied RMR and Q classification methods, along with RSR and RMi methods, to 22 km of a 27.4 km long tunnel driven in the Higher Himalayas, India. This tunnel encountered many challenges related to geothermic heavy inflows of groundwater, excessive overburden, flowing, slabbing and squeezing ground conditions. The tunnel was driven through metamorphic rock formations comprising schists, gneisses and amphibolites. In all 685 tunnel sections, inclusive of 50 sections in squeezing ground conditions and 69 in shear zones, were studied. The total length of tunnel facing squeezing ground conditions is more than 1 km with individual lengths varying from 3 to 63 m. Shear zones have a total length of 1 km with individual lengths varying from 2 to 63 m. The tunnel passes under a rock cover of more than 1000 m with a maximum of 1430 m over a distance of 800 m. In addition to RMR, Q, RSR and RMi, Kumar et al. also applied RCR (*RMR* without *IRS* and *RA* parameters) and N (*Q* with *SRF*=1). The *RCR* and *N* are as defined by Goel et al. (1995). Kumar et al. also attempted to develop correlations between these methods and assessed the *SRF* values recommended in the Q system for stress related problems in moderately jointed rocks.

In the Q system, for "Competent rock, rock stress problems" (Table 4.11 Section b) *SRF* values are recommended on the basis of $\sigma_c/\sigma_1$ and $\sigma_\theta/\sigma_c$ for six different conditions denoted as H, J, K, L, M and N (where of $\sigma_c$, $\sigma_1$ and $\sigma_\theta$ are intact rock strength, major principal stress and maximum tangential stress, respectively). The recommended *SRF* values for conditions H, J and K (low, medium and high stresses, respectively) are understood to be applicable for both massive as well as moderately jointed rock. On the other hand, in conditions L, M and N (moderate slabbing, slabbing and rock burst, and heavy rock burst, respectively) the recommended *SRF* values pertain to massive rocks only, which should really be the case since slabbing, rock burst etc are associated with competent massive rocks only. There are, however, situations in which ratios $\sigma_c/\sigma_1$ and $\sigma_\theta/\sigma_c$ lie in ranges corresponding to conditions L, M and N, but the rock is moderately jointed and not massive. For these situations Table 4.11 does not provide *SRF* values. If *SRF* values are selected for "rock stress problems" purely on the basis of $\sigma_c/\sigma_1$ and $\sigma_\theta/\sigma_c$ ratios, the results might be correct for massive rocks but are bound to be incorrect for jointed rock masses. In order to address this problem Kumar et al. attempted to estimate appropriate *SRF* values for moderately jointed rock experiencing high stresses, and proposed new guidelines for estimating *SRF*. New correlations have also been proposed for estimating support pressure using RMi system support recommendations and prediction of ground conditions based on joint roughness and alteration. The conclusions relevant to RMR and Q that may be drawn from the study were:

- For "rock stress problems" *Q* values determined by selecting *SRF* ratings purely on the basis of $\sigma_c/\sigma_1$ and $\sigma_\theta/\sigma_c$ ratios, the only option available to the user, might be erroneous.

- New *SRF* values have been proposed for "rock stress problems" in moderately jointed rocks. The proposed *SRF* values range from 1.5 to 3.0, which are significantly smaller than the range of values (5 to 400) given by Barton and Grimstad (1994) for slabbing and bursting in competent rocks.

- Correlations for predicting ground behaviour based on joint roughness (*Jr*) and alteration (*Ja*) of *Q* have been developed as given in Table 5.4.

Table 5.4 Correlations for predicting ground behaviour based on *Jr* and *Ja*

| Ground condition | Correlation |
|---|---|
| Moderate slabbing with noise | $Jr \geq 0.5Ja$ |
| Mild squeezing | $0.5Ja \geq Jr \geq 0.5Ja - 2$ |
| Moderate squeezing | $0.5Ja - 2 \geq Jr \geq 0.5Ja - 3$ |
| High squeezing | $Jr \leq 0.5Ja - 3$ |

Further, Kumar et al. (2004) presented three *RMR-Q* relationships for moderately jointed rocks facing "rock stress problems". The first (Equation 5.20) assumed that *SRF*=1, the second (Equation 5.21) used the revised *SRF* values, and the third (Equation 5.22) used the *SRF* values recommended in the Q system.

$$RMR = 4.7 \, ln \, Q + 56.8 \tag{5.20}$$

$$RMR = 8.3 \, ln \, Q + 42.5 \text{ (with } SRF = 1\text{)} \tag{5.21}$$

$$RMR = 6.4 \, ln \, Q + 49.6 \text{ (with revised } SRF \text{ values)} \tag{5.22}$$

Note that none of the three *RMR-Q* relationships presented by Kumar et al. (2004) is identical to Equation 5.1 proposed by Bieniawski (1976, 1989).

### 5.2.22 Critical Review Comments by Palmstrom and Broch

Palmstrom and Broch (2006) reviewed rock mass classification systems with particular reference to the Q system focussing on its structure and different input parameters. The relevance and limitations of the Q input parameters to different rock mass conditions were examined. They also examined in some detail the kinds of rock masses and ground conditions the Q system covers and the applicability of its support recommendations for different stages of a project. The main conclusions arising from the review were:

- Classification systems, and not least the Q system, may be useful tools for estimating the need for tunnel support at the planning stage, particularly for tunnels in hard and jointed rock masses without overstressing. There are, however, a number of restrictions that should be applied if and when the system is going to be used in other rock masses and in complicated ground conditions.

- Potential users of the Q system should carefully study the limitations of this system as well as other classification systems they may want to apply, before using them.

- For practical use the Q system works best for the approximate range $0.1<Q<40$, which represents Very Poor to Good Rock mass classes of the system. This range of *Q* values might be referred to as normal hard rock conditions.

- On the recently introduced $Q_{TBM}$ to estimate "penetration rate" for TBM, the review found that the Q input parameters are irrelevant or even misguiding for TBM performance, and the total effects are difficult to follow in the model.

### 5.2.23 Australian Experience

Pells and Bertuzzi (2008) compared the actual support installed in 14 major tunnels and caverns with those deduced from RMR and Q classification methods, and commented on the reliability of the two empirically methods for support design. Eleven of the projects are in Sydney, two are in Melbourne and one is in Brisbane. The data presented include (a) the support designed (or would have designed) using RMR or Q, (b) the actual support installed, and (c) the performance of the tunnels and caverns. The Sydney area is underlain by a sequence of near horizontal sandstone and shale. The projects presented by Pells and Bertuzzi are:

- Three ocean outfall tunnels: 4 m wide 3.5 km long at North Head; 3 m wide 1.3 km long at Bondi; and 4 m wide 4 km long at Malabar.

- The Sydney Opera House underground car park, doughnut-shaped in plan view with an 18 m span and 6 to 8 m rock cover.

- The M2 tollway: two 11.9 m wide 450 m long twin two-lane tunnels separated by a 5.5 m wide pillar, with typical cover of 16 m and 22 m. The predominant rock type is very good quality sandstone with some 1 to 1.5 m horizons of poor quality sandstone.

- The New Southern Railway: only about 2.3 km of this 11 m wide 10 km long tunnel was driven in rock. The results of the application of the Q system to about 1.1 km is provided by Asche and Quigley (1999) and discussed earlier in Section 5.2.18 of the thesis.

- The Eastern Distributor tollway: a 50 m length of >20 m span tunnel with a total cover of about 30 m. The project comprised a 15 to 24 m span 2.4 km long three-lane double-decker tunnel and about 1 km of associated ramp tunnels with spans up to 24 m.

- The M5 Motorway: 8.6 m wide 4 km long twin two-lane tunnels excavated in Hawkesbury sandstone with an overburden of between 15 and 70 m. In the ramp bifurcation areas at the ends of the main tunnels spans increase up to 19 m. The tunnels were excavated by roadheaders. The average strength of sandstone intact rock was between 25 and 30 MPa. The predominant rock mass defects were gently undulating bedding planes, with the typical regional near vertical joints being quite sparse where depth of cover was greater than about 40 m. The major principal stress in the sandstone was near horizontal and oriented approximately NNE with a range of 4 to 7.5 MPa for overburden depths between 25 and 70 m.

- The North-side sewer storage tunnels, comprising 3.8 m wide 6.5 km long, 6 m wide 3.7 km long and 6.3 m wide 3.5 km long TBM driven tunnels. All tunnels were in sandstone with depth of cover ranging from approximately 20 to 80 m.

- Three parallel gas storage caverns, 14 m wide 11 m high and 120 m long with a depth of about 120 m beneath Botany Bay.

- The Epping-Chatswood rail link, 7 m wide 14 km long twin tunnels and four train stations with spans up to 20 m.

- Two three-lane road tunnels in the Melbourne City Link project. The 1.6 km long Domain Tunnel driven at a maximum depth of about 25 m beneath a parkland hill and a cut and cover length (not included in the study) beneath the Yarra River. The 3.5 km long Burnley Tunnel driven at a depth of 60 m beneath the Yarra River. Two tunnels were excavated primarily in Melbourne mudstone. The tunnels were driven using full face top headings excavated by a 100 tonne roadheader, followed by removal of 4 to 5 m high bench using a combination of roadheader, impact breaker and a small amount of blasting.

- Approximately 3.6 km of a 3 m diameter S1 main sever tunnel in Brisbane driven through hard rocks comprising metasediments and occasional meta-basalts.

The conclusions drawn by Pells and Bertuzzi from the Sydney projects were:

- For the North Head and Malabar ocean outfall tunnels, excavated by drill and blast methods and road header respectively, the RMR system was conservative in terms of support requirements, particularly for machine excavated tunnels. The Q system provided a reasonable prediction for machine excavation but was non-conservative where drill and blast was concerned.

- In the Sydney Opera House car park cavern, the support measures installed were significantly more than those predicted by the RMR and Q methods. Pells and Bertuzzi acknowledge that, in a way, this cavern is an unfair test of the classification systems because of its low rock cover. However, there is nothing in the relevant publications (particularly in the Q system) suggesting it should not be applied to such structures.

- In the M2 tollway tunnels, the RMR predicted support measures were comparable to those installed, whereas Q predictions could have been undesirably non-conservative.

- In the Eastern Distributor tunnel, a 50 m length with a span of >20 m was studied. Since the RMR support recommendations are for 10 m diameter tunnels, they are not considered applicable to this tunnel. However, it was obvious that the adopted support design is substantially greater than that deduced from the Q system.

- For Botany Bay gas storage caverns with spans of about 14 m the RMR support recommendations may not be considered applicable. During construction two roof collapses occurred when the support installed was in accordance with the Q system. The actual support installed subsequently was of a substantially higher capacity than the Q recommendations.

- In the North-side sewer storage project TBM driven tunnels, the initial primary support, comprising rock bolts and mesh, was designed using the Q system. The actual density of rock bolting (bolts per metre) which proved necessary to install following inadequate performance of the initial design ranged between 5 and 9 times the initial design densities (Pells, 2002, Pells and Bertuzzi, 2008).

- In the M5 Motorway tunnels, the actual support installed were substantially more than those deduced using the Q system.

- In the New Southern railway tunnel, the Q predicted support only occasionally matched the actual support installed, as discussed earlier under Asche and Quigley (1999).

- In the Epping-Chatswood rail link tunnels there is no correlation between the Q value (or the recommended support) and the installed support.

- The primary support used in the two Melbourne tunnels comprised rock bolts, shotcrete, mesh and steel sets. Other than for the very poor quality rock (fault zones and weathered igneous intrusions) there is no correlation between the Q value (or the recommended support) and the installed support.

- In the Brisbane S1 main sewer tunnel, apart from the very poor and extremely poor rock classes, much greater support was installed than that recommended by Q. According to the Q system the bulk (~70%) of the tunnel would require no support, but during excavation some of these areas required spot bolting and the remaining areas required systematic bolting and occasional wire mesh.

The main conclusions drawn from the comparison based on the factual data from the 14 projects were:

- The design correlations published in the various papers on the Q and RMR systems should be used with great caution in geological environments significantly different from those comprising the original case studies.

- Cognisance must be taken of the fact that use of the general classification design approach is contrary to normal engineering design process. It is not a proper application of the scientific method. There are no applied mechanics calculations of stress or displacement, no computations, or information, as to loads, strains and stresses in the support elements (shotcrete, rock bolts and sets), and therefore nothing against which to compare field monitoring data. The position of the classification design approach in relation to modern limit state design is unknown and unknowable. It covers neither ultimate no serviceable limit states.

Pells and Bertuzzi (2008) further stated that "*Classification systems are good for communication and in many cases good for producing correlations in particular geological environments. However, … … they should not be used as the primary tool for the design of primary support*".

### 5.2.24 Experience from TBM Driven Tunnels

Regarding TBM (tunnel boring machine) driven tunnels, Barton (2000a, 2000b, 2001 and 2002) implied that, for a wide range of rock properties, less support would be required and suggested adjustments to the Q support chart of Barton and Grimstad (1994) to enable its use for TBM tunnels (Figure 5.2). He maintains that a reduced level of disturbance is caused by TBM compared to drill and blast. Accordingly the "no-support" boundary in the Q support chart would move to the left. This may be the case if Q values were determined from borehole data obtained during investigation. On the other hand, Barton maintains that by logging the as-excavated tunnel walls a higher Q value may be obtained due to the way that the TBM wall appears to the logger and that the tendency for TBM is to be gentler on the rock mass resulting in less support requirements. This moves the "no-support" boundary in the Q support chart to the left.

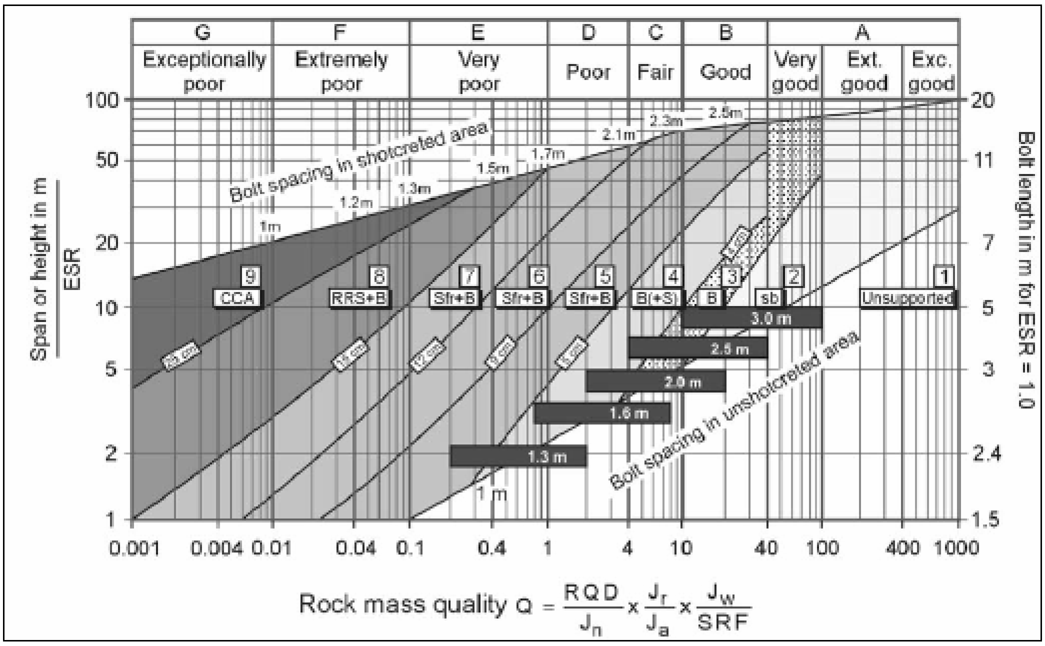


Figure 5.2 Modified Q support chart for TBM tunnels (Barton, 2000)

Asche and Cooper (2002) presented a discussion on the use of empirical methods to estimate support requirements for TBM driven tunnels, with specific reference to the Q system. They opined that the currently used empirical methods of tunnel support design such as Q were originally based on databases comprising drill and blast excavated tunnels and may not necessarily be applicable to TBM driven tunnels, without due consideration of the conditions specific to the latter. Their discussion focused on the designs based on Q values determined from bore core data obtained during investigation, for which Barton made adjustments to the support chart by shifting the "no-support" limit to the left (Figure 5.2).

In their study, Asche and Cooper highlighted several points specific to TBM tunnels:

- TBM cuts an exact circular profile without taking any account of pre-existing planes of weakness. In contrast, drill and blast or roadheader excavations will automatically cut down or knock down loose blocks.

- TBM tunnelling is very rapid and can lead to time dependent effects such that failure can occur behind the face and mucking system. This rapidity also has an effect on the decision making process for selecting support, as the review of rock mass condition is often significantly delayed.

- Support installation is often delayed due to limited access.

- Type of support that can be used is limited, i.e. shotcrete is not popular immediately behind the face because of dust generation. Further, it requires diligent cleanup and is susceptible to damage by TBM gripper pads. Similarly, placement of bolts is constrained in location and direction by conveyor belt etc.

- TBM excavation is less damaging.

Despite the fact that TBM excavation is less damaging, Asche and Cooper pointed out that a combination of the above factors lead to:

- In blocky grounds, small blocks often require bolting and meshing, which would not even exist in drill and blast or roadheader driven tunnels. These blocks are often insignificant from a gross tunnel stability point of view, but they cannot be handled in any other way.

o The speed of TBM tunnelling, the limited access for viewing the rock at the face and the organisational aspects often lead to heavier support installation than actually necessary.

o In some cases, movement on joints can happen behind the face due to the speed of excavation.

Asche and Cooper (2002) compared the Q predicted support and installed support in the North Side Storage Tunnel in Sydney. This project involved the construction more than 16 km of tunnels. The support design was based in part on the Q system and included rock bolts, mesh and steel sets as listed in Table 5.5. They compared the support predicted using Q values obtained by logging the tunnel and the actual support installed and the results are presented in Figure 5.3.

Table 5.5 NSTP Support types

| Support type | Designed Q range | Support |
|---|---|---|
| ST1 | >5 | Random bolts |
| ST2 | 3-5 | 2 bolts + mesh |
| ST3 | 2-3 | 4 bolts + mesh |
| ST4 | 1-2 | 6 bolts + mesh |
| ST5 | 0.2-1 | 8 bolts + mesh |
| ST6 | <0.2 | Steel sets |

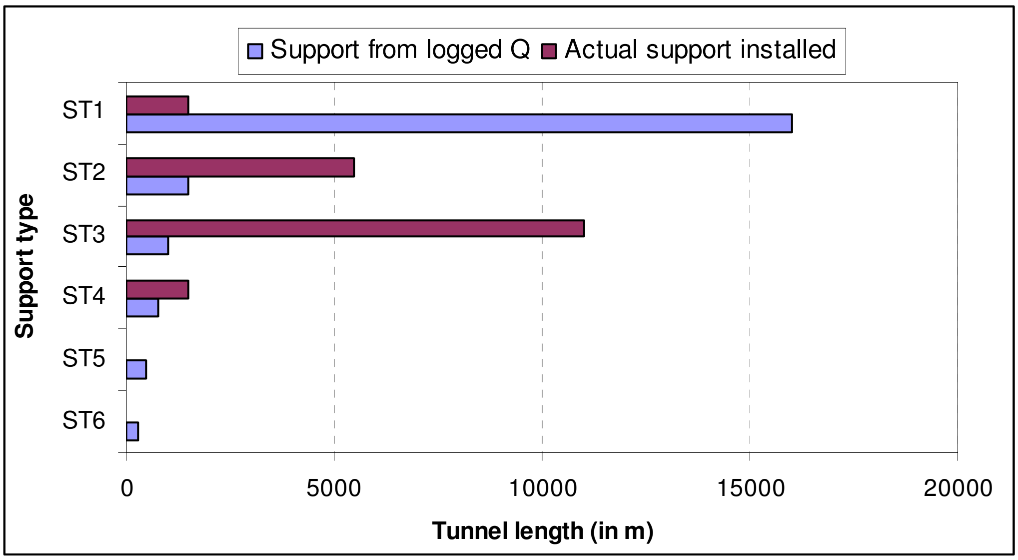


Figure 5.3 Q predicted and installed support (after Asche and Cooper, 2002)

Referring to Figure 5.3 Asche and Cooper commented that the tunnel would appear to have been significantly over-supported according to the un-modified support chart in Barton and Grimstad (1994), and even more so if the black highlighted bars shown in Figure 5.2 are taken into account. However, in reality this was not so, and all of the effects described earlier were considered to have caused this result. The effect of jointing (blocky rock) and stress concentration were evident throughout the tunnel.

Asche and Cooper proposed a new support type for the instability in TBM tunnels driven in jointed rock and suggested that this be included in the "no-support" zone of the Q support chart. Further, they recommended that "no-support" boundary be shifted to the right (Figure 5.4) instead of shifting it to the left as proposed by Barton (2000a, 200b, 2001 and 2002). The conclusions drawn from their study were:

- For estimating support quantities for TBM tunnels, the "no-support" line suggested by them should be used (Figure 5.4).

- The theory that TBM does less damage to the rock mass than drill and blast tunnels is worth pursuing.

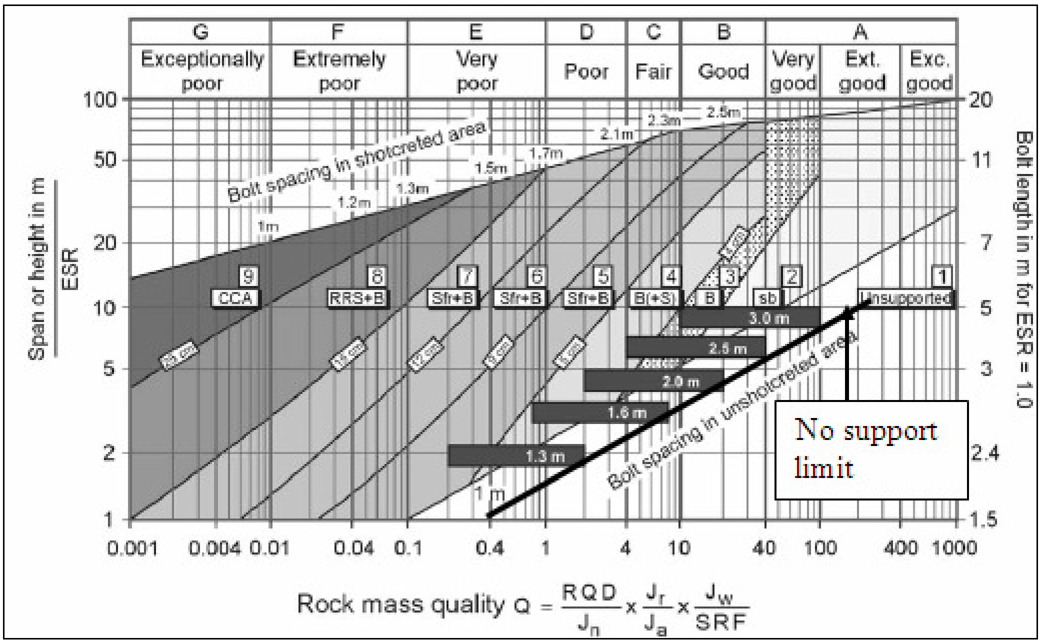


Figure 5.4 Q support chart for TBM tunnels (Asche and Cooper, 2002)

## 5.3 The Reliability of RMR and Q Correlations

It has been noted mention in Section 5.1 that some of the previous studies had attempted to correlate the RMR and Q indices with the intention of linking the two rating systems. They contemplate that the primary aim of the two systems is to divide the rock mass into distinct classes of similar characteristics, with the common objective of predicting rock mass behaviour and support requirements for excavations. It is, therefore, reasonable to expect a correlation between the ratings assigned to a rock mass by the two methods.

In the previous sections of this chapter some of the published correlations between the RMR and Q have been already discussed. These were presented by Bieniawski (1976), Rutledge and Preston (1978), Cameron-Clarke and Budavari (1981), Moreno Tallon (1982), Baczynski (1983), Udd and Wang (1985), Kaiser et al. (1986), Choquet and Charette (1988), Rawlings et al. (1995) and Kumar et al. (2004). For the present study, a correlation was obtained from the data presented by Sheorey (1993) and discussed in Section 5.2.15. Apart from these correlations, other researchers have also established *RMR-Q* correlations including the work of Abad et al. (1983), Celada Thamames (1983), Al-Harthi (1993), Asgari (2001), Sunwoo and Hwang (2001) and Sari and Pasamehmetoghu (2004). The reliability of these is discussed below.

### 5.3.1 Background of the first RMR-Q correlation

As cited in Section 5.2, not long after the advent of the RMR and Q classification methods, the correlation given in Equation 5.1 was proposed by Bieniawski (1976) by linear regression analysis of 111 data sets obtained from Scandinavian, South African, North American, European and Australian case histories.

$$RMR = 9\ ln\ Q + 44 \tag{5.1}$$

By adding the Indian case histories compiled by Jethwa et al. (1982), Bieniawski (1989) supplemented the database used for Equation 5.1. Since the data used in deriving the correlation was widely scattered about the regression line, Bieniawski

(1976) also provided the 90% confidence limits (Equation 5.1a) within which 90% of the data used did fall and indicated the limitations of the relationship.

$$RMR = 9\ ln\ Q + 44 \pm 18 \qquad (5.1a)$$

When the correlation was first published by Bieniawski (1976), a plot of *RMR-Q* data pairs used was also presented as given in Figure 5.5, which shows that the range of values represented by the 90% confidence limits covers almost two RMR ground classes, and as a result Equation 5.1 was of little practical value. In subsequent publications (Bieniawski, 1979, 1989, 1993; Barton, 1995; Barton and Bieniawski, 2008), the 90% confidence limits were omitted when referring to the relationship. Consequently, some practitioners in the field of rock engineering assumed that this relationship is universally applicable for transforming the ratings assigned by one system to the ratings of the other. This assumption is erroneous and deserves scrutiny. A review of the previous studies on the reliability of the RMR and Q systems will therefore not be complete without discussing the validity of the correlations obtained by regression analysis of the *RMR* and *Q* values.

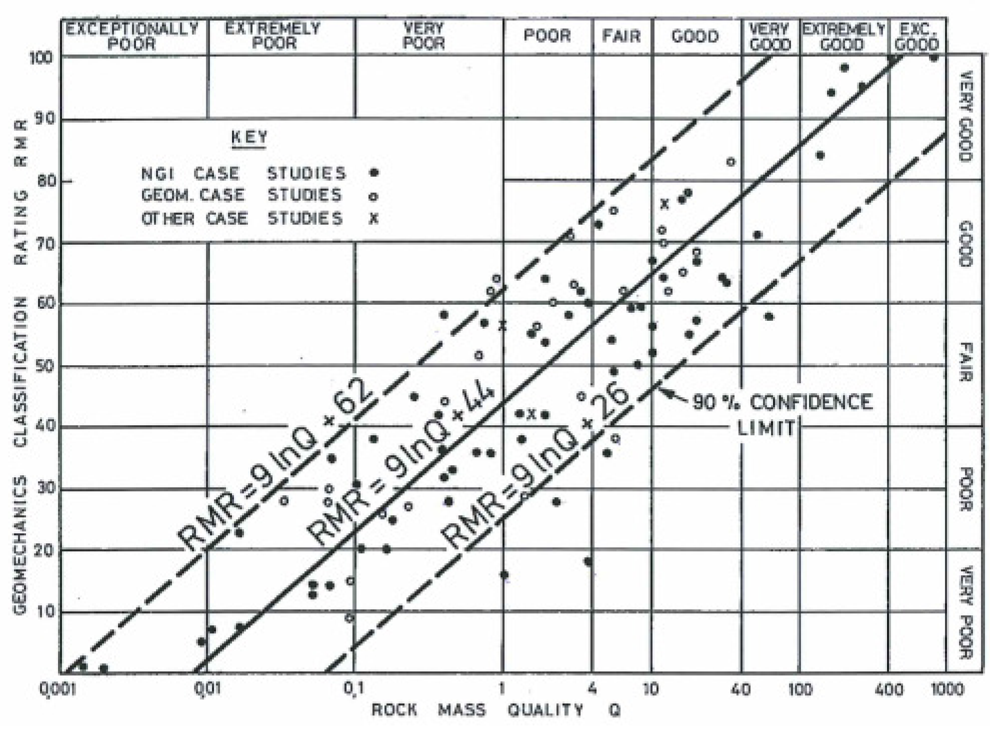


Figure 5.5 *RMR-Q* correlation with 90% confidence limits (Bieniawski, 1976)

Table 5.6 Correlations between RMR and Q

| Correlation | Source | Eq. No. |
|---|---|---|
| $RMR = 9\ ln\ Q + 44$ | Bieniawski (1976) | 5.1 |
| $RMR = 13.5\ log\ Q + 43 = 5.9\ ln\ Q + 43$ | Rutledge and Preston (1978) | 5.2 |
| $RMR = 5\ ln\ Q + 60.8$ (from in situ data) | Cameron-Clarke & Budavari (1981) | 5.3 |
| $RMR = 4.6\ ln\ Q + 55.5$ (from bore core data) | Cameron-Clarke & Budavari (1981) | 5.4 |
| $RMR = 12.5\ log\ Q + 55.2 = 5.4\ ln\ Q + 55.2$ | Moreno Tallon (1982) | 5.5 |
| $RMR = 7.5\ ln\ Q + 42$ | Baczynski (1983) | 5.6 |
| $RMR = 12.11\ log\ Q + 50.81 = 5.3\ ln\ Q + 50.81$ | Udd and Wang (1985) | 5.7 |
| $RMR = 10\ ln\ Q + 39$ | Choquet & Charette (1988) | 5.8 |
| $RMR = 6.3\ ln\ Q + 41.6$ | Kaiser et al. (1986) | 5.9 |
| $RMR = 8.7\ ln\ Q + 38 \pm 18$ (probability theory)[a] | Kaiser et al. (1986) | 5.10 |
| $RMR = 6.8\ ln\ Q + 42$[b] | Sheorey (1993) | 5.13 |
| $RMR = 10.3\ ln\ Q + 49.3$ (when $Q \leq 1$, $SRF = 1$)[c] | Rawlings et al. (1995) | 5.16 |
| $RMR = 6.2\ ln\ Q + 49.2$ (when $Q > 1$, $SRF = 1$)[c] | Rawlings et al. (1995) | 5.17 |
| $RMR = 6.6\ ln\ Q + 53$ (when $Q \leq 0.65$)[c] | Rawlings et al. (1995) | 5.18 |
| $RMR = 5.7\ ln\ Q + 54.1$ (when $Q > 0.65$)[c] | Rawlings et al. (1995) | 5.19 |
| $RMR = 4.7\ ln\ Q + 56.8$ | Kumar et al. (2004) | 5.20 |
| $RMR = 8.3\ ln\ Q + 42.5$ (with $SRF = 1$) | Kumar et al. (2004) | 5.21 |
| $RMR = 6.4\ ln\ Q + 49.6$ (with revised $SRF$ values) | Kumar et al. (2004) | 5.22 |
| $RMR = 10.5\ ln\ Q + 41.8$ | Abad et al. (1983) | 5.23 |
| $RMR = 7\ ln\ Q + 36$ | Tugrul (1998) | 5.24 |
| $RMR = 5.97\ ln\ Q + 49.5$ | Sunwoo & Hwang (2001) | 5.25 |
| $RMR = 3.7\ ln\ Q + 53.1$ | Sari & Pasamehmetoglu (2004) | 5.29 |
| $RMR = 43.89 - 9.19\ ln\ Q$ | Celada Thamames (1983) | 5.30 |
| $RMR = 9\ ln\ Q + 49$ | Al-Harthi (1993) | 5.31 |
| $RMR = 4.2\ ln\ Q + 50.6$ | Asgari (2001) | 5.32 |

[a] assuming *RMR* and *ln Q* are normal variates and satisfy the central limit theory of probability; [b] derived from the data presented by Sheorey (1993); [c] from bore core data

The discussions presented in Section 5.2 show that in addition to Bieniawski (1976), several other researchers also found *RMR-Q* correlations, but they are not identical to that given in Equation 5.2. Table 5.4 presents a list of *RMR-Q* correlations including those discussed in Section 5.2. It is evident from Table 5.6 that there is no unique correlation between the two methods, and in fact, it is apparent that different *RMR-Q* relationships can be obtained for different ground conditions. Further, the relationships obtained by regression analysis of *RMR* and *Q* values are not reliable for practical use as the data used for deriving them are widely scattered around the regression line as discussed in the following section.

### 5.3.2 Data scattering

Information available from relevant publications shows wide scattering of the data used in deriving the equations listed in Table 5.6. The wide scattering for the first correlation (Equation 5.1) can be seen from Figure 5.6, which according to Bieniawski (1989), plots the data used in 1976 and Jethwa et al. (1982). According to the data in Figure 5.6, when the *Q* value is 1.1 (poor rock), the corresponding *RMR* value can range from <20 (very poor rock) to >61 (good rock), while Equation 5.1 transforms it to a *RMR* value of 45 (fair rock). This equation is not valid when $Q<0.008$ and $Q>500$. In other words, if $Q<0.008$, $RMR<0$ and if $Q>500$, $RMR>100$; such *RMR* values are undefined. It is worth mentioning here that, Palmstrom (2009) noted that, *"… this correlation is a very crude approximation, involving an inaccuracy of ±50% or more*".

Rutledge and Preston (1978) did not present a plot of *RMR* and *Q* values obtained from the New Zealand tunnels used in deriving Equation 5.2. However, the *RSR* and *Q* data plot presented in their paper shows wide scattering. Further, regarding the *RMR-Q* correlation the two authors stated that "*There is considerable scatter in the results*" and that, "*It was expected that there would have been a better correlation than that obtained and it is obvious that more development of the systems is required, especially in the weighting given to each parameter making up the classification*". This implies that data points are scattered about the regression line defined by Equation 5.2.

On the two relationships obtained using bore core data and in situ observations in South African tunnels, Cameron-Clarke and Budavari (1981) stated the following: "*The scatter of points about the regression lines is greater for the in situ values than for the bore core values. In both cases, however, it is probably too great to indicate any meaningful correlation between the two classification systems.*"

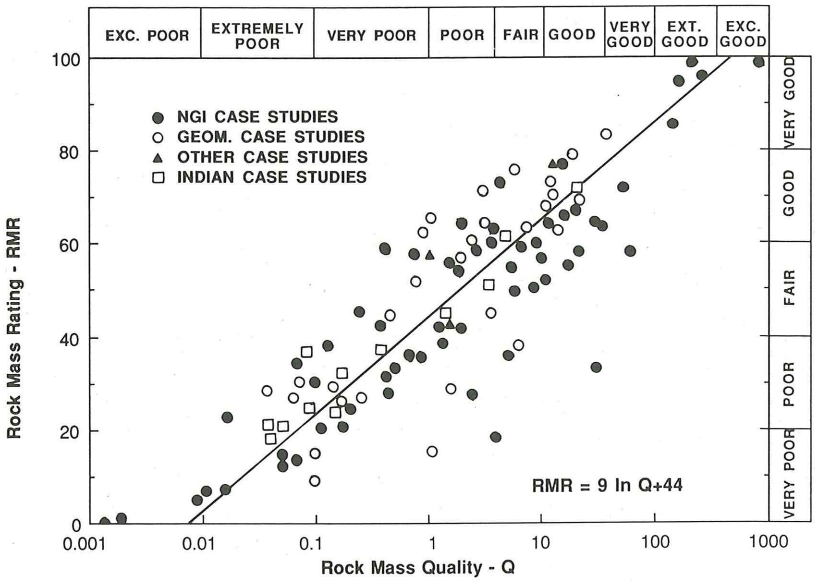


Figure 5.6 *RMR-Q* correlation (after Bieniawski, 1989)

The *RMR-Q* correlation (Equation 5.6) obtained by Baczynski (1983) assumed that *SRF*=2 in the *Q* values. Despite the fact that his correlation is in close agreement with that proposed by Bieniawski (1976), Baczynski stated that, "*However, it must be strongly emphasised that the correlations are stress dependent. The relationship will be significantly altered, if, for example, different SRF values are assumed in the determination of the Barton's Q rating. It is therefore important that any relationship for the transformation from one classification rating to another is not assumed to have universal application*".

The *RMR-Q* correlation given in Equation 5.7 obtained by Udd and Wang (1985) used 153 data pairs. As mentioned in Section 5.2, the *RMR-Q* data plot they presented shows wide scattering of the data about the regression line. Consequently, based on the plot the two authors commented that there is not always good agreement between the RMR and Q methods.

Kaiser et al. (1986) used two different approaches to obtain *RMR-Q* correlations. Firstly, they used the least square linear regression analysis as used by the creators of all the other *RMR-Q* correlations listed in Table 5.4, and obtained Equation 5.8. Secondly, they used a probabilistic approach assuming that *RMR* and *ln Q* are normal variates and satisfy the central limit theorem of probability theory, and obtained Equation 5.9. Despite the use of a probabilistic approach, Kaiser et al. observed wide scattering of the data and the correlation was presented as two equations representing 90% confidence limits within which 90% of the data used for their study fall. However, Kaiser et al. (1986) noted that the range of values represented by the two equations is of little practical value as it covers almost two RMR ground classes, as in the case of 90% confidence limits given by Bieniawski (1976).

Since the RMR and Q data obtained from the 44 case studies representing coalmine roadways were widely scattered, Sheorey (1993) did not perform a regression analysis to obtain a linear correlation. Nevertheless, for the present study, a regression analysis of this data was performed to obtain the correlation in Equation 5.13 (Section 5.2.15). Due to the wide scattering of data, this correlation is also of very limited practical value.

Rawlings et al. (1995) analysed *RMR* and *Q* values assigned to some 1700 m of bore core from a geological formation comprising volcanic rocks. Two sets of Q values were considered: the first assumed *SRF*=1 and the second used the *SRF* values recommended in the Q system. By correlating the two sets of *Q* values with the relevant *RMR* values, Rawlings et al. obtained three equations for each set of Q values as described in Section 5.2.16. Although Rawlings et al. did not provide a plot of the data used in the analysis the apparent need for obtaining bilinear correlations from each of the two data sets indicate wide scattering of the data used.

Kumar et al. (2004) from observations in major tunnelling projects in the Himalaya, India, found that the *SRF* values provided in the Q system are not applicable to overstressed moderately jointed rocks that are subject to rock slabbing and bursting, and proposed a revised set of *SRF* values for such rock stress problems. As mentioned in Section 5.2.21, the proposed range of *SRF* values is 1.5 to 3.0, which is significantly smaller than the range of values (5 to 400) given by Barton and Grimstad (1994) for slabbing and bursting in competent rocks. Kumar et al. (2004) then presented three *RMR-Q* relationships for moderately jointed rocks facing "rock stress problems". The first (Equation 5.20) assumed that *SRF*=1, the second (Equation 5.21) used the revised *SRF* values, and the third (Equation 5.22) used the *SRF* values recommended in the Q system. Note that none of the three *RMR-Q* relationships presented by Kumar et al. (2004) is identical to Equation 5.1 proposed by Bieniawski (1976, 1989).

In addition to the *RMR-Q* correlations presented in Section 5.2, several other researchers have also obtained correlations based on linear regression analysis of *RMR* and *Q* data gathered from various projects in different parts of the world. These are also of limited practical value as the data used in deriving them are also widely scattered.

Abad et al. (1983), in a study conducted on roof support design for coal mine roadways in Spain, applied amongst other tools, the RMR and Q classification methods. They correlated the *RMR* and *Q* values and obtained the following relationship with a correlation coefficient $r^2$ =0.93.

$$RMR = 10.53 \ln Q + 41.83 \tag{5.23}$$

Despite the high correlation coefficient, the *RMR-Q* plot provided by Abad et al. shows significant scattering of the data used.

Tugrul (1998) applied RMR, Q and RSR classification indices to two grouting galleries in Turkey and compared their support predictions. The galleries were driven through very closely to moderately jointed limestone with low *RQD* values and some karstic features. The tunnels are 4.1 m wide with a total length of 2113 m. By linear regression analysis of *RMR* and *Q* values Tugrul found a correlation in the form of

$$RMR = 7 \ln Q + 36 \tag{5.24}$$

In deriving the above correlation, Tugrul used only 35 data sets, yet the data are well represented by the regression line with relatively low scattering.

Sunwoo and Hwang (2001) applied RQD, RMR and Q systems to several tunnels in Korea and collected approximately 300 data sets from widely different geological environments representing igneous, metamorphic and sedimentary rocks. By linear regression analysis of the *RMR* and *Q* data they obtained four correlations, one for the combined data (Equation 5.25) and one each for igneous, metamorphic and sedimentary rocks (Equations 5.26, 5.27 and 5.28, respectively).

$$RMR = 5.97 \ ln\ Q + 49.5 \quad (5.25)$$

$$RMR = 5.69 \ ln\ Q + 47.0 \quad (5.26)$$

$$RMR = 6.04 \ ln\ Q + 49.6 \quad (5.27)$$

$$RMR = 6.07 \ ln\ Q + 50.1 \quad (5.28)$$

It is clear that the four correlations are similar. Since the combined data sample used is statistically significant and represents a wide range of rock mass conditions, Equation 5.25 may be considered to represent a reliable correlation between *RMR* and *Q*. Nevertheless, the *RMR-Q* data plot provided by Sunwoo and Hwang (2001) shows wide scattering about the linear regression line. For instance, according to the plot when the *Q* value is 1 (poor rock) the corresponding *RMR* value can vary from <30 (poor rock) to >61 (good rock). Similarly, when the *RMR* is 35 (poor rock), the corresponding *Q* value can range from <0.1 (extremely poor rock) to >10 (good rock).

Sari and Pasamehmetoglu (2004) correlated *RMR* and *Q* values collected during the preliminary design stage of a 2.5 km highway tunnel in Turkey. The main rock type along the proposed alignment is limestone olistolites, with occasional spilite and sandstones intrusions. The majority of the rock mass is moderately jointed with four discontinuity sets. Spilite and sandstone are characterised by faults. When constructed the tunnel will be 12.7 m wide and 9.6 m high and have a maximum overburden of 300 m. The rock mass parameters were collected from rock core samples recovered from bore holes. They found that the RMR system was less sensitive to the weak featured rock mass than the Q system which showed better performance in defining the weak rock mass present along the tunnel alignment. By linear regression analysis of the *RMR* and *Q* values, they obtained the following correlation:

$$RMR = 3.7 \ln Q + 53.1 \tag{5.29}$$

Equation 5.29 is different (its gradient is flatter) from the other equations listed in Table 5.4 and may be attributed to the type and condition of the rock mass present along the tunnel alignment.

<u>Celada Tamames (1983)</u> presented a discussion on 14 years of rock bolting experience, particularly from coalmine roadways, in Spain. Based on the experience acquired from rock bolting in coalmine roadways in carboniferous rock formations not affected by creep phenomena, he obtained a correlation between the *RMR* and *Q* values as given in Equation 5.30, which has a correlation coefficient of 0.94.

$$RMR = 43.89 - 9.19 \ln Q \tag{5.30}$$

The data used to obtain Equation 5.28 were not provided in Celada Tamames (1983). Nevertheless, he maintains that for coalmine road headings in carboniferous rock formations in northern Spain, *Q* values could be obtained from *RMR* values using this equation. He further maintains that the road headings produce an *RMR* index of 30. This indicates that the correlation given in Equation 5.30 is probably valid for *RMR* and *Q* values of around 30 and 4.5, respectively.

<u>Al-Harthi (1993)</u> applied RMR and Q classification systems to a road tunnel at Al-Dela Descent in the Kingdom of Saudi Arabia. The tunnel was to be driven through weak Precambrian slate and chlorite schist intercalated by meta-basaltic rock affected by three sets of structural discontinuities. The two methods were applied to the 180 m long tunnel during investigations using the data collected by surface mapping and bore core logging. Al-Harthi correlated the *RMR* and *Q* values

obtained from this study and three other tunnels in the area and obtained Equation 5.31, which is similar to that presented by Bieniawski (1976).

$$RMR = 9\ ln\ Q + 49 \tag{5.31}$$

The $RMR$-$Q$ data plot used for obtaining the above equation shows a relatively better correlation. However, the plot has only 16 data points with a $Q$ value range of between 0.1 and 5 and a $RMR$ range of 30 to 75. The small sample size and the narrow range of ratings reduce its statistical significance.

Asgari (2001) applied RMR and Q to Iranian geological conditions and used 47 data pairs to obtain the correlation given as Equation 5.32.

$$RMR = 4.2\ ln\ Q + 50.6 \tag{5.32}$$

Since his work was published in Persian and only an abstract is available in English the details of the study could not be reviewed.

### 5.3.3 Correlation of modified (or truncated) RMR and Q values

In an attempt to reduce data scattering and to obtain better $RMR$-$Q$ correlations, Sheorey (1993), Goel et al. (1996) and Kumar et al. (2004) used modified or truncated versions of the RMR and Q systems. Sheorey (1993) defined $RMR_{mod}$ as $RMR$ without ratings for $IRS$ and $JA$, and $Q_{mod}$ as $Q$ with $SRF$=1, and found a reduction in the data scattering compared to the original $RMR$ and $Q$ values. Goel et al. (1995, 1996) also noted that RMR and Q are not truly equivalent, and defined $RCR$ (rock condition rating) as $RMR$ without $IRS$ and $JA$, and $N$ as $Q$ with $SRF$=1, which are essentially the same as $RMR_{mod}$ and $Q_{mod}$ defined by Sheorey (1993). Kumar et al. (2004) also used $RCR$ and $N$ in their study. By regression analysis of the truncated versions of the two methods, Sheorey (1993), Goel et al. (1996) and Kumar et al. (2004) obtained the relationships given by Equations 33, 34 and 35, respectively.

$$RCR = 9.5\ ln\ N + 31 = 21.8\ log\ N + 31 \tag{5.33}$$

$$RCR = 8.0 \ln N + 30 \tag{5.34}$$

$$RCR = 8.0 \ln N + 42.7 \tag{5.35}$$

Goel et al. (1996) hold the view that Equation 5.34, which has a correlation coefficient of 0.92, provides a reliable correlation between the two systems and can be used for obtaining *RMR* from *Q* and vice-versa. Based on their study, the following conclusions were drawn regarding the correlations between *RMR* and *Q*:

- Available correlations between *Q* and *RMR* have high scatter because the two systems are not truly equivalent.

- Equation 5.34 should be used for determining the interrelationship between *N* and *RCR* before interchanging *Q* and *RMR*.

- For mine roadways through coal measures, the correlation between *RCR* and *N* proposed by Sheorey (1993) should be used.

It should be noted, however, the data plot presented by Goel et al. shows scattering of data about the regression line given by Equation 5.34. Despite the relatively high correlation coefficients ($r^2$=0.87, 0.92 and 0.88, respectively) of Equations 5.33, 5.34 and 5.35, the relevant data plots show that the data are still scattered around the regression lines of the three equations. For instance, according to the data provided by Goel et al. (1996) when *N* is 3, the corresponding *RCR* can be between 25 and 45. Further, Sari and Pasamehmetoglu (2004) found that regression analysis of *RCR* and *N* values does not always yield high correlation coefficients. Their *RMR-Q* correlation (Equation 5.29) with $r^2$=0.86 is better than their *RCR-N* correlation given as Equation 5.36 with $r^2$=0.65, showing a distinction from the three equations given above.

$$RCR = 1.7 \ln N + 51.5 \tag{5.36}$$

Based on their analysis, Sari and Pasamehmetoglu stated that the correlation between *RCR* and *N* cannot be generalised.

Further, Asgari (2001), using Iranian case studies, obtain the *RCR-N* correlation given as Equation 5.36. This again is different to the four *RCR-N* correlations listed earlier.

$$RCR = 6 \ln N + 45 \qquad (5.37)$$

Tzamos and Sofianos (2007) correlated four classification methods, RMR, Q, GSI and RMi, using their common parameters, namely joint surface conditions (*JC*) and the block size (*BS*). They provided charts for selecting ratings for *JC* and *BS* parameters of the four systems and defined a rock mass fabric index *F*, a function of *JC* and *BS*, and is expressed as

$$F = f(JC, BS) \qquad (5.38)$$

The rock mass fabric indices of the four classification methods were denoted as $F_{RMR}$, $F_Q$, $F_{GSI}$ and $F_{RMi}$, respectively, and a common chart was prepared for all four rock mass fabric indices. The required index value can be estimated by direct measurements of appropriate parameters (i.e. to determine $F_Q$ index, $JC_Q$ and $BS_Q$ should be measured) or by estimating other system parameters. In the RMR system *BS* is represented by *RQD* and *JS* (joint spacing). Therefore,

$$F_{RMR} = RQD + JS + JC \qquad (5.39)$$

In the Q system *BS* is represented by *RQD* and *Jn* (joint set number) and *JC* is represented by *Jr* and *Ja*. Therefore,

$$F_Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \qquad (5.40)$$

For validation of the chart, Tzamos and Sofianos applied the four rock mass fabric indices to several case tunnels using the data sourced from the literature. Similarly, they defined $F_{GSI}$ and $F_{RMi}$ for GSI and RMi classification methods, respectively, as well. Since this thesis deals with only the RMR and Q methods, GSI and RMi are excluded from this discussion.

The $F_{RMR}$ and $F_Q$ values obtained by applying the rock mass indices to several case tunnel data sets sourced from the literature allowed them to identify a correlation between the two indices as given below:

$$F_{RMR} = 6.52 \ln F_Q + 32 = 15 \log F_Q + 32 \tag{5.41}$$

Although this relationship has a correlation coefficient of 0.96, the database used has only about 30 data pairs and is not statistically significant. Further, the $F_{RMR}$-$F_Q$ data plot shows data scattering about the line defined by the above equation. For instance, according to the data plot presented by the two authors, for a given $F_Q$ value the corresponding $F_{RMR}$ value can have a range of up to 15 points. Similarly, for a given $F_{RMR}$ value the corresponding $F_Q$ can vary by about one order of magnitude. This clearly shows that the attempts made to correlate the *RMR* and *Q* values serve no useful purpose from the point of view of rock engineering.

### 5.3.4 Choice of the independent variable and method of analysis

The relationships listed in Table 5.4 are based on least square linear regression analysis of *RMR* and *Q* values with *Q* as the independent variable (abscissa of the *RMR-Q* plot as in Figure 5.2). Kaiser et al. (1986) pointed out that the correlations developed using linear regression analysis should be viewed with caution because the results depend on the choice of the dependent variable. By linear regression analysis of the data collected from the Wolverine West Tunnel in Canada they derived two relationships; the first (Equation 5.8) used *Q* as the independent variable, and the second (Equation 5.8a) used *RMR* as the independent variable.

$$RMR = 6.3\ ln\ Q + 41.6 \tag{5.8}$$

$$ln\ Q = 0.087\ RMR - 2.28 \tag{5.8a}$$

In spite of the fact that the two relationships were derived using the same data set, they do not lead to the same result. For example, the first equation would predict an *RMR* value of 40 from a *Q* value of 0.8, while in turn, the second equation would predict a *Q* value of 3.35 from a *RMR* value of 40. This clearly demonstrates the weakness of the conventional least square linear regression analysis. To overcome this weakness, Kaiser et al. (1986) used a probabilistic approach to determine a unique relationship between RMR and Q systems assuming that *RMR* and *ln Q* are normal variants and satisfy the central limit theorem of probability theory. Despite the use of a probabilistic approach, Kaiser et al. (1986) observed wide scattering of the data and therefore proposed Equation 5.9, which represents the 90% confidence limits within which 90% of the data used for their study fall. However, they noted that the range of values represented by the two equations is of little practical value as the range covers almost two RMR ground classes, as in the case of 90% confidence limits given by Bieniawski (1976).

### 5.3.5 Limitations of the Correlations

The possibility of deriving somewhat different *RMR-Q* correlations from different rock masses and the wide scattering of the data used in deriving them may be attributed to the fact that the assessment of some of the rock mass parameters is significantly different in the two methods as detailed in Section 4.5. For instance, in the *Q* system, the intact rock strength is considered as a factor in the *SRF* term only if the stability is likely to be affected by the in situ stress field around the excavation. When the potential instability in the excavation is due to the presence of weak zones, the intact rock strength is not considered in the *Q* value. In contrast, regardless of the critical factor governing the potential instability in the excavation, the intact rock strength is included in the *RMR* value. Similarly, in RMR the in situ stress field is not considered in the classification, although the support recommendations are for tunnels with a vertical stress magnitude of less than 25 MPa. In Q stress is a factor if the excavation stability is likely to be affected by the in situ stress field. Further, as pointed out by Milne et al. (1998), another difference between RMR and Q is evident in the assessment of joint spacing. If three or more joint sets are present and the joints are widely spaced, it is difficult to get the Q system to reflect the competent nature of a rock mass. For widely spaced jointing, the joint set parameter *Jn* in the Q system appears to unduly reduce the resulting *Q* value.

From the foregoing it is clear that there is unlikely to be a universally applicable single formula for linking *RMR* and *Q* values. Any relationship will be specific to the rock mass from which the data were obtained, the potential failure mode assumed in deriving the *Q* values and the orientation of the excavation considered for the *RMR* values. It is also noteworthy that the data used for deriving the *RMR* and *Q* correlations listed in Table 5.4 were obtained by applying different versions of the RMR system. For instance, the correlation given in Equation 5.1 was probably obtained using the pre-1976 version(s) of RMR, while the subsequent correlations may be based on either pre or post 1976 versions. Since different versions of the RMR method use somewhat different ranges of ratings, it is important to state which version is being used when correlating the *RMR* and *Q* values. The lumping of the ratings assigned using different RMR versions to compare and correlate them with the *Q* values has no scientific basis.

### 5.4 Conclusions Drawn from the Previous Studies

From the previous studies presented in this chapter several important conclusions on the reliability of the RMR and Q classification methods for underground excavation support design may be drawn. Since each study deals with a set of rock mass conditions or a geological environment specific to a particular project and the main focus of one study is not exactly the same as that of the next, conclusions do not necessarily always complement each other. While some conclusions complement each other, some conflict with others. In general, they can be divided into three broad groups:

(a) Common conclusions: the conclusions common to most (or all) of the case studies. These are likely to be of relevance to almost any past or future application of the classification approach, and deal with the relative easiness of application, structure, robustness and general applicability of the two systems.

(b) Predictions of support measures and/or support pressures. Prediction of the latter is an indirect method of support design. These conclusions could be specific to a single case or to closely similar cases and may either agree with or conflict with the conclusions drawn from the other cases. They can be further divided into three subsets of support predictions: overconservative, optimistic and adequate.

(c) Findings on the empirical formulas for support pressures. These conclusions directly deal with the empirical formulas given in the two methods for the prediction of support pressures. While some of these conclusions may be case specific and may or may not conflict with each other, they aimed at improving the support pressure formulas.

These conclusions are presented in the following sections.

**5.4.1 The common conclusions**

- Both methods are easier to use with RMR being the easiest.

- The size of the excavation relative to joint spacing is not considered, though this relation can be an important factor.

- The classification systems place strong emphasis on geological or rock mass parameters and not enough emphasis on the method of excavation.

- Rock durability is an important parameter not included in the two systems.

- In general, one should not necessarily rely on any one classification method.

- The classification approach should not be taken too far as a substitute for rock engineering design.

- There is not always good agreement between RMR and Q methods. Despite established correlations, the two systems do not lead to similar conclusions concerning tunnel support requirements.

- Under swelling ground conditions, the reliability of any of the approaches is yet to be established.

- The RMR system is relatively insensitive to minor variations in rock quality and is more so for the weak featured rock masses compared to the Q system.

- The *RMR* and *Q* values estimated from a larger tunnel would be smaller than those obtained from small drifts in a similar rock mass. This is due to the possibility of intersecting more discontinuity sets and weaker rock intrusions in a large opening.

- The RMR support recommendations are for 10 m wide tunnels only, and prediction of support requirements for larger tunnels (span>10 m) or smaller tunnels (span<10 m) is difficult with the current RMR system. The RMR support recommendations are often conservative for small diameter (~3 m) tunnels in better quality (fair to very good) rock masses.

- It is problematic to obtain correct *SRF* values near weakness zones intersecting an excavation. For example, consider two tunnels at depths of 100 and 300 m from surface, indicating different magnitudes of cover pressure, excavated through the same rock mass with a single weakness zone containing clay or chemically disintegrated rock. For both tunnels, the *SRF* value will be 2.5, because the depth of excavation is more than 50 m. This clearly shows that precise weightage to stress condition is missing from *SRF*, thereby indicating inadequacy in the Q system.

- A large range of *SRF* values is suggested when a shear zone only influences but does not intersect the excavation.

- Classification systems may be useful tools for estimating the need for tunnel support at the planning stage, particularly in hard and jointed rock masses without overstressing. There are, however, a number of restrictions that should be applied if and when these systems are to be used in other rock masses and in complicated ground conditions.

- Potential users of these systems should carefully study their limitations before adopting them.

### 5.4.2 Conclusions on the predictions of support measures

Conclusions on the prediction of support requirements can be further divided into three subsets of support predictions: overconservative, optimistic and adequate. In other words, from different case studies, while some researchers found that the support recommendations were conservative, others found that they were under conservative or optimistic. In some cases the recommended support measures were comparable to those installed. These are summarised below:

#### *5.4.2.1 Overconservative support recommendations*

- As a result of the presence of non-continuous joints and bedding planes, the RMR and Q methods are too conservative and therefore unsuitable for chalk rock mass. According to the two methods, the seven tunnels in chalk needed support, yet these in fact had stood up for many years (some of them for nearly 1500 years) with minimal or no support (Polishook and Flexer, 1998).

- Both RMR and Q are conservative for 5 to 5.5 m diameter BC rail tunnels but RMR is more so. This discrepancy must be attributed to the influence of opening size on tunnel performance and cannot be eliminated by simultaneous application of both classification systems to assess the factors that have been neglected (Kaiser et al., 1986).

- Many of the rock support pressures predicted by the Q system were greater than the measured rock loads (in non-squeezing rock) by an excessively conservative margin (Rutledge and Preston, 1978). The Q system overestimates support pressures for coalmine roadways (Sheorey, 1993).

- No support limit of RMR is found to be too conservative (Kaiser et al., 1986).

- The RMR system yields conservative estimates for unsupported stope spans (in hard rock mining). The 20 m upper limit of maximum permissible unsupported span is extremely conservative (Baczynski, 1983).

- The Q system overestimates support pressures for coalmine roadways yet also overestimates un-supported safe spans (Sheorey, 1993). In other words, while predicting high support pressures (or rock loads), the Q systems predicts the opening will be safe without support.

- The RMR estimates of support pressure for non-squeezing rock conditions are overly safe for large tunnels (Goel et al., 1995).

- For the North Head and Malabar ocean outfall tunnels in Sydney, excavated by drill and blast methods and road header respectively, the RMR system was conservative in terms of support requirements, particularly for machine excavated tunnels (Pells and Bertuzzi, 2008).

*5.4.2.2 Optimistic support recommendations*

- The Q predicted support matched only for two of the eleven sites studied, both being small span (~2.8 m) mine tunnels. For the other nine (six of them are from a 7.2 m span hydroelectric tunnel) the Q system underestimated support requirements (Brook and Dharmaratne, 1985). For the 7.2 m span tunnel RMR also underestimated support requirements.

- The Q system overestimates un-supported safe spans for coalmine roadways (Sheorey, 1993).

- The RMR system underestimated support pressures in poorer rock masses in coalmine roadways (Sheorey, 1993).

- The RMR estimated support pressures for tunnels in squeezing rock conditions are unsafe for all sizes of tunnels investigated (Goel et al., 1995).

- The RMR estimates of support pressures for non-squeezing rock are unsafe for small tunnels. When applied to coalmine roadways the estimated support pressures are unrealistically low in poor rock masses (Goel et al., 1995).

- The Q estimated support pressures for squeezing ground conditions were unsafe at least in two 9 m diameter tunnel sections. Limited data showed that Q tends to be unsafe for large tunnels in squeezing ground conditions (Goel et al., 1995).

- In the M2 tollway tunnels in Sydney the Q predictions could have been undesirably non-conservative (Pells and Bertuzzi, 2008).

- In the 50 m length of the Eastern Distributor tunnel in Sydney with a span of >20 m, the adopted support design is substantially greater than that deduced from the Q system (Pells and Bertuzzi, 2008).

- In the 14 m wide Botany Bay gas storage caverns in Sydney, two roof collapses occurred during construction when the support installed was in accordance with the Q system. The actual support installed subsequently was substantially of a higher capacity than the Q recommendations (Pells and Bertuzzi, 2008).

- In the M5 Motorway tunnels, the actual support installed were substantially more than those deduced using the Q system.

- In the Brisbane S1 main sewer tunnel, apart from the very poor and extremely poor rock classes, much greater support was installed than that recommended by Q which predicted that approximately 70% of the tunnel would be unsupported. However, some of these areas were supported with spot bolting and the remaining areas with systematic bolting and occasional mesh.

- In the North-side sewer storage project TBM driven tunnels, the installed support quantities were often different to those derived by the Q system. The actual density of rock bolting (bolts per metre) which proved necessary to install following inadequate performance of the initial design (based on the Q system), ranged between 5 and 9 times the initial densities (Pells, 2002; Pells and Bertuzzi, 2008).

- In the New Southern railway tunnel, Q predicted support using bore core data alone varied from random bolting to pattern bolting and fibre reinforced shotcrete, however, the installed support was limited to pattern bolting to pattern bolting and mesh (Asche and Quigley, 1999).

- In approximately 2 km of the Epping-Chatswood rail link tunnels considered by Pells and Bertuzzi (2008), there is no correlation between the Q value (or the recommended support) and the installed primary support. In most areas the support installed exceeded the Q predictions.

- For shallow tunnels under high horizontal stresses excavated in weak rocks with highly anisotropic behaviour due to schistocity, the RMR under predicted support requirements compared to the support installed. In all the 25 tunnels examined the installed support exceeded the RMR predictions.

In 64% of the cases, the difference between the predicted and installed was two RMR classes and the rest had a difference of one RMR class (Gonzalez de Vallejo, 2002).

- In tunnels excavated in weak rocks with high overburden thickness and high horizontal stresses, the RMR system underestimated support for 89% of the tunnel headings investigated in the 25 tunnels. The difference was one RMR class in 67% of the cases examined and in the remaining 22% the difference was two RMR classes. In these tunnels the difference between the installed and the Q predicted support was one Q support class, i.e. predicted poor class, but actual was very poor class, however, the sample size for the Q system was small (Gonzalez de Vallejo, 2002).

- The primary support used in the two Melbourne tunnels comprised rock bolts, shotcrete, mesh and steel sets. Other than for the very poor quality rock (fault zones and weathered igneous intrusions) there is no correlation between the Q value (or the recommended support) and the installed support.

- For TBM tunnels in jointed rock and laminated rock with high horizontal stress the Q system under estimate support requirements.

- Highly variable correlations were observed between tunnel convergence or deformation and the RMR index (Gonzalez de Vallejo, 2002).

*5.4.2.3 Adequate support recommendations*

- The Q system provides a reasonable estimate of support pressure in non-squeezing conditions and for smaller tunnels under squeezing ground conditions (Goel et al., 1995; Singh et al., 1997).

- For the North Head and Malabar ocean outfall tunnels in Sydney, the Q system provided a reasonable prediction where machine excavation was concerned, but was non-conservative for the drill and blast methods (Pells and Bertuzzi, 2008).

- In the M2 tollway tunnels in Sydney, the RMR predicted support measures were comparable to those installed (Pells and Bertuzzi, 2008).

- In the New Southern railway tunnel, the support deduced by Asche and Quigley (1999) subsequent to the tunnel excavation using the *Q* median values obtained from preconstruction bore core data best reflects the support installed in the tunnel with 7 out of 12 cases of *Q* median support predictions agreeing with the installed support.

- Tunnels excavated in weak rocks under low to moderate horizontal stresses regardless of overburden thickness the support installed were consistent with those predicted by the RMR method. In these tunnels the actual support installed was same as the Q predicted support (Gonzalez de Vallejo, 2002).

### 5.4.3 Findings on the support pressure (rock load) formulas

As discussed in Section 4.4, the empirical support pressure formulas of the Q system (see Equations 4.10 to 4.13) assume that the support pressure is independent of the width of the excavation and is only a function of the rock mass quality (i.e. *Q* value, *Jr* and *Ja*). In contrast, the RMR system takes the width of the excavation into account directly when estimating support pressures (see Equation 4.5). As already mentioned, some conclusions drawn from previous case studies directly deal with the support pressure formulas given in the two methods. Some of the findings from these studies have already been included in the Q system as can be seen from Equation 4.14. These conclusions are presented below:

- For a given rock class (in non-squeezing rock) the support pressure does not increase linearly from zero at zero tunnel width, but shows a less than linear increase with increasing tunnel width (Rutledge and Preston, 1978).

- The support pressures for rock tunnels in non-squeezing ground conditions can be taken as independent of the tunnel size, whereas in squeezing ground conditions the support pressure increases significantly with tunnel size. Furthermore, the size effect increases with tunnel depth. In addition, poorer rock masses experience a higher size effect (Goel et al., 1995).

- Support pressure in tunnels and caverns does not increase directly with span size due mainly to the dilatant behaviour of rock masses, joint roughness and prevention of loosening of rock mass by modern tunnelling technology. However, the support pressure is likely to increase directly with the excavation width for tunnels through slickensided shear zones, thick clay filled fault gouges, weak clay shales and running or flowing ground conditions where interlocking of blocks is likely to be missing (Singh et al. 1997).

- In poor quality brecciated rock masses experiencing squeezing conditions the support pressure increases with tunnel span (Singh et al., 1997).

- Support pressures in squeezing ground conditions decrease with tunnel closure significantly and increase rapidly beyond 6% closure (Singh et al., 1997).

- In the case of flat-roofed mine roadways through coal measures, the support pressure increases directly with the roadway width.

### 5.4.3 Conclusions on correlations between the *RMR* and *Q* values

As mentioned in Section 5.3 and listed in Table 5.4, several researchers have derived RMR and Q correlations by linear regression analysis of data. The review of the published information showed that each correlation is different from the next and the data used in deriving them are often widely scattered. The main reasons for this are the differences in the parameters and the rating methods used and the manner in which the final *RMR* and *Q* values are computed.

It is clear from the available information that a different relationship can be obtained for each case study and that each is applicable only to that particular rock mass and project conditions from which the relationship was obtained. Even for the same rock mass, if the data used are widely scattered, such relationships are of very little practical value and their use for transforming the ratings between the two methods could lead to errors. Further, Kaiser et al. (1986) showed that the results of correlations depend on the choice of the dependent variable. From the foregoing, it is apparent that there is no sound scientific basis to assume a universally applicable linear relationship between the two systems.

When both methods are to be applied to a project, which is desirable, each should always be applied independent of the other, without attempting to convert the ratings of one method to that of the other using the relationships published in the literature. Such relationships, bearing in mind their obvious limitations, may be used as a crude guide for checking the general accuracy of the ratings derived by the two systems.

# CHAPTER 6

# ANALYSIS OF CASE STUDIES

## 6.1 Introduction

In this chapter the details of the application of the RMR and Q to several case studies and an assessment of the reliability of their support predictions are presented. This research considers that under a given set of conditions, the reliability of the RMR and Q derived support for an underground excavation can be assessed by comparing them with those derived by other applicable methods and also with the actual support installed. Such an assessment can best be carried out during excavation of an underground opening because representative data can be collected by direct observation of the intersected ground conditions and monitoring the performance of the support installed. In this context, ten case studies were analysed for the present study. For nine of which the RMR and Q systems were applied by detailed mapping during excavation and for the remaining case they were applied during design stage using the data obtained by site investigation. For the present study the geotechnical data obtained and the ratings assigned to the RMR and Q input parameters during the construction of the case tunnels were reviewed and where deemed necessary minor adjustments were made to reflect the extreme ground conditions reported in the case studies. The effectiveness of the support predictions of the two classification methods was then evaluated against the potential failures that can be predicted by some of the applicable rational methods. Both structurally controlled gravity driven failures and stress induced failures in jointed rocks were considered depending on their relevance to the rock mass conditions intersected in the case tunnels. The structurally controlled gravity driven failures were analysed using limit equilibrium methods of analysis. For some of the selected case tunnels, the stress controlled failures in jointed rocks were also analysed by numerical modelling using the discontinuum approach.

The main mode of structurally controlled instability analysed in this study was tetrahedral rock wedge failure caused by three intersecting joints in rock masses.

Beam failure was also analysed when horizontally bedded or laminated rocks were intersected in the case tunnels. Tetrahedral rock wedge analysis was undertaken using UNWEDGE software code (Rocscience, 2003), developed based on the block theory proposed by Goodman and Shi (1985). UNWEDGE provides an effective means of identifying all kinematically unstable tetrahedral wedges in a rock mass, provided discontinuity orientations are known. The stability of kinematically unstable tetrahedral rock wedges identified in the rock mass was then assessed by limit equilibrium analysis, and the support measures required to stabilise potentially unstable rock wedges were determined. The UNWEDGE analysis assumes that the geological discontinuities are ubiquitous. For the present study this is acceptable because the application of the two rock classification indices also assumed that the joints were ubiquitous in each sector (or structural domain) of the case tunnels. Rock wedge stability is not only a function of discontinuities but also of the stress field in the rock mass. The induced stress field around the excavation can have a stabilising influence, particularly for narrow and deep rock wedges, but its effect may be reduced if wedges have loosened during excavation, i.e. due to blast vibration. In some cases the stress field can have a destabilising influence by forcing them out of their sockets, particularly if the wedges are broad and shallow. UNWEDGE does not accurately model the wedge failure caused by the stress field around the tunnel. However, it allows identification of wedges that have no restraining effect from the in situ stress field. For such wedges the stability analysis under gravity loading alone (unstressed state) may be considered applicable (Rocscience, 2008).

UNWEDGE can model the effect of both mechanically (point) anchored and full column grouted rock bolts installed in a rock mass. It can also analyse the effect of shotcrete by computing punching shear capacity of shotcrete along the edge of a rock wedge. In this study the bolts considered were the cement grouted type with 100% bond efficiency and an ultimate tensile strength of 180 kN installed normal to the rock face. The shotcrete was assumed to have a nominal compressive strength of 30 MPa and a tensile strength of 3 MPa. The effect of mesh or fibre reinforcement was analysed by doubling these values. A factor of safety (FOS) of 1.5 for walls and 2 for roof were selected for long term stability.

The beam analysis was undertaken using the suspended beam concept presented by Stilborg (1994) and Brady and Brown (2004), discussed in Chapter 3. It is a simplified method of analysis which ignores the in situ stress field around the excavation. The same type of rock bolts installed normal to the rock face was considered for the beam analysis. A FOS of 2 was used for long term stability. The effect of shotcrete was considered only implicitly.

Both tetrahedral wedge analysis and suspended beam analysis are based on three dimensional models and may be considered to represent a close approximation of the actual failure modes in a jointed rock mass around an underground opening.

In some of the case studies, the in situ stress field is considered to be high enough to warrant numerical analysis of stress induced failures. Since the rock masses in these case tunnels are moderately jointed, the discontinuum approach using UDEC developed by Itasca (2004) was adopted for numerical modelling of the stress induced rock mass behaviour around the case tunnels. UDEC is a two dimensional numerical modelling software package based on the distinct element method in which a rock mass is represented as an assembly of discrete blocks and discontinuities are viewed as interfaces between distinct bodies. UDEC can simulate the response of jointed rock masses subjected to either static or dynamic loading. It allows modelling of rock mass failure along discontinuities as well as through intact rock material. In this study UDEC analysis was applied only to the case studies in which the stability is considered to be governed by both discontinuities and the stress field. Only the static loading conditions were modelled using UDEC and the same rock bolts and shotcrete parameters were used as in the case of the limit equilibrium analyses. The adhesive strength and elastic modulus of shotcrete were assumed to be 0.5 MPa and 30 MPa, respectively. The elastic modulus of mesh/fibre reinforced shotcrete was assumed to be 35 MPa.

## 6.2 The Case Studies

The case studies used in this research comprised civil engineering project tunnels in different parts of the world representing a range of geological, geotechnical and project conditions. The tunnels in the database were constructed for hydroelectric, water supply, road and railway projects, underground power station access, exploration and grouting and drainage of a major dam foundation. For most of the case tunnels, detailed data were available from research reports, conference and journal papers or project specific documentation. The tunnels included in the database and the relevant data sources are listed in Table 6.1 and the details of each case tunnel are discussed in the following sections.

The tunnel mapping and the application of the RMR and Q methods to the case tunnels were conducted by several researchers with the participation of experienced site based geotechnical professionals. It is therefore considered that the data presented are devoid of individual bias and judgemental errors.

Table 6.1 Case tunnels included in the database

| Case No. | Project | Purpose of tunnel(s) | Length studied | Data source |
|---|---|---|---|---|
| 1 | Chiew Larn Hydropower, Thailand | River diversion/ irrigation | 493 m | Ratanasatayanont (1984) |
| 2 | Chiew Larn Hydropower, Thailand | Hydropower | 240 m | Ranasooriya (1985) |
| 3 | Huai Saphan Hin, Thailand | Hydropower | 732 m | Lasao (1986) |
| 4 | Central Tunnel, South Link Railway, Taiwan | Railway tunnel | 78 m | Yu-Shan (1987) |
| 5 | Lam Ta Khong pump storage, Thailand | Exploratory tunnel | 1230 m | Praphal (1993); Tran (1994) |
| 6 | Lam Ta Khong pump storage, Thailand | Underground access | 885 m | Sriwisead (1996) |
| 7 | Klong Tha Dan Dam, Thailand | Grouting/drainage (five tunnels) | 1657 m | Swe (2003) |
| 8 | Namroud Hydro Project, Iran | River diversion/irrigation | 740 m | Site personnel |
| 9 | Boztepe Hydro Project, Turkey | River diversion | 565 m | Gurocak et al. (2007) |
| 10 | Ramboda Pass Tunnel Project, Sri Lanka | Highway Tunnel | 222 m | Project report/maps (2006) |

### 6.3 CASE STUDY 1:
### The Chiew Larn Diversion (CLD) Tunnel, Chiew Larn Hydro Project, Thailand

The Chiew Larn project is a multipurpose water resources development project located in the Southern Province of Thailand, constructed between 1984 and 1986. The project comprised a 95 m high rock/earth main dam built across the Klong Saeng River, a 240 MW power station, a 493 m long diversion tunnel, and a 240 m long hydropower tunnel. The main purposes of the project are power generation, irrigation and groundwater salinity control by maximising the use of the Klong Saeng River flow. Additional benefits of the project include flood control, river pollution control, transportation, recreation and fishing.

The excavated diameter of the 493 m long CLD tunnel was 11.3 with an internal finished diameter of 10 m. Located in a ridge and running parallel to the ridge line, the tunnel has an overburden of 40 to 80 m with an average of about 60 m. Its general alignment is NW-SE with a 0.2% down gradient towards SE.

The horseshoe shaped CLD tunnel was constructed for two purposes: (a) to temporarily divert the Klong Saeng River to facilitate the construction of the main dam; and (b) as an irrigation water supply tunnel for downstream users. To fulfil the second purpose, after the completion of the main dam construction, the tunnel was plugged at approximately 105 m from the inlet and below the centreline of the main dam, which crosses the tunnel alignment. An irrigation outlet valve was provided in the plug. After filling the reservoir, the tunnel length upstream of the plug functions as a pressure tunnel under a hydraulic head equivalent to the reservoir level, which has a maximum elevation of 95 m RL (The tunnel invert level is approximately 10 m RL). In contrast the tunnel length downstream of the plug has an external water pressure equivalent to the groundwater level artificially elevated by the reservoir, and functions as a groundwater sink. During construction groundwater flow into the tunnel was generally nil to low, except for some isolated areas of water flow during the wet season.

### 6.3.1 Project Site Geology

The project area consists of clastic rocks of the Kanchanaburi group which includes greywacke, sandstone, pebbly sandstone, shale, mudstone and quartzite of Silurian to Permocarboniferous age. Intrusive quartz veins are widespread in the rock formation. The regional geological structures include major transcurrent faults which occurred in the Jurassic-Cretaceous period with 20 to 150 km displacements. These major regional faults do not traverse through the project area, but they caused other minor geological structures including minor faults striking NW-SE in the project area.

### 6.3.2 CLD Tunnel Rock Mass Data

Two rock types are present along the CLD tunnel: greywacke and subarkosic sandstone, with the former being the main rock type. Greywacke composed of silts, very fine sand and clay matrix with megaclasts of quartz, feldspar, chert, calcareous and granite rock fragments of 1 to 70 mm in diameter. Subarkosic sandstone is present as layers or lenses of less than 30 m in thickness within greywacke and has sharp contacts with the latter. This rock is composed of very fine to medium grained quartz and feldspar which are fused together with siliceous cement. The top 5 m of the 40 to 50 m thick overburden comprised completely weathered rock and residual soil materials.

The tunnel is located below the regional groundwater table, but pockets of perched water appeared to be present. During the dry season groundwater flow into the tunnel was generally low and did not affect construction. An increase in the groundwater inflow through crushed zones and very persistent discontinuities in the rock mass was evident during the wet season. This was interpreted as an indication of relatively high hydraulic conductivity of the rock mass surrounding the tunnel.

During the excavation, a suite of tests was conducted to determine the engineering properties of intact rock material in the tunnel, the results of which are given in Table 6.2. Based on the testing the main rock type (greywacke) is described as fresh, medium strong to strong rock with an average UCS of 60 MPa and an average intact rock Young's Modulus of 55 GPa. The average unit weight of intact rock is 26.4 kN/m$^3$ and Poisson's Ratio is 0.20. Subarkosic sandstone is stronger than greywacke and has an average UCS of 140 MPa, intact rock Young's Modulus of 60 GPa, intact rock unit weight of 26.3 kN/m$^3$ and an average Poisson's Ratio of 0.16. In addition to the testing reported in Table 6.2, conventional direct shear testing was also conducted on 28 greywacke and 10 subarkosic sandstone saw-cut core samples, and found that the average basic friction angle ($\phi_b$) for the two rock types is 32$^o$.

Table 6.2 Intact rock properties along the Chiew Larn Diversion Tunnel

| Property | Range | Mean | Std | # of tests |
|---|---|---|---|---|
| *Greywacke* | | | | |
| UCS (MPa) | 33-89 | 60 | 18 | 14 |
| E Modulus (GPa) | 20-90 | 55 | 17 | 14 |
| Joint wall strength (MPa) | - | 68 | 14 | 908 |
| Poisons Ratio | 0.13-0.29 | 0.20 | 0.05 | 14 |
| Density (kN/m$^3$) | - | 26.4 | - | 88 |
| *Subarkosic Sandstone* | | | | |
| UCS (MPa) | 72-185 | 140 | 48 | 4 |
| E Modulus (GPa) | 40-90 | 60 | 20 | 4 |
| Joint wall strength (MPa) | - | 114 | 12 | 83 |
| Poisson's Ratio | 0.11-0.21 | 0.16 | 0.04 | 4 |
| Density (kN/m$^3$) | - | 26.3 | - | 20 |

Three discontinuity types were observed in the tunnel: joints, shears and bedding. Joints are the most common and are ubiquitous. Shear zones usually contain crushed rock and are up to several centimetres in thickness. They strike more or less normal to the tunnel alignment with a dip of less than 45$^o$. Bedding planes are rare. From the discontinuity orientation data collected by mapping, six discontinuity sets were identified. The average orientations of the discontinuity sets are shown in Table 6.3.

In any given interval of the tunnel, with some exceptions, typically three joint sets are prominent with other sets present at random. The exceptions are the fractured zones, where closely spaced four or more sets may be present. The orientations of the prominent sets may change in different tunnel intervals. The discontinuity spacing, aperture size and their surface characteristics vary. The *RQD* along the tunnel is generally high. However, the presence of narrow fractured zones reduces the *RQD* value locally. Statistical distribution of the parameters concerning the discontinuities is shown in Figure 6.1.

Table 6.3 Average orientation of discontinuity sets

| Set No. | Dip angle (deg) | Dip direction (deg) | Remarks |
|---|---|---|---|
| 1 | 70 | 002 | Major set |
| 2 | 48 | 251 | Major set |
| 3 | 77 | 170 | Major set |
| 4 | 30 | 207 | Minor set |
| 5 | 41 | 044 | Minor set |
| 6 | 88 | 295 | Minor set |

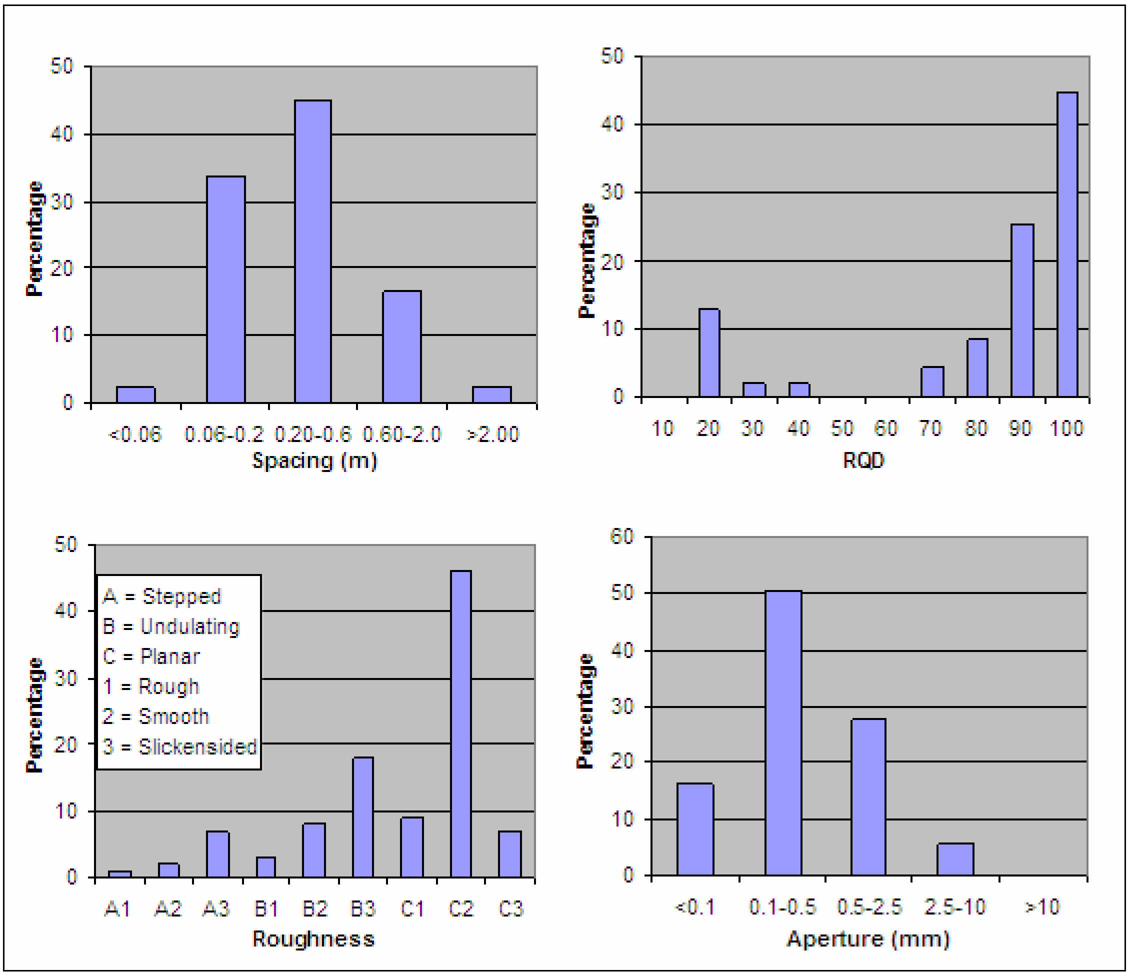


Figure 6.1 Distribution of joint spacing, aperture size, roughness and *RQD* in the CLD tunnel

The presence of these discontinuities creates a structural set up where rock blocks, particularly tetrahedral wedges, can theoretically be formed by several combinations of intersecting joints. Since the in situ stresses were low, movement of some of these blocks under gravity was possible.

### 6.3.3 Support Prediction for the CLD Tunnel using RMR and Q

Except for a total length of about 11 m, which was already covered by the inlet and outlet structures, Ratanasatayanont (1984) prepared a detailed engineering geological map for the 493 m long tunnel. Scan line mapping was also carried out to determine *RQD* using the method proposed by Priest and Hudson (1976). He divided the CLD tunnel into 47 sectors (geotechnical domains) taking into account the number of discontinuity sets, discontinuity spacing and their surface characteristics as well as groundwater conditions. The sector lengths varied from 4 to 54 m. Some sectors have virtually the same rock mass characteristics, but all adjoining sectors have different rock mass conditions. He applied $RMR_{79}$ and $Q_{74}$, which were current at the time, to the average rock mass conditions within each sector.

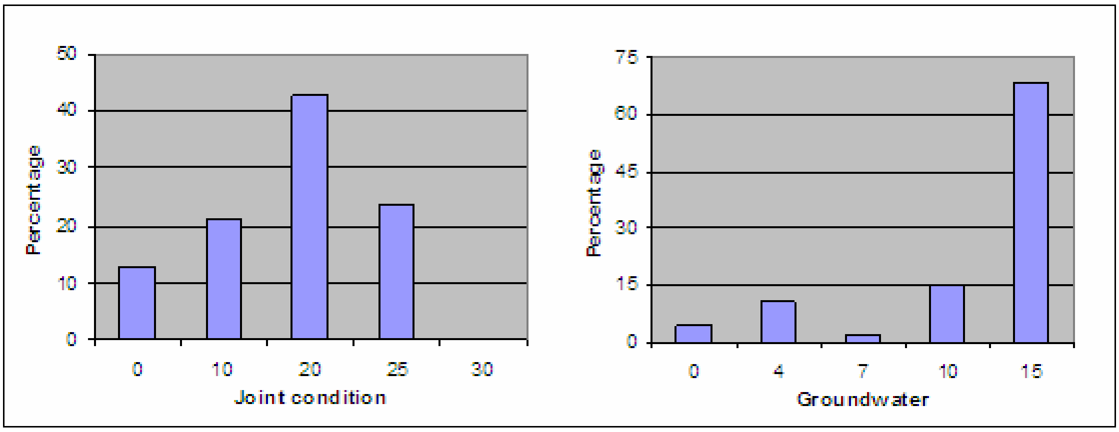


Figure 6.2 Distribution of *JC* and *GW* ratings of the CLD tunnel

The distribution of the ratings for the *RQD* and *JS* parameters of the RMR system may be visualised from Figure 6.1. Figure 6.2 presents the distribution of the ratings assigned to *JC* and *GW* parameters of the RMR method. Figure 6.3 presents the distribution of the ratings assigned to the *Jn*, *Jr*, *Ja* and *Jw* parameters of the Q system. Since the stress levels are low and favourable, and no major weak zones cut through or run parallel to the tunnel, for most of the tunnel length, *SRF*=1. For the tunnel sectors affected by the zones of weakness, *SRF* values of 2.5 and 7.5 were used. Table 6.4 presents a summary of the ratings assigned to the input parameters of the two classification methods.

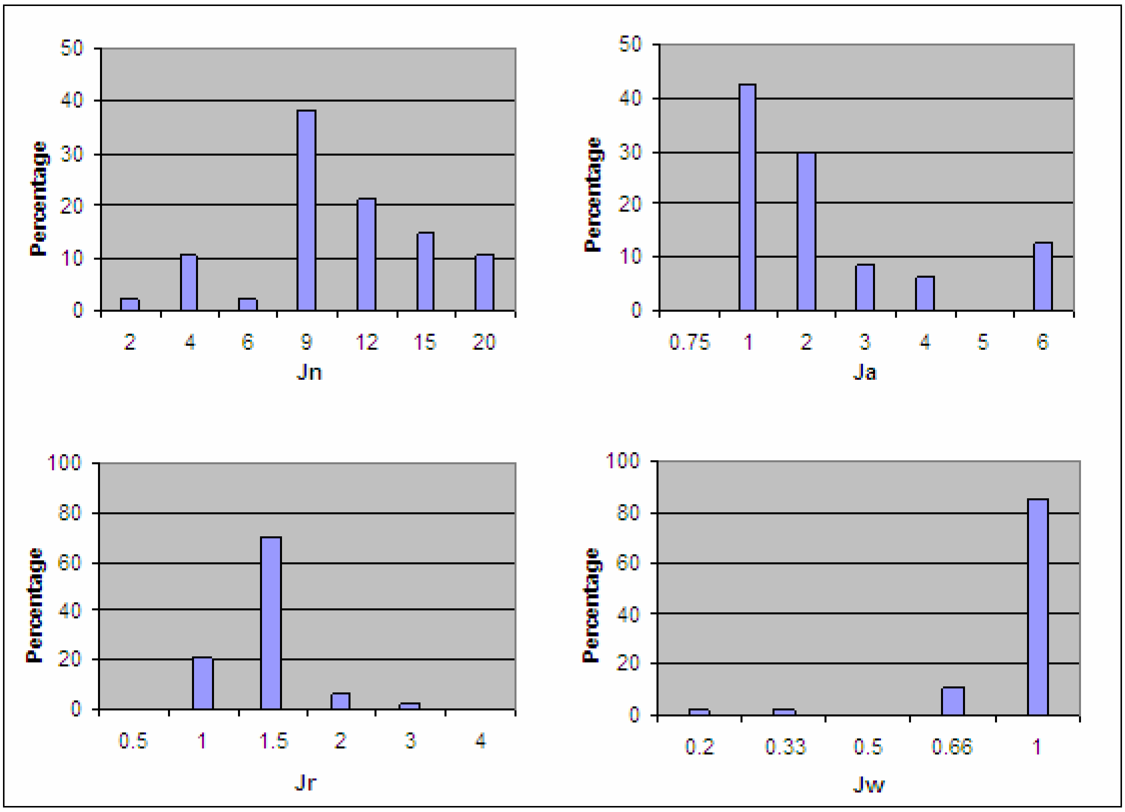


Figure 6.3 Distribution of the *Jn, Jr, Ja* and *Jw* ratings of the CLD tunnel

Table 6.4 RMR and Q input ratings for the CLD tunnel

| RMR | | Q | |
|---|---|---|---|
| ***Parameter*** | Ratings range | ***Parameter*** | Ratings range |
| *IRS* | 7-12 | *RQD* | 15-100 |
| *RQD* | 3-20 | *Jn* | 2-20 |
| *JS* | 5-15 | *Jr* | 1.5-4 |
| *JC* | 0-25 | *Ja* | 1-6 |
| *GW* | 0-15 | *Jw* | 1-0.2 |
| *RA (-)* | 0-12 | *SRF* | 1-7.5 |
| *RMR* value | 14-80 | *Q* value | 0.02-37.5 |

Under average rock mass conditions, RMR classified 67% of the rock mass in the tunnel as good rock and 10%, 12% and 11% as fair, poor and very poor rock, respectively (Table 6.5). For the tunnel roof and walls in poor and very poor classes of rock, RMR recommended rock bolts, mesh reinforced shotcrete and steel set support. For the roof in fair and good classes of rock, the recommended support measures were rock bolts and mesh reinforced shotcrete. No support was recommended for the tunnel walls in good rock. This means according to $RMR_{79}$ no support was required for 67% of the tunnel walls, and rock bolts plus a nominally 30 mm layer of shotcrete for further 10% of the walls (Table 6.5).

Table 6.5 The $RMR_{79}$ recommended support for the CLD tunnel

| RMR value | 80-61 | 60-41 | 40-21 | <20 |
|---|---|---|---|---|
| Rock mass class | Good | Fair | Poor* | Very poor# |
| Amount in each class | 67% | 10% | 12% | 11% |
| *Roof support* | | | | |
| Bolts (m) | L=3 S=2.5 | L=4 S=1.5-2 | L=4-5 S=1-1.5 | L=5-6 S=1-1.5 |
| Shotcrete (mm) | 50 (mr)+ | 50-100 (mr) | 100-150 (mr) | 150-200 (mr) |
| Steel sets (m) | None | None | S=1.5* | S=0.75# |
| *Wall support* | | | | |
| Bolts (m) | None | As above | As above | As above |
| Shotcrete (mm) | None | 30 | 100 (mr) | 150-200 (mr) |

L=length, S=spacing, mr=mesh reinforced; + Where required, * Light to medium steel sets where required. # Medium to heavy steel sets with lagging & fore-poling, if required, and bolt & close invert.

Table 6.6 shows that Q classified 38%, 34%, 5%, 6% and 17% of the rock mass as good, fair, poor, very poor and extremely poor rock, respectively. To determine the support requirements according to $Q_{74}$, an *ESR* value of 1.6 (for water tunnels) was selected, which gives the following equivalent dimension, *De*, values for the tunnel roof and walls:

$$De \text{ (roof)} = (Span/ESR) = 7 \text{ m}$$

$$De \text{ (walls)} = (Height/ESR) = 3.5 \text{ m}$$

The Q index recommended only spot bolting for the tunnel roof in good rock and systematic bolting plus 40 to 50 mm of shotcrete for that in fair rock. For the walls in both good rock and fair rock classes Q recommended no support. This means 72% of the tunnel walls required no support (Table 6.6). Note that wall support is derived using $Q_{wall}$ ($Q_{wall}$=2.5$Q$, when 0.1<$Q$<10).

For this study $Q_{94}$, the current version, was also applied using the same rock mass data compiled by Ratanasatayanont and required support measures were determined. With this version an *ESR* value of 1.8 was used as recommended by Barton and Grimstad (1994). A summary of the $Q_{94}$ derived support is presented in Table 6.7 which shows that no support was recommended for the tunnel walls in both good and fair rock.

Table 6.6 The $Q_{74}$ recommended support for the CLD tunnel

| Q value | 10-40 | 4-10 | 1-4 | 0.1-1 | 0.01-0.1 |
|---|---|---|---|---|---|
| Rock mass class | Good | Fair | Poor | Very Poor | Ext poor* |
| Amount in each class | 38% | 34% | 5% | 6% | 17% |
| *Roof support* | | | | | |
| Bolts (m) | L=3# (utg) S=1.5-2 | L=3 (utg) S=1-1.5 | L=3 (utg) S=1 | L=3 (tg) S=1 | L=3 (tg) S=1 |
| Shotcrete (mm) | None | 20-30 | 25-50 (mr) | 75-250 (mr) | 150-250 (mr) |
| *Wall support* | | | | | |
| Bolts (m) | None | None | L=3 (utg) S=1 | L=3 (utg) S=1 | L=3 (tg) S=1 |
| Shotcrete (mm) | None | None | 20-30 | 25-50 | 150-250 (mr) |

Note: L=length, S=spacing, sb=spot bolting, utg=un-tensioned grouted, tg=tensioned grouted, mr=mesh reinforced. #=spot bolting also recommended for roof in good rock. * For extremely poor rock steel reinforced cast concrete arch also recommended.

Table 6.7 The $Q_{94}$ recommended support for the CLD tunnel

| Q value | 10-40 | 4-10 | 1-4 | 0.1-1 | 0.01-0.1 |
|---|---|---|---|---|---|
| Rock mass class | Good | Fair | Poor | Very Poor | Ext poor+ |
| Amount in each class | 38% | 34% | 5% | 6% | 17% |
| *Roof support* | | | | | |
| Bolts (m) | Spot bolting | L=4 S=2-2.3 | L=4 S=1.7-2.2 | L=4 S=1.3-1.7 | L=4 S=1-1.3 |
| Shotcrete (mm) | None | 40-50 | 50-90 (Fr) | 90-120 (Fr) | 120-200 (Fr) |
| *Wall support* | | | | | |
| Bolts (m) | None | None | L=3 S=1.7-2 | L=3 S=1.3-1.7 | L=3 S=1-1.3 |
| Shotcrete (mm) | None | None | 40-50 | 50-120 (Fr) | 120-150 (Fr) |

Note: L=length, S=spacing, Fr=fibre reinforced, + Reinforced ribs shotcrete are also recommended.

Classification of the rock mass using $RMR_{89}$ yielded the same ratings as those of $RMR_{79}$ because the changes in the former have no direct effect on the rock mass interested in the CLD tunnel. Therefore the support recommendations of the current version, $RMR_{89}$, are the same as those of $RMR_{79}$.

### 6.3.4 Tetrahedral Wedge Stability Analysis

As mentioned in Section 6.2.2, the intersecting sets of joints in the rock mass could create kinematically unstable tetrahedral rock wedges at the periphery of the tunnel. Since the tunnel is shallow and the in situ stresses are low, movement of these rock blocks under gravity is possible. The kinematically unstable rock wedges were identified and their stability under the empirically recommended support measures was analysed using UNWEDGE. Since the tunnel overburden is between 40 and 80 m, two stress scenarios were considered in the analysis. The first assumed that the wedges are subjected to gravity loading only with no effect from the in situ stress field. The second included an inferred in situ stress field; the stress field was assumed to be due to the weight of the overlying rock with the horizontal to vertical stress ratio $k=\sigma_h/\sigma_v=1.5$.

#### *6.3.4.1 Shear strength parameters of joints*

Potentially unstable rock wedges may be present in either the most favourable or the most unfavourable ground conditions in the tunnel. To take this into account, three joint shear strength scenarios representing different rock mass conditions were considered in the wedge analysis. The first scenario shear strength parameters, representing a best case (with no clay filling in joints), were estimated using the shear strength relationship of Barton and Choubey (1977) given in Equation 2.7 and discussed in Section 2.5.2.

$$\tau_p = \sigma_n \tan\{\Phi_b + JRC \,\mathrm{Log}_{10}(JCS/\sigma_n)\} \qquad (2.7)$$

where $\tau_p$=shear strength; $\sigma_n$=joint normal stress; $\Phi_b$=basic friction angle; *JRC*=joint roughness coefficient; and *JCS*=joint compressive strength.

From the data presented by Ratanasatayanont (1984) the following values were selected: $JCS$=60 MPa, $JRC$=2.5, $\Phi_b$=32$^o$, and $\sigma_n$=1 MPa (based on vertical stress due to gravity). Using these input values and the Mohr-Coulomb relationship $\tau_p=\sigma_n \tan \Phi + c$, joint shear strength parameters c=50 kPa and $\Phi$=35$^o$ were obtained for a best case (most favourable) joint conditions.

The second strength scenario was estimated using the frictional component only (c=0) relationship $\Phi=\tan^{-1}(Jr/Ja)$ suggested by Barton (2002). For this purpose the joint roughness ($Jr$) and joint alteration ($Ja$) parameters shown in Figure 6.3 were used. Figure 6.3 shows that the most common $Jr$ value is 1.5 (70%) and the next is 1.0 (21%). The most common $Ja$ value is 1.0 (43%) and the next is 2.0 (30%). Accordingly, when c=0 the most common friction angle is $\Phi=\tan^{-1}(1.5/1.0)=56^o$. The other possible values are $\Phi=\tan^{-1}(1.5/2.0)=37^o$, $\Phi=\tan^{-1}(1.0/1.0)=45^o$, and $\Phi=\tan^{-1}(1.0/2.0)=27^o$. From these four values, $\Phi$=56$^o$ was selected as a possible best case scenario, which also represents unfilled joints with no cohesion component (c=0).

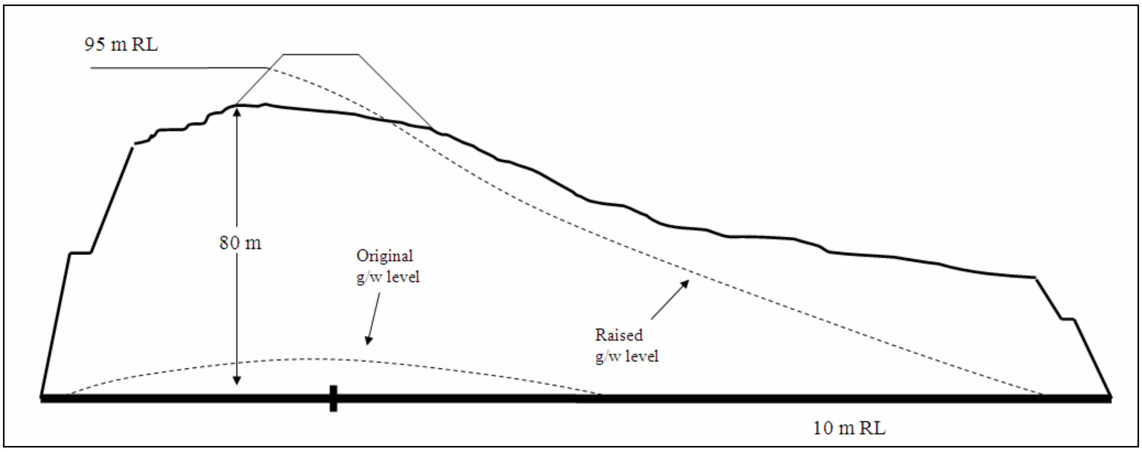


Figure 6.4 Indicative groundwater level along the CLD tunnel alignment

For the filled joints, c=10 kPa and $\Phi$=25$^o$ were selected based on the data compiled by Barton (1974) and the suggestions given by Barton and Grimstad (1994) for filled joints, and also considering the potential for water saturation of the joints. The filled joints represent a worst case joint shear strength scenario in the rock mass. The lowest friction angle, $\Phi$=27$^o$, obtained by $\Phi=\tan^{-1}(Jr/Ja)$ formula indicates that the estimated worst case joint shear strength parameters for filled joints c=10 kPa and $\Phi$=25$^o$ are reasonable estimates.

*6.3.4.2 Changing groundwater conditions*

As mentioned earlier, after the completion of the main dam construction, the tunnel was plugged and converted to an irrigation outlet by providing a valve in the plug, approximately 105 m from the inlet. During the operation of the project the groundwater level around the tunnel length downstream of the plug is likely to be elevated by the reservoir (Figures 6.4 and 6.5). The anticipated change in groundwater level was included in the wedge stability analysis discussed in the following sections.

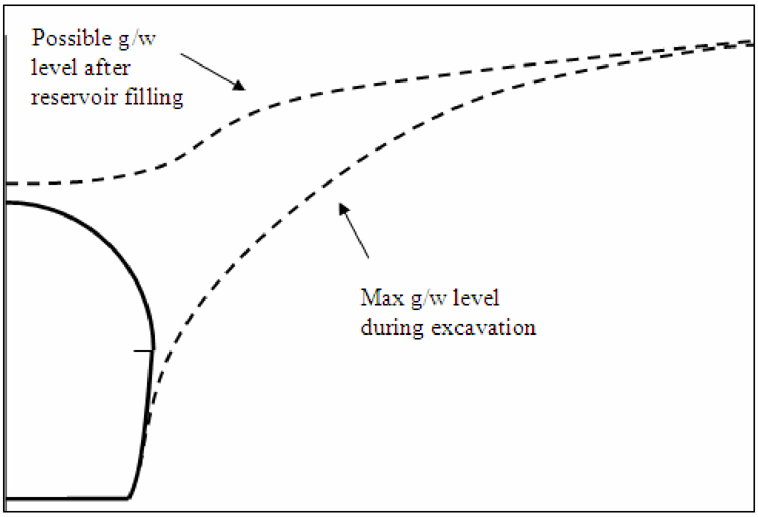


Figure 6.5 Indicative groundwater level in a CLD tunnel section

*6.3.4.2 The results of the UNWEDGE analysis*

The analysis showed that there were several kinematically unstable tetrahedral wedges in the tunnel roof and walls. Since the potentially unstable smaller wedges in the tunnel periphery are likely to fall during blasting or be removed by scaling, only the wedges with an apex height greater than 1 m were selected for stability assessment. The stability of these wedges was analysed for the three shear strength conditions with and without the effect of the in situ stresses in the rock mass.

The empirically derived support measures were then included in the UNWEDGE model to examine their adequacy to stabilise the potentially unstable rock wedges.

First the effect of rock bolts recommended by the two classification methods was analysed. Then the effect of empirically derived shotcrete layers was also analysed.

The results of the analysis show that the RMR recommended support measures for fair, poor and very poor classes of rock are sufficient to stabilise the theoretically possible rock wedges in the tunnel roof and walls. Similarly, Q recommended support measures for poor, very poor and extremely poor classes of rock are sufficient for the roof and walls. The Q recommendations for the tunnel roof in fair rock are also sufficient for the theoretically possible tetrahedral rock wedges.

For the tunnel roof in good rock, RMR recommended rock bolts plus mesh and 50 mm of shotcrete where required (shotcrete is probably for fractured ground, if present, and not for the entire roof), and Q recommended spot bolting. The RMR recommendation is only marginally acceptable. For instance, if the effect of in situ stress field is ignored in the analysis, with the recommended rock bolts, roof wedges formed by joint sets 1-3-6 and 3-5-6 with maximum possible weights of 121 kN and 686 kN may have FOS values as low as 0.72 and 0.52, respectively, depending on the position of the rock bolts with respect to the perimeter of the wedge. This is because the RMR recommended bolt spacing is too wide to ensure a sufficient number of bolts to penetrate through the potentially unstable rock wedges. Unless the bolt spacing is reduced or such wedges are visually identified and spot bolted, these recommendations may not meet the FOS requirements. This is illustrated in Figure 6.4 which shows the wedge formed by joints sets 3-5-6 with rock bolts installed at the RMR recommended 2.5 m spacing. With this bolt system the number of bolts penetrated through the wedge is insufficient to provide an adequate safety margin. In this instance the FOS is 0.52 when the joint shear strength parameters were taken as c=50 kPa and $\Phi=35^{o}$ with no contribution from the in situ stress field.

Further, RMR classified 67% of the rock mass as good rock, and recommended no support for the tunnel walls. Similarly, Q classified 38% and 34% of the tunnel as good rock and fair rock respectively, and recommended no support for the walls in these two classes of rock (total of 72% of the tunnel).

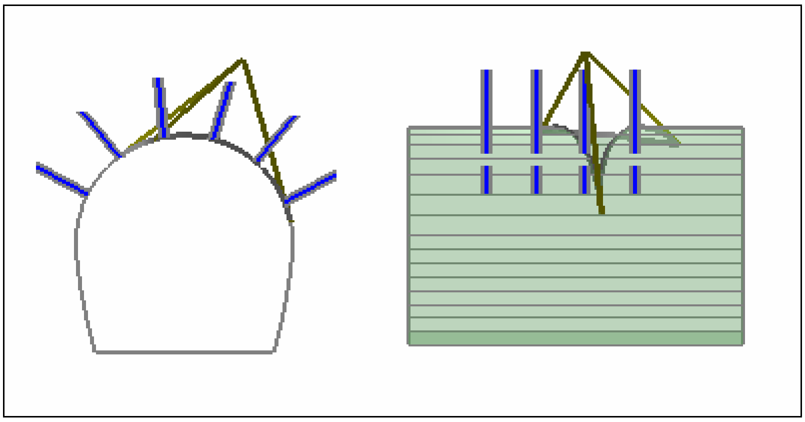

Figure 6.6 A wedge formed by J3, J5 and J6 with RMR derived rock bolts.

The wedge analysis showed that there is the potential for several different tetrahedral rock wedges in the tunnel walls, and that the stability (or the FOS) of these wedges is sensitive to both joint shear strength and the stress field around the tunnel. The stress field around the tunnel increases the FOS of deep narrow wedges by clamping the wedges in place. In the case of shallow flat wedges, the stress field reduces the FOS by forcing the wedges out. As previously mentioned UNWEDGE does not accurately model the wedge failure caused by the stress field, however, it does allow identification of wedges that have no restraining effect from the stress field. In this study the rock wedges with sufficient clamping effect from the in situ stress field to provide an acceptable FOS were not analysed further. The flat wedges that have no stabilising effect from the stress field were further analysed to assess the effect of the changing groundwater conditions described earlier (Figures 6.4 and 6.5).

The elevated groundwater level around the tunnel interval downstream of the plug at 105 m could cause erosion of discontinuity infill materials and contribute to wedge failure in the longer term. Hence the UNWEDGE analysis was extended to examine the effect of the elevated groundwater level on the stability of the theoretically possible rock wedges. This was modelled using the elevation water pressure option in UNWEDGE. Several groundwater elevations were modelled ignoring the in situ stress field. The results of the analysis showed that when the groundwater level is elevated to about mid height of the tunnel, the FOS of several rock wedges fall well below unity. When the groundwater table is about 7 m above the invert level (about half the tunnel height), which is considered likely, the FOS of several rock wedges becomes zero or near zero indicating potential instability. For each wedge analysed the FOS for seven cases are presented in Table 6.8. The details presented in Table 6.8 are for wedges that have no stabilising effect from the in situ stress field. The seven cases are explained below the table.

Since the analysis assumed that the joints are ubiquitous, these wedges may or may not be present. If they are present in the tunnel walls, some form of support will be required to ensure their long term stability. As previously mentioned, RMR recommended no support for 67% of the tunnel walls and Q recommended no support for 72% of the tunnel walls.

Table 6.8 Potentially sliding rock wedges in the CLD tunnel walls

| Wedge | Sets | Wall | Apex (m) | Weight (kN) | FOS1 | FOS2 | FOS3 | FOS4 | FOS5 | FOS6 | FOS7 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 125 | Left | 1.7 | 402 | 4.25 | 0.85 | 0.00 | 1.14 | 0.53 | 1.33 | 0.00 |
| 2 | 125 | Right | 1.8 | 513 | 4.29 | 0.83 | 0.00 | 1.23 | 0.52 | 1.70 | 0.00 |
| 3 | 126 | Left | 1.2 | 73 | 4.60 | 1.04 | 0.00 | 1.22 | 0.65 | 1.34 | 0.00 |
| 4 | 146 | Right | 1.8 | 158 | 5.25 | 1.24 | 0.00 | 1.22 | 0.78 | 0.78 | 0.00 |
| 5 | 146 | Left | 2.1 | 189 | 3.91 | 1.23 | 0.00 | 1.35 | 0.79 | 2.56 | 0.00 |
| 6 | 245 | Left | 3.2 | 2336 | 2.69 | 0.97 | 0.00 | 1.10 | 0.61 | 2.57 | 0.00 |
| 7 | 245 | Right | 3.1 | 2656 | 3.19 | 0.96 | 0.00 | 1.01 | 0.60 | 1.70 | 0.00 |
| 8 | 256 | Left | 2.9 | 974 | 5.40 | 1.39 | 0.60 | 1.23 | 0.88 | 3.42 | 0.58 |
| 9 | 256 | Right | 2.7 | 1151 | 4.51 | 1.35 | 0.61 | 1.40 | 0.85 | 2.28 | 0.69 |
| 10 | 356 | Right | 1.7 | 113 | 3.50 | 1.35 | 0.00 | 1.07 | 0.86 | 1.70 | 0.00 |

FOS1: c=50 kPa $\Phi=35^o$ $\sigma_h=\sigma_v=0$ $h_w=0$

FOS2: c=50 kPa $\Phi=35^o$ $\sigma_h=1.5\sigma_v$ $h_w=0$

FOS3: c=50 kPa $\Phi=35^o$ $\sigma_h=\sigma_v=0$ $h_w=7$ m

FOS4: c=10 kPa $\Phi=25^o$ $\sigma_h=\sigma_v=0$ $h_w=0$

FOS5: c=10 kPa $\Phi=25^o$ $\sigma_h=1.5\sigma_v$ $h_w=0$

FOS6: c=0 kPa $\Phi=56^o$ $\sigma_h=\sigma_v=0$ $h_w=0$

FOS7: c=0 kPa $\Phi=56^o$ $\sigma_h=\sigma_v=0$ $h_w=7$ m

### 6.3.5 The Installed Support

During excavation several large potentially unstable rock blocks were identified in the roof and walls of the tunnel. These were temporarily stabilised using 6, 4 and 3 m long mechanically anchored and resin grouted rock bolts and wire mesh. At the outlet portal, steel rib support was also installed. The tunnel was fully lined with a 700 mm nominal thickness in situ cast concrete liner. Among the issues considered in selecting the final support were fluctuating water levels during river diversion and an increase in groundwater pressure around the tunnel after the reservoir filling. Both could cause erosion of joint filling material, which in turn could lead to instability in the tunnel.

### 6.3.6 Discussion

In general, both rock mass classification methods recommended adequate support measures for the entire tunnel roof. The two methods did not recommend any support for a significant length of the walls in this tunnel. Tetrahedral rock wedge analysis showed that several wedges are kinematically possible in the tunnel walls. The stability of these wedges depends on the joint surface characteristics and the magnitude and direction of stresses in the rock mass. While these wedges may be stable in general, the sensitivity analysis showed that when the groundwater level is artificially elevated by the creation of the reservoir, the wedges listed in Table 6.8 could become unstable. This is a changing loading scenario which needs consideration for support design.

The limitations of the rock mass classification methods for tunnels subjected to changing loading conditions, in both civil and mining projects, are known and were discussed in detail by Speers (1992). He concluded that “the use of empirical support design methods such as the RMR-method and Q-method will lead to under designs …” and consequently recommended analytical approaches for such situations. For mining excavations with changing stress conditions, Mikula and Lee (2003) showed that Q can be applied by adjusting the *SRF* rating to reflect the expected future stress conditions. Similarly, the change in groundwater level subsequent to the construction of a tunnel may be accounted for by adjusting the *Jw* rating of Q (and ground water rating of RMR) to reflect the anticipated groundwater pressure. However, the artificially elevated groundwater head considered in the analysis of the CLD tunnel is not high enough to downgrade the *Jw* rating. It falls within the minor inflow/pressure range and receives a *Jw* rating of 1, which means no change. With RMR, the inferred groundwater pressure lowers the overall rating by three points. But this does not significantly change the recommended support.

### 6.3.7 Conclusion

Both RMR and Q methods recommended adequate support measures for the rock wedge instability in the tunnel roof. An exception is the roof in good rock where RMR recommended bolt spacing is too wide for some of the potentially unstable rock wedges identified by wedge analysis.

The two methods did not recommend any support for a significant length (RMR 67% and Q 72%) of the tunnel walls where rock wedges were identified during excavation and supported with rock bolts. The tetrahedral rock wedge analysis confirmed the presence of kinematically unstable rock wedges in the tunnel walls. The analysis showed that the RMR and Q recommendations were not sufficient to stabilise the potentially unstable rock wedges in the CLD tunnel walls, particularly under the artificially elevated groundwater levels.

For long term stability and project specific requirements, the support installed exceeded those recommended by the RMR and Q methods.

## 6.4 CASE STUDY 2:
## The Chiew Larn Hydropower (CLH) Tunnel, Thailand

The Chiew Larn hydropower (CLH) tunnel, a major part of the Chiew Larn Project described in Section 6.3, was constructed to feed three 80 MW power generating units. The tunnel located in a limb of a hill is 240 m long and was excavated to a horseshoe shape with a 13 m span. Its final shape is circular with an internal diameter of 11.2 m and its alignment is $140^{o}$ E with a plunge of $10^{o}$. The ground surface above the tunnel alignment is uneven, but has an overall slope of about $10^{o}$ towards downstream. The tunnel overburden varies between 25 and 50 m above crown level, with an average of approximately 30 m.

### 6.4.1 Project Site Geology

The general geology of the project site has been described in Section 6.2.1.

### 6.4.2 CLH Tunnel Rock Mass Data

The tunnel was driven entirely through dark grey, fine to medium grained greywacke sandstone, in which intact rock material can be described as fresh but some of the discontinuity surfaces are weathered. The intact rock material test results are given in Table 6.9. The RQD values determined by the method proposed by Priest and Hudson (1976) ranged from 60 to 100 with a mean value of 80 and a standard deviation of 10.

Table 6.9 Intact rock material properties (CLH tunnel)

| Property | Range | Mean | Std | # of tests |
|---|---|---|---|---|
| UCS (MPa) | 102 – 172 | 138 | 24 | 7 |
| E Modulus (GPa) | 42 – 57 | 51 | 6 | 5 |
| Poisson's Ratio | N/A | 0.23 | N/A | 5 |
| Density ($kN/m^3$) | N/A | 26.5 | N/A | 6 |

N/A=not available

From the analysis of discontinuity orientation data, three major discontinuity (joint) sets (Figure 6.7) and two minor sets were identified.  The average orientations of joint sets and their surface characteristics are presented in Table 6.10.

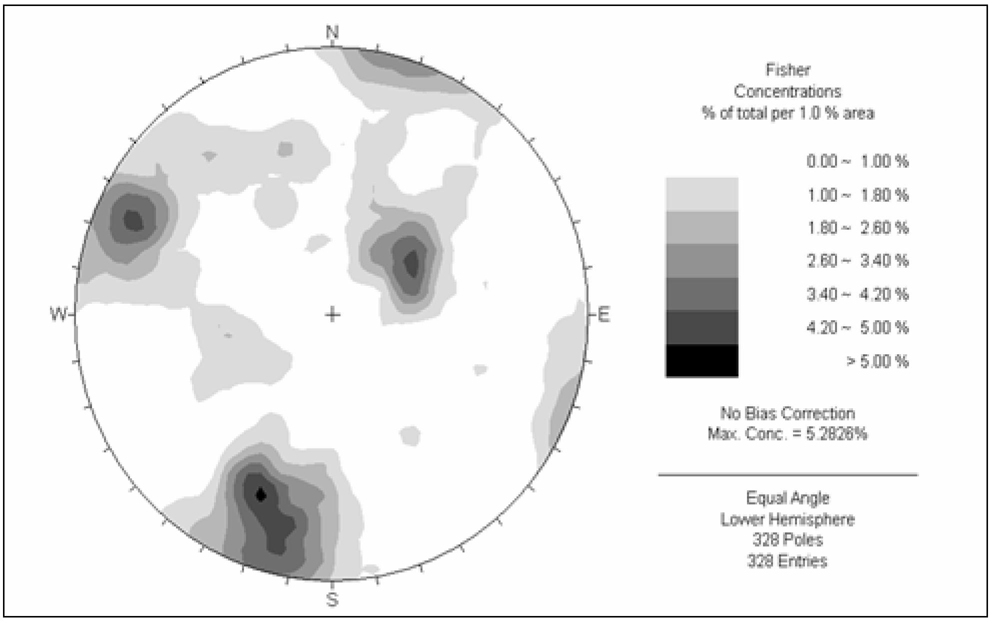


Figure 6.7 CLH tunnel discontinuity orientations

Not all three major sets are prominent along the entire tunnel length.  Only two major sets are prominent within any selected length of the tunnel, with the third set occurring at random.  In the first 80 m of the tunnel, Sets 1 and 2 are prominent with Set 3 occurring at random.  From 80 to 130 m, Sets 2 and 3 are prominent and Set 1 is at random.  From 130 to 240 m Sets 1 and 3 are prominent and Set 2 is at random.  Sets 4 and 5 are present randomly with no recognisable patterns.

Table 6.10 CLH tunnel joint orientations and their surface characteristics

| | Set 1 | Set 2 | Set 3 | Set 4 | Set 5 |
|---|---|---|---|---|---|
| Roughness | | | rough to slickensided | | |
| Waviness | | | undulating to planar | | |
| Aperture (mm) | 0.25-0.5 to 2.5-10 | 0.25-0.5 to 2.5-10 | 2.5-10 to 10-100 | 2.5-10 | 0.25-0.5 to 10-100 |
| Filling | coated to clay filled | coated to clay filled | coated to sandy clay/clayey sand | coated | Coated to sandy clay/clayey sand |
| Dip | 76 | 79 | 37 | 62 | 44 |
| Direction | 016 | 112 | 231 | 151 | 067 |
| Remarks | Major set | Major set | Major set | Minor set | Minor set |

Discontinuity Sets 1 and 2 are partly open (0.25-0.5 mm) to moderately wide open (2.5-10 mm) and Set 3 is moderately wide to widely open (>10 mm). Some of the joint surfaces are slightly weathered and some are coated with clayey material. Sets 1 and 2 joints are either filled or coated with clayey material. In the first 100 m of the tunnel, Sets 3 and 5 are slickensided with an aperture of less than 5 mm. From 100 m onwards, shear movement was evident in these two sets with an aperture of 10 to 100 mm filled with sandy clay or clayey crushed rock. Joint surface roughness, waviness and aperture size etc vary from one extreme to the other within each set (Table 6.10).

Minimum spacing between members of the major joints sets is approximately 0.6 m and the maximum is greater than 2 m. However, the presence of narrow fractured zones parallel to some of the major joints reduces the *RQD* value locally. An important feature is that, when the joint surface conditions (roughness, aperture and filling) are at the worst observed state, the joint spacing is at its best state (i.e. >2 m). Set 3 has very high persistence with joint traces extending for several tens of meters along the tunnel. The persistence of Set 1 is also high with almost all its members through-going. Set 2 also has a high persistence, most of its members having a trace length of at least 10 m. The persistence of Sets 4 and 5 varies between 3 and 20 m. Although the joints are relatively open, the tunnel was mostly dry, with only negligible water inflow in some places, partly because the tunnel is below the regional groundwater level.

Table 6.11 General features of the three CLH tunnel sections

| Tunnel Section | Prominent joint sets | RQD | In situ vertical stress | Water inflow |
|---|---|---|---|---|
| 1 (ch040-110 m) | Set 1, Set 2 | 68-98 | 1.30 MPa | Dry |
| 2 (ch110-170m) | Set 2, Set 3 | 62-80 | 1.04 MPa | Dripping |
| 3 (ch170-280m) | Set 1, Set 3 | 60-96 | 0.81 MPa | Dripping |

### 6.4.3 Support Predictions by the RMR and Q Methods

During the excavation, Ranasooriya (1985) prepared a detailed engineering geological map for the entire tunnel. To apply the two rock mass classification methods, the CLH tunnel was divided into three sectors (geotechnical domains) along the tunnel axis based on major geological discontinuities, rock quality designation, estimated vertical in situ stress level and water seepage into the tunnel. The three sectors of the tunnel and the variations in the above parameters are given in Table 6.11 (note that tunnel intake portal is located 40 m from the reference point).

Table 6.12 Ratings assigned for RMR and Q (CLH tunnel)

| | Sector 1 (Ch40–110 m) | | Sector 2 (Ch 110-170m) | | Sector3 (Ch170-280m) | |
|---|---|---|---|---|---|---|
| Rock mass scenario | Best | Worst | Best | Worst | Best | Worst |
| **RMR** | | | | | | |
| *IRS* | 12 | 12 | 12 | 12 | 12 | 12 |
| *RQD* | 20 | 13 | 17 | 13 | 20 | 13 |
| *JS* | 15 | 10 | 20 | 20 | 20 | 20 |
| *JC* | 20 | 10 | 10 | 0 | 10 | 0 |
| *GW* | 15 | 10 | 15 | 10 | 15 | 10 |
| *RA* | -5 | -5 | -10 | -12 | -12 | -12 |
| *RMR value* | 77 | 50 | 64 | 43 | 65 | 43 |
| Rock mass class | II | III | II | III | II | III |
| **Q** | | | | | | |
| *RQD* | 100 | 65 | 90 | 60 | 100 | 65 |
| *Jn* | 6 | 12 | 6 | 9 | 6 | 9 |
| *Jr* | 3 | 1.5 | 2 | 1.5 | 2 | 1.5 |
| *Ja* | 2 | 3 | 3 | 6 | 3 | 6 |
| *Jw* | 1 | 1 | 1 | 1 | 1 | 1 |
| *SRF* | 2.5 | 2.5 | 5 | 5 | 5 | 5 |
| *Q (roof)* | 10 | 1.08 | 2 | 0.33 | 2.22 | 0.36 |
| Support category (roof) | 13 | 23 | 22 | 31 | 22 | 31 |
| *Q (walls)* | 25 | 2.7 | 5 | 0.83 | 5.55 | 0.9 |
| Support category (wall) | None | 18 | 18 | 27 | 18 | 27 |

During construction, the two classification methods $RMR_{79}$ and $Q_{74}$ were applied to the rock mass intersected in the tunnel. Since the discontinuity characteristics (roughness, waviness, aperture and filling material) vary from one extreme to the other within each set and also within a selected tunnel section, two extreme conditions were considered for classifying the rock mass, i.e. the best (most favourable) and the worst (most unfavourable) combinations of ground conditions observed along the tunnel. The results of the ratings assigned as per RMR and Q are presented in Table 6.12.

The support measures recommended by the two methods are presented in Tables 6.13 and 6.14. The RMR recommended support (Bieniawski, 1979, 1989) for 10 m diameter tunnels were assumed to be applicable to the CLH tunnel, except for bolt lengths which were increased to take into account its 13 m diameter in subsequent stability analysis of rock wedges. To determine support measures according to the Q system an *ESR* value of 1.6 was selected as recommended by Barton et al (1974) for water tunnels. The equivalent dimension, *De,* for roof and walls are then as follows:

$$De \text{ (roof)} = (Span/ESR) = 8.1 \text{ m}$$
$$De \text{ (walls)} = (Height/ESR) = 4.1 \text{ m}$$

From Tables 6.13 and 6.14 it is evident that the supports recommended by the two empirical methods are generally in agreement. Their main differences are in bolt lengths and spacing as well as in shotcrete thickness.

Table 6.13 $RMR_{79}$ recommended support for the CLH tunnel

| Rock mass class | II | | III | |
|---|---|---|---|---|
| | Roof | Walls | Roof | Walls |
| Bolts (m) | S=2.5 L=3 locally | None | S=1.5-2 L=4 | S=1.5-2 L=4 |
| Wire mesh | Occasional | None | Yes | None |
| Shotcrete (mm) | 50 where required | None | 50-100 | 30 |

L=length, S=spacing.

Table 6.14 $Q_{74}$ recommended support for the CLH tunnel (*ESR*=1.6)

| Support category | 13 | 18 | 22 | 23 | 27 | 31 |
|---|---|---|---|---|---|---|
| Bolts (m) | Sb L=3.2 | S=1-1.5 L=3.2 | S=1 L=3.2 | S=1-1.5 L=3.2 | S=1 L=3.2 | S=1 L=3.2 |
| Wire mesh | None | Clm | Yes | Yes | Yes | Yes |
| Shotcrete (mm) | None | None | 25-50 | 50-100 | 50-75 | 50-125 |

L=length, S=spacing, Sb=spot bolting, Clm=chain link mesh.

For the present study the tunnel support requirements were also determined according to $Q_{94}$, the current version of Q, using an *ESR* value of 1.8, which gives *De* (roof) = 7.2 m and *De* (walls) = 3.6 m. The support recommendations are presented in Table 6.15. The current version $Q_{94}$ also recommended similar support, but no support is recommended for the tunnels walls in the best rock mass conditions. Note that, $Q_{94}$ recommended fibre reinforced shotcrete, whereas the earlier version recommended wire mesh reinforcement for shotcrete.

Table 6.15 $Q_{94}$ supports recommendations for the CLH tunnel (*ESR*=1.8)

| | Section 1 (40–110 m) | | Section 2 (110-170m) | | Section 3 (170-280m) | |
|---|---|---|---|---|---|---|
| Rock mass scenario | Best | Worst | Best | Worst | Best | Worst |
| *Q (roof)* | 10 | 1.08 | 2 | 0.33 | 2.22 | 0.36 |
| Support category (roof) | 3 | 5 | 5 | 6 | 5 | 6 |
| Bolts (m) | L=4.5 S=2 | L=4.5 S=1.7 | L=4.5 S=1.7 | L=4.5 S=1.5 | L=4.5 S=1.7 | L=4.5 S=1.5 |
| Shotcrete (mm) Fr | | 50-90 | 50-90 | 90-120 | 50-90 | 90-120 |
| *Q (walls)* | 25 | 2.7 | 5 | 0.83 | 5.55 | 0.9 |
| Support category (walls) | 1 | 4 | 1 | 4 | 1 | 4 |
| Bolts (m) | | L=3.5 S=2 | | L=3.5 S=2 | | L=3.5 S=2 |
| Shotcrete (mm) | | 40-100 | | 40-100 | | 40-100 |

L – length; S – spacing; NA – not applicable; FR – fibre reinforced.

### 6.4.4 Tetrahedral Wedge Stability Analysis

It has been mentioned earlier in Section 6.4.2 that two major joints sets are prominent within any given length of the tunnel with the third major joint set present at random. The two minor sets are also present at random. Overall, the joint persistence is relatively high.

Several combinations of these high persistence and intersecting joints can form kinematically unstable tetrahedral rock wedges. Since the tunnel is shallow and the in situ stresses are low, movement of these rock wedges under gravity is the most significant stability concern. The kinematically unstable wedges were identified by the UNWEDGE analysis. The ubiquitous joint method was adopted in the analysis because the classification of the rock mass using the two empirical methods also assumed that joints were ubiquitous.

The analysis showed that 19 different combinations of joints had the potential to form tetrahedral rock wedges with an apex height greater than 1 m. Table 6.16 shows the joint set combination, location, maximum apex height and maximum weight of the 19 rock wedges. The factor of safety (FOS) against failure of each wedge was computed using joint shear strength parameters representative of the best and the worst joint surface conditions considered for RMR and Q methods. The Joint shear strength parameters (best: c=10 kPa, $\Phi=30^o$) and (worst: c=0 kPa, $\Phi=20^o$) were estimated taking into account the joint surface conditions and the presence of filling materials discussed in Section 6.4.2. In estimating these parameters, the suggestions given in Section 4 of Table 1 of the paper by Barton & Grimstad (1994) and the potential for water saturation of the joints were also taken into account. Since the tunnel overburden is between 25 and 50 m and the tunnel is to be operated with an internal water pressure, it was assumed that the wedges are subjected to gravity loading only. The FOS of unsupported rock wedges in the best and the worst ground conditions, denoted as F-B and F-W respectively, are given in Table 6.16.

The analysis was extended to examine whether the empirically predicted support could stabilise the 19 theoretically possible rock wedges. Since the wedges may be present in the best or the worst ground conditions, the analysis considered both the best and worst joint shear strength scenarios mentioned earlier. Table 6.16 also shows the FOS assuming that only the bolts recommended by the two empirical methods were installed. The bolts considered were the cement grouted type with 100% bond efficiency and an ultimate tensile strength of 180 kN installed normal to the rock face.

The effect of empirically recommended shotcrete layers was also analysed. The results of the analysis (not provided here) showed that shotcrete increased the FOS of large rock wedges beyond the desired level. (A FOS of 1.5 and 2 for the walls and roof, respectively, were selected for long term stability.) The shotcrete layers would also provide the necessary support for the smaller rock blocks in between bolts and for fractured ground and would meet the desired FOS. The analysis shows that the support predicted by the two methods for the worst ground conditions were adequate to stabilise the theoretically possible tetrahedral rock wedges in the roof and walls.

For the tunnel walls in the best rock mass, the two empirical methods recommended no support. However, the analysis showed that there is potential for 14 different tetrahedral rock wedges in the tunnel walls (Table 6.16). Four of these wedges (# 8, 10, 11 & 15) have a FOS of less than or equal to one indicating potential instability in the best rock mass. Three more wedges (# 9, 12 & 17) have a FOS of less than 1.2, which is considered to be below the acceptable level.

For the tunnel roof in the best rock mass conditions, RMR recommended rock bolts plus mesh and 50 mm of shotcrete where required (mesh and shotcrete are probably for fractured ground, if present, and not for the entire roof). As can be seen from Table 6.16, there is potential for five rock wedges with zero or near zero FOS in the roof. The first three of these wedges, with maximum possible weights of 983, 261 and 585 kN, will have a FOS of less than or equal to one, if RMR recommended rock bolting pattern for the best rock mass conditions is used (Table 6.16). With the same bolting pattern the fourth wedge with a maximum possible weight of 628 kN will have a FOS of only 1.18. The analysis showed that shotcrete would increase the FOS of these possible rock wedges to an acceptable level. However, RMR did not recommend shotcrete for large rock wedges in the roof.

With the Q recommendation for the tunnel roof in the best rock mass conditions the FOS for wedge nos. 1, 3 and 4 are below the acceptable level for long term stability of large rock blocks in the tunnel roof. The Q system did not recommend shotcrete for the roof in the best rock mass.

Table 6.16 Results of UNWEDGE analysis of the CLH tunnel

| Wedge # | Wedge characteristics | | | | | FOS-Best Conditions | | | FOS-Worst Conditions | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sets | Location | Failure mode | Apex(m) | Weight(kN) | F-B | F-BR | F-BQ | F-W | F-WR | F-WQ |
| 1 | 1, 2, 3 | Roof | Falling | 3.9 | 983 | 0.00 | 0.77 | 1.39 | 0.00 | 1.49 | 2.17 |
| 2 | 1, 3, 4 | Roof | Falling | 2.2 | 261 | 0.00 | 1.02 | 1.96 | 0.00 | 2.14 | 2.53 |
| 3 | 1, 2, 4 | Roof | Sliding on 2 | 5.2 | 585 | 0.39 | 0.84 | 1.64 | 0.07 | 1.41 | 1.38 |
| 4 | 2, 4, 5 | Roof | Sliding on 2 | 4.2 | 628 | 0.41 | 1.18 | 1.49 | 0.07 | 1.65 | 2.31 |
| 5 | 1, 3, 5 | Roof | Sliding on 1 | 1.2 | 254 | 0.27 | 2.52 | 4.57 | 0.09 | 6.16 | 7.51 |
| 6 | 2, 3, 5 | R/wall | Sliding on 2/5 | 6.2 | 9140 | 1.66 | 2.30 | 2.60 | 0.82 | 1.96 | 2.34 |
| 7 | 2, 3, 5 | L/wall | Sliding on 2/3 | 6.0 | 7829 | 1.64 | 2.23 | 2.53 | 0.79 | 1.94 | 2.36 |
| 8 | 3, 4, 5 | R/wall | Sliding on 5 | 5.2 | 7328 | 0.87 | 1.55 | 1.85 | 0.37 | 1.51 | 1.85 |
| 9 | 3, 4, 5 | L/wall | Sliding on 3/4 | 5.1 | 6339 | 1.18 | 2.05 | 2.64 | 0.53 | 2.25 | 2.84 |
| 10 | 1, 3, 5 | R/wall | Sliding on 5 | 3.9 | 4227 | 0.88 | 1.51 | 2.16 | 0.38 | 1.76 | 2.25 |
| 11 | 1, 3, 5 | L/wall | Sliding on 3 | 3.9 | 4166 | 1.08 | 2.45 | 3.05 | 0.48 | 2.86 | 3.66 |
| 12 | 1, 3, 4 | L/wall | Sliding on 3 | 3.6 | 1025 | 1.13 | 2.21 | 3.20 | 0.48 | 2.71 | 3.43 |
| 13 | 1, 3, 4 | R/wall | Sliding on 1/4 | 3.6 | 962 | 1.83 | 2.71 | 3.23 | 0.73 | 2.26 | 2.82 |
| 14 | 1, 2, 3 | L/wall | Sliding on 3 | 1.9 | 293 | 1.26 | 2.65 | 4.11 | 0.48 | 2.62 | 3.31 |
| 15 | 2, 4, 5 | R/wall | Sliding on 5 | 2.6 | 278 | 1.02 | 1.58 | 1.57 | 0.38 | 1.39 | 1.88 |
| 16 | 2, 4, 5 | L/wall | Sliding on 2/4 | 2.4 | 260 | 1.94 | 3.21 | 5.01 | 0.51 | 3.44 | 3.78 |
| 17 | 1, 2, 3 | R/wall | Sliding on 1/2 | 1.6 | 196 | 1.14 | 2.44 | 2.55 | 0.17 | 1.88 | 3.24 |
| 18 | 1, 2, 4 | L/wall | Sliding on 2/4 | 1.2 | 47 | 3.05 | 5.59 | 5.82 | 0.51 | 2.56 | 3.68 |
| 19 | 1, 2, 4 | L/wall | Sliding on 1/2 | 1.1 | 36 | 2.25 | 2.29 | 3.15 | 0.17 | 1.30 | 1.30 |

Note: F-B = FOS for best ground conditions with no artificial support; F-W = FOS for worst ground conditions with no artificial support; F-BR = FOS for best ground with 4m long bolts in a 2.5m x 2.5m pattern (as per RMR); F-BQ = FOS for best ground with 5m long bolts in a 2.0m x 2.0m pattern (as per Q); F-WR = FOS for worst ground with 5m long bolts in a 1.5m x 2.0m pattern (as per RMR); F-WQ = FOS for worst ground with 5m long bolts in a 1.5m x 1.5m pattern (as per Q).

### 6.4.5 Assessment of Internal Water Pressure Effects

Two important design considerations for pressurised water tunnels are hydraulic jacking and water leakage. Hydraulic jacking or uplift of the surrounding ground can occur if water pressures imposed within a rock mass are greater than the in situ compressive stresses in the rock mass (Benson, 1989).

In the case tunnel the internal water pressure along the centreline ranges from 0.45 to 0.82 MPa, and the gravity induced vertical stress at crown level (assuming a rock mass density of 25 $kN/m^3$) ranges from 0.61 to 1.07 MPa depending on the overburden thickness. At any given point along the tunnel the internal water pressure is less than the confinement stresses due to overburden. However, since the natural groundwater pressure along the tunnel alignment is less than the internal water pressure, water loss by seepage is possible through open interconnected joints. Further, the seepage may cause instability at the ground surface, particularly on the hill slope. This may be demonstrated by a simple two dimensional numerical model using UDEC.

For this purpose, and also to examine the type of support required to prevent the adverse effects of the internal water pressure on the rock mass around the tunnel, a two dimensional numerical model was run using UDEC. The model was constructed assuming jointed rock from the natural ground surface ignoring the near surface soil profile. Two major joint sets (Sets 2 and 3), which are sub-parallel to the tunnel axis, were included in the model. The model assumed that joint spacing is 2 m and that joints are fully persistent with the best joint shear strength parameters considered earlier. Joint aperture size was varied to reflect the observed site conditions given in Table 6.10.

In situ stresses were assumed to be due to gravity only. The intact rock blocks were assumed to be elastically deformable. Based on the laboratory determined intact rock Young’s modulus and Poisson’s ratio mentioned earlier in Section 6.4.2, intact rock bulk modulus of 30 GPa and shear modulus of 20 GPa were assumed. Estimated joint normal and shear stiffness values of 800 MPa/m and 100 MPa/m, respectively, were used, but were varied to investigate the sensitivity of the model and it was found that the higher the joint stiffness, the lower the maximum displacement of rock blocks. A sample UDEC data file is presented in Appendix B.

Four sections across the tunnel representing different ground profiles and internal water pressures were considered. Two cases were modelled for each section. Case 1: bolts installed as per empirical recommendations for the best ground conditions with steady-state seepage through the rock mass surrounding the tunnel. Case 2: a fully impermeable liner installed covering the entire tunnel periphery.

The results of Case 1 modelling showed that seepage would occur through the rock mass in the four sections considered, even if the joint apertures were at their observed lowest range (0.25-0.5 mm). The Case 1 modelling also showed that, instability may occur at the ground surface (on the hill slope) when the overburden thickness is about 30 m (site average). The possible seepage paths and displacement vectors are shown in Figures 6.8 and 6.9 for a section where the overburden thickness above the crown is 32 m (vertical stress is 0.78 MPa) and internal water pressure is 0.6 MPa.

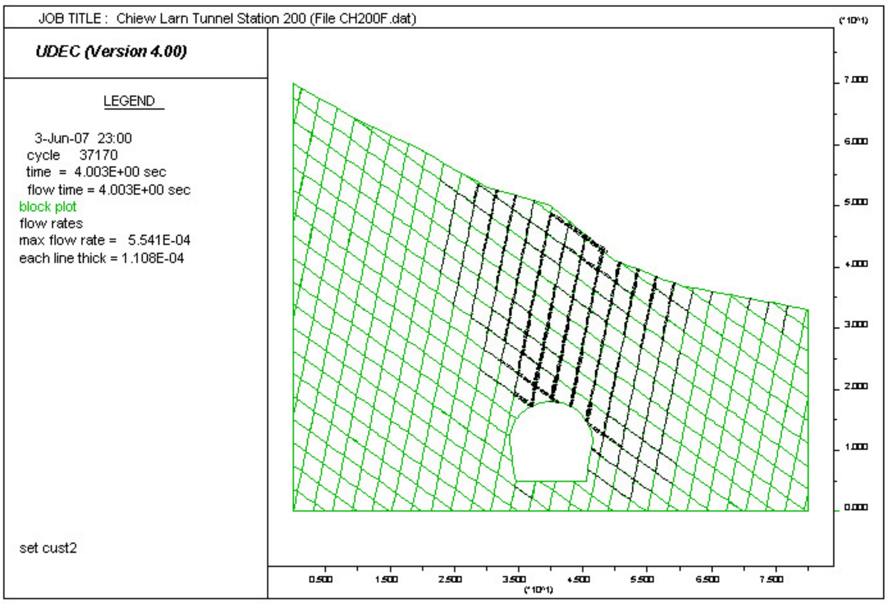


Figure 6.8 Possible seepage paths from the CLH tunnel

Case 2, which assumed no flow through the tunnel periphery, showed insignificant movement at the ground surface. This demonstrates that an impermeable liner is required to prevent water losses from the tunnel and to minimise the risk of instability on the hill slope above the tunnel.

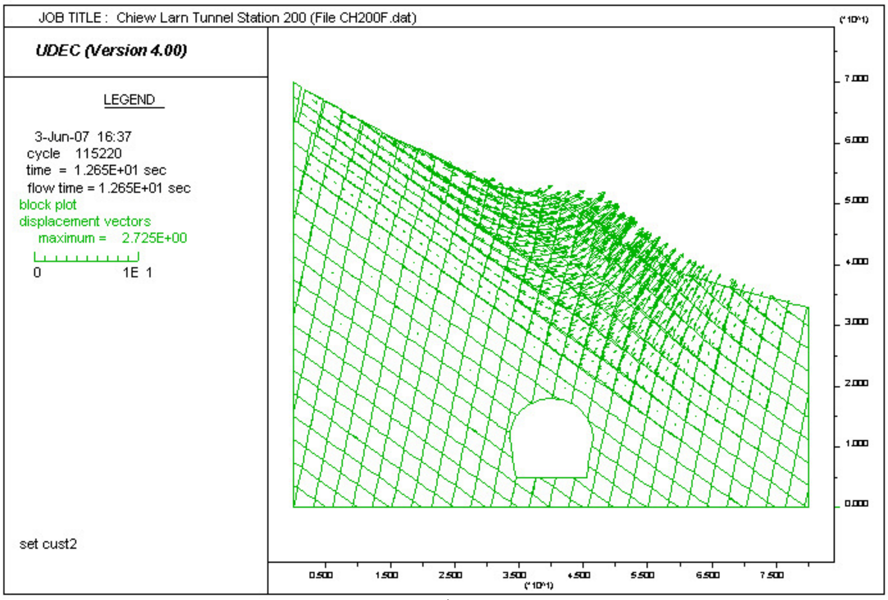


Figure 6.9 Displacement vectors showing ground movement at the surface above the CLH tunnel

### 6.4.6 The Installed Support

During excavation of the tunnel, several potentially unstable large rock blocks were identified within both the best and worst ground conditions described earlier. These blocks were temporarily stabilised using 6, 4 and 3 m long mechanically anchored and resin grouted rock bolts. Considering the potential for leakage losses and hydraulic jacking, the tunnel was fully steel lined with the annulus between the tunnel and the lining filled with concrete. Rock bolting was used only as a temporary rock mass stabilisation measure. No shotcrete was used.

### 6.4.7 Discussion

The support measures predicted by RMR and Q classification methods represent permanent (or long term) requirements and the predictions take into account the purpose of the excavation. For instance, Q (Barton & Grimstad, 1994) recommends an *ESR* value of between 1.6 and 2.0 for hydropower tunnels and a value of 1.8 was used for the case tunnel. Bieniawski (1989) presents an example application of RMR to a shallow water tunnel with an overburden of between 15.3 and 61 m above crown level. This overburden thickness range is comparable to that of the case tunnel.

As mentioned earlier the empirical methods recommended rock bolts and shotcrete for the worst ground conditions intersected in the tunnel. For the best rock mass, no support was recommended for the walls and only bolts were recommended for the roof. Clearly, the empirically derived supports are unlikely to eliminate seepage, which could eventually lead to instability at the ground surface. The application of shotcrete along the entire tunnel including the invert, additional to the empirical recommendations for the worst ground conditions, may be an option to control seepage. However, since shotcrete is known to have numerous incipient cracks formed by shrinkage, expansion and shear movement etc, it may not completely eliminate seepage and the risk of instability at the ground surface. It would perhaps delay the problem, as the pressures tend to build up slowly due to decreased flow through the shotcrete.

Given the project specific requirement to prevent seepage losses the final lining design was not entirely based on stability concerns and is therefore not a reasonable test to evaluate the reliability of the empirical recommendations. However, purely from a stability point of view, the most favourable (best) rock mass surrounding this tunnel provides an interesting example. If a tunnel with similar diameter were to be constructed in this best rock mass for low pressure water conveyance, i.e. for flood or river diversion, the predictions of “no support” by the two rock mass classifications methods would not meet long term stability requirements against potentially unstable large rock wedges, particularly on the tunnel walls.

### 6.4.8 Conclusion

From purely a stability point of view the support recommendations of RMR and Q for the worst rock mass conditions in the tunnel were adequate to prevent structurally controlled failures. However, the recommendations for walls in best rock mass were found to be inadequate for stabilising potentially unstable large rock wedges.

## 6.5 CASE STUDY 3:
## The Huai Saphan Hin Power (HSHP) Tunnel, Thailand

The Huai Saphan Hin project is a multipurpose water resources development project located in the eastern seaboard of Thailand. The 3.5 m wide, 732 m long D-shaped HSHP tunnel feeds a 12.2 MW powerhouse. It has an overburden of between 35 m and 90 m and was excavated by conventional drill and blast methods. Its alignment is N55$^{o}$E with a 1% gradient.

### 6.5.1 Project Site Geology

The Huai Saphan Hin river basin is surrounded by moderately high mountains composed mainly of meta-sedimentary rocks. The major geological structures at the project site are faults and shears, which trend 20 to 40$^{o}$ west of north with a dip range of 60 to 90$^{o}$ towards southwest or northeast. Three major rock formations are present: (1) Shale and sandstones belonging to the Kanchanaburi formation of Silurian, Devonian and Carboniferous ages; (2) Basalts of Tertiary and Pleistocene ages; and (3) Residual soil, alluvium and colluviums of Quaternary to Recent age.

### 6.5.2 Rock Mass Data

Two rock types are present along the tunnel alignment: greywacke and shale of the Kanchanaburi Formation. Greywacke is the main rock type representing about 95% of the tunnel. It consists of fine to medium grained quartz and feldspar fragments cemented in a fine matrix. Shale is present in relatively small amounts and is usually inter-bedded within greywacke. The cumulative thickness of shale is about 35 m (approximately 5% of the tunnel). Along the tunnel alignment, from surface elevation to approximately 28 m depth the ground profile comprises residual soil and completely weathered rock. On average, sound rock is present only from about 28 m below the natural ground surface along the tunnel alignment.

The main rock type, greywacke, can be described as a strong rock with an average intact UCS of 104 MPa. The results of greywacke intact rock material testing are given in Table 6.17.

Table 6.17 Intact rock properties along the HSHP Tunnel

| Property | Range | Mean | Std | # of tests |
|---|---|---|---|---|
| UCS (MPa) | 76-141 | 104 | 27 | 8 |
| Point load index (MPa) | 64-163 | 111 | 26 | 12 |
| E Modulus (GPa) | 42-92 | 66 | 13 | 15 |
| Poisson’s Ratio | 0.17-0.38 | 0.28 | 0.07 | 15 |
| Density (kN/m$^3$) | | 27.2 | | |

Four major geological discontinuity sets (1, 2, 3 and 4) and four minor sets (5, 6, 7 and 8) were identified by mapping. The general orientations of the discontinuity sets are given in Table 6.18.

Table 6.18 Discontinuity orientation data (HSHP tunnel)

| Set # | Dip | Direction | Remark |
|---|---|---|---|
| 1 | 89 | 238 | Major |
| 2 | 89 | 297 | Major |
| 3 | 72 | 087 | Major |
| 4 | 71 | 199 | Major |
| 5 | 38 | 310 | Minor |
| 6 | 11 | 331 | Minor |
| 7 | 09 | 177 | Minor |
| 8 | 88 | 150 | Minor |

The discontinuities present in the tunnel belong to three types: joints, faults and bedding planes. Joints are the most common structural features. Of the 739 discontinuity measurements, 649 are joints. Joint spacing varies from moderate (200-600 mm) to very close (20-60 mm). The joint aperture size ranges from tight (0.1-0.5 mm) to very wide (>10 mm) and they were usually filled with silica and clayey materials. Faults are the next most common structural feature with displacements of 5 to 18 cm. The fault zones contain gouge and fault breccia and bounding surfaces are slickensided. Three subsets of faults, namely, Crushed zones (CZ), Sheared zones (SZ) and Shattered zones (ShZ), are also present. They have fault characteristics, but their offset was not visible. CZ consists of zones of angular rock fragments and plastic clayey material. SZ represent closely spaced, sub-parallel and slickensided shear planes often coated with clay. ShZ are closely fractured and shattered rock zones consisting of angular rock fragments with minor amounts of clay. Generally the faults (including subsets) observed were less than 10 cm in thickness, occasionally up to 30 cm. A total of 72 faults were observed at regular intervals. Bedding planes are rare with only 18 measurements and present only along the contact between the two rock types. Their openings are usually filled with clay and surfaces are generally smooth. The faults (including CZ, SZ and ShZ) and beddings strike northwest (more or less normal to the tunnel axis) and dip 60 to 90$^o$ towards northeast or southwest. They represent discontinuity set 1.

Based on a detailed engineering geological map prepared during the construction, Lasao (1986) divided the tunnel into five geotechnical sectors considering the major discontinuities and groundwater conditions: Sector 1: 011 to 150 m, Sector 2: 150 to 220 m, Sector 3: 220 to 320 m, Sector 4: 320 to 550 m and Sector 5: 550 to 733 m. In Sector 1 heavy water inflow was common due to open joints, and in Sectors 2, 3 and 4 water inflow was limited to dripping. Sector 5 was generally dry with wet surfaces and occasional dripping. During heavy rains a significant increase in water inflow in Sector 1 and a significant increase in dripping in Sectors 3 and 4 were observed. The increase in groundwater inflow was interpreted as an indication of high permeability of the rock mass due to the presence of open discontinuities.

During mapping, Lasao (1986) observed that the most common modes of failure in the tunnel were loosening of the rock mass around major discontinuity zones such as CZ, SZ and ShZ mentioned earlier, wedge failures due to intersecting joints and slab failure from the roof due to flat dipping joints.

### 6.5.3 Support Predictions by the RMR and Q Methods

Lasao (1986) applied $RMR_{79}$ and $Q_{74}$, which were current at the time, to the most favourable (best) and most unfavourable (worst) rock mass conditions in each of the five sectors (domains) of the tunnel. A review of the *RMR* ratings assigned and the brief explanations given by Lasao showed that the ratings accurately represent the rock mass conditions recorded in the tunnel map. Nevertheless, for the present study the worst case *JC* rating for tunnel Sectors 2 and 4 was downgraded to “0”, instead of “6” used by him, so that the selected value is consistent with the ratings recommended by Bieniawski (1979). However, this did not change the RMR recommended support measures for the worst case rock in tunnel Sectors 2 and 4. The RMR ratings are presented in Table 6.19.

Table 6.19 The RMR ratings for the HSHP tunnel

| Sector | Condition | *IRS* | *RQD* | *JS* | *JC* | *GW* | *RA* | *RMR* |
|---|---|---|---|---|---|---|---|---|
| 1 | Best | 12 | 20 | 20 | 20 | 10 | 0 | 82 |
| | Worst | 12 | 13 | 10 | 10 | 4 | 0 | 39 |
| 2 | Best | 12 | 20 | 20 | 20 | 10 | -12 | 70 |
| | Worst | 12 | 13 | 10 | 0 | 4 | -12 | 27 |
| 3 | Best | 12 | 20 | 10 | 12 | 10 | 0 | 64 |
| | Worst | 12 | 8 | 5 | 0 | 4 | 0 | 29 |
| 4 | Best | 12 | 20 | 20 | 25 | 10 | 0 | 87 |
| | Worst | 12 | 17 | 10 | 0 | 4 | 0 | 43 |
| 5 | Best | 12 | 20 | 20 | 25 | 10 | -5 | 82 |
| | Worst | 12 | 17 | 10 | 0 | 7 | -5 | 41 |

Table 6.20 The Q ratings for the HSHP tunnel

| Sector | Condition | *RQD* | *Jn* | *Jr* | *Ja* | *Jw* | *SRF* | *Q* |
|---|---|---|---|---|---|---|---|---|
| 1 | Best | 96 | 9 | 3 | 2 | 1 | 2.5 | 6.40 |
| | Worst | 70 | 15 | 2 | 3 | 0.5 | 2.5 | 0.62 |
| 2 | Best | 96 | 9 | 3 | 2 | 1 | 2.5 | 6.40 |
| | Worst | 70 | 12 | 2 | 3 | 0.66 | 5 | 0.50 |
| 3 | Best | 95 | 9 | 2 | 2 | 1 | 5 | 2.11 |
| | Worst | 32 | 9 | 1 | 6 | 0.66 | 10 | 0.04 |
| 4 | Best | 99 | 9 | 3 | 2 | 1 | 2.5 | 6.60 |
| | Worst | 77 | 12 | 2 | 3 | 0.66 | 2.5 | 1.13 |
| 5 | Best | 98 | 9 | 3 | 2 | 1 | 2.5 | 6.53 |
| | Worst | 77 | 12 | 1.5 | 3 | 1 | 2.5 | 1.28 |

A review of Lasao's tunnel map indicated that some of the *Q* rating values he selected could be downgraded to better represent the rock mass conditions in the tunnel. The reassigned values reduced the final *Q* rating for both best and worst conditions considered in all five sectors except for the worst conditions in Sector 5. The Q ratings are presented in Table 6.20.

Table 6.21 $RMR_{79}$ and $Q_{74}$ recommended support for the HSHP tunnel

| Sector | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| Best case *RMR* value | 82 | 70 | 64 | 87 | 82 |
| Bolts (m) | None | L=2 locally | L=2 locally | None | None |
| Shotcrete (mm) | None | 50 (mr) | 50 (mr) | None | None |
| Worst case *RMR* value | 34 | 27 | 24 | 43 | 41 |
| Bolts (m) | L=2 S=1-1.5 | L=2 S=1-1.5 | L=2 S=1-1.5 | L=2 S=1.5-2 | L=2 S=1.5-2 |
| Shotcrete (mm) | 100-150 (mr) | 100-150 (mr) | 100-150 (mr) | 50-100 (mr) | 50-100 (mr) |
| Steel ribs (m) | S=1.5* | S=1.5* | S=1.5* | None | None |
| Best case *Q* value | 6.40 | 6.40 | 2.11 | 6.60 | 6.53 |
| Bolts/shotcrete | None | None | None | None | None |
| Worst case *Q* value | 0.62 | 0.50 | 0.04 | 1.12 | 1.28 |
| Bolts (m) | L=2 S=1 | L=2 S=1 | L=2 S=1 | None | None |
| Shotcrete (mm) | 50 (mr) | 50 (mr) | 25-50 (mr) | 25-50 (mr) | 25-50 (mr) |

Note: L=length, S=spacing, mr=mesh reinforced, *=light to medium ribs where required.

The RMR recommended support measures are shown in Table 6.21. {Note that since the RMR recommendations are for 10 m diameter tunnels only, the bolt lengths were reduced using the empirical formula proposed by the Norwegian Institute of Rock Blasting Techniques, L=1.4+0.184a, where "L" is bolt length and "a" is the tunnel span. Ref. Stilborg (1994)}. To determine support requirements according to $Q_{74}$ an *ESR* value of 1.6 (for water tunnels) was used. Then the equivalent dimension *De*=(*Span/ESR*)=2.2 m. The $Q_{74}$ recommended support measures are also shown in Table 6.21. Since the tunnel span is 3.5 m and the wall height is only 1.5 m, the support measures recommended for the tunnel roof may be extended to cover the walls. Hence no attempt was made to determine support requirements for the walls.

Table 6.22 $Q_{94}$ recommended support for the HSHP tunnel

| Sector | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| Best case *Q* value | 6.40 | 6.40 | 2.11 | 6.60 | 6.53 |
| Bolts/shotcrete | | No | support | | |
| Worst case *Q* value | 0.62 | 0.50 | 0.04 | 1.12 | 1.28 |
| Bolts (m) | L=2.7 S=1.5-1.7 | L=2.7 S=1.5-1.7 | L=2.7 S=1.5 | None | None |
| Shotcrete (mm) | 50 | 50 | 50-90 (fr) | None | None |

Note: L=length, S=spacing, fr=fibre reinforced

For the present study, $Q_{94}$ (the current version of Q) was also applied and required support measures were determined. A summary of the support measures recommended by $Q_{94}$ is presented in Table 6.22. $RMR_{89}$ was not applied as support measures recommended by the two versions are the same.

### 6.5.4 Tetrahedral Wedge Stability Analysis

As discussed earlier, four major joints sets and four minor joint sets were present in the rock mass. In any given length of the tunnel, generally three joint sets are prominent with the other sets present at random. Intersecting members of these joint sets have the potential to create kinematically unstable tetrahedral rock wedges in the tunnel. The analysis performed using UNWEDGE confirmed that several rock wedges were kinematically unstable in both the roof and walls. The stability of these rock wedges was then analysed. As for Case Study 1 (CLD tunnel), two stress scenarios were considered taking into account that the overburden is between 35 and 90 m with 90% of the tunnel having more than 50m of overburden. The first scenario assumed that the wedges are subjected to gravity loading only; the second scenario included an assumed in situ stress field corresponding to the weight of the overburden and $k=\sigma_h/\sigma_v=1.5$. In order to assess the sensitivity of wedge stability (wedge FOS) to the shear strength parameters of discontinuities, two shear strength scenarios were considered. Since the rock type and its joint filling materials are similar to those of the CLD tunnel (Case Study 1) discussed earlier, it was assumed that the same shear strength values c=50 kPa, $\Phi=35^o$ and c=10 kPa, $\Phi=25^o$ represent, respectively, the best case and the worst case ground conditions considered for classifying the rock mass using RMR and Q. The results of the analysis showed that several rock wedges would be unstable in both roof and walls of the tunnel. The FOS of each wedge under the two joint shear strength scenarios with and without the effect of stress field, along with the joint set combination, location, maximum apex height and maximum weight of the rock wedges, are presented in Table 6.23.

The analysis showed that the RMR recommended support measures for the worst case ground conditions are sufficient for theoretically possible tetrahedral rock wedges in the tunnel. The RMR recommendations for the best rock mass conditions in tunnel Sectors 2 and 3 are also sufficient for tetrahedral rock wedges. RMR did not recommend any support for the best case rock mass in Sectors 1, 4 and 5. The Q recommended support measures for the worst case ground conditions are also sufficient to stabilise the theoretically possible tetrahedral rock wedges. However, Q did not recommend any support for the best case ground conditions considered in the study.

Table 6.23 The potentially unstable rock wedges in the HSHP tunnel

| Wedge | Sets | Location | Failure mode | Apex (m) | Weight (kN) | FOS1 | FOS2 | FOS3 | FOS4 |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 158 | Roof | Sliding | 2.0 | 164 | 0.80 | 1.58 | 0.17 | 1.08 |
| 2 | 168 | Roof | Sliding | 0.7 | 71 | 0.44 | 0.70 | 0.09 | 0.49 |
| 3 | 168 | Roof | Sliding | 0.8 | 96 | 4.86 | 0.74 | 0.98 | 0.51 |
| 4 | 246 | L/wall | Sliding | 0.9 | 30 | 2.38 | 0.99 | 0.54 | 0.65 |
| 5 | 246 | R/wall | Sliding | 0.7 | 15 | 4.12 | 0.88 | 0.83 | 0.83 |
| 6 | 247 | L/wall | Sliding | 1.0 | 29 | 2.96 | 1.18 | 0.66 | 0.77 |
| 7 | 258 | Roof | Sliding | 1.3 | 67 | 2.28 | 1.49 | 0.46 | 0.97 |
| 8 | 268 | Roof | Sliding | 0.7 | 84 | 0.80 | 0.67 | 0.17 | 0.45 |
| 9 | 268 | Roof | Falling | 0.8 | 105 | 4.24 | 0.69 | 0.86 | 0.46 |
| 10 | 358 | Roof | Falling | 1.6 | 105 | 0.00 | 1.38 | 0.00 | 0.89 |
| 11 | 368 | Roof | Sliding | 0.9 | 124 | 0.76 | 0.75 | 0.26 | 0.52 |
| 12 | 368 | Roof | Falling | 0.7 | 67 | 0.00 | 0.66 | 0.00 | 0.45 |
| 13 | 468 | Roof | Sliding | 0.8 | 80 | 5.32 | 0.99 | 1.06 | 0.65 |
| 14 | 568 | R/wall | Sliding | 1.5 | 522 | 3.02 | 1.11 | 1.02 | 0.79 |

FOS1 - c=50 kPa $\Phi=35^o$ $\sigma_h=\sigma_v=0$; FOS2 - c=50 kPa $\Phi=35^o$ $\sigma_h=1.5\sigma_v$;
FOS3 - c=10 kPa $\Phi=25^o$ $\sigma_h=\sigma_v=0$; FOS4 - c=10 kPa $\Phi=25^o$ $\sigma_h=1.5\sigma_v$.

As shown by the wedge analysis, several tetrahedral rock wedges are possible in the tunnel. The FOS of some of these wedges under the two joint shear strength scenarios are well below the acceptable level when the effect of in situ stress field around the tunnel is ignored. On the other hand there are shallow relatively flat wedges which may become unstable due to the effect of the in situ stress field even when the best case joint shear strength scenarios were assumed. If these wedges are present in the tunnel, some form of restraint will be required to prevent failure. For the best case ground conditions, this observation contradicts the Q predictions for the entire tunnel and RMR predictions for Sectors 1, 4 and 5.

During excavation of the HSHP tunnel, Lasao (1986) observed that structurally controlled failures were the main modes of instability which included wedge failures due to intersecting discontinuities, slab failure from the roof due to flat dipping joints and loosening of the rock mass around weakness zones such as faults, which cut across the tunnel alignment. The results of the wedge analysis agree with these observations.

### 6.5.5 The Installed Support

The actual support installed in the tunnel included steel ribs with steel/timber lagging, liner plates and invert struts near the portals and at four intervals inside the tunnel in the worst ground conditions (a 5 m interval each in Sectors 3 and 4, and a 4 m and an 8 m interval in Sector 5). In other areas, rock bolts and wire mesh were used as necessary to provide safety during construction. No shotcrete was used. After the completion of the excavation, a 500 mm thick cast in place concrete lining was constructed, partly for hydraulic reasons. While the rock bolts and wire mesh quantities installed were minimal, the final support (lining) installed was substantially greater than the support recommended by the two empirical methods.

### 6.5.6 Discussion

Both methods recommended rock bolts and shotcrete as the main types of rock mass stabilisations measures for the HSHP tunnel. However, the recommended rock bolt quantities and shotcrete thicknesses vary between the two methods. For the best rock mass conditions in all five sectors of the tunnel, Q recommended no support, whereas RMR recommended spot bolting and 50 mm of shotcrete with occasional wire mesh for Sectors 2 and 3. For the worst rock mass conditions, RMR recommended pattern bolting with 100 to 150 mm of mesh reinforced shotcrete and light to medium steel ribs at 1.5 m spacing where required. $Q_{74}$ recommended pattern bolting with 50 mm of mesh reinforced shotcrete for the first three sectors of the tunnel, but only 50 mm of mesh reinforced shotcrete for the last two sectors. $Q_{94}$ also recommended a similar support system for the first three sectors but no support for the last two. This illustrates that for small diameter tunnels the RMR and Q derived support measures are not always comparable, and depending on the rock mass conditions they differ considerably.

One of the reasons for the above mentioned difference is that RMR support recommendations are for 10 m diameter tunnels only. No guidelines are provided for selection of support quantities for tunnels with dimensions other than 10 m span. Some difficulties may arise when support measures are designed for small diameter tunnels such as the HSHP tunnel. In this situation, other empirical guidelines should be used for selecting bolt lengths, shotcrete thickness and steel ribs spacing etc. While bolt lengths may be easily adjusted using other empirical guidelines or rules of thumb as in this case, the adjustment of bolt and rib spacing, shotcrete thickness and mesh reinforcement etc using other relevant empirical guidelines could result in significant deviation from the RMR recommendations.

The Q system also has some major limitations for small diameter (i.e. 3.5 m) water tunnels such as this, for which $ESR$=1.6 to 2.0 and $De$=1.75 to 2.2. When $De \leq 2$, the Q system recommends no support if $Q>1$. As seen in the HSHP case tunnel, support could be warranted when the $Q$ value is as high as 6.

In this 3.5 m diameter HSHP tunnel, the RMR and $Q_{74}$ derived support measurers were sufficient to stabilise the potentially unstable tetrahedral rock wedges in the worst case ground conditions. However, the $Q_{94}$ recommendations were adequate only for the first three sectors of the tunnel. It did not recommend any support for the last two sectors where wedge instability was possible.

For the best scenario ground conditions, both versions of Q did not recommend any support for the entire tunnel and RMR recommended no support for Sectors 1, 4 and 5. Nevertheless, the wedge analysis showed that even under the best case joint shear strength parameters considered, tetrahedral rock wedge instability was possible. During construction several structurally controlled failures occurred and the potential failures identified in advance were stabilised using rock bolts. Steel ribs were also used in the poor rock mass conditions, for which RMR recommended steel ribs, rock bolts and shotcrete and Q recommended rock bolts and shotcrete.

### 6.5.6 Conclusion

The results of the analysis of the data available from the HSHP tunnel show that the support systems recommended by the two empirical methods were significantly different to the final support (concrete lining) installed for rock mass stability and hydraulic reasons.

This case study also showed that for small diameter tunnels RMR and Q derived support measures are not always comparable and depending on the rock mass conditions they differ considerably.

Both methods have some major limitations for small diameter (i.e. 3.5 m) water tunnels such as the HSHP tunnel. Depending on the condition of the rock mass the RMR system may be overconservative, whereas the Q system could lead to under design.

For the best scenario ground conditions in the entire tunnel and the worst case ground conditions in two of the five sectors, Q did not recommend any support, yet the wedge stability analysis showed that support would be warranted in these areas. During construction primary support measures (rock bolts) were installed in areas where Q did not even recommend permanent support. For the best scenario ground conditions, RMR recommended no support for Sectors 4 and 5, where support would be required for wedge stabilisation.

## 6.6 CASE STUDY 4:
## The Central Tunnel, Taiwan, Republic of China

The Central Tunnel is part of the Southern Link Railway Project (SLRP) in the Southern part of Taiwan, Republic of China. The Central Tunnel, the longest double track railway tunnel in the SLRP, is 8070 m long with a 10 m diameter horseshoe shape. It has a general alignment of N75$^{o}$E and passes underneath Mt Chaliu at the southern end of the Central Mountain Range. Excavated by drill and blast techniques, its construction was completed in 1990. The present study deals with only a 78 m length of the tunnel with difficult ground conditions. The average overburden of the tunnel length is 218 m.

### 6.6.1 Project Site Geology

The project area is characterised by the Lushan Formation of Miocene age, which can be divided into two members: the upper member is mainly argillite with massive meta-sandstone and the lower member consists of argillite with a lesser amount of thinly layered meta-sandstone. The rocks are slightly metamorphosed and rock beds steeply dip to the east and partly overturned to the west. Well developed bedding planes, tight folds, drag folds and cleavages are common in the rock formations in the project area. The surface topography is rough with steep scarps, and the regional tectonic trend has a north-south orientation.

### 6.6.2 Rock Mass Data

The rock types intersected in the tunnel are argillite and meta-sandstone of Lushan Formation. Argillite is the main rock type and is massive. Meta-sandstones are thick and massive or inter-bedded with argillite. Yu-Shan (1987) undertook a detailed rock mass survey of the 78 m length of the tunnel from chainage 25K+904.8m to 25K+982.4m (western portal is at Ch. 23K+258m and eastern portal is at 31K+328m). The survey was conducted by examining and recording all the relevant rock mass parameters after each excavation round. In the 78 m tunnel interval selected for Yu-Shan's study, there were 59 excavation rounds varying in length from 1 to 2.9 m, with the majority having a length of only 1 m. The rock mass survey included measuring and recording of discontinuity orientations, spacing, persistence, wall rock hardness, roughness and aperture size. Discontinuity filling materials, groundwater conditions, rock weathering and *RQD* in each excavation round were also recorded. The *RQD* values estimated using the method suggested by Palmstrom (1982) varied between 10 and 80 with a mean value of 50 as shown in Figure 6.10.

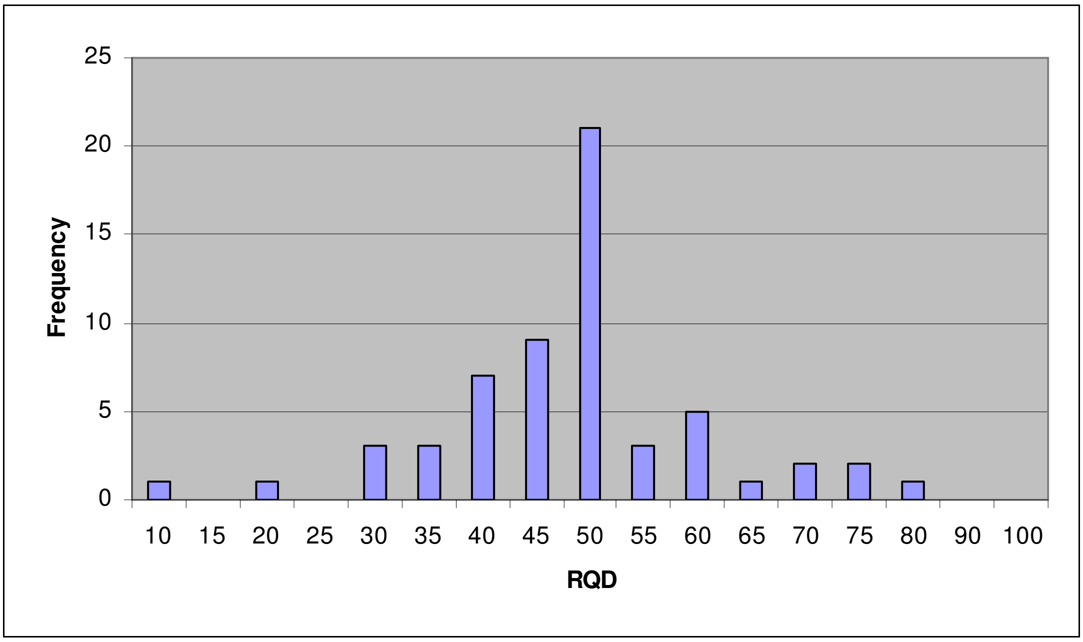


Figure 6.10 Frequency distribution of RQD in the Central Tunnel

Yu-Shan (1987) identified four major joints sets (Sets 1, 2, 3 and 4) and several minor joint sets in the selected tunnel interval. The general orientations (dip and dip direction) of Sets 1, 2, 3 and 4 are 71/020, 72/255, 64/285 and 74/205, respectively. In the 78 m study length of the tunnel, joint spacing varies from close (60-200 mm) to moderate (200-600 mm). Joint aperture size ranges between tight (0.1-0.5 mm) and moderately wide (2.5-10 mm). Joints are usually filled with silica and clay, and their wall surfaces are smooth planar to rough undulating. The tunnel length under consideration was mostly damp except for the last few meters where heavy water inflow caused considerable construction delays.

Yu-Shan also undertook a limited program of laboratory testing on rock samples collected from the tunnel, the results of which are presented in Table 6.24.

Table 6.24 Intact rock material properties of the Central Tunnel

| Property | Range | Mean | Std | No. of tests |
|---|---|---|---|---|
| UCS (MPa) | 41.4 – 74.2 | 54.1 | 14.8 | 4 |
| UTS (MPa) | 2.9 – 6.7 | 4.7 | 1.5 | 6 |
| Point Load Index (MPa) | 1.6 – 3.9 | 2.6 | 0.8 | 10 |
| E Modulus (GPa) | 13 – 25 | 18.3 | 5.4 | 4 |
| Poisons Ratio | 0.14 – 0.25 | 0.21 | 0.05 | 4 |
| Density ($kN/m^3$) | 26.9 - 27.7 | 27.2 | 0.2 | 10 |

### 6.6.3 Support Predictions Using the RMR and Q Methods

As already mentioned, Yu-Shan (1987) geotechnically surveyed each excavation round, the majority of which were only 1 m in length, in the 78 m length of the tunnel. He applied $RMR_{79}$ and $Q_{74}$ to each excavation round so that the most relevant rock mass conditions could be closely observed to assign rating values for the input parameters of the classification methods. The range of rating values assigned to the input parameters of the two classification systems are presented in Table 6.25.

Table 6.25 RMR and Q ratings for the Central Tunnel

| RMR | | Q | |
|---|---|---|---|
| *Parameter* | *Ratings range* | *Parameter* | *Ratings range* |
| *IRS* | 4-7 | *RQD* | 10-80 |
| *RQD* | 3-17 | *Jn* | 9-15 |
| *JS* | 8-10 | *Jr* | 1-3 |
| *JS* | 0-20 | *Ja* | 4-8 |
| *GW* | 0-10 | *Jw* | 1.0-0.33 |
| *RA* | 0 | *SRF* | 2.5 |
| *RMR* | 13-44 | *Q* | 0.042-0.82 |

Table 6.25 shows that despite the fact that only a relatively short length of the tunnel was studied, the ratings assigned to the RMR and Q input parameters vary. Therefore the final *RMR* and *Q* values along the tunnel length also vary. This can be seen from Figure 6.11 which shows the spatial distribution of the *RMR* and *Q* values along the tunnel length. Figure 6.11 also shows that the variation of the *RMR* and *Q* values along the tunnel length resonates with each other.

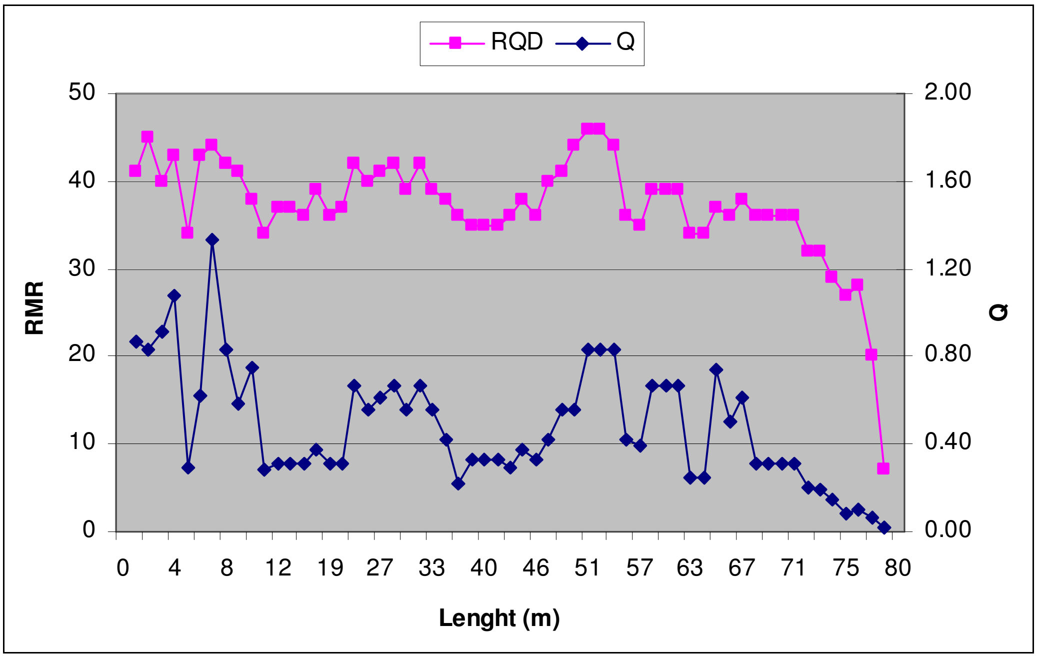


Figure 6.11 Spatial distribution of the RMR and Q values along the Central Tunnel

On the basis of the results of the rock mass classification, Yu-Shan divided the tunnel interval under consideration into seven geotechnical sectors. The seven sectors and their final RMR and Q ratings are given in Table 6.26.

Table 6.26 Sectors and RMR and Q ratings of the Central Tunnel

| Tunnel sector | Length | RMR | RMR Class | Q | Q Class |
|---|---|---|---|---|---|
| (1) 25+904.8 – 913.8 m | 9.0 | 41 | III – Fair | 0.818 | Very Poor |
| (2) 25+913.8 – 925.0 m | 11.2 | 37 | IV – Poor | 0.371 | Very Poor |
| (3) 25+925.0 – 935.9 m | 10.6 | 41 | III – Fair | 0.621 | Very Poor |
| (4) 25+935.9 – 951.9 m | 16.0 | 37 | IV – Poor | 0.367 | Very Poor |
| (5) 25+951.9 – 958.4 m | 6.5 | 44 | III – Fair | 0.722 | Very Poor |
| (6) 25+958.4 – 980.4 m | 22.0 | 35 | IV - Poor | 0.373 | Very Poor |
| (7) 25+980.4 – 982.4 m | 2.0 | 13 | V - Very poor | 0.042 | Extremely poor |

The tunnel support measures derived by RMR for each sector are presented in Table 6.27. In order to determine support measures according to $Q_{74}$, an *ESR* value of 1.0 (for rail tunnels) was used, which gives the following equivalent dimension, *De*, values: *De (roof)*=(*Span/ESR*)=10 m; *De (walls)*=(*Height/ESR*)=5 m. The support measures derived by $Q_{74}$ for each sector are presented in Table 6.28. For the present study $Q_{94}$ was also applied and support requirements were determined (Table 6.29).

Table 6.27 The RMR recommended support for the Central Tunnel

| Tunnel sector | 1, 3 & 5 | 2, 4 & 6 | 7 |
|---|---|---|---|
| RMR value & class | 41-44 (III) | 35-37 (IV) | 13 (V) |
| *Roof support* | | | |
| Bolts (m) | L=4 S=1.5-2 | L=4-5 S=1-1.5 | L=5-6 S=1-1.5 |
| Shotcrete (mm) | 50-100 (mr) | 100-150 (mr) | 150-200 (mr) |
| Steel sets (m) | None | S=1.5* | S=0.75# |
| *Wall support* | | | |
| Bolts (m) | L=4 S=1.5-2 | L=4-5 S=1-1.5 | L=5-6 S=1-1.5 |
| Shotcrete (mm) | 30 | 100 (mr) | 150-200 (mr) |
| Steel sets (m) | None | S=1.5* | S=0.75# |

Note: L= length (m), S=spacing (m), mr=mesh reinforced, * light to medium steel where required.
# medium to heavy sets with steel lagging and fore-poling, if required, and bolt & close invert.

Table 6.28 $Q_{74}$ recommended support for the Central Tunnel

| Tunnel sector | 1, 3 & 5 | 2, 4 & 6 | 7 |
|---|---|---|---|
| Q value & class | 0.62-0.82 (Very Poor) | 0.37 (Very Poor) | 0.042 (Ext poor*) |
| *Roof support* | | | |
| Bolts (m) | L=3 (utg) S=1 | None | L=3 (tg) S=1 |
| Shotcrete (mm) | 50-75 (mr) | 75-250 (mr) | 200-750 (mr) |
| *Wall support* | | | |
| Bolts (m) | L=3 (utg) S=1 | None | L=3 (tg) S=1 |
| Shotcrete (mm) | 25-50 (mr) | 50-75 (mr) | 150-250 (mr) |

Note: L=length, S=spacing, utg=un-tensioned grouted, tg=tensioned grouted. mr=mesh reinforced,
* steel reinforced cast concrete arch and several bolt lengths also recommended.

Table 6.29 $Q_{94}$ recommended support for the Central Tunnel

| Tunnel sector | 1, 3 & 5 | 2, 4 & 6 | 7 |
|---|---|---|---|
| Q value & class | 0.62-0.82 (Very Poor) | 0.37 (Very Poor) | 0.042 (Ext poor*) |
| *Roof support* | | | |
| Bolts (m) | L=3 S=1.5-1.7 | L=3 S=1.5-1.7 | L=3 S=1.2 |
| Shotcrete (mm) | 90-120 (Fr) | 90-120 (Fr) | >150 (Fr, RR) |
| *Wall support* | | | |
| Bolts (m) | L=2.4 S=1.5 | L=2.4 S=1.5-1.7 | L=2.4 S=1.2 |
| Shotcrete (mm) | 50-90 (Fr) | 90-120 (Fr) | 120-150 (Fr) |

Note: L=length, S=spacing, Fr=fibre reinforced, RR=rib reinforced.

### 6.6.4 Tetrahedral Wedge Stability Analysis

The tetrahedral rock wedge analysis performed using UNWEDGE showed that several rock wedges are kinematically unstable, however, only three are large enough to warrant a stability analysis. To assess the sensitivity of the stability of these wedges to joint shear strength, two strength scenarios were considered: the best scenario c=20 kPa and $\Phi$=35$^o$ and the worst scenario c=10 kPa and $\Phi$=20$^o$. The best scenario was determined using the empirical relationship of Barton and Choubey (1977) discussed in Chapter 2 and given below:

$$\tau_p = \sigma_n \tan \{\Phi_b + JRC \operatorname{Log}_{10}(JCS/\sigma_n)\} \qquad (2.7)$$

where $\tau_p$ = shear strength; $\sigma_n$=joint normal stress; *JRC*=joint roughness coefficient; *JCS*=joint compressive strength. From the data presented by Yu-Shan (1987), the following values were selected for the determination of joint shear strength: *JCS*=54 MPa, *JRC*=2.5, $\Phi_b$=30$^o$ (assumed based on test results reported in Case Study 1), and $\sigma_n$=5.4 MPa (vertical stress due to gravity). These values return c=20 kPa and $\Phi$=31$^o$. These joint shear strength values were validated using the "frictional component only" (c=0) relationship $\Phi=\tan^{-1}$(Jr/Ja) suggested by Barton (2002). For the tunnel interval studied, the *Jr* parameter was given a rating from 1 to 3 inclusive and the *Ja* parameter was given a rating from 4 to 8 inclusive. For the best case *Jr* and *Ja* ratings $\Phi$(best)=$\tan^{-1}$(3.0/4.0)=37$^o$. This is comparable to the above quoted strength values. For the worst and average *Jr* and *Ja* ratings $\Phi$(worst)=$\tan^{-1}$(1.0/8.0)=7$^o$ and $\Phi$(avg)=$\tan^{-1}$(2.0/6.0)=18$^o$. These joint friction angles show that the selected worst case joint shear strength parameters are not unrealistically low for the purpose of this study.

The analysis also considered two field stress scenarios. The first assumed that the wedges are subjected to gravity loading only with no effect from the in situ stress field. The second included an inferred in situ stress field assumed to be due to the weight of the overlying rock/soil only with $k=\sigma_h/\sigma_v=1.5$. The FOS of the three wedges under the best and worst joint shear strength scenarios with and without the effect of in situ stress field are presented in Table 6.30. The effect of the empirically derived support on the three wedges was also analysed. The results showed that the

RMR and Q recommended support (i.e. rock bolts and shotcrete) are sufficient to stabilise the largest possible tetrahedral rock wedges in the Central Tunnel.

Table 6.30 Potentially unstable wedges in the Central Tunnel

| Wedge | Sets | Failure mode | Apex (m) | Weight (kN) | FB | FBS | FW | FWS |
|---|---|---|---|---|---|---|---|---|
| 1 | 124 | Sliding | 2.6 | 114 | 0.90 | 2.67 | 0.46 | 1.39 |
| 2 | 134 | Sliding | 3.3 | 232 | 0.85 | 2.87 | 0.43 | 1.49 |
| 3 | 234 | Falling | 15.5 | 1772 | 1.06 | 8.63 | 0.53 | 4.49 |

FB: FOS when c=20 kPa $\Phi=35^o$ $\sigma_h=\sigma_v=0$ FBS: FOS when c=20 kPa $\Phi=35^o$ $\sigma_h=1.5\sigma_v$
FW: FOS when c=10 kPa $\Phi=20^o$ $\sigma_h=\sigma_v=0$ FWS: FOS when c=10 kPa $\Phi=20^o$ $\sigma_h=1.5\sigma_v$

### 6.6.5 The Installed Support

The support measures installed in the studied interval of the tunnel were designed based on the New Austrian Tunnelling Method (NATM). The design included a support performance monitoring system comprising pressure cells, convergence measuring points and borehole extensometers. In all the seven sectors, the support installed during excavation included rock bolts, wire mesh, shotcrete and steel sets. In sectors 2, 4 and 6 forepoling was also used occasionally. In sector 7 forepoling was installed at 300 mm spacing where required. The support measures installed are given in Table 6.31. The forepoling used were 420 mm diameter steel bars.

Table 6.31 Support measures installed in the Central Tunnel

| Tunnel sector | 1, 3, 5 | 2, 4, 6 | 7 |
|---|---|---|---|
| Bolts (m) | L=3 S=1.5-1.7 | L=4 S=1.2-1.5 | L=5 S=1-1.5 |
| Wire mesh (mm) | 6x100x100 | 6x100x100 | 6x100x100 |
| Shotcrete (mm) | 150 | 150 (face 50) | 200 (invert 100, face 50) |
| Steel sets (m) | H150 S=1.5-1.8 | H150 S=1.2-1.5 | H150 S=0.75-1.0 |
| Forepoling (mm) | None | Occasionally | 300 c/c where required |

Note: L= length (m), S=spacing (m).

### 6.6.6 Discussion

In sectors 1, 3 and 5 of the tunnel length studied the support installed exceeded the RMR recommendations. For these three subsections, RMR did not recommend steel ribs, but steel ribs were installed during construction. Further, RMR recommended a shotcrete layer considerably thinner than the installed shotcrete thickness of 150 mm. Only the RMR recommended rock bolting pattern is comparable to the installed bolting pattern. The support measures installed in tunnel sector 7 are virtually the same as those recommended by RMR. In sectors 2, 4 and 6 the RMR recommendations are comparable to the support measures installed (see Tables 6.27 and 6.31).

The $Q_{74}$ recommendations on the other hand are not comparable to the actual support installed except for the sector 7 for which $Q_{74}$ recommends rock bolts plus 200 to 750 mm of mesh reinforced shotcrete. $Q_{74}$ also recommended steel reinforced cast concrete arch as an option for sector 7. In the case of sectors 1, 3 and 5, bolt lengths and spacing recommended by $Q_{74}$ were comparable to the actual bolting pattern installed. Nevertheless, $Q_{74}$ did not recommend steel ribs. Furthermore $Q_{74}$ recommended a shotcrete thickness much less than the installed shotcrete thickness. For sectors 2, 4 and 6, $Q_{74}$ recommended only shotcrete and no rock bolts or steel sets. As can be seen from Table 6.31, both rock bolts and steel sets were installed during construction.

$Q_{94}$ recommendations for Sectors 1 to 6 inclusive comprised 3 m long rock bolts at 1.5 to 1.7 m spacing and 90 to 120 mm of fibre reinforced shotcrete (Table 6.29). For Sector 7 in addition to the bolts and shotcrete, reinforced ribs were also recommended. As can be seen from Tables 6.29 and 6.31 the actual support measures installed far exceeded those recommended by $Q_{94}$.

Although the UNWEDGE analysis showed that the empirically determined support measures are sufficient to stabilise the potentially unstable rock wedges, instability in the tunnel during construction was not controlled by large rock wedges. The main mode of failure during construction was loosening and unravelling due to the fractured nature of the rock mass. This is clearly evidenced from the fact that the 78 m length of the tunnel required 59 excavation cycles with lengths varying from 1 to 2.9 m, the majority having a length of only 1 m. Moreover, the tunnel sectors 2, 4, 5 and 7 required forepoling.

### 6.6.7 Conclusion

The support measures installed in approximately 65% the tunnel section studied are generally comparable to those recommended by RMR. For the remainder (approximately 34%) of the tunnel length studied, the support installed exceeded the RMR predictions. The Q recommendations are not comparable to the RMR recommended support and the actual support measures installed far exceeded the Q recommendations.

## 6.7 CASE STUDY 5:
## The Lam Ta Khong Exploratory (LTKE) Tunnel, Thailand

The LTKE tunnel is part of the Lam Ta Khong pumped storage project constructed between 1992 and 1997. The project is situated on the Lam Ta Khong River about 200 km northeast of Bangkok and involved the construction of the first underground power station in Thailand. The major project components included several caverns and tunnels. During the detailed design stage of the project, the D-shaped 3 m high, 3 m wide and 1340 m long exploratory tunnel was excavated by conventional drill and blast methods to investigate the rock mass conditions in the project site. The tunnel is horizontal with an alignment of 110$^{o}$ to the north, and the tunnel intervals from 83 to 103 m and 158 to 179 m were located beneath a winding highway. The tunnel overburden at the portal is about 15 m, at the far end is about 275 m, and under the highway is between 20 and 30 m. Only the tunnel length from 110 m to the far end at 1340 m was considered in the present study.

### 6.7.1 LTKE Project Site Geology

The bedrock in the project area composed of Phu Kradung Formation and Phra Wihan Formation. The former comprised siltstone, fine-grained sandstone and conglomerate, and the latter comprised coarse-grained sandstone, claystone and an alternating sequence of fine-grained sandstone and siltstone. The rock beds are sub-horizontal with a strike direction of N30$^{o}$W to N70$^{o}$W and a dip of 5 to 10$^{o}$ NE. The bedrock formation is overlain by a several metres thick residual soil cover. Up to 30 m thick colluvial and talus deposits are present in the lower reaches of the hill slopes where the exploratory tunnel portal is located.

### 6.7.2 LTKE Tunnel Rock Mass data

From the portal to about 110 m, the materials intersected in the tunnel are colluviums and talus deposits consisting pieces of sandstone and siltstone ranging from gravel size to large boulders in a very stiff to hard clay matrix. In the tunnel interval from 110 m to the far end at 1340 m an alternating sequence of muddy siltstone and sandy siltstone is present. The average intact rock UCS of muddy siltstone is 32 MPa and that of sandy siltstone is 68 MPa. The rock intersected in the tunnel can be described as medium strong to strong rock according to ISRM suggested methods (ISRM, 1978).

The siltstone and sandstone intersected from 110 m onwards are bedded rocks with well developed bedding planes. From the data collected by mapping four discontinuity sets were identified. The orientations of the discontinuity sets are given in Table 6.32.

Table 6.32 Major discontinuity sets in the LTKE tunnel

| Set | Dip | Direction |
|---|---|---|
| North dipping | 00 – 35 | 000 – 035 |
| South dipping | 00 – 35 | 165 – 225 |
| West dipping | 00 – 45 | 255 – 315 |
| East dipping | 00 – 30 | 090 – 135 |

North dipping and south dipping joints are consistently present throughout the tunnel. They occur as single joints or as zones of several joints. The zones are usually 200 to 500 mm thick. Since the tunnel alignment is approximately 110$^{o}$ to the north, these north and south dipping joints strike sub-parallel to the tunnel axis. They are closely spaced, continuous, slickensided, undulating and have clay, calcite or gypsum infilling materials with a general thickness of 1 to 5 mm with an occasional maximum of 20 mm. They represent the bedding plane joint set. The west dipping joint set has three sub sets with dip ranges from 0 to 30$^{o}$, 30 to 45$^{o}$ and >45$^{o}$. Persistence of these joints varies from 3 to 10 m. Their surfaces are planar, fresh to slightly weathered and mostly tight, but sometimes slickensided and has a clay gouge, calcite or gypsum filling of less than 50 mm in thickness. Often steeply dipping members of this set splay off from the flatter ones dipping more or less in the same direction. The east dipping joint set has a smaller joint population compared to the others and may be a part of the west dipping set. Their surface characteristics are similar to those of the west dipping set.

All these four joint systems are consistent throughout the tunnel with spacing ranging from 300 to 1500 mm with the majority having spacing between 300 to 700 mm.

These joints are thought to have been formed by or undergone shearing. In addition to these shear joints, a sub-vertical tension joint set striking east-west is also present in the rock mass. These tension joints are discontinuous, rough, tight and generally clean with no filling.

### 6.7.3 Support Predictions for the LTKE Tunnel Using RMR and Q

In the LTKE tunnel the two classification methods were applied from Ch. 110 m to the far end at 1340 m using the data collected by engineering geological mapping after each round of excavation and testing of intact rock samples. The mapping was carried out by project site personnel. From 110 m to 275 m, Prapphal (1993) classified the rock mass and Tran (1994) continued from 275 m onwards. However, they presented only part of the tunnel map. Ratings given to the RMR and Q input parameters in each excavation round were not provided. Only the final *RMR* and *Q* values for each round are presented. They used $RMR_{89}$ and $Q_{74}$. Spatial variation of the *RMR* and *Q* values along the tunnel alignment is shown in Figure 6.12.

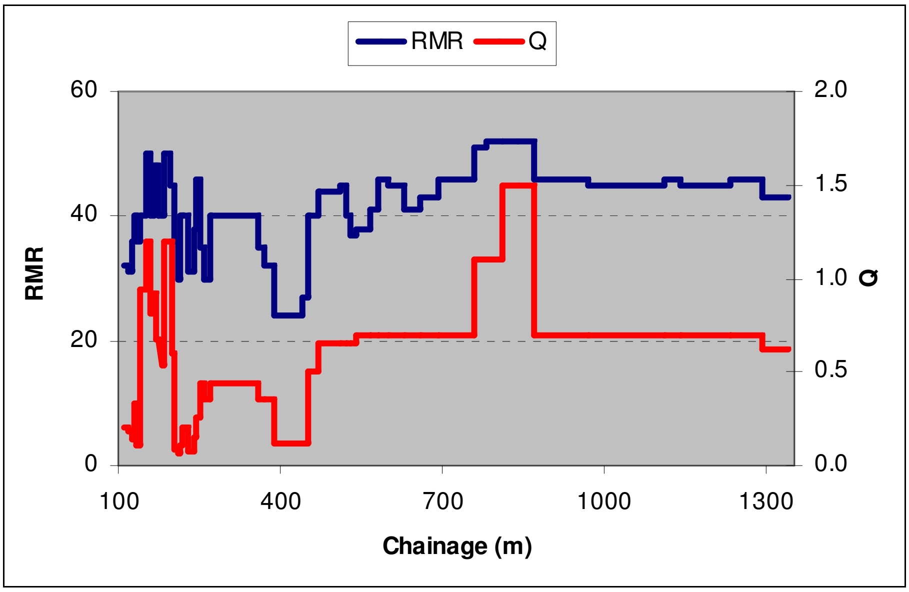


Figure 6.12 Spatial variation of the RMR and Q values along the LTKE tunnel

Figure 6.12 shows that in general the *RMR* and *Q* values along the tunnel resonate with each other. The figure also shows that the variation of both the *RMR* values (24 to 52) and the *Q* values (0.2 and 1.5) is restricted to relatively narrow ranges. The percentage of rock mass falling into each relevant RMR and Q classes are shown in

Figure 6.13 (Note that the descriptive terms poor, fair and good are used in both methods, but they do not necessarily mean identical rock mass conditions.).

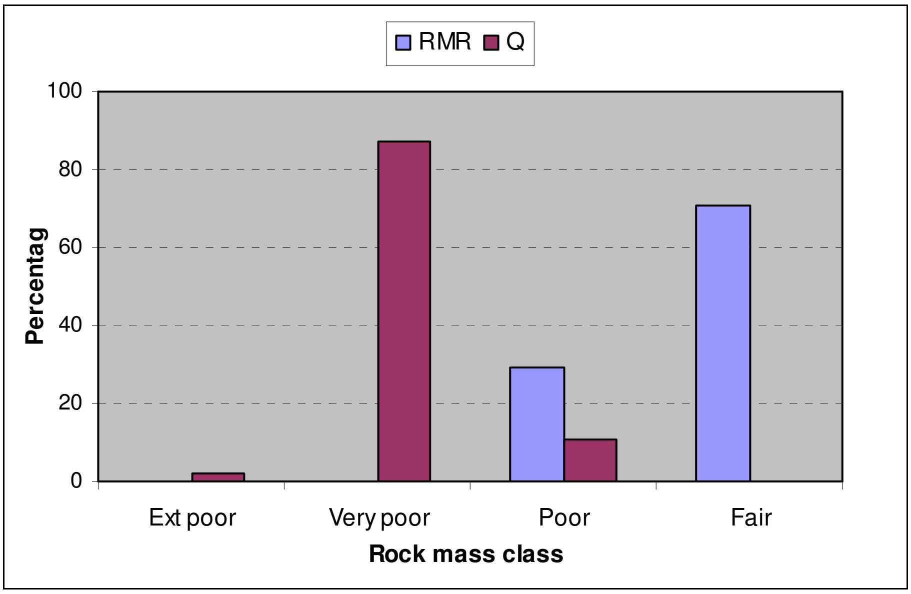


Figure 6.13 Percentage of RMR and Q classes in the LTKE tunnel

After classifying the rock mass, the adjoining excavation rounds falling into the same rock mass classes were grouped. This led to the division of the 1230 m length of the tunnel (from 110 to 1340 m) into ten geotechnical sectors (domains). The ten sectors and the results of the classification of the rock mass according to $RMR_{89}$ and $Q_{74}$ are presented in Table 6.33 and Table 6.34, respectively.

Table 6.33 Application of $RMR_{89}$ to the LTKE Tunnel

| Chainage (m) | RMR | Rock Mass Class | Bolts (m) | Shotcrete (mm) |
|---|---|---|---|---|
| 110 – 150 | 31 – 40 | IV - Poor rock* | L=2 S=1-1.5 | 100-150 (mr) |
| 150 – 160 | 50 | III - Fair rock | L=2 S=1.5-2 | 50-100 (mr) |
| 160 – 165 | 40 | IV - Poor rock | L=2 S=1-1.5 | 100-150 (mr) |
| 165 – 175 | 48 | III - Fair rock | L=2 S=1.5-2 | 50-100 (mr) |
| 175 – 185 | 40 | IV - Poor rock | L=2 S=1-1.5 | 100-150 (mr) |
| 185 – 205 | 45 – 50 | III - Fair rock | L=2 S=1.5-2 | 50-100 (mr) |
| 205 – 470 | 24 – 40 | IV - Poor rock | L=2 S=1-1.5 | 100-150 (mr) |
| 470 – 530 | 41 – 46 | III - Fair rock | L=2 S=1.5-2 | 50-100 (mr) |
| 530 – 565 | 37 – 38 | IV - Poor rock | L=2 S=1-1.5 | 100-150 (mr) |
| 565 – 1340 | 41 – 52 | III - Fair rock | L=2 S=1.5-2 | 50-100 (mr) |

* For poor rock, light to medium steel where required, mr=mesh reinforced.

RMR classified 29% of the rock mass in the tunnel as poor rock and 71% as fair rock. Q classified 7% of the rock mass in the tunnel as extremely poor rock, 82% as very poor rock and the remaining 11% as poor rock.

Table 6.34 Application of $Q_{74}$ to the LTKE Tunnel

| Chainage (m) | Q | Rock Mass Class | Rock Bolts (m) | Shotcrete (mm) |
|---|---|---|---|---|
| 110 – 150 | 0.11 – 0.94 | Very poor | L=2 S=1 (tg) | 50 (mr) |
| 150 – 160 | 1.2 | Poor | None | None |
| 160 – 185 | 0.54 – 0.81 | Very poor | L=2 S=1 (tg) | 50 (mr) |
| 185 – 200 | 1.2 | Poor | None | None |
| 200 – 205 | 0.6 | Very poor | L=2 S=1 (tg) | 50 (mr) |
| 205 – 215 | 0.06 – 0.09 | Extremely poor | L=2 S=1 (tg) | 25-50 (mr) |
| 215 – 230 | 0.11 – 0.2 | Very poor | L=2 S=1 (tg) | 50 (mr) |
| 230 – 240 | 0.07 | Extremely poor | L=2 S=1 (tg) | 25-50 (mr) |
| 240 – 760 | 0.12 – 0.7 | Very poor | L=2 S=1 (tg) | 50 (mr) |
| 760 – 870 | 1.13 – 1.5 | Poor | None | None |
| 870 – 1340 | 0.62 – 0.7 | Very poor | L=2 S=1 (tg) | 50 (mr) |

Note: tg=tensioned grouted, mr=mesh reinforced.

For the poor rock class, $RMR_{89}$ recommended systematic bolting at 1 to 1.5 m spacing plus 100 to 150 mm of mesh reinforced shotcrete and light to medium steel ribs spaced at 1.5 m. For fair rock, $RMR_{89}$ recommended the same bolting pattern plus 50 to 100 mm of mesh reinforced shotcrete. Since the RMR recommended bolts lengths are for 10 m diameter tunnels only, for the case tunnel, bolt lengths were estimated using the formula L=1.4+0.184a, where "L" is bolt length and "a" is tunnel span, proposed by the Norwegian Institute of Rock Blasting Techniques.

In order to determine the support requirements according to $Q_{74}$, it was assumed that *ESR*=3 (temporary opening), hence the equivalent dimension, *De*=1. For extremely poor rock, which represents only about 7% of the tunnel, depending on the block size (*RQD/Jn*), $Q_{74}$ would recommend 2 m long tensioned grouted systematic bolts in a 1 m grid plus 25 to 50 mm of wire mesh reinforced shotcrete, or 50 to 100 mm of wire mesh reinforced shotcrete without rock bolts. For very poor rock $Q_{74}$ would recommend the same bolt pattern plus 50 mm of wire mesh reinforced shotcrete. According to $Q_{74}$ no support is required for the poor rock, which represents approximately 11% of the tunnel. Even if a lower *ESR* value is used (i.e. 1.8 for pilot tunnels) the recommended support measures would not have been significantly changed.

Table 6.35 Application of $Q_{94}$ to the LTKE Tunnel

| Chainage (m) | Q | Rock Mass Class | Rock Bolts (m) | Shotcrete (mm) |
|---|---|---|---|---|
| 110 – 150 | 0.11 – 0.94 | Very poor | L=1.5 S=1.3 | 50 (Fr) |
| 150 – 160 | 1.2 | Poor | None | None |
| 160 – 185 | 0.54 – 0.81 | Very poor | None | None |
| 185 – 200 | 1.2 | Poor | None | None |
| 200 – 205 | 0.6 | Very poor | None | None |
| 205 – 215 | 0.06 – 0.09 | Extremely poor | L=1.5 S=1.2 | 50-90 (Fr) |
| 215 – 230 | 0.11 – 0.2 | Very poor | None | 50 (mr) |
| 230 – 240 | 0.07 | Extremely poor | L=1.5 S=1.2 | 50-90 (Fr) |
| 240 – 760 | 0.12 – 0.7 | Very poor | None | None |
| 760 – 870 | 1.13 – 1.5 | Poor | None | None |
| 870 – 1340 | 0.62 – 0.7 | Very poor | None | None |

Note: Fr=fibre reinforced

In the present study $Q_{94}$ was also applied. For the extremely poor class of rock, which represents approximately 7% of the tunnel, $Q_{94}$ recommended pattern bolting in a 1.2 m grid and 50 to 90 mm of fibre reinforced shotcrete (Table 6.35). For the remainder of the tunnel, $Q_{94}$ did not recommend any support when $ESR$=3. If $ESR$ is taken as 1.8, $Q_{94}$ would recommend pattern bolting and fibre reinforced shotcrete for the extremely poor class and also for the very poor class of rock when $Q$<0.4. It should be noted that the total length with $Q$<0.4 represents about 22.5% of the tunnel. This means $Q_{94}$ would recommend no support for 77.5% of the tunnel.

### 6.7.4 Rock Instability in the LTKE Tunnel

The kinematic rock wedge analysis of the four discontinuity sets listed in Table 6.32 showed that no tetrahedral wedges are possible in the roof of the tunnel and the wedges formed in the side walls are inherently stable due to the gentle dip of the discontinuities. The tension joints present in the tunnel were not included in the analysis as these joints are not continuous. Nevertheless, the presence of sub-horizontal and sub-vertical joints in the rock mass has the potential to create polyhedral rock blocks or slabs. Tran (1994) observed that the main modes of instability in the exploratory tunnel were loosening of rock from tunnel crown and walls due to the presence of these discontinuities. Very often rock falls occurred not in the form of true wedges (Tran, 1994). In the crown, slab or block falls occurred through intact rock failure while flat dipping bedding planes provided separation (release plane). The sub-vertical tension joints parallel to the tunnel axis (striking east-west), although discontinuous, may also have contributed to the slab or block falls from the tunnel roof as illustrated in Figure 6.14.

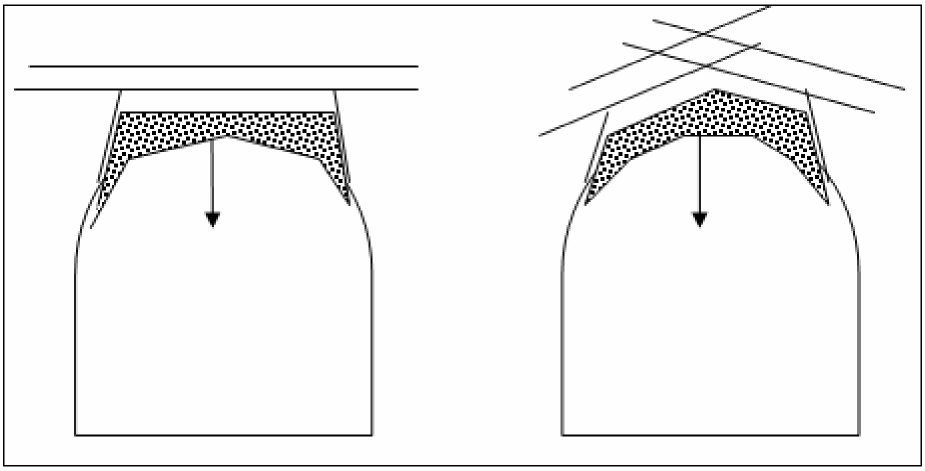

Figure 6.14 Modes of rock falls in the LTKE tunnel

Other modes of instability in the exploratory tunnel include immediate and delayed stress induced rock slabbing in side walls and water dependent slaking and swelling. Stress slabbing was more noticeable when the tunnel depth exceeded 150 m below ground surface, i.e. from Station 945 m onwards where inferred in situ stress levels were higher than in the rest of the tunnel. The vertical stress at Stations 945 and 1340 m was estimated to be 4 and 7 MPa and corresponding $\sigma_H$ normal to the tunnel axis is 8 and 14 MPa respectively (dry unit weight of the rock is 2.7 t/m$^3$, k=2 and the respective overburden is 150 and 275 m). In this length the tunnel walls below the spring line were not covered with shotcrete. Thin rock slabs of 5 to 100 mm thickness were formed up to a depth of about 0.5 m into the side walls. Stress slabbing was also seen in the tunnel face. Slaking was observed in siltstone when in contact with water or moisture. In general, the tunnel was mostly dry, except for some isolated wet patches, yet slaking occurred apparently due to the moisture in the air. Water used for drilling is considered to have contributed to the high moisture content during excavation. However, the slaking was restricted to walls not covered with shotcrete and not significant to compromise safety during the required operating life of the LTKE tunnel.

### 6.7.5 The Actual Support Installed

In the first 110 m of the tunnel, which was driven through colluvial and talus deposits, heavy steel support plus timber lagging were used. From 110 to 1340 m depending on the rock mass conditions the support included steel sets, shotcrete with or without wire mesh and rock bolts. A summary of the support installed in this length of the tunnel is presented in Table 6.35. From 110 to 187 m the support installed included steel sets with 0.6 to 1.2 m spacing and shotcrete with a nominal thickness of 70 mm. For the tunnel intervals from 95 to 116 m and 158 to 179 m, located under the highway, 0.6 m spaced steel sets and a 250 mm thick shotcrete layer were used. The closer steel set spacing and thicker shotcrete layer were to reduce the risk of long term ground settlement and damage to the highway. Therefore the support installed in these two intervals cannot be directly compared with those predicted by the classification approach. (Note that part of the first tunnel interval beneath the highway was located within colluvial material.)

Table 6.35 Support installed in the LTKE Tunnel

| Chainage (m) | Rock bolts (m) | Shotcrete (mm) | Steel sets (m) | Remarks |
|---|---|---|---|---|
| 110-116 | | 250 | H125 S=0.6 | Transition zone |
| 116-136 | | 70 | H125 S=0.6 | |
| 136-143 | | 70 | H125 S=1.2 | |
| 143-179 | | 250 | H125 S=0.9 | Highway above |
| 179-187 | | 70 | H125 S=1.2 | |
| 187-378 | | 70 (mr) | | |
| 378-389 | | 70 | H125 S=0.6 | |
| 389-400 | L=1.5; S=1-1.5 | 70 | H125 S=1.2 | |
| 400-420 | L=1.5; S=1-1.5 | 70 | | |
| 420-430 | | 70 | H125 S=1.2 | |
| 430-780 | L=1.5; S=1-1.5 | 70 | | |
| 780-1340 | L=1.5; S=1-1.5 | 70 | | Walls unsupported |

Note: L= length (m), S=spacing (m), mr=mesh reinforced.

From chainage 187 to 378 m (191 m interval) the actual support installed were 70 mm of shotcrete with wire mesh reinforcement, from 378 to 389 m and from 420 to 430 m steel sets and 70 mm of shotcrete, from 389 to 400 m steel sets rock bolts and shotcrete, from 400 to 420 m and from 430 to 1340 m bolts and shotcrete (Table 6.35).

### 6.7.6 Discussion

From chainage 187 to 378 m the actual support installed were 70 mm of shotcrete with wire mesh reinforcement without rock bolts. Both $RMR_{89}$ and $Q_{74}$ recommended support for this interval included systematic rock bolting. In addition, for poorer sections of this interval, $RMR_{89}$ also recommended light to medium steel ribs spaced at 1.5 m. This means the $RMR_{89}$ and $Q_{74}$ recommendations for the 191 m interval were more conservative than those actually installed. For tunnel intervals from 378 to 400 m and 420 to 430 m support installed included steel ribs, rock bolts and shotcrete. These support measures are closely comparable to the $RMR_{89}$ recommended support. The $Q_{74}$ recommended support does not include steel sets and can be considered insufficient for this interval. Apart from the above, the support measures installed from 430 to 1340 m and from 400 to 420 m (76% of the tunnel) were comparable to those predicted by $Q_{74}$. The only difference is that although $Q_{74}$ recommended wire mesh reinforced shotcrete, no wire mesh was used with shotcrete for this length (76% of the total length) of the tunnel. Again the $RMR_{89}$ recommended support measures are closely comparable to the support installed within this length.

For the extremely poor class of rock, which represents approximately 7% of the tunnel, $Q_{94}$ recommended pattern bolting in a 1.2 m grid and 50 to 90 mm of fibre reinforced shotcrete (Table 6.35). For the remainder of the tunnel, $Q_{94}$ did not recommend any support when $ESR$=3. If $ESR$ is taken as 1.8, $Q_{94}$ would recommend pattern bolting and fibre reinforced shotcrete for the very poor class of rock when $Q$<0.4. It should be noted that the total length with $Q$<0.4 is about 22.5% of the tunnel. This means $Q_{94}$ would recommend no support for 77.5% of the tunnel. From the foregoing it is apparent that for small diameter (~3 m) tunnels such as

LTKE the current version of the Q system, $Q_{94}$, recommended less support compared to the earlier version, $Q_{74}$.

### 6.7.7 Conclusion

In the exploratory tunnel, support measures installed from 430 to 1340 m and from 400 to 420 m (76% of the tunnel) were comparable to those predicted by $RMR_{89}$ and $Q_{74}$ the only difference being that although $Q_{74}$ recommended wire mesh reinforced shotcrete, no wire mesh was used with shotcrete in these intervals. In the remaining 24% of the tunnel, the support measures recommended by the two empirical methods were more conservative than those installed. The only exceptions were the intervals from 378 to 400 m and 420 to 430 m, for which the $Q_{74}$ recommendation falls short of the support installed. However, the comparison showed that $Q_{94}$ did not recommend any support for 77.5% of the tunnel. Even if *ESR* is taken as 1.8 (pilot or water tunnel), $Q_{94}$ would recommend pattern bolting and fibre reinforced shotcrete for only 22%. For this small diameter tunnel the current version of the Q system, $Q_{94}$, is less conservative compared to the earlier version, $Q_{74}$, and did not recommend adequate support.

## 6.8 CASE STUDY 6:
## The Lam Ta Khong Powerhouse Access (LTKPA) Tunnel, Thailand

The LTKPA tunnel is the main access route to the underground power station of the Lam Ta Khong pumped storage project discussed in Section 6.7. The tunnel is 1060 m long and has a D-shape. The excavated diameter of the tunnel is 6.8 m and the finished diameter is 6 m. The tunnel alignment is approximately 107$^{o}$ with a downward slope of 1:12 (V:H) towards the power station. The tunnel was excavated by conventional drill and blast methods. Its overburden varies from about 15 m at the entrance portal to approximately 350 m at the powerhouse end. Construction of the tunnel took place between October 1994 and October 1995. The tunnel interval from 94 m to 125 m is located under a highway. For this study only an 885 m length of the tunnel (from 180 m to 1065 m) was considered.

### 6.8.1 Project Site Geology

The geology of the project area is described in Section 6.7.1.

### 6.8.2 LTKPA Tunnel Rock Mass Data

According to Sriwisead (1996), from the portal to 180 m, the tunnel was driven through talus material and highly weathered siltstone. Slightly weathered siltstone intersected in the tunnel interval from 180 to 220 m. From 220 m to the powerhouse end, a fresh rock sequence consisting of sandy siltstone and sandstone is present. Sandstone contains 2 to 20 mm thick micaceous seams and occasional 5 to 30 mm peat or lignite bearing layers. Sandstone beds are 200 to 1000 mm thick. Sandy siltstone is an inter-bedded rock consisting of 100 to 1500 mm thick beds of siltstone with occasional beds of sandstone. The results of the intact rock material testing conducted at various stages of the project were reported by Sriwisead (1996) and Praphal (1993). A summary of the relevant results is presented in Table 6.36.

Five discontinuity sets are present in the rock mass. In any selected interval of the tunnel, typically two to three sets are present with others at random. The discontinuity orientations vary along the tunnel. The average orientations of the five sets are presented in Table 6.37. The flat dipping bedding plane set (Set 1) with a dip range of 3 to 10$^{o}$ is the most prominent in the entire length of the tunnel. Bedding is well developed with a consistent orientation. Bedding planes are defined by thin alternating seams of mica and mud with smooth planar to slickensided planar surface characteristics (Sriwisead, 1996).

Table 6.36 Intact rock material properties of the LTKPA tunnel

| Property | Sandstone | Siltstone |
|---|---|---|
| UCS (MPa) | 20-100 | 20-80 |
| E Modulus (GPa) | 25-33 | 20-22 |
| Poisson's Ratio | 0.29 | 0.16 |
| Density (kN/m$^3$) | 25.5 | 25.9 |
| Tensile strength (MPa) | 7.6 | 9.3 |

Table 6.37 Orientations of discontinuity sets (LTKPA tunnel)

| Set # | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| Dip | 7 | 83 | 82 | 29 | 32 |
| Direction | 285 | 207 | 298 | 320 | 231 |

Sets 2 and 3 are major joint sets consistently present throughout the 885 m length of the tunnel. Their spacing varies from 500 to 2000 mm. The two sets are continuous across the tunnel and are mostly tight and undulating to planar. In siltstone, they are sometimes slickensided with a 2 to 8 mm thick calcite infilling, and their surfaces are slightly weathered to fresh. Joint Sets 4 and 5, developed only in siltstone, are mostly tight, slickensided and planar with calcite or gypsum infill material. Set 1 has a significant effect on the stability of the tunnel as it forms rock slabs, particularly in the crown. The sub-vertical joints (Sets 2 and 3) in combination with Set 1 were responsible for flat roofs, stepped over-breaks and block falls (Gurung, and Iwao, 1998). The in situ stress levels at relevant depths were determined by hydraulic fracturing tests conducted in boreholes. The tests indicated that the major principal stress is horizontal ($\sigma_H$) and normal to the tunnel axis (Sriwisead, 1996; Nitaramorn, 1997). The intermediate principal stress is vertical ($\sigma_v$) and is slightly greater than the minor principal stress ($\sigma_h$). The $\sigma_H$:$\sigma_v$ ratio, k, is interpreted to be in the vicinity of 2. The tunnel was mostly dry, except for localized wet areas with water dripping along sub-vertical discontinuities, mostly in sandstone.

Generally, the rock mass behaviour was favourable for tunnelling and excavation progressed steadily without major delays. However, some rock mass instability occurred during excavation. This was governed by the geological structure, in situ stress conditions, intact rock strength, and to a lesser extent by blast damage (Sriwisead, 1996; Gurung, and Iwao, 1998). The main mode of instability reported during excavation of the tunnel was rock block and wedge failure due to the presence of discontinuities, particularly sub-horizontal bedding planes and near vertical joints with slickensided surfaces. Stress induced rock slabbing also occurred on side walls located between Sta. 570 and 880 m and formed in siltstone where shotcreting was initially limited only to the tunnel crown. Minor slaking and swelling were also observed in siltstone when in contact with water.

Table 6.38 Ranges of RMR and Q ratings assigned to the LTKAP tunnel

| RMR | | Q | |
|---|---|---|---|
| Parameter | Ratings range | Parameter | Ratings range |
| *IRS* | 2-7 | *RQD* | 60-98 |
| *RQD* | 13-20 | *Jn* | 6-12 |
| *JS* | 10-15 | *Jr* | 1-2 |
| *JC* | 10-25 | *Ja* | 1-3 |
| *GW* | 7-15 | *Jw* | 0.66-1 |
| *RA* | 5-10 | *SRF* | 1-5 |
| *RMR* value | 44-67 | *Q* value | 1.91-30 |

### 6.8.3 Application of RMR and Q to the LTKPA Tunnel

During excavation of the tunnel, the rock mass intersected was classified according to the RMR and Q methods. At the time $RMR_{89}$ and $Q_{74}$ were applied. Since the portal to 180 m the material encountered were talus deposits and highly weathered rocks, the two methods were not applied for that interval. The results of the rock mass mapping and classification were compiled by Sriwisead (1996). Based on the rock types and conditions intersected in the 885 m length of the tunnel, it was divided into 26 geotechnical sectors (domains). The available data were reviewed and, where necessary, the classification parameter ratings were downgraded to better reflect the poor rock conditions described in the tunnel map. The ranges of ratings assigned are presented in Table 6.38 and spatial variation of the final RMR and Q values along the tunnel are shown in Figure 6.15.

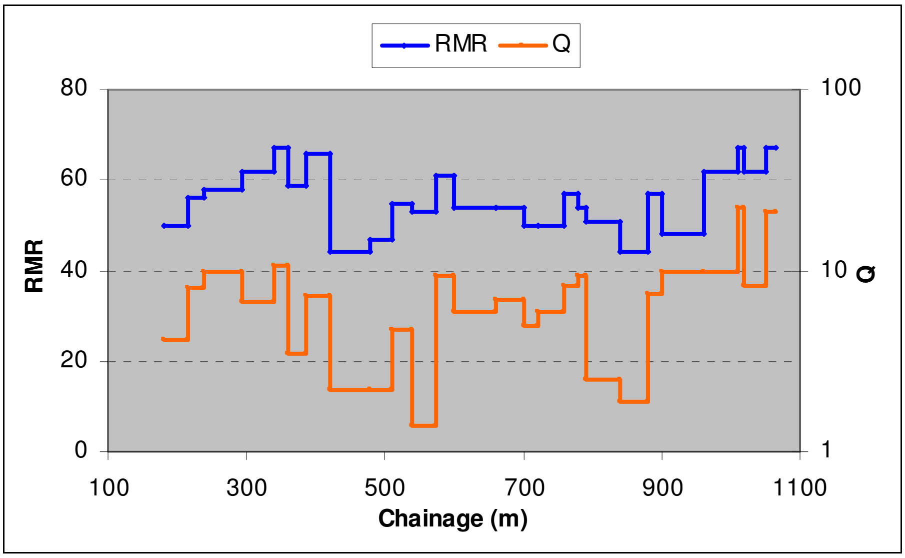


Figure 6.15 RMR and Q values along the LTKPA tunnel

Figure 6.15 shows that in general the RMR and Q values along the tunnel resonate with each other. The figure also shows that while the RMR values range is narrow (44 to 67) the Q values range is relatively wider (1.6 and 40). The amount of rock mass falling into each relevant RMR and Q classes are shown in Figure 6.16 (Note that the descriptive terms poor, fair and good are used in both methods, but they do not necessarily mean identical rock mass conditions.). From Figure 6.16, it is evident that RMR classified 74% (655 m length) of the tunnel as fair rock and the remaining 26% (230 m) as good rock. Q classified 27% (238 m) as poor rock, 56% (497 m) as fair rock and the remaining 17% (150 m) as good rock.

For each relevant class of rock along the tunnel, support measures were derived using $RMR_{89}$, $Q_{74}$ and $Q_{94}$ versions. Table 6.39 presents summaries of the $RMR_{89}$, $Q_{74}$ and $Q_{94}$ recommended support for the different rock mass classes in the tunnel. Since RMR recommendations given in the literature are for 10 m span tunnels only, the bolt lengths (L) were adjusted using the empirical formula: L=1.40+0.184a, where "a" is tunnel span. With Q, an Excavation Support Ratio (*ESR*) of 1.2 (for access tunnels) was used. Hence the equivalent dimension *De*=5.66. The detailed listing of the support derived by $RMR_{89}$, $Q_{74}$ and $Q_{94}$ for tunnel intervals with different *RMR* and *Q* values are presented in Tables 6.40, 6.41 and 6.42 respectively.

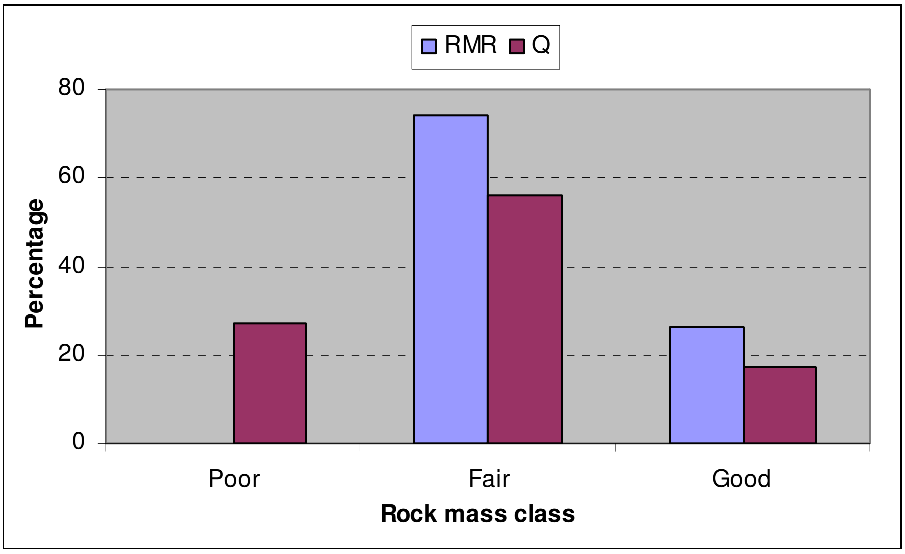


Figure 6.16 The amount of rock in each RMR and Q class (LTKAP tunnel)

Table 6.39 Summary of the $RMR_{89}$, $Q_{74}$ and $Q_{94}$ derived support for the LTKPA tunnel

| | | | | |
|---|---|---|---|---|
| Tunnel length (%) | 26% | 74% | N/A | N/A |
| $RMR_{89}$ Bolts (m) | L=2 S=2.5 | L=2 S=1.5-2 | | |
| $RMR_{89}$ Shotcrete (mm) | 50 (mr)* | 50-100 (mr) | | |
| Tunnel length (%) | 19% | 24% | 46% | 11% |
| $Q_{74}$ Bolts (m) | None | L=2.8 S=1-1.5 | None | L=2.8 S=1 |
| $Q_{74}$ Shotcrete (mm) | None | None | 25-50 | 25-50 (mr) |
| Tunnel length (%) | 21% | 47% | 21% | 11% |
| $Q_{94}$ Bolts (m) | None | L=2.5 S=1.6-2 | L=2.5 S=1.7-2.1 | L=2.5 S=1.7-2.1 |
| $Q_{94}$ Shotcrete (mm) | None | None | 40-100 | 40-100 (Fr) |

L=length, S=spacing, mr=mesh reinforced, NA=not applicable; Fr=fibre reinforcement; *=occasional

It can be seen from Table 6.39 that RMR recommended a support system comprising rock bolts and mesh reinforced shotcrete for the entire tunnel length. $Q_{74}$ and $Q_{94}$ recommended support for only 81% and 79% of the tunnel and their recommendations somewhat differ. (The use of fibre reinforcement instead of mesh in the current version is not considered a major difference as it only reflects the advanced technology.)

Table 6.40 $RMR_{89}$ recommended support for the LTKPA tunnel

| Chainage (m) | RMR value | Rock Mass Class | Rock bolts (m) | Shotcrete (mm) |
|---|---|---|---|---|
| 180-295 | 50-58 | III Fair | S=1.5-2 | 50-100 (mr) |
| 295-360 | 62-67 | II Good | S=2.5 | 50 |
| 360-385 | 59 | III Fair | S=1.5-2 | 50-100 (mr) |
| 385-420 | 66 | II Good | S=2.5 | 50 |
| 420-575 | 44-55 | III Fair | S=1.5-2 | 50-100 (mr) |
| 575-600 | 61 | II Good | S=2.5 | 50 |
| 600-960 | 44-57 | III Fair | S=1.5-2 | 50-100 (mr) |
| 960-1065 | 62-67 | II Good | S=2.5 | 50 |

L=length, S=spacing, mr=mesh reinforced

Table 6.41 $Q_{74}$ recommended support for the LTKPA tunnel

| Chainage (m) | Q value | Rock Mass Class | Rock bolts (m) | Shotcrete (mm) |
|---|---|---|---|---|
| 180-340 | 4.2-8.0 | Fair | L=2.8 S=1-1.5 | Nil |
| 340-360 | 10.7 | Good | L=2.8 S=1.5-2 | Nil |
| 360-385 | 3.5 | Poor | | 25-75 |
| 385-420 | 7.2 | Fair | L=2.8 S=1.5-2 | Nil |
| 420-480 | 1.6 | Poor | L=2.8 S=1 | 20-50(mr) |
| 480-510 | 2.2 | Poor | | 25-50 |
| 510-792 | 4.7-9.4 | Fair | | 20-30 |
| 792-840 | 2.5 | Poor | | 25-50 |
| 840-880 | 1.9 | Poor | L=2.8 S=1 | 20-50(mr) |
| 880-900 | 7.6 | Fair | | 20-30 |
| 900-1065 | 12.5-22.5 | Good | | Nil |

L=length, S=spacing, mr=mesh reinforced

Table 6.42 $Q_{94}$ recommended support for the LTKPA tunnel

| Chainage (m) | Q value | Rock Mass Class | Rock bolts (m) | Shotcrete (mm) |
|---|---|---|---|---|
| 180-215 | 4.2 | Fair | L=2.5 S=2.1 | 40-100 |
| 215-340 | 6.7-8.0 | Fair | L=2.5 S=2 | |
| 340-360 | 10.7 | Good | Nil | |
| 360-385 | 3.5 | Poor | L=2.5 S=2.1 | 40-100 |
| 385-420 | 7.2 | Fair | L=2.5 S=1.8 | |
| 420-480 | 1.6 | Poor | L=2.5 S=2.1 | 50-90 (Fr) |
| 480-540 | 2.2-4.7 | Poor | L=2.5 S=2.1 | 40-100 |
| 540-700 | 5.9-9.4 | Fair | L=2.5 S=1.8-2 | |
| 700-720 | 5.0 | Fair | L=2.5 S=2.1 | 40-100 |
| 720-792 | 6.0-9.4 | Fair | L=2.5 S=1.8-2 | |
| 792-840 | 2.5 | Poor | L=2.5 S=2.1 | 40-100 |
| 840-880 | 1.9 | Poor | L=2.5 S=2.1 | 50-90 (Fr) |
| 880-900 | 7.6 | Fair | L=2.5 S=1.8 | |
| 900-1065 | 12.5-22.5 | Good | Nil | |

L=length, S=spacing, Fr=fibre reinforcement

### 6.8.4 The Support Measures Installed

For this tunnel, the support measures were designed using the CRIEPI empirical system discussed in Section 4.1.6. The support measures installed in the 885 m tunnel length were 2 m long rock bolts plus shotcrete with or without mesh reinforcement. The installed support measures are listed in Table 6.43.

From Sta. 180 to 230 m driven in slightly weathered siltstone, the bolt spacing used was 1.2 m and the mesh reinforced (MRF) shotcrete thickness was 150 mm. From Sta. 230 to 280 m in fresh siltstone, the bolt spacing was 1.5 m and the MRF shotcrete thickness was 100 mm. From Sta. 280 to 540 m in good quality sandstone, relatively less support quantities were used. Notable in this area was the absence of mesh reinforcement. In the interval between Sta. 370 to 417 m the shotcrete thickness was reduced to 70 mm and between Sta. 435 to 454 m, no rock bolts were installed. Again between Sta. 945 to 1035 m in good quality sandstone, no mesh was installed. In the remainder of the tunnel length studied the main rock type was siltstone, where rock bolts and MRF shotcrete were installed.

Table 6.43 Support measures installed in the LTKPA tunnel

| Chainage (m) | Rock bolts | Shotcrete (mm) | Chainage (m) | Rock bolts | Shotcrete (mm) |
|---|---|---|---|---|---|
| 180-230 | L=2 S=1.2 | 150 (mr) | 480-514 | Sb | 100 |
| 230-280 | L=2 S=1.5 | 100 (mr) | 514-540 | L=2 S=2* | 100 |
| 280-370 | L=2 S=1.8 | 100 | 540-600 | L=2 S=1.5 | 100 (mr) |
| 370-390 | L=2 S=2 | 70 | 600-700 | L=2 S=1.8 | 100 (mr) |
| 390-410 | L=2 S=2.5 | 70 | 700-815 | L=2 S=1.5 | 100 (mr) |
| 410-417 | L=2 S=2.5* | 70 | 815-900 | L=2 S=1.8 | 100 (mr) |
| 417-435 | L=2 S=2 | 100 | 900-945 | L=2 S=1.8 | 100 (mr) |
| 435-454 | | 100 | 945-965 | L=2 S=1.8 | 100 |
| 454-458 | L=2 S=2 | 100 | 965-1035 | L=2 S=2* | 100 |
| 458-480 | L=2 S=2* | 100 | 1035-1065 | L=2 S=2 | 100 (mr) |

S=bolt spacing in meters, mr=mesh reinforced, *=bolting in crown only, bolt length 2 m

Apart from the above mentioned variations, the applied shotcrete thickness was 100 mm. Typically the number of bolts per section was 6 (mainly in the crown), but varied between 4 and 8. The bolt spacing varied from 1.2 to 2.5 m with a typical spacing range of 1.5 to 2 m. Shotcrete was applied to the entire 885 m length of the tunnel and 60% was initially reinforced with welded wire mesh. The bolt spacing, shotcrete thickness and mesh reinforcement along the tunnel are graphically presented in Figure 6.17. Note that zero bolt spacing in Figure 6.17 means no bolts were installed.

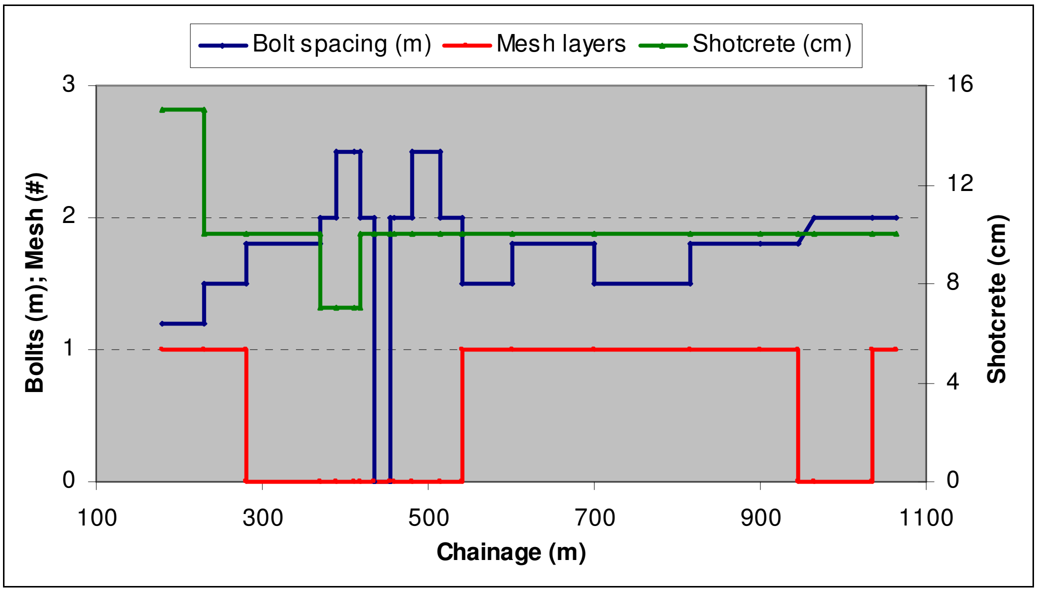


Figure 6.17 The support installed in the LTKPA tunnel

*6.8.4.1 Performance of the Installed Support*

The support system used in the tunnel worked satisfactorily (Gurung, and Iwao, 1998), except for some short intervals between Sta. 280 and 540 m where no mesh reinforcement was installed with shotcrete. In this area the shotcrete layer was damaged and additional support measures were installed to repair it. The damage was manifested by both longitudinal and transverse cracks in shotcrete, and was interpreted to be due to overloading of the support system. Notable shotcrete damage was observed in the following areas: Sta. 280 to 295 m, Sta. 340 to 374 m, Sta. 390 to 410 m, Sta. 462 to 469 m, Sta. 523 to 526 m and at Sta. 532 m Sriwisead (1996).

Areas with significant shotcrete damage were re-spayed with an additional 100 mm layer of shotcrete and depending on the severity of cracking, mesh reinforcement and additional rock bolts were also installed. The areas where additional support measures were installed are listed in Table 6.44.

Table 6.44 Areas where additional support installed

| Station (m) | Initial support | Additional support |
|---|---|---|
| 280-288 | B; S 70 mm of URF | S 100 mm URF |
| 288-291 | B; S 70 mm of URF | S 100 mm MRF |
| 291-295 | B; S 70 mm of URF | S 100 mm URF |
| 340-350 | B; S 100 mm URF | S 100 mm URF |
| 350-354 | B; S 100 mm URF | S 100 mm MRF |
| 354-374 | B; S 100 mm URF | S 100 mm URF |
| 390-416 | S 70 mm URF | B; S 100 mm URF |
| 462-469 | B; S 100 mm URF | S 100 mm MRF |
| 514-526 | SB; S 100 mm URF | B; S 100 mm MRF |

B=pattern bolts; SB=spot bolts; S=shotcrete; MRF=mesh reinforced; URF=un-reinforced

### 6.8.5 Assessment of the RMR and Q Derived Support Measures

To assess the adequacy of the RMR and Q derived support for stabilizing the potential rock instability in the tunnel, the following approaches were used:

- Suspended beam analysis,
- Tetrahedral wedge analysis,
- Numerical simulation, and
- Comparisons with the installed support.

*6.8.5.1 Suspended beam analysis*

The presence of sub-horizontal bedding planes and near vertical joints parallel to and normal to the tunnel axis creates a structural setup for rock beams to be formed in the roof. The support required for stabilizing the potentially unstable rock beams was determined by a conservative analysis using the suspended beam concept discussed in Section 3.4.1.2. The analysis assumed that the weight of the rock in the unstable zone is supported entirely by the force developed in the rock bolts anchored in the overlying solid rock. Ignoring the effect of shear and flexural strengths of the strata and the in situ stress field around the tunnel, the required bolt spacing was determined by Equation 3.3.

$$s = (T/\gamma\ h\ F)^{1/2} \tag{3.3}$$

where $s$=rock bolt spacing on both longitudinal and transverse directions, $T$=ultimate load capacity per rock bolt, $h$=thickness of unstable rock beam, $\gamma$=unit weight of the rock, and $F$=required factor of safety against failure for long term roof stability, which is assumed to be 2.0 for this study.

As mentioned in Section 6.7.2, sandstone and sandy siltstone beds are up to 1 m and 1.5 m in thickness, respectively. Their respective unit weights are 25.5 and 25.9 kN/m$^3$. If 100% bond efficiency cement grouted bolts with an ultimate tensile strength of 180 kN were to be installed normal to the rock face, the required bolt spacing for 1.5 m thick sandy siltstone beds is 1.5 m and that for 1 m thick sandstone beds is 1.8 m.

It can be seen from Table 6.39 that the bolt spacing recommended by $RMR_{89}$ for 74% of the tunnel and that by $Q_{94}$ for 83% of the tunnel are comparable to the range of bolt spacing (1.5 to 1.8 m for the maximum bed thicknesses) determined by the beam analysis. The $Q_{74}$ recommended rock bolts for 64% of the tunnel with somewhat conservative spacing. For 26% of the tunnel RMR recommended a bolt spacing of 2.5 m. Since RMR also recommended 50 mm of mesh reinforced shotcrete, the combined support system is likely to be sufficient to control beam instability. For 46% of the tunnel in poor class of rock $Q_{74}$ recommended only shotcrete. This may be considered adequate for preventing beam failures in this rock class. However, for 19% and 21% of the tunnel in good rock mass class $Q_{74}$ and $Q_{94}$ recommended no support although beam instability is possible in this rock class.

*6.8.5.2 Tetrahedral rock wedge stability analysis*

A tetrahedral rock wedge stability analysis of the LTKPA tunnel rock mass was undertaken using UNWEDGE. To compute wedge factors of safety (FOS) the shear strength parameters of the discontinuities were estimated considering their surface characteristics described earlier. The selected parameters were c=20kPa and $\Phi=30^{o}$. Two stress scenarios were considered. First the analysis assumed that the wedges are subjected to gravity loading only with no effect from the in situ stress field. It then included an in situ stress field assumed to be due to the weight of the overlying rock with k=2.

Table 6.45 Kinematically possible rock wedges in the LTKPA tunnel roof

| Joint sets | Apex height (m) | Weight (kN) | FOS1 | FOS2 |
|---|---|---|---|---|
| 123 | 1.1 | 73 | 0.63 | 0.80 |
| 234 | 1.3 | 53 | 0.98 | 1.09 |
| 235 | 0.6 | 13 | 1.32 | 0.83 |

The analysis showed that several rock wedges are kinematically possible and most of them are stable under the joint shear strength and in situ stress conditions considered. The details of three roof wedges that have no significant stabilizing effect from the in situ stress field are listed in Table 6.45. The wedge factors of safety without and with the effect of the stress field are given as FOS1 and FOS2, respectively, when the overburden thickness is about 100 m.

The stability of these wedges was then examined under the support recommended by the two classification methods. The analysis showed that, RMR derived support for the entire 885 m length of the tunnel and Q derived support for poor and fair classes of rock are sufficient for stabilizing the possible rock wedges. However, $Q_{74}$ and $Q_{94}$ recommended no support for approximately 19% and 21% of the tunnel in good rock class where wedge instability was possible.

It should be noted that the analysis considered only the maximum size rock wedges that can be formed. The actual wedges may be smaller and may fall between the rock bolts installed in a standard pattern. To address this possibility, spot bolting and/or shotcreting may be required. However, the Q recommendation for fair rock is pattern bolting only (no shotcrete) and that for good rock class is no support. These two rock mass classes represent 73% of the tunnel.

*6.8.5.3 Numerical modelling using UDEC*

For jointed rock formations with Q values between 0.1 and 100, UDEC is considered suitable for two dimensional simulations of the behaviour of rock mass around an underground opening (Barton, 1996). In the case tunnel, the *Q* values ranged between 1.9 and 40, hence UDEC (Version 4.0) could be used for verifying the adequacy of the support derived by the two classification methods.

Four sections of the tunnel resembling the conditions at Stations 410, 529, 670 and 830 m, reported by Sriwisead (1996), were modelled. The section details are presented in Table 6.46 and two dimensional representations of the discontinuities at each section are presented in Figures C1 to C4 in Appendix C. The simulation assumed k=2, the intact rock blocks are elastically deformable and the joints follow the Coulomb slip area contact failure model. Relevant intact rock material properties were selected from the data presented in Table 6.36. As in the case of wedge analysis the joint shear strength parameters were estimated to reflect their surface characteristics described earlier.

Table 6.46 UDEC model section details of the LTKPA tunnel

| Station (m) | Depth (m) | Rock type | No. of joint sets |
|---|---|---|---|
| 410 | 80 | Sandstone | 2 |
| 529 | 110 | Sandstone | 3+ random |
| 670 | 160 | Siltstone | 3 |
| 830 | 210 | Siltstone | 2 |

The following cases were modelled for each section:

- Case 1: unsupported tunnel.
- Case 2: with bolts and URF shotcrete support.
- Case 3: with bolts and MRF shotcrete support.

The URF and MRF shotcrete were modelled using the shotcrete parameters listed in Table 6.47. It should be noted that as the effect of welded wire mesh reinforcement installed with shotcrete is difficult to model, the values listed for MRF shotcrete in Table 6.47 were assumed to represent the lower bound effect of wire mesh reinforcement. The bolts included in the model were the cement grouted type as used earlier in the limit equilibrium analyses.

Table 6.47 Shotcrete parameters used in the UDEC analysis

| Property | URF | MRF |
|---|---|---|
| Compressive strength *MPa* | 30 | 60 |
| Tensile strength *MPa* | 3 | 6 |
| Adhesive strength *MPa* | 0.5 | 0.5 |
| Elastic modulus *GPa* | 30 | 35 |

Case 1 showed that rock block instability is possible both in the roof and walls, particularly when kinematically feasible blocks are present at the tunnel periphery (Figures 6.18 and 6.19). This is consistent with the observed rock mass behaviour in the tunnel. Further, Case 1 showed that tensile stress zones developed behind the side walls (Figure 6.20) and therefore tensile failure is also possible in tunnel walls when the in situ stress levels are relatively high (i.e. when the depth of tunnel is >100 m). This is in agreement with the stress induced rock slabbing observed in the walls between Sta. 570 and 880 m.

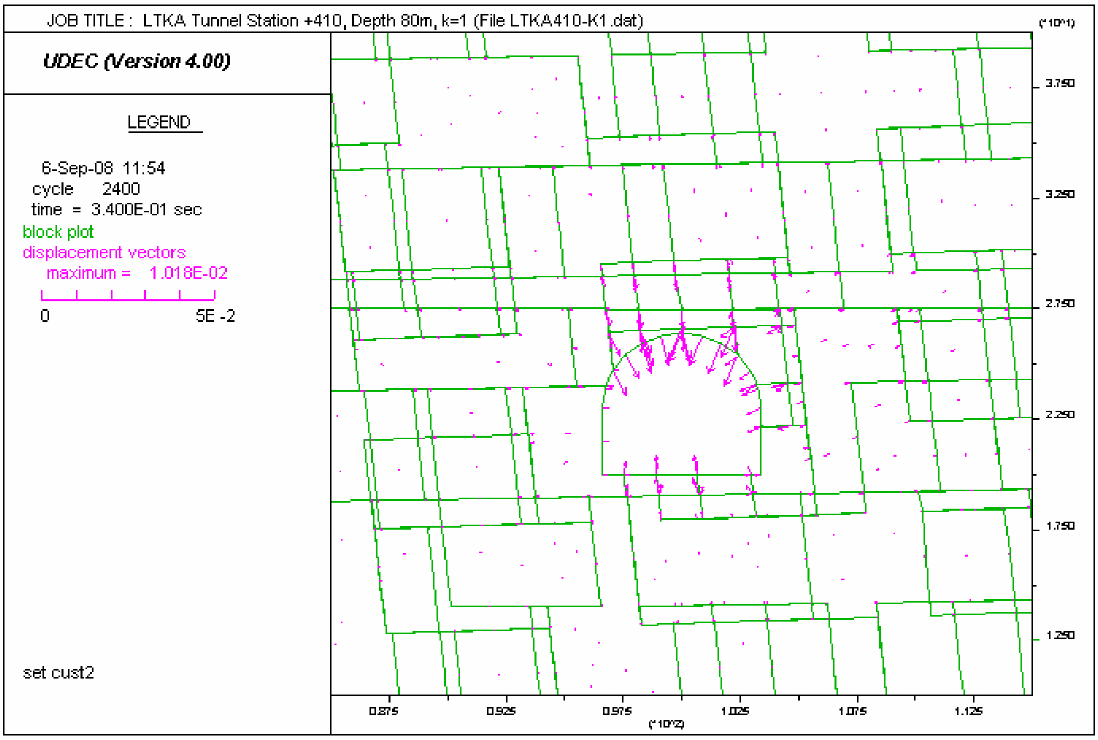


Figure 6.18 Displacement vectors showing potential roof instability

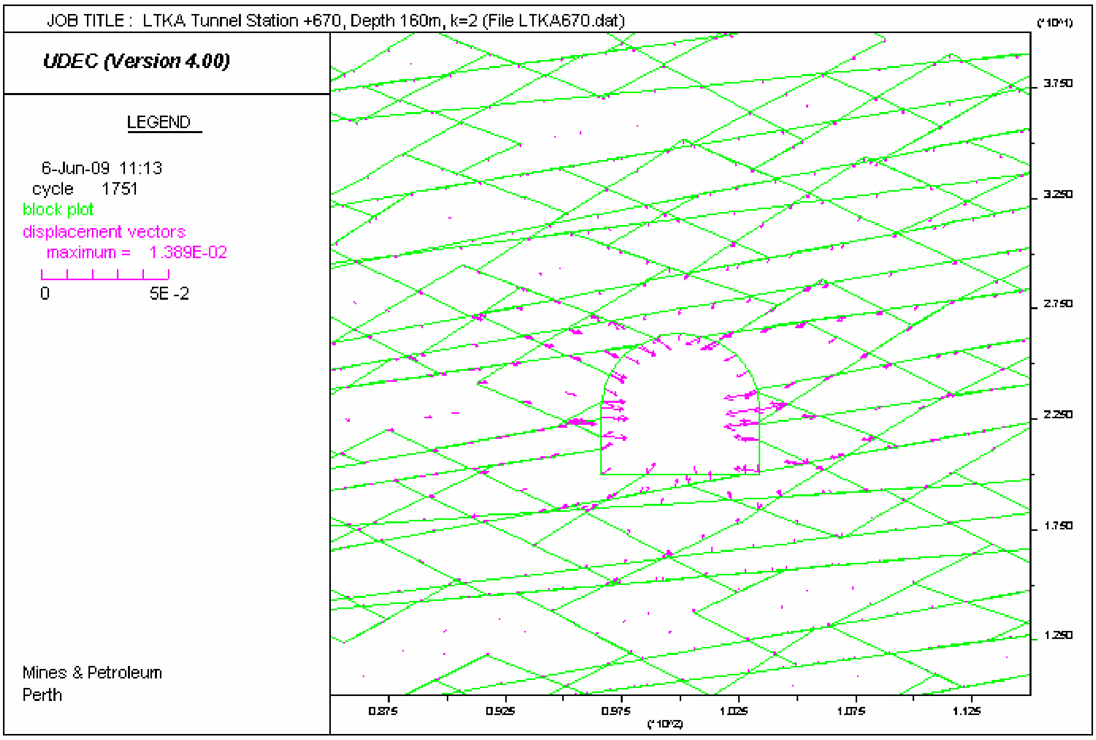


Figure 6.19 Displacement vectors showing potential wall instability

Case 2 indicated compressive failure and axial yielding of URF shotcrete under the conditions simulated in all four sections. As mentioned in Section 6.7.4.1, some damage occurred in the URF shotcrete layer installed from Sta. 280 to 540 m. Interestingly, the predicted failure zones in the tunnel periphery (Figure 6.21) compare well with the actual locations of shotcrete damage reported by Sriwisead (1996). Case 3 showed that the extent of predicted shotcrete damage can be reduced significantly by using MRF shotcrete (Figure 6.22). The simulation also showed that a marginal increase in the MRF shotcrete strength parameters above those listed in Table 6.47 would be sufficient to eliminate the predicted failure zone. Since no damage was reported in the areas supported with MRF shotcrete, it may be deduced that URF shotcrete is not the best option for the tunnel, although $Q_{94}$ recommended URF shotcrete with rock bolts for 37% of the tunnel and no shotcrete for the remaining 63%. In the case of $Q_{74}$, URF and MRF shotcrete was recommended for only 46% and 11% of the tunnel, respectively.

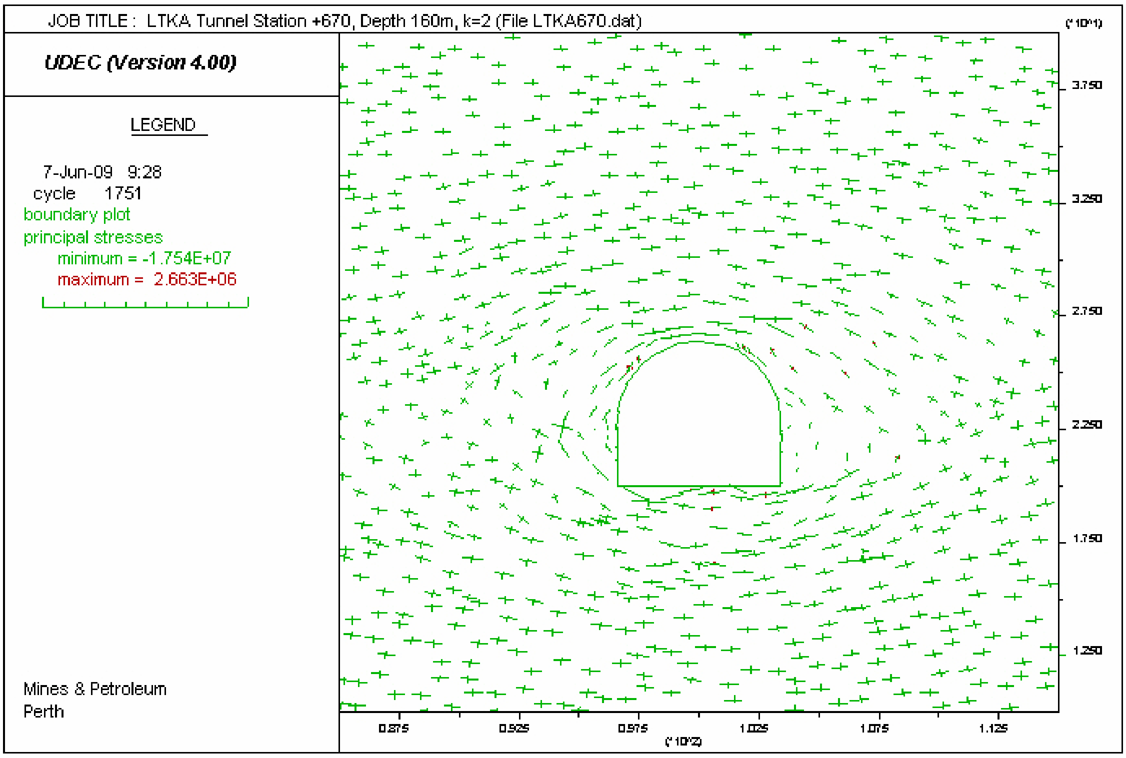


Figure 6.20 Tensile stress zones on side walls

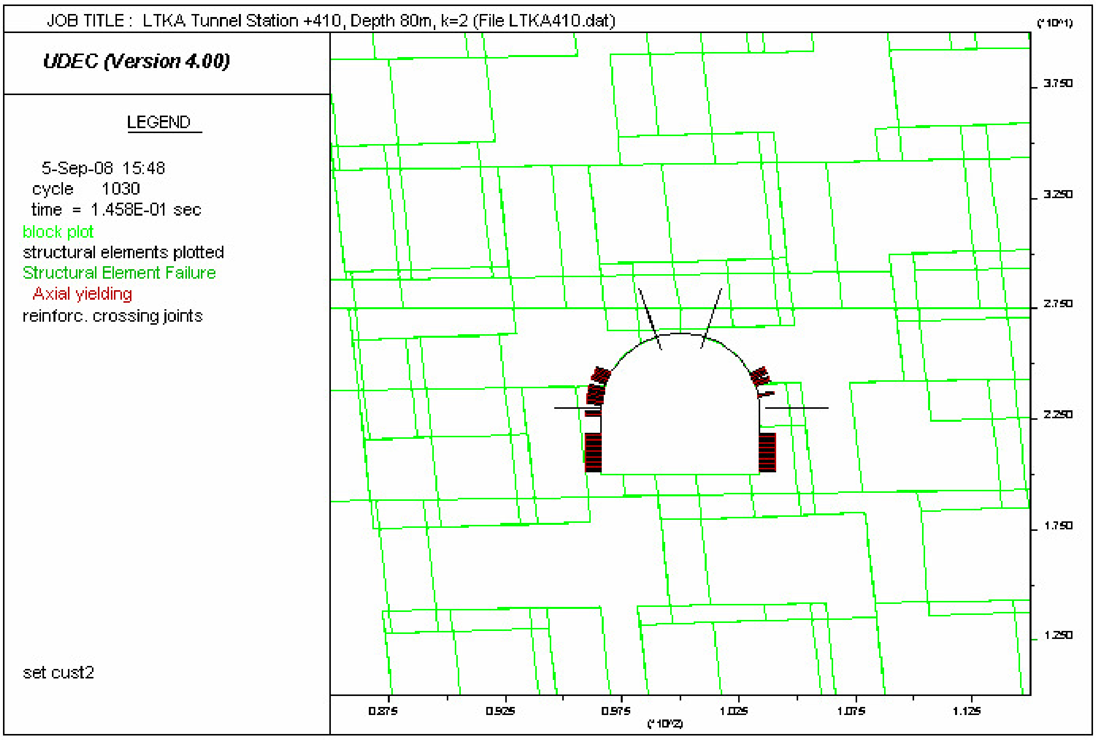


Figure 6.21 Failure of URF shotcrete at Sta. 410 m when k=2 (rock bolts installed)

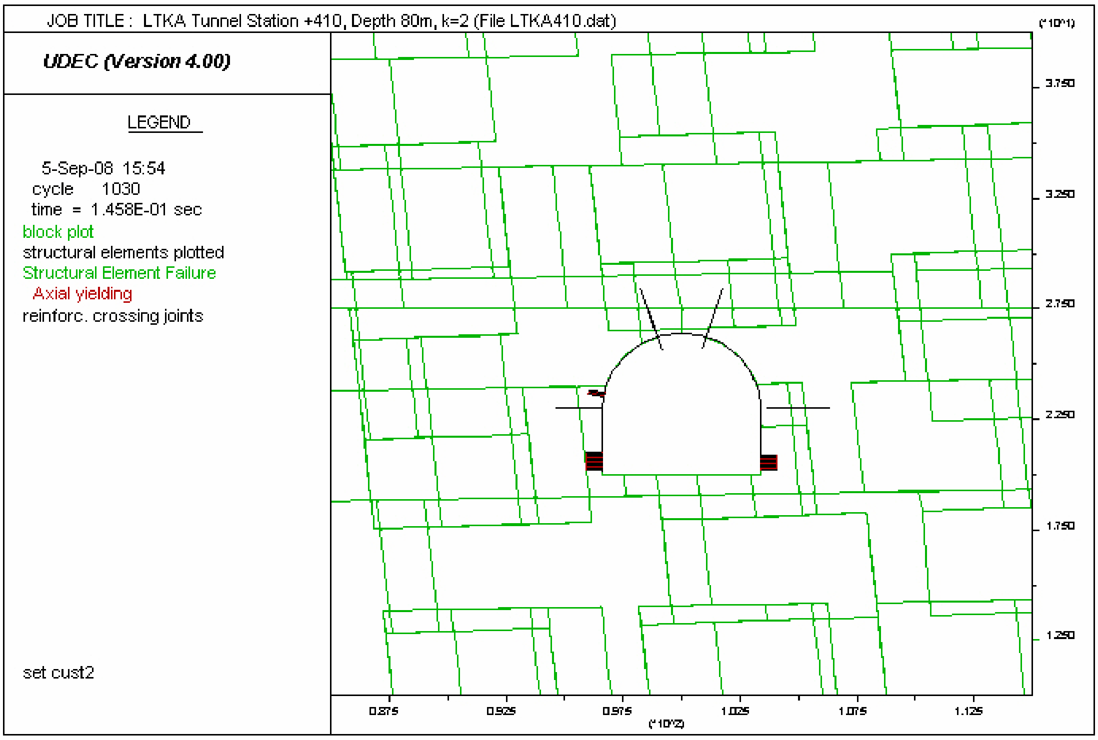


Figure 6.22 Reduction in failure at Sta. 410 m when mesh reinforcement was added

### 6.8.6 Discussion

From Tables 6.40 to 6.43 it can be seen that, in general, the recommended and the actual bolt spacing may be considered comparable, except for some areas where RMR recommended bolt spacing is greater than those used. Despite the fact that 96% of the tunnel length was rock bolted, $Q_{74}$ recommended rock bolting for only 35% of the tunnel. $Q_{94}$ on the other hand recommended rock bolts for 79% of the tunnel.

The applied shotcrete thickness was mostly 100 mm except for two areas with 150 mm and 70 mm thicknesses (Table 6.43). As can be seen from Table 6.40, the thickness of the RMR recommended shotcrete layer was less than the applied shotcrete thickness. In the case of $Q_{74}$ and $Q_{94}$, shotcrete was recommended for only 57% and 32% of the tunnel, but the entire tunnel was shotcreted.

The RMR system recommended mesh reinforcement for 74% of the tunnel. Compared to this, the two Q versions recommended mesh/fibre reinforcement for 11% (only when Q<2) and 60% of the tunnel was initially supported with mesh. As discussed in Section 6.7.4.1, the reported shotcrete damage was limited to the tunnel interval from 280 to 532 m, where no mesh reinforcement was used initially, but additional support was installed subsequently. This may mean that the RMR recommendation for mesh is more in line with the requirements of this tunnel and the $Q_{74}$ and $Q_{94}$ recommendations are not. It should be noted that the RMR recommendations given in the literature are for 10 m diameter tunnels and in this study these recommendations are assumed to be applicable to the 6.8 m diameter tunnel

### 6.8.7 Conclusion

Beam analysis, which ignored the in situ stress field, showed that RMR and Q derived support measures are adequate for stabilizing the potentially unstable rock beams in the tunnel. An exception to this is that both Q versions recommended no support for the good rock mass class representing approximately 20% of the tunnel where beam instability was considered possible.

The RMR derived support measures for the entire 885 m tunnel length and the Q derived support measures for the poor rock class were adequate for the potentially unstable tetrahedral rock wedges. Q recommended only pattern bolting for fair rock and no support for good rock. The recommended pattern bolting is adequate for stabilizing the largest possible tetrahedral rock wedges in the tunnel, however, in fair rock class, shotcrete was warranted as there was the potential for small rock block instability in between the installed rock bolts. Further, Q recommended no support for good rock class which represents approximately 20% of the tunnel length studied where wedge instability was possible.

The numerical simulation showed that the RMR derived support measures meet the numerically predicted support requirements. The simulation indicated that instead of the URF shotcrete recommended by $Q_{94}$, MRF (or fibre reinforced) shotcrete is a better option to reduce the risk of damage to shotcrete. Both Q versions recommended reinforced shotcrete only for 11% of the tunnel

The study showed that the RMR recommended support types are in agreement with the support installed which consisted of rock bolts, shotcrete and mesh reinforcement. There were differences in the RMR recommended and installed bolt spacing and shotcrete thickness, but in general, they were comparable. $Q_{94}$ recommended rock bolts for 79% of the tunnel, shotcrete for 32% and fibre reinforcement for only about 11%. Although the $Q_{94}$ derived bolt pattern may be considered comparable to the installed bolt pattern, its recommended shotcrete and fibre reinforcement fall well short of the extent of shotcrete and mesh installed in the tunnel. The support recommended by $Q_{74}$ also falls well short of the actual support installed, i.e. it recommended bolts for 35% whereas 96% was actually supported with rock bolts.

## 6.9 CASE STUDY 7:
## The Klong Tha Dan (KTD) Project Tunnels, Thailand

The Klong Tha Dan (KTD) water resources development project located in the Nakhon Nayok province of Thailand consists of a 93 m high and approximately 2.7 km long roller compacted concrete (RCC) dam with a maximum base width of 86 m. Together with a rock-earth saddle embankment dam, it creates a water storage reservoir of 224 million cubic meters at full supply level of 110 m RL. As part of the dam foundation treatment work, five small diameter tunnels with a total length of 1590 m were constructed. Four are drainage tunnels and the fifth is a grouting cum drainage gallery. At the design stage, rock support measures for the five tunnels were based on the results of site investigations and project-specific requirements. Several approaches, including rock mass classification methods were considered for the final design.

The dam was founded on a solid rock surface with its lowest elevation at approximately 19 m RL. Strictly speaking the KTD RCC dam consists of two adjoining dams built between three hills: Hill A (right abutment), Hill B (middle) and Hill C (left abutment). The first dam connects Hills A and B and the second connects Hills B and C (Figure 6.23). A few meters below the crest level at 112 m RL the two dams join each other and become a single continuous structure making it one of the longest RCC dams in the world. The five tunnels constructed within the three hills are key components of the dam to control water loss through the foundation and also to ensure the stability of the dam by relieving uplift pressures. A summary of the tunnel details is provided in Table 6.48. The approximate locations of the tunnels are shown in Figure 6.23.

Table 6.48 KTD project tunnels

| Name | Location | Length (m) | Shape | W x H (m) | Depth (m) | Purpose |
|---|---|---|---|---|---|---|
| TBR-D1 | Hill A | 227.80 | D-shape | 2.9 x 3.0 | 10 – 65 | Drainage |
| TSB-D1 | Hill B | 388.00 | D-shape | 2.9 x 3.0 | 12 – 87 | Drainage |
| TSB-D3 | Hill B | 158.50 | D-shape | 2.9 x 3.0 | 12 – 45 | Drainage |
| TSL-D2 | Hill C | 501.20 | D-shape | 2.9 x 3.0 | 10 – 52 | Drainage |
| TSB-P | Hill B | 381.40 | Horseshoe | 4.0 x 3.6 | 12 – 87 | Grouting/drainage |

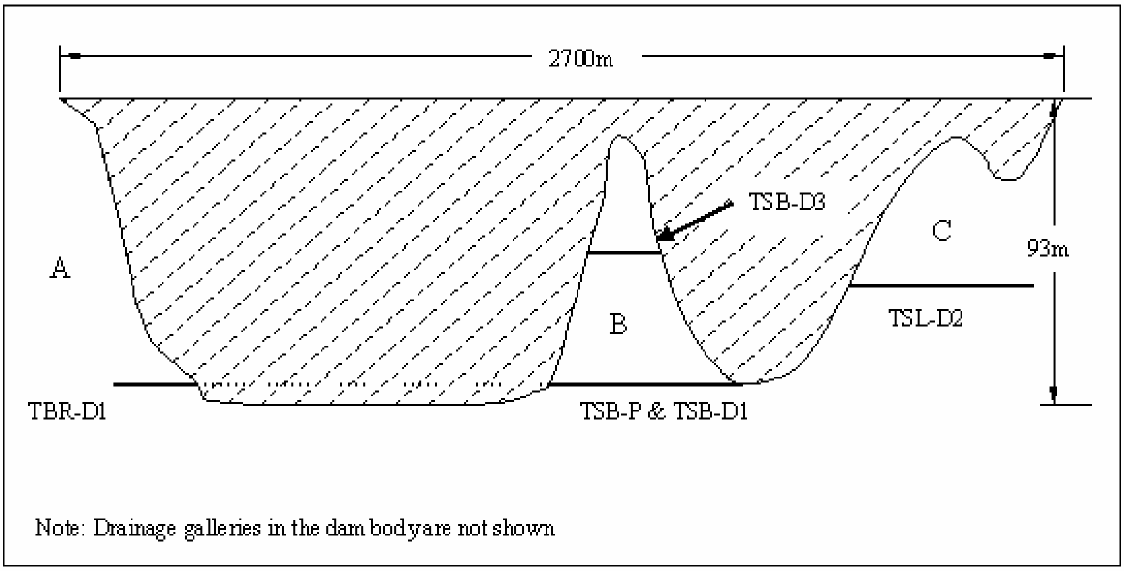


Figure 6.23 A long section of the dam showing the tunnel locations (upstream view)

### 6.9.1 Project Site Geology

The KTD dam site mainly comprised undifferentiated Permo-Triassic volcanic rocks of the Khao Yai Volcanic Formation, which consists of rhyolite, andesite, rhyolitic and andesitic tuff and agglomerate and basalt (Phuntumat, 1997; Swe, 2003). Within the site, the predominant geological discontinuities in the rocks are flow bands, joints and minor faults. No major structures were present.

The main rocks of Hill A are pyroclastic consisting of agglomerate and tuff with basalt and andesite present at random. Hill C comprised lava rocks, namely rhyolite, andesite and basalt and in Hill B (middle hill), the main rock type is rhyolite interrupted by basalt dykes.

### 6.9.2 Rock Mass Data Intersected in the Five Tunnels

In the five tunnels rhyolite and tuff were the main rock types. Andesite, basalt and dacite were also present in small amounts usually as intrusions. A series of laboratory tests conducted by Phuntumat (1997) showed that the average UCS values of rhyolite, andesite, agglomerate and tuff intact rock materials were 76, 134, 102 and 110 MPa, respectively. No UCS data are available for basalt, but it is generally known as a very strong rock. Basalt represents less than 10% of the total tunnel length. The geological discontinuities intersected in the tunnels were flow bands, joints and minor faults. Their general orientations (dip and dip direction) are listed in Table 6.49. All five tunnels were excavated well above the natural groundwater level and were dry during excavation. A summary of the rock types and the notable weakness zones intersected in the five tunnels is presented in Table 6.50.

Table 6.49 General orientations of discontinuities in the KTD tunnels

| | | Set 1 | Set 2 | Set 3 | Set 4 | Set 5 | Set 6 | Set 7 |
|---|---|---|---|---|---|---|---|---|
| TSB-D3 | Dip | 81 | 81 | 75 | 78 | 81 | 49 | |
| | Direction | 074 | 302 | 339 | 275 | 170 | 057 | |
| TBR-D1 | Dip | 79 | 80 | 79 | 64 | 47 | 30 | 23 |
| | Direction | 277 | 055 | 230 | 146 | 015 | 227 | 336 |
| TSL-D2 | Dip | 69 | 78 | 66 | 79 | 81 | 43 | |
| | Direction | 210 | 018 | 248 | 307 | 062 | 060 | |
| TSB-D1 | Dip | 78 | 76 | 78 | 27 | 15 | 55 | 77 |
| | Direction | 314 | 080 | 230 | 201 | 110 | 172 | 269 |
| TSB-P | Dip | 78 | 76 | 78 | 27 | 15 | 55 | 77 |
| | Direction | 314 | 080 | 230 | 201 | 110 | 172 | 269 |

Table 6.50 Summary of the geological conditions of the five tunnels

| Tunnel | Rock types | Weakness zones/structures |
|---|---|---|
| TBR-D1 | Tuff 90%, basalt & adesite 10% | Five minor fault/fractured zones at regular intervals |
| TSB-D1 | Rhyolite 90%, basalt 10% | Twelve minor fault zones & three closely jointed zones |
| TSB-D3 | Rhyolite 100% | Four minor fault zones & a closely jointed zone |
| TSL-D2 | Rhyolite 80%, basalt & dacite 20% | Six minor faults, seven closely jointed zones with an average thickness of 2 m |
| TSB-P | Rhyolite 90%, basalt 10% | Nine fault/fractured zones of les than 1 m thickness. A 30 m wide & two 5 m wide closely jointed zones. |

The predominant form of ground response in the five tunnels was structurally controlled loosening. In the beginning, delays in support installation and inadequate support measures aggravated loosening and caused unnecessary over-breaks. As excavation progressed, controlled blasting and proper and timely installation of support improved the excavation process without any undue over-breaks (Swe, 2003). Since the tunnels were shallow and the in situ stresses were low, no stress related ground instability was observed. No water related effects were reported during construction as the natural groundwater level along the tunnel alignments was below the invert level. Structurally controlled failures, however, occurred both in the crown and walls of the tunnels.

### 6.9.3 Application of RMR and Q to the KTDP Tunnels

During excavation of the tunnels, the two rock mass classification methods were applied independently of each other and records of as-excavated rock mass conditions were prepared by site personnel. These included a graphic log of engineering geology, a description of the rock mass, the minimum and maximum *RMR* and *Q* ratings for each 20 m length of the five tunnels, support recommended by the two methods and a record of the support installed. The minimum and maximum *RMR* and *Q* ratings in each 20 m tunnel length represent the worst and the best case ground conditions within that length.

In the five tunnels, *RMR* values ranged from 27 (poor rock) to 84 (very good rock) and *Q* values ranged from 0.2 (very poor rock) to 62 (very good rock). The worst case and the best case *RMR* values ranged from 27 to 62 and 53 to 84, respectively, and the corresponding *Q* values ranged from 0.2 to 17 and 6 to 62, respectively.

In this study for comparison and correlation of the RMR and Q methods, the worst case and the best case ratings assigned to each 20 m length of the tunnels are treated as two separate data sets, each representing a 20 m length of a tunnel. Histograms of the percentages of rock mass falling into different RMR and Q classes under both the worst case and best case scenarios are shown in Figure 6.24.

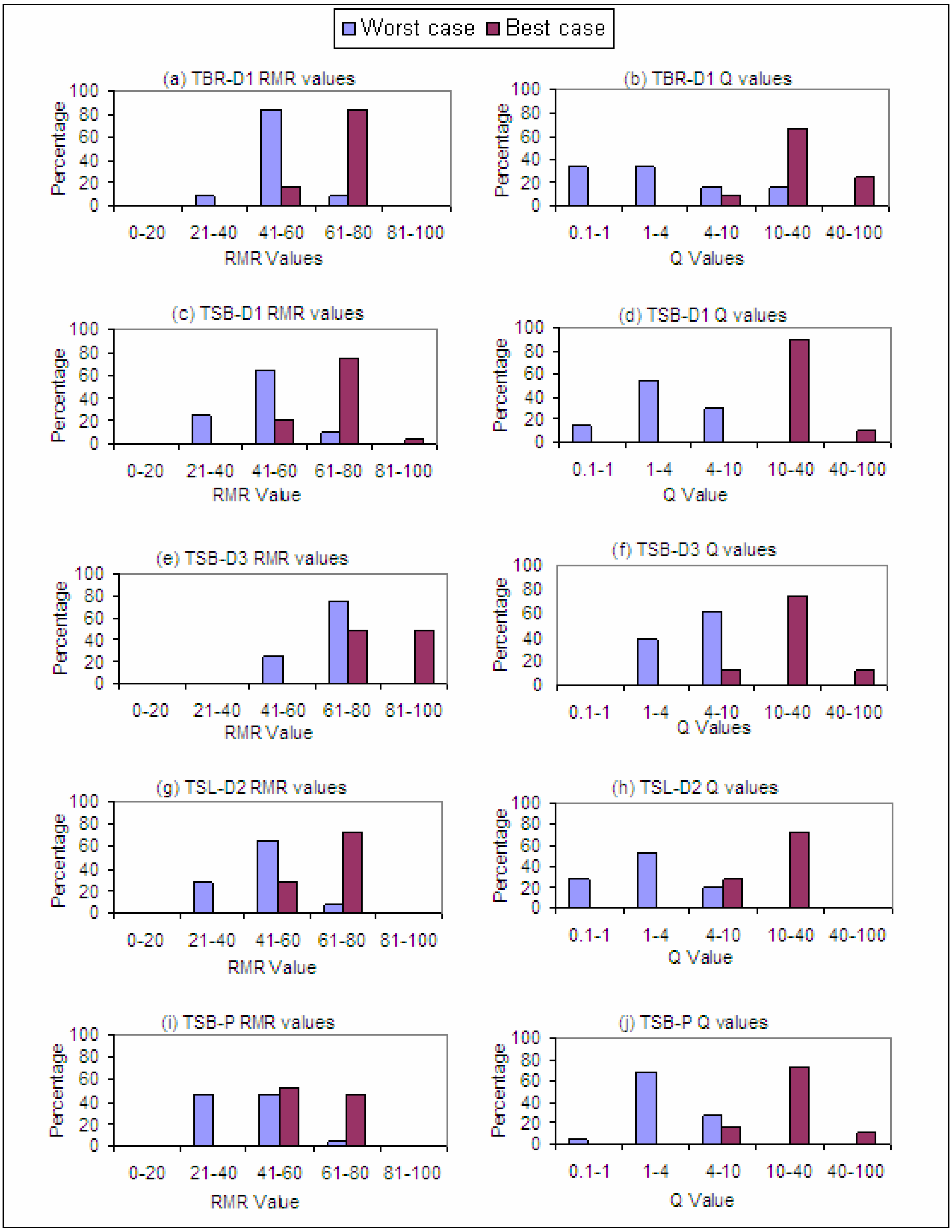


Figure 6.24 Percentages of rock mass classes in the KTD tunnels

It is apparent from the *RMR* data presented in Figure 6.21 that the difference between the worst case and the best case ground conditions is usually one RMR class. In other words, the *RMR* values increase (shift to the right in Figure 6.24) by only one rock mass class. In some instances the increase is less than one class as evident from the *RMR* values of TSB-D3 tunnel. In the case of the Q system, the difference between the worst and the best conditions is often more than one rock mass class. This is an indication that compared to the Q system, the RMR system is less sensitive to the variations in the ground conditions intersected in the five tunnels. This is expected for two reasons. Firstly, the RMR system has only five rock mass classes compared to the nine in the Q system. Secondly, the *RMR* value is derived by summing the ratings given to the six input parameters, while the *Q* value is the product of the three quotients (see Equation 4.8) defined by the six input parameters. Hence any variation in the ratings assigned to the Q input parameters would result in a notable variation in the *Q* value.

### 6.9.4 Comparison of RMR and Q derived support with those installed

During construction of the five tunnels, the $RMR_{89}$ and $Q_{74}$ versions were applied to the KTD tunnels. Therefore, in this section, the $RMR_{89}$ and $Q_{74}$ derived support measures were compared with the actual support installed. The support measures recommended by $Q_{94}$ are discussed separately. With the Q system, considering the need for regular access to the five tunnels, an *ESR* of 1.3was used. Accordingly $D_e$ varies from 2.3 to 2.8 depending on the tunnel span.

In the KTD tunnels five predefined support classes denoted as Classes I to V were used. In order to compare the installed support with the RMR and Q support predictions, it was thought useful to establish a set of support classes common to all three. The RMR method also has five support classes, Classes I to V which can be directly compared with the five support classes used. Although $Q_{74}$ has 38 support categories, for the *Q* value range of 0.2 to 62 and a $D_e$ value range of 2.3 to 2.8 applicable to the KTD tunnels, only four support categories are relevant: the no support category and categories 21, 25 and 29. Since the support types and quantities of Categories 25 and 29 are essentially the same, these two categories were combined. The resulting three $Q_{74}$ support categories were then renumbered as Class I (no support category), Class II (Category 21) and Class III (Categories 25 and 29). These can now be compared with the actual used and RMR predicted support classes. The support types in the relevant classes of $Q_{74}$ and $RMR_{89}$ and in the actual support classes used are given in Table 6.51. The percentages of rock mass falling into each relevant support class of Q and RMR and the actual support classes used are shown in Figure 6.25. Note that the percentages of Q and RMR support classes shown in this figure are based on the worst case ratings assigned to the rock masses in the five tunnels. The best case support recommendations are excluded from the comparison and are less than the worst case support.

Table 6.51 Support types in the five $RMR_{89}$, $Q_{74}$ classes and the actual used in the KTD tunnels

| Support class | I | II | III | IV | V |
|---|---|---|---|---|---|
| *$Q_{74}$ system* | | | | | |
| Bolts pattern | None | Systematic | Systematic | N/A | N/A |
| Shotcrete (mm) | None | 25-50 | 50 mm (mr) | | |
| Steel set spacing (m) | None | None | None | | |
| *RMR system* | | | | | |
| Bolts pattern | None | Spot/local | Systematic | Systematic | Systematic |
| Shotcrete (mm) | None | 50, if required | 50-100 (mr) | 100-150 (mr) | 150-200 (mr) |
| Steel set spacing (m) | None | None | None | 1.5 | 0.75 |
| *Actual used* | | | | | |
| Bolts pattern | None | Systematic | Systematic | Systematic | Systematic |
| Shotcrete (mm) | None | None | 30 (mr)* | 50 (mr) | 50 (mr) |
| Steel set spacing (m) | None | None | None | None | 1.5-2 |

mr=mesh reinforced, *occasional mesh, N/A – not applicable

As can be seen from Figure 6.25 and Table 6.51, the actual support measures installed were significantly more than the support requirements predicted by the Q system, whereas the RMR predicted support classes were comparable to the actual support classes used. However, it will be seen from Table 6.51 that the RMR recommended support, shotcrete and mesh in particular, are excessive compared to those of actual support Class III, which is the most commonly used in the five tunnels. This is partly due to the fact that the RMR support recommendations are for 10 m span tunnels and not necessarily for small span tunnels as in this project. In contrast the Q system has the flexibility to recommend support requirements virtually for any span size. Nevertheless, as can be seen from Figure 6.25, for KTD tunnels the $Q_{74}$ system underestimated the support requirements when compared to the actual support installed.

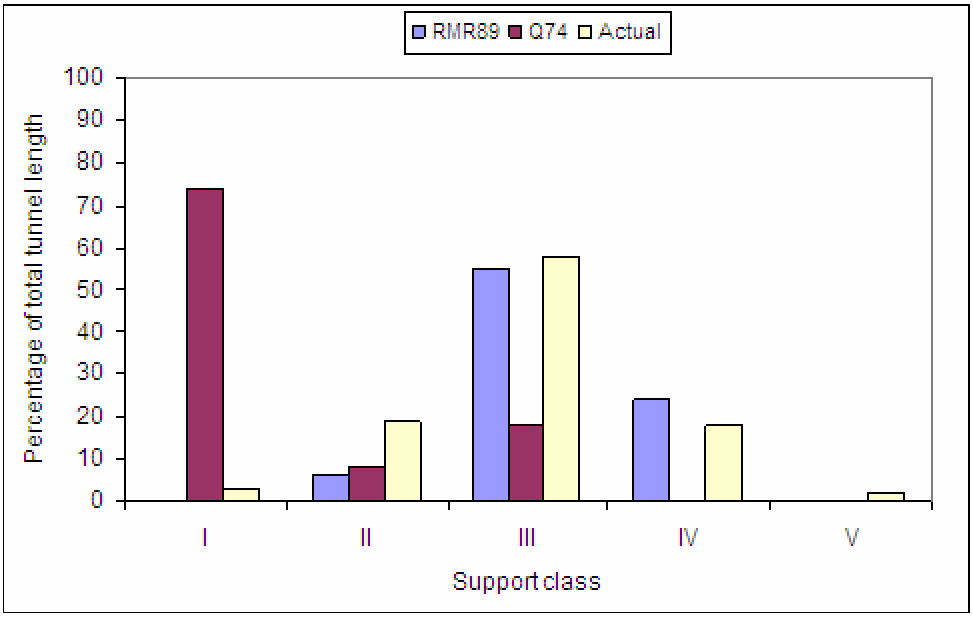


Figure 6.25 Percentages of $RMR_{89}$, $Q_{74}$ and actual support classes in the KTD tunnels

According to the $Q_{94}$ support chart, a tunnel with a *De* of 2.3 to 2.8 would not require rock support if $Q \geq 2$. Congruously, approximately 90% of the rock mass in the KTD tunnels requires no support. Only about 5% of the rock mass in the five tunnels requires Category 5 support (systematic bolts plus 50 to 90 mm of fibre reinforced shotcrete) and another 6% require Category 4 support (systematic bolts plus 40 to 100 mm of un-reinforced shotcrete). In effect $Q_{94}$ also recommended significantly less support than those installed in the five tunnels.

It should be noted that the KTD project design required concrete lining of the grouting gallery. Since the purpose of the lining was not necessarily to deal with the as-excavated rock mass instability, it was not included in the present study for comparison with the RMR and Q derived support. The main purpose of the concrete lining was to ensure: (a) the stability of the tunnel during high pressure grouting, (b) an efficient grouting operation by preventing grout leakage into the tunnel, and (c) long term stability of the tunnel after the creation of the reservoir that generates a hydraulic head of more than 80 m immediately above the tunnel, which was dry during excavation. These aspects are not covered in the two classification methods.

Table 6.52 Potentially falling roof wedges in the KTD tunnels

| Tunnel | Joint sets | Apex height (m) | Weight (kN) | FOS1 |
|---|---|---|---|---|
| TBR-D1 | 1,2,6 | 2.9 | 107 | 0 |
| TSB-D1 | 1,2,3 | 3.8 | 183 | 0 |
| | 1,3,7 | 1.9 | 68 | 0 |
| | 2,6,7 | 1.4 | 29 | 0 |
| TSB-D3 | 1,2,5 | 4.4 | 117 | 0 |
| | 1,4,5 | 5.6 | 429 | 0 |
| | 4,5,6 | 1.6 | 45 | 0 |
| TSB-P | 1,2,3 | 4.2 | 247 | 0 |
| | 1,2,6 | 1.5 | 33 | 0 |
| | 1,4,7 | 0.7 | 64 | 0 |
| | 2,3,7 | 4.2 | 741 | 0 |
| | 2,4,7 | 1.4 | 296 | 0 |
| | 2,5,7 | 2.8 | 486 | 0 |
| | 2,6,7 | 4.4 | 996 | 0 |
| | 3,4,7 | 1.0 | 130 | 0 |
| TSL-D2 | 1,3,5 | 2.1 | 180 | 0 |
| | 1,4,5 | 2.8 | 86 | 0 |

### 6.9.5 Tetrahedral Rock Wedge Analysis

A tetrahedral rock wedge analysis undertaken using UNWEDGE showed that several wedges are kinematically unstable in the five tunnels. For wedge stability analysis the discontinuity shear strength parameters were estimated taking into account their surface characteristics observed during tunnel mapping. They ranged from c=0 kPa and $\Phi$=35° to c=50 kPa and $\Phi$=45°. With the higher shear strength values the FOS of the potentially sliding rock wedges are high enough to prevent sliding failures in the five tunnels, only the potentially falling rock wedges in the crown are therefore of concern. The largest possible wedges are listed in Table 6.52 with the FOS values computed ignoring the effect of the in situ stress field on wedge stability. Since the tunnels are shallow (Table 6.48), the effect of in situ stress field may be ignored for stability assessment. The stability of these wedges was then examined under the support recommended by the two classification methods. Analysis showed that the RMR derived support measures were sufficient to stabilise the potentially falling rock wedges in the five tunnels. The $Q_{74}$ derived support Classes II and III (Table

6.51) were also sufficient for stabilizing the possible rock wedges. However, $Q_{74}$ recommended no support for 74% of the tunnel length, where wedge instability was possible. According to $Q_{94}$ version 90% of the tunnel required no support.

### 6.9.6 Comparison of predicted support pressures

Both RMR and Q systems provide empirical formulas for estimating support pressures required to stabilise the rock mass surrounding an excavation. These formulas may be used to determine the support pressure required or the rock load needed to be supported in an excavation with a given *RMR* or *Q* value.

On the basis of the work of Unal (1983) on prediction of support pressures using the RMR system, Bieniawski (1989) provided the following equation:

$$P = (100 - RMR)\gamma B/100 \tag{4.5}$$

where, $P$ is the support load in kN, $B$ is the tunnel width in meters and $\gamma$ is the rock density in kg/m$^3$. As can be seen from Equation 4.5, the RMR system assumes that the support pressure not only depends on the rock mass quality (or the *RMR* value), but also on the width of the opening. This means that although the RMR recommended support quantities are primarily aimed at 10 m span tunnels, the *RMR* versus support pressure relationship can be applied to any tunnel regardless of its span. Naturally, as would be expected, according to Equation 4.5 different support pressures will be required for different tunnel spans in the same rock mass.

With reference to the Q system, Barton et al. (1977) and Grimstad and Barton (1993) provided the following empirical formula for estimating the permanent radial support pressures required to stabilise the roof of an excavation:

$$P = 200Jn^{1/2}Q^{-1/3}/3Jr \tag{4.13}$$

where $P$ is in kPa, $Jn$ and $Jr$ are as defined earlier. From the above equation, it is clear that the Q system assumes that the support pressure is a function of only the rock mass quality ($Q$ value, $Jn$ and $Jr$). The width or span of the excavation is ignored. This is somewhat surprising because in determining the support requirements using the support chart, the Q system explicitly takes into consideration the span (or diameter) of the excavation. For instance, for a tunnel of 2 m span driven in a rock mass with a *Q* value of 0.2 the recommended bolt length is 1.5 m, if *ESR*=1, whereas for a tunnel of 5 m span in the same rock mass with the same *ESR* the recommended bolt length is 2.4 m. This implies that increasing tunnel span increases the thickness of the potentially unstable rock zone, and is rightly so, particularly for jointed rocks. It follows that with an increase in span, the rock load for a unit surface area (hence the required support pressure) also increases. The corollary is that when estimating support requirements and support pressures, seemingly, the Q system contradicts itself.

Notwithstanding the above, the support pressures estimated using the two empirical formulas were analysed to note any relationships or trends between the two. Both the worst case and best case support pressures estimated by site personnel for each 20 m length of the five tunnels were included in the analysis. This showed that there is no direct correlation between the RMR and Q derived support pressures. Figure 6.26 shows a plot of *P-Q* (Q derived support pressure) versus *P-RMR* (RMR derive support pressure). As can be seen from Figure 6.26, the data are widely scattered and as a result no meaningful linear relationship can be expected.

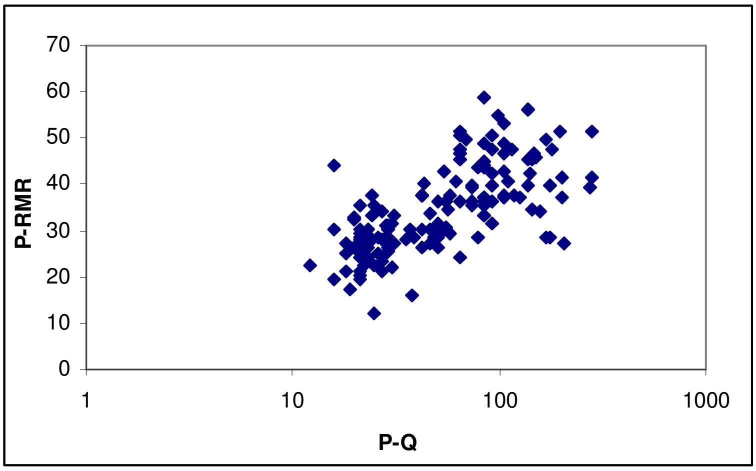


Figure 6.26 RMR and Q derived Support pressure for the KTD tunnels

The data showed that the two empirical formulas do not always yield the same result for the same rock mass intersected in a tunnel. While in some instances the RMR and Q derived support pressures are almost the same, in other instances the Q derived support pressures are significantly higher. Further examination of the data indicated that the ratio of Q derived support pressure (*P-Q*) to RMR derived support pressure (*P-RMR*) is approximately one when $Q$>10. When $Q$≤10, the ratio (*P-Q:P-RMR*) is always greater than one, and it increases rapidly with decreasing $Q$ value. This is shown in Figure 6.27, where *P-Q:P-RMR* ratios are plotted as ordinate and the corresponding $Q$ values are plotted as abscissa. This observation can be summarised as follows:

$$P_Q/P_{RMR} >> 1.0, \text{ when } Q \leq 10$$
$$P_Q/P_{RMR} \approx 1.0, \text{ when } Q > 10$$

where $P_Q$ and $P_{RMR}$ are the support pressures derived by the Q and RMR methods. It should be noted that the above observation is made from the support pressures estimated for approximately 3 m span tunnels. For larger span tunnels this observation is not valid. For instance, for a 6 m span tunnel in the same rock mass, the $P_{RMR}$ values will be twice as high as those of the 3 m span tunnels used in the present study, while $P_Q$ values remain the same.

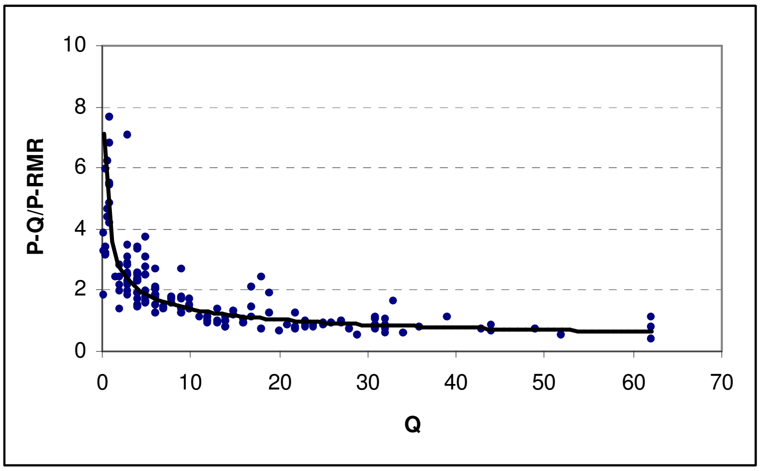


Figure 6.27 $Q$ value versus $P_Q:P_{RMR}$ ratio in the KTD tunnels

Since the actual support pressures were not measured in the KTD tunnels, it is not possible to compare the empirically predicted support pressures with the actual site conditions. A comparison of the Q predicted support pressures with the rock loads measured in several tunnels in New Zealand (Rutledge and Preston, 1978) showed that many of the rock pressures predicted by the Q system are greater than the measured rock loads by an excessively conservative margin. When $Q \leq 10$ the support pressures estimated for the KTD tunnels agree with that observation. The study conducted by Rutledge and Preston (1978) did not include a comparison of RMR derived support pressures with the measured values. Based on the available information, it is not possible to comment whether the RMR system underestimates $P_{RMR}$ for poor rock mass conditions (i.e. when $Q \leq 10$).

An important point that should be mentioned here is that when $Q \leq 10$, the Q system predicts higher support pressures than those predicted by the RMR system, yet it does not recommend support for the KTD case tunnels if the $Q$ value is more than 2. This is because, according to the Q support chart, the small span tunnels with a $D_e$ of around 2 do not require support if the $Q$ values is greater than 2. While this may be the case for the case studies included in the database used in developing the Q system, this is not always the case for other excavation projects as seen from the KTD tunnels and previously shown by Ranasooriya and Nikraz (2008a and 2008b).

Note that since the KTD tunnels are mostly shallow the RMR versus support pressure correlation (Equation 4.6) proposed by Goel and Jethwa (1991) for predicting support pressures in tunnels at depths greater than 50 m was not used. Even with the 87 m maximum depth of the KTD tunnels this equation would yield only a negative support pressure and therefore it is not applicable in this case. Similarly, the Q versus support correlation (Equation 4.14) proposed by Bhasin and Grimstad (1996) was not used because this equation is for crushed and brecciated rock masses and not for jointed rock masses as in the case tunnels.

### 6.9.7 Conclusions

The study showed that for the five tunnels, the Q system under estimated support requirements, whereas, the RMR system overestimated the same. In the five tunnels more support was installed than those derived by the Q system.

The predicted support pressures showed that the two empirical formulas do not always yield the same result for the same rock mass intersected in a tunnel. Further examination of the data indicated that for small diameter (~3 m) tunnels, the ratio of Q derived support pressure ($P_Q$) to RMR derived support pressure ($P_{RMR}$) is approximately one when $Q>10$. When $Q\leq10$, the ratio ($P_Q$:$P_{RMR}$) is always greater than one, and it increases rapidly with a decreasing $Q$ value.

When $Q\leq10$, the Q system predicted higher support pressures than those of the RMR system, yet did not recommend support for the KTD case tunnels if the $Q$ value is greater than 2. For these small diameter tunnels the Q system underestimated support requirements.

## 6.10 CASE STUDY 8:
## The Namroud Water Resources Project Diversion (NWRPD) Tunnel, Iran

The Namroud water resources development project is currently being built in Firouz Kouh, Tehran province, Iran. It comprises a 652 m long 82 m high rock fill embankment dam with a clay core. The project also comprises a 5.5 m wide 740 m long horseshoe shaped diversion tunnel constructed for two purposes: (a) to temporarily divert the Namroud River to facilitate the construction of the dam; and (b) as a bottom outlet during project operation to provide drinking and irrigation water for downstream users. The tunnel located in the left abutment has an overburden of between 30 and 90 m and was driven through weak sedimentary rocks comprising limestone, marlstone and limy shale.

### 6.10.1 Project Site Geology

The regional geology of the project area is characterised by sedimentary rock formations that have been subjected to a series of folding and faulting. Several major geological structures are present in the general area including the Seleh Bon syncline, the Nachoostan anticline, the Namroud and the Masha Fasham faults, and the Barijan, Alborz, Garmsar, Namroud and Firoozkooh thrusts. The site is located on the southern limb of the Seleh Bon syncline.

To the downstream of the dam axis is a region of several sedimentary rock units which have been subjected to different tectonic events. Along the dam axis and to the upstream is a group of soft sedimentary rocks of Karaj Formation. The tectonic activities in the region caused several splay faults and shears and as a result some of the rock units along the tunnel alignment are shattered and sheared. In general the project area is overlain by recent sediments and most of the dam foundation is located on alluvial deposits. The Namroud riverbed is composed of an alluvium deposit, therefore the dam includes a cut-off wall to reduce the potential for water loss through its foundation.

### 6.10.2 Rock Mass Data Along the NWRPD Tunnel

The main rock types along the tunnel alignment comprise limestone, marlstone and limy shale, most of which are tectonically disturbed. A summary log of rock types intersected in the tunnel is given in Table 6.53.

Table 6.53 Summary log of rock types in the NWRPD tunnel

| Chainage (m) | Rock type |
|---|---|
| 000 – 145 | Tuff marl |
| 145 – 244 | Limy shale/shaly limestone |
| 244 – 304 | Tuff marl |
| 304 – 504 | Tuffmarl/shaly limestone/limy shale |
| 504 – 585 | Marly limestone |
| 585 – 670 | Sand marlstone |
| 670 – 693 | Limestone |
| 693 – 728 | Sandy marlstone |
| 728 – 740 | Limestone |

Several geological discontinuity sets are present along the tunnel alignment including bedding planes, joints and shears. The vast majority of the discontinuities are filled with calcite infill material while some are either clean or coated with oxide material. The discontinuity orientation, spacing and surface characteristics vary along the tunnel. The general orientation and spacing of bedding and joint sets in tuff marl are given in Table 6.54.

Table6.54 Orientation of discontinuity sets in the NWRPD tunnel

| | | Spacing (% in each range) | | | |
|---|---|---|---|---|---|
| Set. | Dip/direc | >2.0m | 0.6-2.0m | 0.2-0.6m | 0.06-0.2m |
| B1 | 70/310 | 38 | 17 | 27 | 18 |
| J1 | 83/211 | 9 | 18 | 55 | 18 |
| J2 | 73/269 | 10 | 38 | 38 | 14 |
| J3 | 60/070 | 37 | 40 | 12 | 11 |

Of particular concern was the tuff marl rocks of the Karaj formation which are highly shattered and weak due to tectonic activity. In the tunnel length from chainage 80 m to 120 m, the weakness of tuff marl was further exasperated by the presence of a minor shear zone in this area. As per the ISRM suggested methods, the intact rock materials intersected in the tunnel can be described as weak with a typical UCS range of 5 to 11 MPa. The tunnel is located below the groundwater table and was wet during excavation.

### 6.10.3 Excavation Methods and Primary Support Measures Used

In the original design the proposed excavation method was drilling and blasting and the proposed primary support measures comprised a 1 m x 1 m pattern of rock bolts, 100 mm thick layer of mesh reinforced shotcrete and lattice girders made of three 25 mm diameter steel bars. In accordance with this design, the entire tunnel was to be pattern bolted, part of it was to be shotcreted with wire mesh reinforcement and lattice girders installed for almost half its length. As this design was based on the data collected from a limited program of site investigation involving exploration drilling, surface mapping and rock sample testing, it needed revision and updating based on more detailed information available at the construction stage.

Subsequent to the commencement of construction, it was observed that the proposed drill and blast excavation method was not the best option for some of the rocks intersected because blasting, even when well controlled, caused unnecessary rock mass damage and instability in the tunnel. After considering the available options, jackhammer and drum-cutter techniques were used for the excavation of approximately 340 m of the 740 m long tunnel to reduce rock mass damage.

At a very early stage of excavation, in light of the additional information collected from direct observation of the rock mass, the original support design was reviewed and it was found that the proposed support, rock bolting in particular, was not appropriate especially for areas where rock mass was weaker than expected. It was, therefore, decided to rely on surface support, i.e. mesh reinforced shotcrete and light steel ribs etc. A comparison of the proposed excavation and support methods with the actual methods used is presented in Figure 6.28.

As can be seen from Figure 6.28, rock bolting was not used in this tunnel. Mesh reinforced shotcrete was the most common support system over a total tunnel length of 406 m. In the weaker rock zones, representing a cumulative length of approximately 200 m, light steel ribs and shotcrete with or without mesh, locally made steel shield and concrete, mass concrete and pre-bolting (forepoling) followed by mesh reinforced shotcrete were used. The steel ribs were connected by welding 25 mm steel bars parallel to the tunnel axis and shotcreted with or without mesh. Despite the fact that the initial design based on rock mass classification methods required pattern bolting of the entire tunnel, approximately 100 m of it from the downstream portal was unsupported.

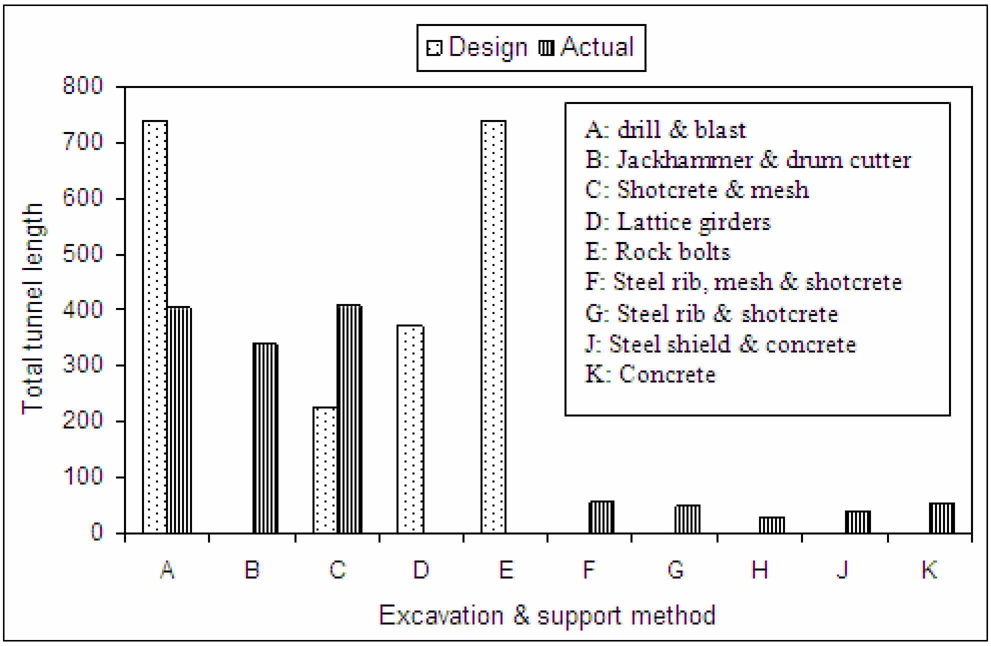


Figure 6.28 Proposed and actual excavation and support method (NWRPD tunnel)

After the completion of excavation, the tunnel was fully concrete lined as it is to be converted to a bottom outlet after the dam construction. The primary support measures were, therefore, kept to the required minimum.

### 6.10.4 Application of RMR and Q During Construction

During construction of the tunnel, $RMR_{89}$ was applied to the entire length by detailed mapping of the exposed rock mass conditions. Based on the rock type and its condition, the tunnel was divided into 11 geotechnical sectors (domains) so that different conditions in each sector could be accounted for in classifying the rock mass according to the RMR system. For comparison purposes, the $Q_{94}$ index was also applied by indirect means using the *RMR-Q* linear correlation (Equation 5.1) proposed by Bieniawski (1976, 1989, 1993).

$$RMR = 9\ ln\ Q + 44 \tag{5.1}$$

The above equation was used because, due to time constraints, ratings for the Q input parameters were not determined during excavation. It is recognised that the correlation given by Equation 5.1 may not necessarily be applicable to the rock mass conditions in the tunnel. Further, as noted by Palmstrom (2009), this correlation is a very crude approximation involving an inaccuracy of ±50% or more. Nevertheless, for the present study, it was assumed that the equation would be accurate enough for comparing the support measures predicted by the two classification systems. Table 6.55 presents a summary of the RMR and Q values and the relevant rock mass classes representing the 11 tunnel sectors.

Table 6.55 Summary of RMR and Q values in the NWRDP tunnel

| Tunnel interval (m) | RMR value | RMR class | Q value | Q class |
|---|---|---|---|---|
| 000 – 060 | 17 | Very poor | 0.05 | Extremely poor |
| 060 – 145 | 18 | Very poor | 0.06 | Extremely poor |
| 145 – 244 | 20 | Very poor | 0.07 | Extremely poor |
| 244 – 304 | 24 | Poor | 0.11 | Very poor |
| 304 – 504 | 18 | Very poor | 0.06 | Extremely poor |
| 504 – 540 | 26 | Poor | 0.14 | Very poor |
| 540 – 585 | 17 | Very poor | 0.05 | Extremely poor |
| 585 – 670 | 35 | Poor | 0.37 | Very poor |
| 670 – 693 | 37 | Poor | 0.46 | Very poor |
| 693 – 728 | 40 | Poor | 0.64 | Very poor |
| 728 – 740 | 33 | Poor | 0.29 | Very poor |

As expected, the RMR system classified the majority (66%) of the rock mass into the very poor class (the lowest in the RMR rating scale) and the remainder (33%) into the poor class. When the RMR values were transformed into Q values using

Equation 5.1, the corresponding Q rock mass classes were extremely poor (66%) and very poor (33%). In general the rating values of any given RMR class would not directly transform into a single rock mass class of the Q system because the latter has nine classes against the five in the former. However, due to the relatively narrow range of RMR values obtained for this tunnel, each class falls into a single Q class when transformed using Equation 5.1. It should be noted that RMR classified the rock mass as very poor and poor, the two lowest classes in this system, and Q classified it as extremely poor and very poor, the second and third lowest classes of the Q system. It would, therefore, be expected that heavier support would be required for the tunnel.

Table 6.56 RMR and Q derived support measures for the NWRDP tunnel

| RMR class | RMR derived support | Q class | Q derived support |
|---|---|---|---|
| Poor (33%) | Bolts at 1-1.5 m spacing with mesh. Shotcrete 100-150 mm in crown & 100 mm in walls. Light ribs spaced at 1.5 m where required | Very poor (33%) | Bolts at 1.3-1.5 m spacing. Fibre reinforced shotcrete 50-90 mm. |
| Very poor (66%) | Bolts at 1-1.5 m spacing with mesh. Shotcrete 150-200 mm in crown, 150 mm in walls & 50 mm on face. Medium to heavy ribs spaced at 0.75 m with steel lagging and forepoling if required, close invert | Extremely poor (66%) | Bolts at 1.2-1.3 m spacing. Fibre reinforced shotcrete 90-120 mm. |

Using the RMR and Q values in Table 6.55 for comparison with those installed in the tunnel, support requirements were determined according to the two rock classification systems. Although the RMR support measures recommended by Bieniawski (1989, 1993) are for 10 m wide tunnels, it was assumed that they are applicable to the 5.5 m wide Namroud diversion tunnel except for bolt lengths which needed adjustment to match its width. This may be justified because the previous RMR versions (Bieniawski, 1974, 1975) recommended the same support measures for 5 to 12 m wide tunnels. With the Q system, an excavation support ratio (*ESR*) of 1.8 (for water tunnels) was used as suggested by Barton and Grimstad (1994). This gives a *De* value of 3.0 for the 5.5 m diameter tunnel. The relevant $Q_{94}$ support categories, therefore, are 5 and 6 for very poor and extremely poor classes of rock respectively. The relevant RMR and Q support measures are presented in Table 6.56. (Note that these are for permanent support.)

As can be seen from Table 6.56, both RMR and Q recommended rock bolts and shotcrete with mesh or fibre reinforcement. The RMR system also recommended steel ribs for the very poor rock class and also for the poor class, if required. Additionally, it also recommended forepoling (pre-bolting) for the very poor class, if required. The steel sets and forepoling recommendations comply with some of the support methods used in the weaker rock zones.

### 6.10.5 Actual Support Installed

As can be seen from Figure 6.28, rock bolting was not used as in the original design. Mesh reinforced shotcrete was used for a total tunnel length of 406 m. Weaker rock zones in a total length of approximately 200 m were supported with light steel ribs and shotcrete with or without mesh, steel shield and concrete, mass concrete and pre-bolting. As stated earlier, although some of the primary support measures used in the weaker zones are comparable to those recommended by the RMR method, they significantly differ from those recommended by the Q system. For the 100 m unsupported length of the tunnel (from ~ Ch. 640 to 740 m), the RMR values ranged from 33 to 40 and the corresponding Q values ranged from 0.29 to 0.64 (Table 6.55). According to the two methods this length required pattern bolts plus mesh or fibre reinforced shotcrete.

Since the RMR and Q recommendations are for permanent support, a direct comparison with the primary support could be open to conjecture. Nevertheless, in this instance heavier primary support measures were required for 200 m of the tunnel than the permanent support recommended by the two methods.

### 6.10.6 Discussion

The rock mass conditions encountered during construction of the 740 m long Namroud diversion tunnel were different to those predicted based on the data collected for project design. Due to the weakness of some of the sedimentary rocks intersected in the tunnel, the conventional drilling and blasting excavation method was found to be problematic for part of the tunnel as blasting caused unnecessary rock mass damage. Despite the fact that the RMR system suggested drill and blast excavation methods for the entire tunnel, jackhammer and drum cutter techniques were adopted for over 300 m thus significantly reducing unnecessary damage to the rock mass.

During the early stages of excavation, it was observed that instead of the initially proposed primary support system mainly consisting of pattern bolting, other support systems could lead to better safety performance in weaker rock zones. Mesh reinforced shotcrete was the most common support system used over a total tunnel length of 406 m. In 200 m of weaker rock zones, light steel ribs and shotcrete with or without mesh, steel shield and concrete, mass concrete and forepoling plus mesh reinforced shotcrete were used. Approximately 100 m of the tunnel was unsupported despite the fact that the initial design required pattern bolting of the entire tunnel.

The rock mass conditions exposed in the tunnel were mapped and the RMR system was applied directly to the as-excavated rock mass conditions. The Q system was also applied indirectly by converting the RMR values by means of a published *RMR-Q* correlation.

### 6.10.7 Conclusion

Both RMR and Q methods recommended rock bolting and shotcrete with mesh or fibre reinforcement for permanent support. While mesh reinforced shotcrete was used, rock bolts were not.

In weaker rock zones, the RMR derived support generally agreed with the installed heavier support which included light steel ribs and shotcrete with or without mesh, steel shield and concrete, mass concrete and pre-bolting. The Q derived support measures for the weaker rock zones differ except for the mesh (or fibre) reinforced shotcrete, which is only part of the heavier support installed in these zones. The primary support measures installed in approximately 200 m of the tunnel were heavier than those recommended by Q for permanent support.

For the 100 m unsupported tunnel length, both RMR and Q recommended rock bolts and wire mesh or fibre reinforced shotcrete.

## 6.11 CASE STUDY 9:
## The Boztepe Dam Project Diversion (BDPD) Tunnel, Turkey

The Boztepe project is situated 10 km northwest of Yazihan, which is a township located to the north of the city of Malatya in eastern Turkey. The Boztepe dam is built across the Yagca stream to regulate water and irrigate the agricultural areas of the Yazihan plain. The BDPD tunnel, constructed to facilitate the construction of the main dam, is 565 m long, has a 5 m diameter circular shape and a maximum overburden of about 38 m. Its alignment is north-south from the downstream portal to about 400 m and then gradually turns to $40^{o}$ west of north.

### 6.11.1 Project Site Geology

The Boztepe dam site consists of various geological formations ranging from the Upper Miocene to the Quaternary age. Middle Upper Miocen volcano-sedimentary rocks, known as Yamadag Volcanics, are exposed in the region and are part of an extensive Miocene volcanism in the Eastern Anatolian Region. The Yamadag volcanics are represented in the study area by four different rock units: sandstone-claystone, tuffite, basalt and agglomerate. The tuffites are well bedded with bed thickness ranging from 300 to 600 mm in the lower levels and 50 to 200 mm in the upper levels. Joints within the tuffites are generally altered and filled with clay or calcite having 20 to 30 mm thickness. Basalts overlying the tuffites are well jointed. Basalts are mainly pillar lavas in the lower levels and columnar structures in the upper levels. The agglomerate overlies basalt.

### 6.11.2 BDPD Tunnel Rock Mass Data

The tunnel cuts through basalt and tuffite. The maximum overburdens of basalts and tuffites above the tunnel crown are about 38 m and 27 m respectively.

From the data collected by surface mapping and logging of 20 cored bore holes, Gurocak et al. (2007) prepared an engineering geological map of the dam site and a geological section along the tunnel alignment. The geological section shows that approximately the first 100 m of the tunnel was driven through tuffite, and the next

465 m was driven through basalt. The results of basalt and tuffite intact rock material testing conducted during site investigation are summarised in Table 6.57.

Table 6.57 BDPD tunnel intact rock material properties

| Rock type | Property | Range | Mean | Std |
|---|---|---|---|---|
| *Basalt* | UCS (MPa) | 8.7 - 76.5 | 40.6 | 19.7 |
| | Young's modulus (GPa) | 1.6 - 96.7 | 30.9 | 47.2 |
| | Poisson's ratio | 0.24 – 0.29 | 0.27 | 0.02 |
| | Unit weight ($kN/m^3$) | 23.1 – 28.1 | 25.6 | 1.5.6 |
| | Cohesion (MPa) | - | 12 | - |
| | Internal friction angle (deg) | - | 42 | - |
| *Tuffite* | UCS (MPa) | 2.0 – 21.2 | 8.2 | 5.7 |
| | Young's modulus (GPa) | 0.6 – 10.5 | 2.2 | 2.6 |
| | Poisson's ratio | 0.17 – 0.22 | 0.20 | 0.03 |
| | Unit weight ($kN/m^3$) | 12.0 – 22.1 | 16.5 | 0.04 |
| | Cohesion (MPa) | - | 1.8 | - |
| | Internal friction angle (deg) | - | 33 | - |

Both rock units are jointed, each having four major geological discontinuity sets. By surface mapping and bore core logging, Gurocak et al. (2007) recorded 388 bedding plane orientations and 520 joints surface orientations from the two rock units. The average orientations (dip and dip direction) of the bedding and major joint sets in tuffite and basalt are shown in Table 6.58. Note that the two rock types have two different discontinuity systems and bedding plane set is present only in tuffites.

Table 6.58 Orientations of discontinuity sets in the BDPD tunnel

| Rock type | Discontinuity type | Dip | Direction |
|---|---|---|---|
| *Tuffites* | Bedding Set 1 | 14 | 100 |
| | Joint Set 2 | 80 | 220 |
| | Joint Set 3 | 87 | 259 |
| | Joint Set 4 | 77 | 305 |
| *Basalts* | Joint Set 1 | 78 | 192 |
| | Joint Set 2 | 71 | 003 |
| | Joint Set 3 | 67 | 287 |
| | Joint Set 4 | 72 | 099 |

The discontinuity properties and their percentage distributions in basalt and tuffite are presented in Table 6.59. Although the properties vary within both rock types, the majority of the joints in basalt have close to very close spacing, low persistence, moderately wide apertures and moderately weathered rough planar surfaces. The majority of the discontinuities in tuffite have close spacing, medium to high persistence, moderately wide apertures and weathered rough planar surfaces (Table 6.59).

Table 6.59 BDPD tunnel joint properties

| Property | Descriptive class | Values range | Basalt (%) | Tuffite (%) |
|---|---|---|---|---|
| *Spacing (mm)* | Extremely close | <20 | 5 | 2 |
| | Very close | 20-60 | 33 | 16 |
| | Close | 60-200 | 42 | 69 |
| | Moderate | 200-600 | 20 | 10 |
| | Wide | 600-2000 | - | 3 |
| *Persistence (m)* | Very low | <1 | 33 | 8 |
| | Low | 1-3 | 56 | 9 |
| | Medium | 3-10 | 11 | 34 |
| | High | 10-20 | - | 31 |
| | Very high | >20 | - | 14 |
| *Aperture (mm)* | Very tight | <0.1 | 8 | 12 |
| | Tight | 0.1-0.25 | 14 | - |
| | Partly open | 0.25-0.50 | 10 | 2 |
| | Open | 0.50-2.50 | 16 | 20 |
| | Moderately wide | 2.5-10 | 48 | 51 |
| | Wide | >10 | 4 | 15 |
| *Roughness* | Rough undulating | IV | 11 | 5 |
| | Smooth undulating | V | 3 | 7 |
| | Slickensided undulating | VI | 10 | - |
| | Rough planar | VII | 61 | 88 |
| | Smooth planar | VIII | 6 | - |
| | Slickensided planar | IX | 9 | - |
| *Weathering ($w_c$)* | Fresh/unweathered | | 22 | - |
| | Moderately weathered | | 67 | 2 |
| | Weathered | | 11 | 98 |

### 6.11.3 Support Predictions Using Classification Methods

In their study, Gurocak et al. (2007) applied RMR and Q to the average or most common rock mass conditions observed from basalt and tuffite bore cores. Extreme values of rock engineering parameters were excluded from the study. For intact rock strength and RQD, their average values were used and for discontinuity spacing and surface conditions etc, their most common values were used in assigning ratings as per RMR and Q. The ratings assigned for the relevant input parameters and the final RMR and Q ratings are listed in Table 6.60.

Table 6.60 Summary of the RMR and Q ratings for the BDPD tunnel

| RMR parameter | Basalt | Tuffite | Q parameter | Basalt | Tuffite |
|---|---|---|---|---|---|
| *IRS* | 5 | 2 | *RQD* | 62 | 25 |
| *RQD* | 12 | 6 | *Jn* | 15 | 12 |
| *JS* | 7.3 | 6 | *Jr* | 1.5 | 1.5 |
| *JC* | 17 | 10 | *Ja* | 6 | 8 |
| *GW* | 15 | 15 | *Jw* | 1 | 1 |
| *RA* | 0 – (-5) | -5 | *SRF* | 1 | 2.5 |
| RMR value | 56.3 -51.3 | 34 | Q value | 1.03 | 0.156 |

As can be seen from Table 6.60, RMR classified basalt and tuffite along the tunnel alignment as fair rock (Class III) and poor rock (Class IV), respectively, and Q classified them as poor rock and very poor rock, respectively. Table 6.61 presents the tunnel support determined by the two classification methods. Since the RMR support recommendations given in the literature are for 10 m diameter tunnels only, the bolt lengths presented in Table 6.61 were reduced to suit the 5 m diameter of the tunnel. To determine support as per $Q_{94}$, an *ESR* value of 1.6 (for water tunnels) was used to derive the equivalent dimension, *De*=(*Span/ESR*)= 5/1.6 = 3.125. For this case study, only the $RMR_{89}$ and $Q_{94}$ were applied. {Note that the bolt lengths listed by Gurocak et al., (2007) are erroneous.}

Table 6.61 The RMR and Q recommended support for the BDPD tunnel

| | RMR | | Q | |
|---|---|---|---|---|
| | Basalt | Tuffite | Basalt | Tuffite |
| Rock mass class | Fair | Poor | Poor | Very poor |
| Bolts (m) | L=4 S=1.5-2 | L=4-5 S=1-1.5 | L=4 S=1.7 | L=4 S=1.3-1.5 |
| Shotcrete (mm) | 50-100 (mr) | 100-150 (mr) | 40-100 | 90-120 (Fr) |
| Steel ribs (m) | | S=1.5* | | |

L=length, S=spacing, mr=mesh reinforced, Fr=fibre reinforced, *=light to medium set where required

### 6.11.4 Tetrahedral Wedge Stability Analysis

The tetrahedral rock wedge analysis carried out using UNWEDGE showed that several rock wedges were kinematically possible in both basalt and tuffite present along the tunnel alignment. The potentially unstable significant rock wedges are listed in Table 6.62. The analysis showed that since the dip angles of the discontinuity sets in basalts are steep, the potentially unstable rock wedges are limited in number and their exposed surface area in the tunnel periphery is small. They can, therefore, be stabilised by spot bolting. On the other hand, one of the major discontinuity sets (bedding plane set) that form the rock wedges in tuffites is flat dipping (see Table 6.58). The rock wedges formed by such discontinuities have large surface areas exposed in the tunnel roof and require systematic bolting for stabilisation. As can be seen from Table 6.62 the flat dipping bedding plane set (Set 1) contributes to all three significant rock wedges in tuffites. The stability of these wedges were analysed using a nominal joint shear strength parameter of c=0 and $\Phi=30^{o}$. Two stress scenarios were considered; the first assumed that the wedges are subjected to gravity loading only with no effect from the in situ stress field, the second included an in situ stress field assumed to be due to the weight of the overlying rock with k=1. Their FOS with and without the stress field are given as FB and FBS in Table 6.62.

The analysis also showed that the RMR and Q predicted rock bolts are sufficient to stabilise both potentially unstable falling and sliding rock wedges in basalts and tuffites, providing that bolts are installed to intersect them.

Table 6.62 Tetrahedral rock wedges in the BDPD tunnel

| Rock type | Wedge # | Sets | Failure mode | Apex (m) | Weight (kN) | FB | FBS |
|---|---|---|---|---|---|---|---|
| Basalt | 1 | 123 | Sliding | 2.8 | 60 | 0.24 | 0.00 |
| | 2 | 134 | Falling | 1.6 | 84 | 0.00 | 0.00 |
| | 3 | 234 | Sliding | 2.0 | 119 | 0.20 | 0.00 |
| Tuffite | 1 | 123 | Sliding | 1.7 | 261 | 0.32 | 1.12 |
| | 2 | 124 | Sliding | 1.2 | 126 | 0.10 | 0.68 |
| | 3 | 124 | Falling | 0.9 | 75 | 0.00 | 0.60 |

### 6.11.5 Numerical Analysis of the BDPD Tunnel

In their study Gurocak et al. (2007) conducted a detailed finite element method of analysis using the Phase2 software package developed by Rocscience (1999) to simulate the behaviour of the tunnel. Phase2 is a 2D elasto-plastic finite element stress analysis program for underground or surface excavations in rock or soil. It models the rock mass as a continuum. Two models representing basalt and tuffite were analysed assuming that the rock masses around the tunnel are isotropic and failure occurs according to the Hoek-Brown failure criterion discussed in Chapter 3. The rock mass properties used in the analysis were estimated based on several empirical guidelines including the RMR and Q classification methods.

The analysis showed that induced stress levels around the tunnel were low and the total displacement predicted by the Phase2 models for basalts and tuffites were 0.2 mm and 1.2 mm respectively, indicating that major rock mass instability in the tunnel was unlikely. This is not surprising because the maximum vertical stress due to overburden in basalts and tuffites are 0.97 and 0.44 MPa which are low compared to the strength of both intact rock material and the two rock masses. It also showed that the RMR and Q predicted support would further reduce the rock mass deformation around the tunnel. (Note that the rock bolts used by Gurocak et al. were too long for the diameter of the tunnel and their interpretation regarding the predicted plastic zone around the tunnel is not relevant.)

### 6.11.6 Actual Support Installed

During excavation of the tunnel, Gurocak (2007) found that the actual conditions of tuffite were slightly better than those predicted using bore core data obtained during site investigations. Despite the variations in the actual conditions, the support measures installed in the tuffite rock unit were similar to those determined by the two empirical methods. The actual conditions of basalts were similar to those anticipated based on the results of the site investigation and classification of the rock mass according to RMR and Q. Although systematic rock bolts and shotcrete were recommended by the two methods for basalt, only local (spot) bolting was used during construction of the tunnel (Gurocak, 2007).

### 6.11.7 Discussion

The original support design for the BDPD tunnel was based on the application of the RMR and Q indices using the rock mass data collected primarily from cored boreholes drilled along the tunnel alignment. A total length of 1195 m of core was logged from 20 boreholes and additionally, surface exposures were also mapped (Gurocak et al., 2007). Based on the bore core data the two empirical methods predicted some instability problems in basalts. Both recommended rock bolts and shotcrete for basalts. The two empirical methods indicated that substantial support would be required for tuffites, the weaker of the two rock types. However, during construction, the actual conditions of tuffites were slightly better than those predicted during site investigations. The actual conditions of basalts were similar to those anticipated based on the application of RMR and Q using bore core data obtained during the site investigation. Nevertheless, the actual support measures installed in basalts were less than those recommended by the RMR and Q methods. This indicates that for this tunnel based on the data obtained primarily from bore core, the RMR and Q predicted lower rock mass qualities for tuffites than those actually intersected during excavation, but the support predictions were comparable to those actually installed. In basalt the predicted rock mass conditions were similar to those intersected during construction but the predicted support measures exceeded those actually installed.

### 6.11.8 Conclusion

Both the RMR and Q methods predicted some instability problems in basalts. The RMR system recommended systematic rock bolting and mesh reinforced shotcrete for tunnel roof in basalts. The Q system recommended systematic rock bolting and un-reinforced shotcrete. Although the predicted conditions were similar to those intersected during excavation only spot bolting was used. Thus the two methods could be considered overconservative in this case.

The empirical methods indicated that substantial support would be necessary for tuffites. RMR recommended systematic rock bolting, mesh reinforced shotcrete and occasional light to medium steel sets for the tunnel in tuffites. Q recommended systematic rock bolting and fibre reinforced shotcrete. However, during the construction of the tunnel, the actual conditions of tuffite were slightly better than those predicted during site investigations. Besides the variations in the actual conditions the support measures installed in the tuffite rock unit were similar to those determined by the empirical methods.

The RMR and Q predictions made using data obtained primarily from bore cores, depending on the rock type and its condition, could either be adequate or overconservative.

## 6.12 CASE STUDY 10:
## The Ramboda Pass Highway (RPH) Tunnel, Sri Lanka

The Ramboda Pass Highway tunnel is part of the Gampola to Nuwara Eliya AA005 highway improvement project in the central highlands of Sri Lanka. The tunnel was constructed as a double lane alternative for a narrow single lane portion of the existing highway that traverses around a steep rocky hill slope at the Ramboda Pass. It is located within a hill and has a slightly curved alignment varying between 142$^o$ and 178$^o$ to the north with an overburden of between about 15 m at the upstream portal and about 50 m at mid length. The horseshoe shaped tunnel is 222 m long and was excavated by drill and blast methods. The as-excavated diameter of the tunnel was about 8 m and the finished diameter varies between 7 m at the upstream end and 7.5 m at the downstream end. Its construction was completed in 2007.

### 6.12.1 Project Site Geology

The project is located on a gently dipping limb of a regional fold dominated by Pre-Cambrian crystalline basement rocks, mainly charnockitic and garnetiferous gneisses of the Highland Series rock formation of Sri Lanka. These are high grade metamorphic rocks with well developed foliation planes. The geological structure includes minor faults and well developed joint systems, sub-vertical joints being prominent. In the general area of the project, steep natural rock faces have been formed along the near vertical joints. The sub-vertical joints also contribute to differential weathering of the rock mass.

### 6.12.2 Rock Mass Data

The rock types intersected in the tunnel are charnockitic and garnetiferos gneisses whose conditions vary along the tunnel alignment. From the downstream end to about 177 m, the rocks are mostly fresh. In the next 45 m to the upstream end the rock is weathered. The weathering grade increases from moderately to highly weathered and at the upstream portal, the rock is highly to completely weathered. The UCS of unweathered rock materials varies between 60 and 120 MPa. The tunnel was mostly dry with occasional dripping. The last 50 m at the upstream end was wet with water seepage requiring local drainage control measures before the installation of rock support, particularly shotcrete. In good quality rock, the tunnel was excavated by full face drilling and blasting, and in weaker zones comprising weathered and fractured material where water seepage was also present, the top heading and benching method was adopted. In very weak ground, a smaller pilot drift was advanced and the crown was stabilised before the removal of the remaining rock in several stages.

Four major and two minor geological discontinuity sets were present in the tunnel and their general orientations are presented in Table 6.63. Set 1 represent well developed foliation joints in the gneissic rocks intersected in the tunnel. Sets 2 and 3 are well developed sub-vertical joints present throughout the tunnel. Set 4 is a major set present along the most of the tunnel. Sets 5 and 6 are minor joint sets occurring at random.

Table 6.63 Discontinuity orientations in the RPH tunnel

| Set No, | Dip | Dip direction | Comment |
|---|---|---|---|
| Set 1 | 26 | 246 | Major set |
| Set 2 | 90 | 300/120 | Major set |
| Set 3 | 80 | 215 | Major set |
| Set 4 | 55 | 230 | Major set |
| Set 5 | 78 | 163 | Random set |
| Set 6 | 56 | 123 | Random set |

Joint surface conditions of all sets vary from rough undulating to smooth planar with occasional slickensided surfaces. Most of the joint surfaces are fresh and stained or coated. Some are slightly weathered with clayey infilling material. The minimum spacing between members of the major joints sets is approximately 0.6 m and the maximum is greater than 2 m. Narrow fractured zones parallel to some of the major joints were also present particularly in the last 50 m at the upstream end of the tunnel.

### 6.12.3 Support Predictions by the RMR and Q Methods

During construction of the tunnel, $Q_{94}$ was applied to the entire length by detailed mapping of the exposed rock mass. Based on the rock mass conditions and the Q values assigned, the tunnel was divided into 19 geotechnical sectors. In the present study, for comparison purposes, the $RMR_{89}$ index was also applied by indirect means using the *RMR-Q* linear correlation (Equation 5.1) proposed by Bieniawski (1976, 1989, 1993).

$$RMR = 9 \ ln \ Q + 44 \tag{5.1}$$

This equation was used because ratings for the RMR input parameters were not determined during excavation. Equation 5.1 may not necessarily be applicable to the rock mass conditions in the tunnel. Nevertheless, for the present study it was assumed that the equation would be accurate enough for comparison of the support measures predicted by the two classification systems. Table 6.64 presents a summary of the RMR and Q values and the relevant rock mass classes along the tunnel. The amount of rock mass falling into each Q and RMR class are shown in Figure 6.29.

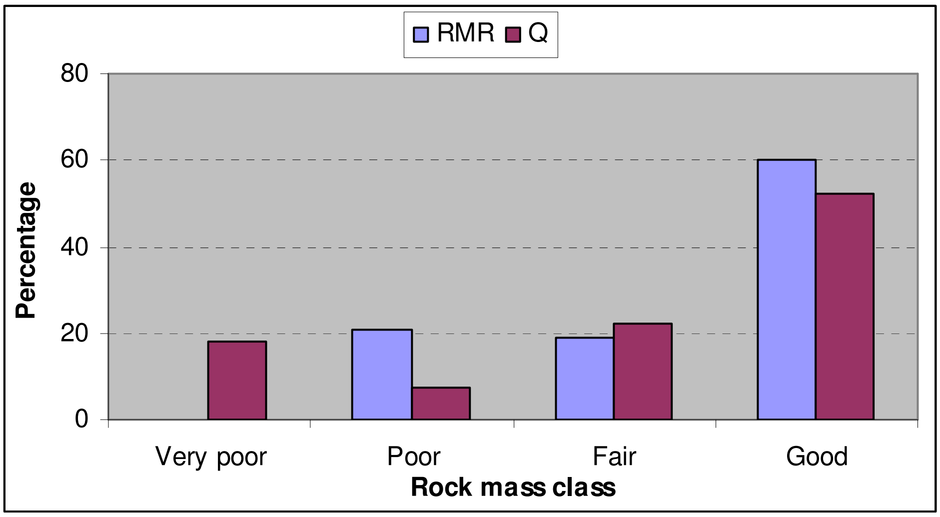


Figure 6.29 Percentage of rock in each relevant class (RPH tunnel)

Table 6.64 RMR and Q ratings for the RPH tunnel

| Sector | Chainage (m) | Q value | Q Class | RMR value | RMR Class |
|---|---|---|---|---|---|
| 1 | 973-976 | 7 | Fair | 62 | II – Good |
| 2 | 976-978 | 10 | Good | 65 | II – Good |
| 3 | 978-983 | 15 | Good | 68 | II – Good |
| 4 | 983-1018 | 20 | Good | 71 | II – Good |
| 5 | 1018-1034 | 11 | Good | 66 | II – Good |
| 6 | 1034-1043 | 5 | Fair | 58 | III – Fair |
| 7 | 1043-1057 | 8 | Fair | 63 | II – Good |
| 8 | 1057-1075 | 20 | Good | 71 | II – Good |
| 9 | 1075-1104 | 14 | Good | 68 | II – Good |
| 10 | 1104-1115 | 24 | Good | 73 | II – Good |
| 11 | 1115-1125 | 6 | Fair | 60 | III – Fair |
| 12 | 1125-1138 | 5.5 | Fair | 59 | III – Fair |
| 13 | 1138-1141 | 0.2 | Very poor | 30 | IV – Poor |
| 14 | 1141-1144.5 | 3 | Poor | 54 | III – Fair |
| 15 | 1144.5-1168 | 0.6 | Very poor | 39 | IV – Poor |
| 16 | 1168-1172.5 | 1.5 | Poor | 48 | III – Fair |
| 17 | 1172.5-1175 | 0.7 | Very poor | 41 | III – Fair |
| 18 | 1175-1185 | 0.5 | Very poor | 38 | IV – Poor |
| 19 | 1185-1195 | 0.3 | Very poor | 33 | IV – Poor |

For each relevant Q and RMR class support measures were determined and the results are presented in Table 6.65. Since the tunnel was constructed between 2006 and 2007, only the current versions of the two classification methods, $RMR_{89}$ and $Q_{94}$, were applied. To determine support according to $Q_{94}$, an *ESR* value of 1.0 (for major road tunnel) was used; then the equivalent dimension, *De*=(*Span/ESR*)= 7.5. The RMR derived support measures are as recommended in the literature for 10 m diameter tunnels. These were assumed to be applicable to the 7.5 m wide Ramboda Pass tunnel because the previous RMR versions (Bieniawski, 1974, 1975) recommended the same support measures for 5 to 12 m wide tunnels.

Table 6.65 RMR and Q derived permanent support for the RPH tunnel

| | Q derived support | | RMR derived support | |
|---|---|---|---|---|
| Chainage (m) | Bolts (m) | Shotcrete (mm) | Bolts (m) | Shotcrete (mm) |
| 973-976 | L=2.8 S=2.2 | 40-100 | L=3 S=2.5 | 50 |
| 976-978 | L=2.8 S=2 | None | L=3 S=2.5 | 50 |
| 978-1018 | SB | None | L=3 S=2.5 | 50 |
| 1018-1034 | L=2.8 S=2 | None | L=3 S=2.5 | 50 |
| 1034-1043 | L=2.8 S=2.2 | 40-100 | L=4 S=1.5-2 | 50-100 (mr) |
| 1043-1057 | L=2.8 S=2 | None | L=3 S=2.5 | 50 |
| 1057-1115 | SB | None | L=3 S=2.5 | 50 |
| 1115-1138 | L=2.8 S=2.2 | 40-100 | L=4 S=1.5-2 | 50-100 (mr) |
| 1138-1141 | L=2.8 S=1.4 | 90-120 (Fr) | L=4-5 S=1-1.5 | 100-150 (mr)* |
| 1141-1144.5 | L=2.8 S=2 | 40-100 | L=4 S=1.5-2 | 50-100 (mr) |
| 1144.5-1168 | L=2.8 S=1.6 | 50-90 (Fr) | L=4-5 S=1-1.5 | 100-150 (mr) |
| 1168-1172.5 | L=2.8 S=2 | 50-90 (Fr) | L=4 S=1.5-2 | 50-100 (mr) |
| 1172.5-1175 | L=2.8 S=1.6 | 50-90 (Fr) | L=4 S=1.5-2 | 50-100 (mr) |
| 1175-1185 | L=2.8 S=1.6 | 90-120 (Fr) | L=4-5 S=1-1.5 | 100-150 (mr)* |
| 1185-1195 | L=2.8 S=1.4 | 90-120 (Fr) | L=4-5 S=1-1.5 | 100-150 (mr)* |

L=length, S=spacing, SB=spot bolting, mr=mesh reinforced, Fr=fibre reinforced, *=light to medium ribs where required

### 6.12.4 Tetrahedral Wedge Stability Analysis

A tetrahedral rock analysis was undertaken using the discontinuity orientation data obtained by rock mass mapping during excavation of the tunnel. It showed that several rock wedges are kinematically unstable in the tunnel roof and walls. To compute wedge FOS, the shear strength parameters of the discontinuities were estimated taking into account their surface characteristics described earlier. The selected parameters were c=10kPa and $\Phi=35^{o}$, which are considered to represent rough planar joint surfaces with non-softening mineral coatings, i.e. $Jr$=1.5 and $Ja$=2.0. Details of the potentially unstable maximum size rock wedges are presented in Table 6.66. The wedge analysis showed that the RMR and Q derived support measures would be sufficient to stabilise the potentially unstable rock wedges in the tunnel.

Table 6.66 Tetrahedral rock wedges in the RPH tunnel

| Wedge # | Sets | Location | Apex (m) | Weight (kN) | FB |
|---|---|---|---|---|---|
| 1 | 123 | Roof | 4.7 | 1318 | 0.34 |
| 2 | 125 | Roof | 6.3 | 988 | 1.12 |
| 3 | 135 | Roof | 4.0 | 585 | 0.90 |
| 4 | 136 | Roof | 2.2 | 283 | 0.45 |
| 5 | 136 | Right wall | 1.6 | 225 | 0.92 |
| 6 | 156 | Roof | 1.2 | 38 | 0.75 |
| 7 | 345 | Roof | 6.0 | 675 | 0.65 |
| 8 | 346 | Roof | 3.3 | 281 | 0.67 |
| 9 | 356 | Roof | 7.4 | 569 | 0.56 |
| 10 | 456 | Roof | 2.8 | 150 | 0.63 |

FB=FOS without support

### 6.12.5 The Primary Support Measures Installed

The primary support measures installed in the RPH tunnel were mainly un-reinforced shotcrete and rock bolts. Rib reinforcement and spilling bars or forepolling were also used in the first 40 m from the upstream end. Table 6.67 presents the Q derived primary support and installed primary support. To determine primary (temporary) support for the tunnel crown, Q values were increased to 5Q as recommended in the Q system. Table 6.67 shows that the first 152 m or 68% of the tunnel was supported with spot bolting and shotcrete, the next 30 m or 14% was supported with pattern bolting and shotcrete and the final 39.5 m or 18% was supported with pattern bolting, rib reinforced shotcrete and spilling bars. According to the Q system, 133 m or 60% of the tunnel required no primary support, 35.5 m or 16% required only spot bolting, 40.5 m or 18% required pattern bolting and shotcrete and the remaining 13 m or 6% required pattern bolting and fibre reinforced shotcrete. It should be noted that the $Q_{94}$ (Barton and Grimstad, 1994) recommend that for temporary support, the *ESR* should be increased to 1.5 x *ESR* in addition to the five fold increase in the Q value. If the increased *ESR* had been used, the Q system derived support would have been much less than those listed in Table 6.67.

As can be seen from Table 6.67, the Q derived primary support measures were less than those actually installed during construction. While the installation of rib reinforced shotcrete in the last 18% of the tunnel may be considered as part of the permanent support system, it would be fair to consider that the installation of spot bolting and plain shotcrete were only meant for construction safety, meaning they are primary or temporary support. This indicates that the Q derived primary support measures were inadequate for 60% of the RPH tunnel. Since RMR recommendations are for permanent support, ideally no comparison should be made with the temporary support. Nevertheless, it can be seen from the two tables the RMR recommendations also differ from the actual support used except for the those installed in weaker rock zones.

Table 6.67 Q derived temporary support and installed primary support for the RPH tunnel

| Chainage (m) | $Q_{primary}$ =5Q | Q Derived primary support | | Primary support installed | |
|---|---|---|---|---|---|
| | | Bolts (m) | Shotcrete (mm) | Bolts (m) | Shotcrete (mm) |
| 973-1034 | 35-100 | None | None | SB | 25 |
| 1034-1043 | 25 | SB | None | SB | 70 |
| 1043-1115 | 40-120 | None | None | SB | 25 |
| 1115-1125 | 30 | SB | None | SB | 25 |
| 1125-1138 | 27.5 | SB | None | L=4 S=2 | 70 |
| 1138-1141 | 1.0 | L=2.8 S=1.3 | 50-90 (Fr) | L=4 S=1.4 | 70 |
| 1141-1144.5 | 15 | SB | None | L=4 S=2 | 70 |
| 1144.5-1155.5 | 3.0 | L=2.8 S=1.4 | 40-100 | L=4 S=1.4 | 70 |
| 1155.5-1168 | 3.0 | L=2.8 S=1.4 | 40-100 | L=4 S=1.4 | 120 (RR & sp) |
| 1168-1172.5 | 7.5 | L=2.8 S=1.8 | 40-100 | L=4 S=1.6 | 120 (RR & sp) |
| 1172.5-1185 | 3.0-3.5 | L=2.8 S=1.4 | 40-100 | L=4 S=1.4 | 120 (RR & sp) |
| 1185-1195 | 1.5 | L=2.8 S=1.3 | 50-90 (Fr) | L=4 S=1.4 | 120 (RR & sp) |

L=length, S=spacing, SB=spot bolting, Fr=fibre reinforced, RR=rib reinforced, sp=spiling bars

### 6.12.5 Permanent Support Measures

The tunnel was fully concrete lined as required by the client. The concrete liner design was not exclusively based on tunnel stability concerns and therefore cannot be directly compared with the support predicted by the two empirical methods. In addition to stability concerns, the liner design criteria included aesthetics, traffic management and pedestrian access etc. However, since the tunnel was to be fully lined, the primary support was kept to the required minimum to ensure safety during construction.

### 6.12.6 Discussion

In general, the permanent rock bolting systems recommended by the two classification methods were comparable, except for two intervals from 978 to 1018 m and 1057 to 1115 m. For these two intervals representing a total of 98 m or 44% of the tunnel, the Q system recommended only spot bolting, whereas RMR recommended pattern bolting plus shotcrete. One major difference in the support recommendations of the two methods is that the RMR recommended shotcrete for the entire tunnel while Q did not recommend shotcrete for 130 m or 56% of the tunnel. For the last 80 m where rock mass conditions were weaker, both classification methods recommended pattern bolting and mesh or fibre reinforced shotcrete. The only difference is that RMR also recommended light to medium steel ribs for the last 20 m. This latter recommendation conforms to the actual support installed in this length. In contrast, for weaker zones comprising weathered rock the Q recommended permanent support measures were less than those actually installed.

It should be noted that RMR was not applied directly to the tunnel during construction and its support estimations were by transforming the Q values into RMR values using Equation 5.1. Thus the RMR values, and therefore the support recommendations, depend on the Q values. Despite this the support requirements predicted by the two methods do not necessarily agree with each other. Further the RMR recommended support for the weaker rocks in this tunnel compared well with the support installed while the Q recommended support differs.

### 6.12.7 Conclusion

The Q derived primary support measures were inadequate for 60% of the RPH tunnel. In poorer rock conditions in the 40 m length from the upstream portal the support installed included pattern bolting, spilling bars and rib reinforced shotcrete. These are comparable to the support recommended by the RMR method. Despite the fact that the RMR system was applied only indirectly by transforming the Q values of the tunnel into RMR values, the support requirements predicted by the two methods did not fully agree with each other.

### 6.13 Correlation of RMR and Q Values

As discussed in Section 5.3, several researchers have correlated the RMR and Q values obtained from different tunnelling projects with the intention of linking the two rating systems. Each of these correlations is somewhat different from the next and it is apparent from the discussions presented in Section 5.3 that different correlations are possible from the RMR and Q values obtained from different rock mass conditions. Despite this possibility, there is a tendency among some practitioners of rock engineering to overly rely on the first correlation published in 1976 (Equation 5.1) and transform ratings between the two systems. This tendency is injudicious and deserves scrutiny.

This section correlates the *RMR* and *Q* values obtained from six of the case studies representing four projects discussed in the preceding sections. In light of the correlations obtained from different rock mass conditions encountered in the case tunnels, it is obvious that there is no sound scientific basis to assume a universally applicable linear relationship between RMR and Q, as alluded to by some research publications.

The four projects considered for this purpose are:

- The Chiew Larn Hydropower Project (CLHP) (two tunnels)
- The Huai Saphan Hin Hydropower Project (HSHP) (one tunnel)
- The Lam Ta Khong Pumped Storage Power Project (LTKP) (three tunnels)
- The Klong Tha Dan Irrigation Project (KTDP) (five tunnels)

The CLHP has two tunnels discussed as Case Studies 1 and 2 in Sections 6.3 and 6.4. A total of 56 $RMR_{79}$ and $Q_{74}$ data pairs were derived from the two tunnels and a $Q$ versus $RMR$ plot is presented in Figure 6.30a. The HSHP has one tunnel referred to as Case Study 3 in Section 6.5. Fifteen $RMR_{79}$ and $Q_{74}$ data pairs were obtained and a plot of the data is presented as Figure 6.30b. The LTKP has three tunnels discussed as Case Studies 5 and 6 in Sections 6.7 and 6.8 and a branch tunnel excavated from the case tunnel discussed in Section 6.8. In all, 114 $RMR_{89}$ and $Q_{74}$ data pairs obtained from the LTKP tunnels representing a total length of 2260 m were used in the present study. A $Q$ versus $RMR$ plot of the 114 data pairs is presented in Figure 6.30c. KTDP has five tunnels discussed as Case Study 7 and for the present study 170 $RMR_{89}$ and $Q_{74}$ data pairs representing the five tunnels were used and plots of the data are shown in Figure 6.30d.

The data used were obtained by mapping exposed rock masses and testing intact rock substances in the tunnels. The ratings assigned to classification parameters are therefore representative of the state of the rock masses intersected in the 11 tunnels.

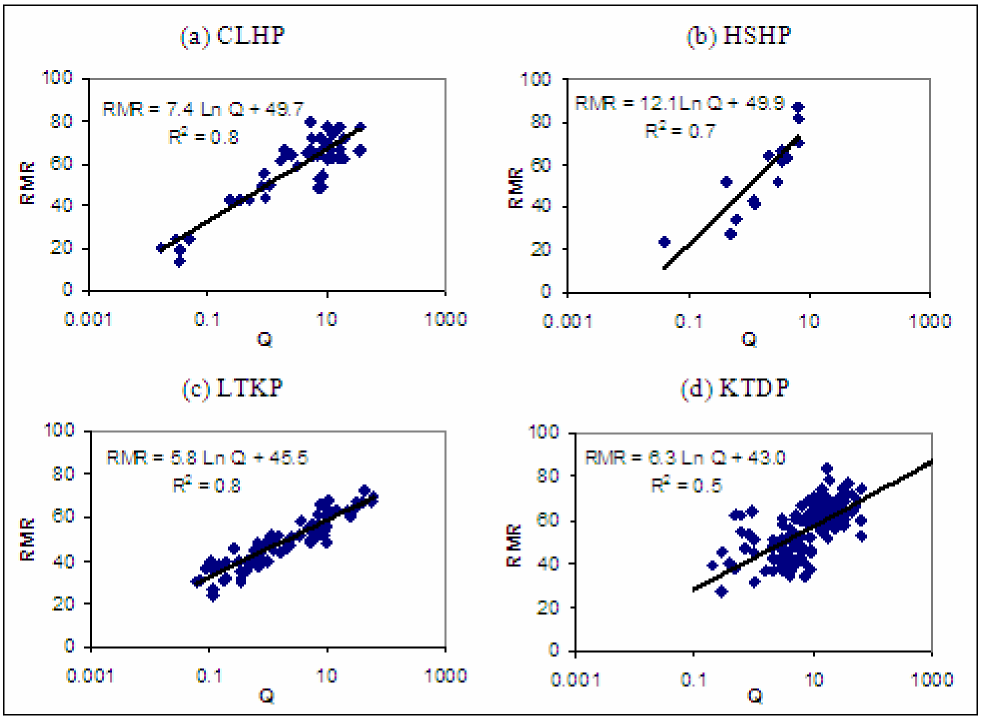


Figure 6.30 RMR and Q correlations for the CLHP, HSHP, LTKP and KTDP tunnels

### 6.13.1 Data analysis

By linear regression analysis of the data presented in Figure 6.30, four relationships similar to that proposed by Bieniawski (1976), with $Q$ as the independent variable, can be obtained (Table 6.68 column 2). If $RMR$ is assumed to be the independent variable, the corresponding $RMR$-$Q$ relationships for the same data sets are those given in the fourth column of Table 6.68.

Table 6.68 New RMR and Q correlations from case studies

| Project | Independent variable Q | Eq. No. | Independent variable RMR | Eq. No. |
|---|---|---|---|---|
| CLHP | $RMR = 7.4 \ln Q + 49.7$ | 6.1 | $Ln\, Q = 0.109\, RMR - 5.169$ | 6.2 |
| HSHP | $RMR = 12.1 \ln Q + 49.9$ | 6.3 | $Ln\, Q = 0.061\, RMR - 2.923$ | 6.4 |
| LTKP | $RMR = 5.8 \ln Q + 45.5$ | 6.5 | $Ln\, Q = 0.145\, RMR - 6.551$ | 6.6 |
| KTDP | $RMR = 6.3 \ln Q + 42$ | 6.7 | $Ln\, Q = 0.073\, RMR - 2.062$ | 6.8 |
| All data | $RMR = 6.2 \ln Q + 45.5$ | 6.9 | $Ln\, Q = 0.102\, RMR - 4.167$ | 6.10 |

It can be shown that although the two equations with *Q* and *RMR* as independent variables were derived from the same data set, they do not yield the same results. For instance, from a *Q* value of 0.8, Equation 6.1 would produce a *RMR* value of 48 while in turn Equation 6.2 would transform a *RMR* value of 48 to a *Q* value of 1.05. From a *Q* value of 10, Equation 6.5 would produce a *RMR* value of 58 while from a *RMR* value of 58, Equation 6.6 would return a *Q* value of 7. Similarly, from a *Q* value of 50, Equation 6.5 would predict a *RMR* value of 68 and in turn from a *RMR* value of 68, Equation 6.6 would predict a *Q* value of 28. Further, as in the case of Equation 5.1 of Bieniawski (1976) the equations given in Table 6.63 are not valid for the full range of Q values. For instance, according to Equation 6.3, if $Q<0.02$, $RMR<0$ and if $Q>45$, $RMR>100$.

As can be seen from Figure 6.30, the main difference between the four equations is the gradient of the regression lines. A review of the rock mass conditions in the case tunnels revealed that more than one rock mass parameter contributes to the variations in the gradient of the regression lines. These include *IRS*, *JS, JC* and *RA* of the RMR system and *Jn*, *Jr*, *Ja* and *SRF* of the Q system. According to the data available from the case studies, the commonly held perception that the use of *SRF*=1 in deriving Q values would result in a better correlation between the two systems is not sustainable.

For instance, the key parameters that influence the slope of the regression line given by Equation 6.7 are the number of joint sets, joint roughness, joint alteration and stress conditions. In deriving *Q* values, these four parameters (i.e. *Jn*, *Jr*, *Ja* and *SRF*) are rated separately and explicitly, whereas in *RMR* the number of joint sets is considered only implicitly in the *JS* parameter, joint roughness and alteration are only two of the three components of the *JC* parameter, and stress is not considered.

Consequently, the variations in *Jn*, *Jr*, *Ja* and *SRF*, without significant variations in the other key parameters of the RMR system (i.e. *IRS*, *JS* and *RA*) would result in a flatter regression line as observed in this study. Similarly, the variations in the *IRS*, *JS* and *RA* parameters, without noticeable variations in the key parameters of the Q system would result in a steeper regression line. Therefore, it may be concluded that a linear relationship obtained by regression analysis of the *RMR* and *Q* data is applicable only to that particular rock mass conditions from which the relationship was obtained. Even for the same rock mass conditions, if the data points in the *RMR-Q* plot are widely scattered, such relationships are of very little practical value and should not be used for transforming the ratings between the two methods.

### 6.13.2 Conclusions

The linear regression analysis of the case tunnel data from four projects confirmed that a different relationship can be obtained for each case study and that each is applicable only to the particular rock mass conditions from which the relationship was obtained. Even for the same rock mass, if the data used are widely scattered, such relationships are of very little practical value and their use for transforming the ratings between the two methods could lead to errors. As observed by Kaiser et al. (1986), the analysis also showed that the results of correlations depend on the choice of the dependent variable. From the available information, it is apparent that there is no sound scientific basis to assume a universally applicable linear relationship between the two.

When both methods are to be applied to a project, which is desirable, each should always be applied independent of the other, without attempting to convert the ratings of one method to that of the other using the relationships published in the literature. Such relationships, bearing in mind their obvious limitations, may be used as a crude guide for checking the general accuracy of the ratings derived by the two systems.

# CHAPTER 7

# CONCLUSIONS AND RECOMMENDATIONS

## 7.1 Introduction

The review of previous studies and the analysis of case studies presented in Chapters 5 and 6, respectively, have resulted in an increased understanding of the RMR and Q rock mass classification systems and some of the situations where tunnel support requirements differ from those predicted by the two systems. The present study has also resulted in several conclusions and identified areas for further research to better understand the reliability of the two systems. The conclusions drawn from the review of the previous studies are outlined in Section 5.4 and those from the case studies are presented in the relevant sections of Chapter 6. In general, these conclusions can be divided into three broad groups:

(a) those applicable only either to RMR or Q
(b) those common to both RMR and Q
(c) those on the RMR-Q correlations.

These are discussed in the following sections.

## 7.2 Conclusions on the RMR System

- The application of the RMR system requires only minimal user experience. After a few applications, a user should be able to easily and confidently apply it provided that he/she has some experience in underground excavation design.

- The six parameters used in the RMR system represent true factors of rock masses. They can be easily determined either during investigations using bore core information or during construction by direct observation of excavations.

- The stability of excavations in jointed rock is significantly influenced by joint spacing, orientation and their surface characteristics. These are all included in the RMR system. Some of the rating allocations, however, are insensitive to minor variations of the parameters that may a have critical influence on the behaviour of a rock mass.

- Joint spacing of the rock mass is counted twice, first directly through the *JS* parameter and then indirectly through the *RQD* parameter. This gives an unnecessary additional weightage to discontinuity spacing.

- The RMR support recommendations are for 10 m wide horseshoe shaped tunnels only and its application to other opening sizes and shapes is user dependent.

- The support recommendations are not tailored to the purpose of the excavations. A one-size-fits-all approach is adopted in recommending support measures. This clearly is a major limitation because different support systems and quantities could be warranted for the same rock mass depending on the purpose of the excavation. This aspect is not considered in the RMR system.

- The system has not been updated since 1989 therefore modern support types i.e. fibre reinforced shotcrete are not included in the recommendations.

- In situ stress field is not considered in the classification of a rock mass. Its support recommendations are for tunnels with less the 25 MPa vertical stress magnitude. Its applicability to stress induced stability problems is open to conjecture. The creator of the system did not recommend its use for such stability problems.

### 7.3 Conclusions on the Q System

- The six parameters used in the Q system represent true factors of rock masses. They can be determined either during investigation or construction. The descriptions used to assess the joint conditions are relatively rigorous, leave less room for subjectivity and are sensitive to minor variations in properties.

- The Q support recommendations reflect the excavation size and its purpose. Different support measures are recommended for the roof and walls of an excavation and also for temporary excavations. The support recommendations have evolved over time and reflect technological developments.

- The Q system does not directly account for the discontinuity orientation in the assessment procedure, although it is included implicitly by classifying the joint roughness and alteration of only the most unfavourably oriented joint sets or discontinuities. No guidance is provided on how to decide on the critical or most unfavourable discontinuity orientation and its selection is user dependent.

- Discontinuity spacing which is an important parameter for the jointed rock masses is not considered directly. The strength of rock material is not considered directly although it is included indirectly in the *SRF* term only if the stability of an excavation is governed by the in situ stress field.

- Where relevant the in situ stress field can be taken into account in classifying a rock mass by selecting a suitable value for the *SRF* term. However, for rock stress problems in competent rock, a rating scale of 1 to 400 is given for the *SRF* term with no guidance on which value to be used. Its selection, therefore, is largely dependent on the site specific experience. Thus for new sites with rock stress problems the reliability of the Q system is yet to be confirmed.

- For small diameter (~ 3 m) water tunnels, for which *ESR*=1.6 to 2.0, the Q system recommends no support if $Q>1$ and $De\leq 2$. However, support could be warranted when the $Q$ value is as high as 6. For temporary mine openings of around 4 m width, for which the recommended *ESR*=2 to 5, this limitation could lead to safety implications.

- The assertion that a reduced level of disturbance results in TBM driven tunnels and less support would be required compared to drill and blast tunnels is not always tenable. In blocky rock and horizontally bedded rock with sub-vertical joints or high horizontal stresses, more support would be required than Q recommendations. For such rock masses the "no support" boundary in the Q support chart should be shifted to the right. Its shifting to the left for TBM driven tunnels as recommended by the creator of the system would lead to under designs compromising safety.

### 7.4 Conclusions Common to Both the RMR and Q Systems

- The two systems may be applied either at the investigation and design stage of a project or during construction. However, the predictions made based on the bore core data obtained for site investigations may not necessarily reflect the actual conditions encountered during excavation. While in some cases bore core data would predict higher rock mass ratings than those derived using in situ observations, in other cases, the opposite could be true. In some case the predictions based on bore core data are accurate. The currently available information does not show any reliable trends. To what extent the bore core data based ratings differ from the in situ data based ratings appears to depend on the type and condition of rock masses.

- There is not always good agreement between the RMR and Q derived support for excavations. Despite attempts to link the two systems by linear correlations they do not always lead to similar conclusions.

- Previous studies as well as the case studies analysed showed examples in which the RMR and Q predicted support could be either overconservative or optimistic compared to the support installed. In some cases, one system agreed with the installed support measure while the other was either overconservative or optimistic.

- In estimating support pressures (which is an indirect method of support design) the two systems differ in their approaches. The RMR system considers that support pressure depends on both rock mass quality (the *RMR* value) and the width of the opening. In contrast, the Q system assumes that the support pressure is a function of only the rock mass quality (*Q* value, *Jn* and *Jr*), the width of the excavation is ignored. On the other hand, the Q support chart explicitly takes into account the width of the excavation. When estimating support requirements and support pressures, seemingly, the Q system contradicts itself.

- The two systems do not always yield the same support pressures for the same rock mass intersected in a tunnel. For small diameter (~3 m) tunnels, the ratio of Q derived support pressure ($P_Q$) to RMR derived support pressure ($P_{RMR}$) is approximately one when $Q>10$. When $Q \leq 10$, the ratio ($P_Q$:$P_{RMR}$) is always greater than one, and it increases rapidly with a decreasing *Q* value.

- For shallow water conveyance tunnels with the potential for water loss through seepage, the support recommendations of RMR and Q could be inadequate.

- For jointed rock under changing loading conditions, due to an increase or decrease in the overburden pressure or due to an increase in the groundwater pressure, the RMR and Q recommended support could be inadequate to deal with rock block or wedge stability problems.

- In some instances, for moderately jointed rock with the potential for large scale wedge instability, the RMR and Q predicted support could be insufficient.

- For tunnels in moderately jointed low strength rock masses, Q predicted support could be insufficient while RMR predictions may be overconservative.

### 7.5 Correlations of the RMR and Q Systems

The literature review revealed 25 different *RMR-Q* correlations obtained by linear regression analysis of the ratings assigned by the two systems. Each of these correlations is different from the next and the data used in deriving them are often widely scattered. The main reasons for this are the differences in the parameters and the rating methods used, and the manner in which the final *RMR* and *Q* values are computed, i.e. the RMR system is additive and the Q system is multiplicative.

The analysis of the RMR and Q values from four projects further confirmed that a different relationship can be obtained for each case study and that each relationship is applicable only to that particular rock mass conditions from which the relationship was obtained. Even for the same rock mass, if the data used in deriving a relationship are widely scattered, such relationships are of very little practical value and their use for transforming the ratings between the two methods could lead to errors. Further, the analysis also showed that the results of correlations depend on the choice of the dependent variable. The available information shows no scientific basis to assume a universally applicable linear relationship between the two.

When both methods are to be applied to a project, which is desirable, each should always be applied independent of the other, without attempting to convert the ratings of one method to that of the other using the relationships published in the literature. Such relationships, bearing in mind their obvious limitations, may be used as a crude guide for checking the general accuracy of the ratings derived by the two systems.

### 7.6 Recommendations for Future Research

The results of this study clearly show that further research is necessary to establish a better understanding of the reliability of the rock mass classifications methods under a wide range of ground conditions. The following items are recommended:

- This study was based on a limited number of case studies. The case study database needs to be expanded by adding more cases.

- Using the existing data in the present case studies database further analysis should be carried out for other possible failure mechanisms that have not been included in the present study.

- The case studies used represent jointed rock masses subject to low to medium in situ stress conditions. The rock instability in the excavations created in these rock masses are primarily structurally controlled. The massive rocks subjected to high in situ stress conditions and weaker rocks with swelling and squeezing conditions have not been covered in the present study. These conditions need to be researched.

- Support pressure measurements were not available for any of the case studies in the present database. Thus the reliability of the support pressures predicted by RMR and Q could not be assessed, except for the commentary on their variations. This aspect needs a detailed assessment using in situ measurements of support pressures.

- More work is required on the reliability of the RMR and Q predictions made using bore core data alone.

- For stress induced failures in high stress environments the Q system uses a *SRF* value of 5 to 400. Selection of an appropriate *SRF* value from such a wide range is user dependent and in effect could lead to significant errors in support predictions. More work is needed to find representative *SRF* values for high stress conditions.

- The assessment of the reliability of the RMR system is difficult due to its one-size-fits-all support recommendations (i.e. for 10 m wide horseshoe shaped tunnels only). The case study database should be expanded by adding more 10 m wide tunnels to overcome this limitation.

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# APPENDIX A

Appendix A contains the publications listed on pages ii and iii of this thesis

# COMPARISON OF ROCK MASS CLASSIFICATION SYSTEMS WITH AN ANALYTICAL METHOD OF UNDERGROUND EXCAVATION DESIGN

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**ABSTRACT:** Rock mass classification systems are useful tools for underground excavation design. These empirical systems avoid the analysis of failure mechanisms and the forces required to stabilise the potentially unstable rocks, and prescribe stabilisation measures based on past experience in similar ground conditions. There are, however, some limitations in this approach. These systems do not have any scientifically proven means to identify failure mechanisms in a rock mass, thus their predictions can sometimes be erroneous. Further, the rock masses at two sites may appear similar and may be broadly classified into the same class, but their failure mechanisms and volumes can be significantly different, so the stabilisation measures installed at one site may not necessarily be adequate for another. One of the ways to identify the limitations and rectify them where possible is to compare the design predictions of rock mass classification systems with those of other methods for the same excavation. This paper compares the predictions made by two rock mass classification systems with that made using block theory, with specific reference to a 13 m diameter hydropower tunnel.

*Keywords*: rock mass; classification; tunnel; rock blocks; stability; RMR; Q-System

## 1. INTRODUCTION

Two empirical rock mass classification systems, RMR [1] and Q-System [2], have gained wide acceptance since their introduction more than three decades ago. Based on the experience gained from the application in underground excavation projects, revised versions of RMR [3, 4, 5, 6] and Q-System [7, 8] were also issued.

Since it is virtually impracticable to determine the exact engineering properties of the entire rock mass involved in an underground excavation project, these classification systems, which describe the rock mass in a qualitative to semi-quantitative manner and prescribe stabilisation measures based on experience in similar ground conditions, are useful tools for underground excavation designers.

Rock mass classification systems, however, have their limitations. First, these systems do not have any scientifically proven means to identify potential failure mechanisms in a rock mass. Second, the rock mass stabilisation measures may be over-designed for the case studies in the database used in developing the classification system, thus the predictions for new projects may be even more conservative and costly than warranted. Put simply, in some instances, the predictions made using rock mass classification systems may not be reliable.

One approach to assess the reliability of empirical rock mass classification systems (under a given set of conditions) is to compare their predictions with those of analytical methods. Such a comparison will be meaningful only if the ground conditions of the excavation site are well enough understood to apply both empirical and analytical methods with confidence. This can best be done after excavation is complete, so that the actual ground conditions can be closely observed to obtain representative rock mass parameters.

This paper compares the ground support predictions of RMR and Q-System with that of an analytical method, with specific reference to a 13 m diameter hydropower tunnel. The analytical method considered is block theory [9] using UNWEDGE software code [10]. In the light of the support predictions made by the two empirical methods, the design requirements to prevent hydraulic jacking in unlined pressure tunnels and the as-excavated rock mass stability conditions of the tunnel are also briefly discussed.

## 2. TUNNEL PROJECT

The 13 m diameter, 240 m long, horseshoe-shaped tunnel is a major part of the Chiew Larn Hydropower Project, in the Southern Province of Thailand. The tunnel feeds three 80 MW power generating units and was driven through dark grey, fine to medium grained greywacke sandstone. The tunnel is shallow and located in a hill slope. Tunnel alignment is $140^o$ E with a plunge of $10^o$. To the east of the tunnel alignment (over the ridge) is the river valley across which the main dam was built to create the project reservoir. To the west of the tunnel alignment is the hill slope. The ground surface above the tunnel alignment is uneven, but has an overall slope of about $10^o$ towards downstream. The tunnel overburden varies between 25 m and 50 m above crown level, with an average overburden of approximately 30 m.

## 3. GEOTECHNICAL DATA COLLECTION

During excavation a detailed engineering geological map was prepared for the entire 240 m length of the tunnel [11]. Geological discontinuities with more than 3 m trace length (persistence) were plotted on the map, together with measurements of the discontinuity orientation and surface features such as roughness, waviness, aperture size, filling material, weathering and water conditions.

In addition, scan line mapping was carried out to determine RQD (rock quality designation) using the method proposed by Priest and Hudson [12]. Scan line mapping recorded all discontinuities including the low persistence discontinuities that were not projected onto the engineering geology map. Using scan line data, RQD was estimated for each five-meter interval of the tunnel. The estimated values of RQD ranged from 60 to 100 with a mean value of 80 and a standard deviation of 10.

From the analysis of discontinuity orientation data, three major discontinuity (joint) sets (Figure 1) and two minor sets were identified. The average orientations of joint sets are shown in Table 1.

Not all three major sets are prominent along the entire tunnel length. Only two major sets are prominent within any selected length of the tunnel, with the third major set occurring at random. In the first 80 m of the tunnel Sets 1 and 2 are prominent with Set 3 occurring at random. From 80 to 130 m, Sets 2 and 3 are prominent and Set 1 is random. From 130 to 240 m Sets 1 and 3 are prominent and Set 2 is random. Sets 4 and 5 are present randomly with no recognisable pattern.

Table 1. Orientations of discontinuity sets

| Set # | Dip | Direction | Remarks |
|---|---|---|---|
| 1 | 76 | 016 | Major set |
| 2 | 79 | 112 | Major set |
| 3 | 37 | 231 | Major set |
| 4 | 62 | 151 | Minor set |
| 5 | 44 | 067 | Minor set |

Discontinuity Sets 1 and 2 are partly open (0.25-0.5 mm) to moderately wide open (2.5-10 mm) and Set 3 is moderately wide to widely open (>10 mm). Joint surface conditions of all sets vary from rough undulating to slickensided planar. Some of the joint surfaces are slightly weathered and some are coated with clayey material. Sets 1 and 2 joints are either filled or coated with clayey material. In the first 100 m of the tunnel, Sets 3 and 5 are slickensided with an aperture of less than 5 mm. From 100 m onwards, shear movement is evident in these two sets with an aperture of 10 to 100 mm filled with sandy clay or clayey crushed rock. Joint surface features such as roughness, waviness and aperture size etc vary from one extreme to the other within each set (Table 2).

Minimum spacing between members of the major joints sets is approximately 0.6 m and the maximum is greater than 2 m. However, the presence of narrow fractured zones parallel to some of the major joints reduces the RQD value locally. An important feature is that, when the joint surface conditions (roughness, aperture and filling) are at the worst observed state the joint spacing is at its best state (i.e. >2 m).

Set 3 has very high persistence with joint traces extending for several tens of meters along the tunnel. Persistence of Set 1 is also high with almost all members of this set through-going. Set 2 also has a high persistence, most of its members having a trace length of at least 10 m. Persistence of Sets 4 and 5 varies between 3 and 20 m.

Throughout the tunnel alignment the intact rock material can be described as fresh, with an average UCS of 138 MPa and an average Elastic Modulus of 51 GPa (Table 3). The density of intact rock is 26.5 kN/m$^3$ and Poisson's Ratio is 0.23.

Although the joints in the rock mass are relatively open the tunnel was mostly dry, with negligible water inflow in some places. This is partly because the tunnel is below the natural groundwater level.

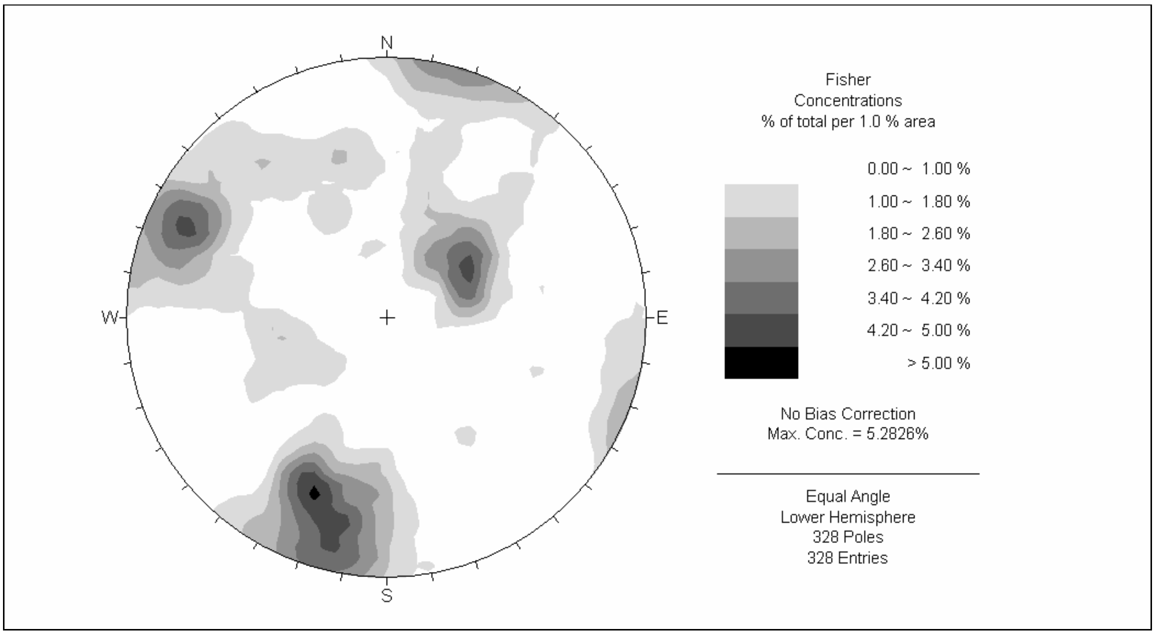


Figure 1. Discontinuity orientations

Table 2. Joint surface features

| | Set 1 | Set 2 | Set 3 | Set 4 | Set 5 |
|---|---|---|---|---|---|
| Aperture | 0.25-0.5mm to 2.5-10mm | 0.25-0.5mm to 2.5-10mm | 2.5-10mm to 10-100mm | 2.5-10mm | 0.25-0.5mm to 10-100mm |
| Roughness | rough to slickensided | | | | |
| Filling | coated to clay filled | coated to clay filled | coated to sandy clay/clayey sand | coated | coated to sandy clay/clayey sand |
| Waviness | undulating to planar | | | | |

Table 3. Intact rock material properties

| Property | Range | Mean | Std | # tested |
|---|---|---|---|---|
| UCS (MPa) | 102 – 172 | 138 | 24 | 7 |
| E Modulus (GPa) | 42 – 57 | 51 | 6 | 5 |

## 4. SUPPORT PREDICTION

In this study two empirical tunnel support design methods, RMR and Q-System, and one analytical method were used. The analytical method used is block theory using UNWEDGE software code. The purpose of using analytical method is to verify whether the supports predicted by the empirical methods are adequate for the failure mechanisms that can be identified by the former.

The process for applying RMR and Q-System are discussed in detail by Bieniawski [6] and Barton and Grimstard [8], respectively, and are briefly outlined below.

### 4.1. RMR – Rock mass rating

RMR is an index of rock mass competency based on six parameters: intact rock strength, RQD, joint spacing, joint surface conditions, groundwater and orientation of joints. Bieniawski [6] provides recommended values (ratings) for the six parameters representing a wide range of ground conditions and describes the procedure for computing the final RMR value. To assess the support requirements, ratings are assigned to the six parameters based on the site conditions and are summed to yield the RMR value, which linearly varies from 0 to 100. The RMR value is then related to five rock mass classes and each class in turn is related to rock support measures.

Bieniawski [6] provides support recommendations for roof and walls of 10m diameter tunnels excavated through the five rock mass classes.

4.2. Q System – Tunnelling quality index

Q-System is also based on six rock mass parameters: RQD, number of joint sets (Jn), roughness of the most unfavourable joint set (Jr), degree of alteration or filling in the weakest joint set (Ja), water inflow along joints (Jw), and stress condition (SRF- stress reduction factor). The Q value or tunnelling quality index is calculated using the equation:

$$Q = (RQD/Jn) \times (Jr/Ja) \times (Jw/SRF) \qquad (1)$$

Barton and Grimstard [8] provide a list of recommended values for Jn, Jr, Ja, Jw and SRF to reflect a range of site conditions. Guidance notes on how to select appropriate values for these parameters to reflect the actual site conditions are also provided in the reference cited. To predict rock mass stabilisation measures, ratings are assigned to Jn, Jr, Ja, Jw and SRF based on site conditions. RQD value is used as estimated during site investigation with no further rating assigned.

The Q value, which varies from 0.001 to 1000 in a logarithmic scale, is then related to predefined ground support categories through an Equivalent Dimension, De, which is defined as:

$$De = (\text{Span or diameter or height})/(ESR) \qquad (2)$$

where ESR is Excavation Support Ratio, which is chosen between 0.8 and 5, and is akin to inverse factor of safety. The details of each ground support category are provided by Barton and Grimstard [8].

4.3. Support Prediction by Empirical Methods

To apply the two rock mass classification methods for this study, two extreme rock mass scenarios were considered: the best and the worst combinations of ground conditions observed along the tunnel. For the observed best and worst ground conditions, ratings were assigned to the relevant RMR and Q input parameters as per the guidelines provided by Bieniawski [6] and Barton and Grimstard [8], respectively. The ratings assigned to each parameter are shown in Table 4. Note that joint spacing is wider when the joint surface conditions are at their worst observed state, which is reflected in the RMR ratings.

Table 4. Ratings assigned for RMR and Q parameters

| RMR | | | Q | | |
|---|---|---|---|---|---|
| Parameter | Best | Worst | Parameter | Best | Worst |
| Strength | 12 | 12 | RQD | 100 | 60 |
| RQD | 20 | 13 | Jn | 6 | 9 |
| Spacing | 15 | 20 | Jr | 3 | 1.5 |
| Condition | 20 | 0 | Ja | 2 | 6 |
| Groundwater | 15 | 10 | Jw | 1 | 1 |
| Adjustment | -5 | -10 | SRF | 2.5 | 5 |
| RMR | 77 | 45 | Q | 10 | 0.33 |

On the basis of the RMR and Q values in Table 4 and the support recommendations provided by Bieniawski [6] and Barton and Grimstard [8], support types were selected for the tunnel roof and walls in both best and worst ground conditions, and are presented in Table 5. For Q-System an ESR value of 1.8 was selected for calculating De. For RMR, the bolt lengths recommended by Bieniawski [6] for a 10 m diameter tunnel are assumed to be applicable to the 13 m diameter tunnel.

From Table 5 it is evident that the supports recommended by the two empirical methods are generally in agreement. Their main differences are in bolt lengths and spacing as well as in shotcrete thickness. Note that, for surface support, RMR recommends wire mesh and shotcrete and not fibre-reinforced shotcrete. This is mainly because RMR method has not been updated since fibre reinforced shotcrete became readily available to the tunnelling industry. The earlier versions of Q-System also recommended wire mesh for surface support, but the current version does not. Instead it recommends fibre-reinforced shotcrete.

4.4. Block Analysis using UNWEDGE

As mentioned earlier, two major joints sets are prominent within any given length of the tunnel with the third major joint set present at random. The two minor sets are also present at random. Overall, the joint persistence is relatively high.

The presence of these high persistence joints creates an environment in which rock blocks can theoretically be formed by several combinations of intersecting joints. Since the tunnel is shallow and the in situ stresses are low, movement of these rock blocks under gravity is the most significant stability concern. Hence, an analysis of rock block stability around the tunnel is necessary for predicting rock mass stabilisation requirements.

Table 5. Supports recommendations by RMR and Q-System

| Support type | RMR | | | | Q-System | | | |
|---|---|---|---|---|---|---|---|---|
| | Best | | Worst | | Best | | Worst | |
| | Roof | Walls | Roof | Walls | Roof | Walls | Roof | Walls |
| Bolts | L=3m S=2.5m locally | None | L=4m S=1.5-2m | L=4m S=1.5-2m | L=4m S=1.5-2m | None | L=5m S=1.5m | L=4m S=1.5m |
| Mesh | Occasional | None | Yes | No | NA | NA | NA | NA |
| Shotcrete | 50mm where required | None | 50-100mm | 30mm | None | None | 90-120mm FR | 50-90mm FR |

L – length; S – spacing; NA – not applicable; FR – fibre reinforced.

Two approaches can be adopted for block stability analysis: (a) specific joint method, in which the exact locations of the intersecting joints in the tunnel are taken into account, and (b) ubiquitous joint method, in which joints are assumed to occur everywhere along the tunnel.

The ubiquitous joint method was adopted for this study using UNWEDGE software code, which can analyse tetrahedral wedges formed by three intersecting joints and the free surface of the excavation, and allows identification of all possible tetrahedral wedges formed by intersecting joints. The ubiquitous joint method for block analysis was justifiable as the classification of rock mass in the tunnel using empirical methods also assumed that joints were ubiquitous.

Analysis of joint set orientation data showed that several combinations of joint sets had the potential to form tetrahedral rock blocks in the roof and walls of the tunnel. Of these, 19 different combinations had the potential to form tetrahedral rock wedges with apex height more than or equal to 1 m. Each of these 19 wedges was further analysed to compute factor of safety (FOS) against failure, taking into account the best and worst joint shear strength parameters, which represented the best and worst joint surface conditions considered for the classification of the rock mass according to RMR and Q-System. The selected joint shear strength parameters were:

Best: c=10 kPa, $\Phi=30^{o}$
Worst: c=0 kPa, $\Phi=20^{o}$

Since the tunnel is shallow with an overburden of between 25 m and 50 m, it was assumed that the wedges are subjected to gravitational loading only. Table 6 shows the failure mode, maximum weight, FOS, maximum apex height and location of the 19 rock wedges.

UNWEDGE analysis was then extended to verify whether the supports predicted by the two rock mass classification methods were adequate to stabilise the 19 theoretically possible rock wedges in the tunnel roof and walls. The analysis considered the best and worst joint shear strength scenarios mentioned earlier. Table 7 shows the FOS assuming that only the bolts recommended by the two empirical methods were installed. The rock bolts are cement grouted type with an ultimate tensile strength of 180 kN installed normal to the excavation face.

The effect of the shotcrete layers recommended by the two empirical methods was also analysed using the UNWEDGE software code, and the results (not provided here) showed that shotcrete increased the FOS of larger rock blocks beyond the desired level. (A FOS of 1.5 and 2 for the walls and roof, respectively, were selected as the design criteria for long term stability.) The recommended shotcrete layers would also provide necessary support for the smaller rock blocks and fractured ground and adequately meet the selected FOS requirement. However, there were some exceptions and limitations of the empirical predictions, which are discussed in Section 5.

### 4.5. Other Design Considerations

Two extremely important design considerations for water tunnels are hydraulic jacking and leakage. If high pressure water tunnels are to be unlined, the existence of major discontinuities must be considered and adequate cover must be provided to prevent hydraulic jacking. Benson [13] recommends that, for tunnels positioned within slopes or valley walls, the cover must be provided by the rock portion only; the required thickness of the rock cover may be estimated using the equation:

$$H_r = (1.3H_w)/(\gamma_r \cos \theta) \qquad (3)$$

where $H_r$ is required rock cover (m); $H_w$ is static head (m); $\gamma_r$ is density of rock (t/m$^3$); and $\theta$ (degrees) is slope angle of the ground surface.

Table 6. Results of wedge analysis using UNWEDGE

| Wedge No. | Joint set combination | Block characteristics | | | | | |
|---|---|---|---|---|---|---|---|
| | | Failure mode | Weight (kN) | FOS-B | FOS-W | Apex (m) | Location |
| 1 | Set 1, 2, 3 | Falling | 981 | 0.00 | 0.00 | 3.9 | Roof |
| 2 | Set 1, 3, 4 | Falling | 265 | 0.00 | 0.00 | 2.2 | Roof |
| 3 | Set 1, 2, 4 | Sliding on Set 2 | 589 | 0.39 | 0.07 | 5.2 | Roof |
| 4 | Set 2, 4, 5 | Sliding on Set 2 | 628 | 0.41 | 0.07 | 4.2 | Roof |
| 5 | Set 1, 3, 5 | Sliding on Set 1 | 255 | 0.27 | 0.09 | 1.2 | Roof |
| 6 | Set 2, 3, 5 | Sliding on Set 2/5 | 9133 | 1.66 | 0.82 | 6.2 | Right wall |
| 7 | Set 2, 3, 5 | Sliding on Set 2/3 | 7819 | 1.64 | 0.79 | 6.0 | Left wall |
| 8 | Set 3, 4, 5 | Sliding on Set 5 | 7318 | 0.87 | 0.37 | 5.2 | Right wall |
| 9 | Set 3, 4, 5 | Sliding on Set 3/4 | 6337 | 1.18 | 0.53 | 5.1 | Left wall |
| 10 | Set 1, 3, 5 | Sliding on Set 5 | 4218 | 0.88 | 0.38 | 3.9 | Right wall |
| 11 | Set 1, 3, 5 | Sliding on Set 3 | 4159 | 1.08 | 0.48 | 3.9 | Left wall |
| 12 | Set 1, 3, 4 | Sliding on Set 3 | 1020 | 1.13 | 0.48 | 3.6 | Left wall |
| 13 | Set 1, 3, 4 | Sliding on1/4 | 961 | 1.83 | 0.73 | 3.6 | Right wall |
| 14 | Set 1, 2, 3 | Sliding on Set 3 | 294 | 1.26 | 0.48 | 1.9 | Left wall |
| 15 | Set 2, 4, 5 | Sliding on Set 5 | 275 | 1.02 | 0.38 | 2.5 | Right wall |
| 16 | Set 2, 4, 5 | Sliding on Set 2/4 | 265 | 1.94 | 0.51 | 2.4 | Left wall |
| 17 | Set 1, 2, 3 | Sliding on Set 1/2 | 196 | 1.14 | 0.17 | 1.6 | Right wall |
| 18 | Set 1, 2, 4 | Sliding on Set 2/4 | 49 | 3.05 | 0.51 | 1.2 | Left wall |
| 19 | Set 1, 2, 4 | Sliding on Set 1/2 | 39 | 2.25 | 0.17 | 1.1 | Left wall |

FOS-B: FOS of wedges in best ground conditions with no artificial support
FOS-W: FOS of wedges in worst ground conditions with no artificial support

Table 7. FOS for empirically selected bolt patterns

| Wedge No. | Joint set combination | Wedge weight (kN) | Best conditions | | Worst conditions | |
|---|---|---|---|---|---|---|
| | | | FOS-B1 | FOS-B2 | FOS-W1 | FOS-W2 |
| 1 | Set 1, 2, 3 | 981 | 0.62 | 1.06 | 1.26 | 1.86 |
| 2 | Set 1, 3, 4 | 265 | 0.96 | 1.47 | 1.75 | 2.74 |
| 3 | Set 1, 2, 4 | 589 | 0.61 | 1.29 | 0.93 | 1.43 |
| 4 | Set 2, 4, 5 | 628 | 1.15 | 1.23 | 1.42 | 1.93 |
| 5 | Set 1, 3, 5 | 255 | 2.49 | 4.51 | 6.07 | 7.41 |
| 6 | Set 2, 3, 5 | 9133 | 2.15 | 2.50 | 1.80 | 2.20 |
| 7 | Set 2, 3, 5 | 7819 | 2.12 | 2.43 | 1.78 | 2.18 |
| 8 | Set 3, 4, 5 | 7318 | 1.41 | 1.73 | 1.36 | 1.69 |
| 9 | Set 3, 4, 5 | 6337 | 1.85 | 2.46 | 1.97 | 2.53 |
| 10 | Set 1, 3, 5 | 4218 | 1.59 | 2.13 | 1.53 | 2.22 |
| 11 | Set 1, 3, 5 | 4159 | 2.41 | 3.01 | 2.81 | 3.60 |
| 12 | Set 1, 3, 4 | 1020 | 2.08 | 2.78 | 2.38 | 3.00 |
| 13 | Set 1, 3, 4 | 961 | 2.59 | 3.01 | 2.11 | 2.42 |
| 14 | Set 1, 2, 3 | 294 | 2.44 | 3.86 | 1.96 | 2.59 |
| 15 | Set 2, 4, 5 | 275 | 1.55 | 1.55 | 1.13 | 1.65 |
| 16 | Set 2, 4, 5 | 265 | 3.16 | 4.12 | 3.13 | 2.52 |
| 17 | Set 1, 2, 3 | 196 | 2.19 | 2.21 | 1.45 | 2.57 |
| 18 | Set 1, 2, 4 | 49 | 5.21 | 7.26 | 2.19 | 2.36 |
| 19 | Set 1, 2, 4 | 39 | 2.25 | 3.96 | 1.27 | 1.86 |

FOS-B1: 3m long bolts in a 2.5m x 2.5m pattern (RMR)
FOS-B2: 5m long bolts in a 2.0m x 2.0m pattern (Q)
FOS-W1: 4m long bolts in a 1.5m x 2.0m pattern (RMR)
FOS-W2: 5m long bolts in a 1.5m x 1.5m pattern (Q)

As mentioned earlier, the tunnel overburden varies between 25 m and 50 m with an average of 30 m. In some places along the tunnel alignment, approximately 5 to 10 m of the overburden was composed of completely weathered rock or residual soil. The maximum normal operating water level in the reservoir is 95 m RL. The tunnel invert level at inlet is 45 m RL and at outlet is 4 m RL approximately. Respective crown levels are 58 m RL and 17 m RL.

As per Eq (3) the available rock cover for most of the tunnel is insufficient to provide an adequate safety margin against hydraulic jacking and uplift, if the tunnel is not fully lined. Leakage can also occur from the tunnel as the rock mass is jointed (pervious) and the internal water pressure is higher than the external groundwater pressure.

## 5. COMPARISON AND DISCUSSION

### 5.1. Supports Predicted

The results of the UNWEDGE analysis show that the supports predicted by the two empirical methods for the worst ground conditions are adequate to stabilise the theoretically possible large tetrahedral rock wedges. The predicted support system includes rock bolts and shotcrete. Additionally, RMR and Q-System recommend wire mesh and fibre reinforcement, respectively, for shotcrete. Both of these increase the capacity of the support system and the FOS against failure of large rock blocks identified by the UNWEDGE analysis. The shotcrete with wire mesh or fibre reinforcement also provides required support for the small rock blocks and fractured zones in the worst ground conditions and adequately meets the selected FOS requirement.

For the tunnel walls in the best rock mass conditions, the two empirical methods do not recommend any artificial supports (Table 5). However, the UNWEDGE analysis showed that there is potential for 14 different tetrahedral rock wedges to be formed in the walls excavated in the best rock mass conditions (Table 6). Of these, four wedges have FOS of less than or equal to one indicating potential instability, if rock wedges are present in the walls. Further, three of the 14 wedges have FOS of less than 1.2, which is considered to be below the acceptable level. Hence the RMR and Q-System recommendations for the walls in the best rock mass conditions can be considered inadequate in the long term.

For the tunnel roof in the best rock mass conditions, RMR recommends rock bolts plus mesh and 50 mm of shotcrete where required (shotcrete is probably for fractured ground, if present, and not for the entire roof). As can be seen from Table 6 there is potential for five rock wedges with zero or near zero FOS in the roof excavated in the best rock mass conditions. Three of these wedges, with maximum possible weights of 265, 589 and 981 kN, will have FOS of less than one, if RMR recommended rock bolting pattern is used (Table 7). A fourth wedge with a maximum possible weight of 628 kN will have a FOS of only 1.15, if the same bolting pattern is used. If shotcrete is applied, FOS of these possible rock wedges can be increased to an acceptable level. However, since RMR does not recommend shotcrete for the entire tunnel roof, the adequacy of the supports predicted is debatable.

Q-System recommendation for the tunnel roof in the best rock mass conditions returns a FOS of one for wedge no. 1. For wedge nos. 3 and 4, FOS is less than 1.3. These FOS values are below the acceptable level for long term stability of large rock blocks in the tunnel roof.

Though the ubiquitous joint method was adopted for the block analysis, only the tetrahedral (or pyramidal) rock wedges formed by three intersecting joints were analysed. Non-pyramidal blocks that may be formed by more than three intersecting joints were excluded. Hence the comparison is limited to the potential tetrahedral rock wedge instability under the selected joint shear strength scenarios.

### 5.2. Other Considerations

The rock mass classification methods, Q-System in particular through ESR, claim to allow for the purpose of the excavation when predicting rock support requirements. As per Eq (3) the tunnel has insufficient rock cover for safe operation without a lining, thus the adequacy of the support requirements predicted by the two empirical methods, particularly for the best rock mass conditions, is debatable.

### 5.3. Actual Supports Installed

During the excavation of the tunnel, several potentially unstable large rock blocks were identified by onsite personnel. Some of these were located within the best ground conditions described earlier. These blocks were temporarily stabilised using 6, 4 and 3 m long mechanically anchored and resin grouted rock bolts.

In view of the potential for hydraulic jacking and leakage losses etc, a steel lining was installed for the full length of the tunnel. The thickness of the lining varied between 25 mm and 31 mm from upstream to downstream, respectively. The annulus between the as-excavated tunnel and the lining was filled with concrete. Since the tunnel was to be lined with steel and concrete, rock bolting was kept to an absolute minimum and was used only as a temporary rock mass stabilisation measure. The tunnel was thoroughly scaled to ensure safety during construction. No shotcrete was used.

## 6. CONCLUSIONS

Based on detailed geotechnical mapping, RMR and Q-System were applied to a 13 m diameter hydropower tunnel. Joint orientation data were used to identify potentially unstable tetrahedral rock wedges using UNWEDGE software code. The adequacy of the support requirements predicted by RMR and Q-System was checked by wedge stability analysis considering the best and worst joint shear strength parameters.

For the worst rock mass conditions considered, the stabilisation measures predicted by RMR and Q-System are generally in agreement with the predictions made by wedge stability analysis. The wedge stability analysis showed that the empirical predictions provide adequate safety factors against tetrahedral wedge failure mechanisms identified within the worst rock mass conditions considered.

For walls in the best rock mass conditions, the two empirical methods do not recommend any artificial support. In contrast, the UNWEDGE analysis showed that rock bolts are warranted for walls in the best ground conditions. The analysis also showed that the adequacy of the RMR recommendation for the tunnel roof in the best ground condition is debatable. The Q-System recommendation for the roof in the best rock mass conditions also does not meet the design FOS selected for this study.

During the excavation, potentially unstable rock blocks were observed, and they were temporarily stabilised with mechanically anchored and resin grouted rock bolts. Considering the potential risk of hydraulic jacking and leakage losses, the tunnel was fully steel-lined and the annulus between the as-excavated tunnel and the steel lining was filled with concrete. In effect, the actual supports installed were significantly different from the supports proposed by RMR and Q-System.

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# Comparison of empirical methods with analytical methods of underground excavation design

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Keywords: rock mass classification; tunnel; rock blocks; stability; RMR; Q-System

**Abstract**

Ground support requirements for a shallow 13 m diameter water tunnel were predicted using two empirical methods (RMR and Q-System). The predictions were compared with the results of a wedge analysis of the rock mass around the tunnel. In addition, a simple two-dimensional numerical model was run to examine the effect of internal water pressure on the rock mass around the tunnel. The study indicates that although empirical predictions may serve as a first pass design, they may not be adequate for stabilising all potentially unstable rock wedges and for preventing adverse effects of internal water pressures in low in situ stress environments.

## 1. INTRODUCTION

Empirical rock mass classification methods are useful tools for underground excavation design, particularly during early stages of a project. These methods semi-quantitatively describe the rock mass, and prescribe rock mass stabilisation measures based on experience gained elsewhere in similar ground conditions. Currently, the most widely used empirical methods are RMR (Bieniawski, 1973) and Q-System (Barton et al., 1974). The experience gained subsequent to the initial introduction of these methods enabled their creators to issue revised versions of RMR (Bieniawski, 1974, 1976, 1979 and 1989) and Q-System (Grimstad & Barton, 1993; and Barton & Grimstad, 1994).

In spite of the revisions, these methods have limitations, which have been discussed by several authors (Speers, 1992; Hudson & Harrison, 1997; Palmstrom et al., 2000; Peck, 2000; Mikula & Lee, 2003; Stille & Palmstrom, 2003; and Palmstrom & Broch, 2006). One approach to assess the limitations of empirical methods and to suggest improvements, if necessary, is to compare their predictions with those of analytical methods.

This paper applies RMR and Q-System to a shallow 13 m diameter hydropower tunnel, and compares their predictions with those made by an analytical method. The analytical method considered is block theory (Goodman & Shi, 1985) using UNWEDGE software code (Rocscience Inc., 2003). In addition, to assess the type of support required to prevent adverse effects of internal water pressure on the rock mass around the tunnel, a simple two dimensional numerical model was run using Universal Distinct Element Code (UDEC) (Itasca Consulting Group Inc., 2004). The results of the numerical modelling are also compared with the empirically derived supports.

## 2. TUNNEL SITE CONDITIONS

The 13 m diameter, 240 m long, horseshoe-shaped tunnel, constructed to feed three 80 MW power generating units, is part of the Chiew Larn Hydropower Project in Thailand. The tunnel is shallow and located in a hill slope. Tunnel alignment is 140$^o$ E with a plunge of 10$^o$. The ground surface above the tunnel is uneven, but has an overall slope of about 10$^o$ towards south (downstream) and about 20$^o$ to the west. The tunnel overburden varies between 25 m and 50 m above crown level, with an average of approximately 30 m.

The tunnel was driven through greywacke sandstone. Along the tunnel the intact rock material is fresh, with an average UCS of 138 MPa and an average intact rock Young's Modulus of 51 GPa. Unit weight of intact rock is 26.5 kN/m$^3$ and Poisson's Ratio is 0.23. Three major discontinuity (joint) sets (Sets 1, 2 & 3) and two minor sets (Sets 4 & 5) are present in the rock mass (Ranasooriya, 1985). Mean orientations of joint sets, based on 328 measurements, are given in Table 1.

Of the three major joint sets only two are prominent within any selected length of the tunnel, with the third major set occurring at random. In the first 80 m of the tunnel, Sets 1 and 2 are prominent with Set 3 occurring at random. From 80 to 130 m, Sets 2 and 3 are prominent and Set 1 is random. From 130 to 240 m Sets 1 and 3 are prominent and Set 2 is random. Sets 4 and 5 are present randomly with no recognisable pattern. Joint surface roughness, waviness, aperture size and filling of all five sets vary as shown in Table 1. Although the joints are relatively open, the tunnel was mostly dry, with water dripping in some places. This is partly because the tunnel is below the regional groundwater level. A notable feature is that, when the joint surface conditions (roughness, aperture and filling) are at the worst observed state the joint spacing is at its best state (i.e. >2 m).

Table 1. Orientations and surface features of discontinuity (joint) sets

| | Set 1 | Set 2 | Set 3 | Set 4 | Set 5 |
|---|---|---|---|---|---|
| Dip Angle | 76 | 79 | 37 | 62 | 44 |
| Dip Direction | 016 | 112 | 231 | 151 | 067 |
| Persistence | >20m | >10m | >20m | 3-20m | 3-20m |
| Aperture | 0.25 - 10mm | 0.25 - 10mm | 2.5 - 100mm | 2.5 - 10mm | 0.25 - 100mm |
| Filling | coated - clay | coated - clay | coated - sandy clay | coated | coated - sandy clay |
| Roughness | rough to slickensided | | | | |
| Waviness | undulating to planar | | | | |
| Spacing | minimum 0.6m – maximum >2m | | | | |

## 3. SUPPORT PREDICTION USING RMR AND Q-SYSTEM

The procedures for applying RMR and Q-System have been well known for more than three decades, and different versions of these methods are documented in the references cited earlier and in many texts on underground excavation design. For this study, RMR (Bieniawski, 1989) and Q-System (Barton & Grimstad, 1994) were used.

Two extreme rock mass scenarios were considered: the best and the worst combinations of ground conditions observed along the tunnel. RMR and Q ratings were assigned to the best and the worst ground conditions and the ratings are given in Table 2. Note that joint spacing is wider when the joint surface conditions are at their worst observed state, which is reflected in the RMR ratings.

Table 2. Ratings assigned for RMR and Q parameters

| RMR | | | Q-System | | |
|---|---|---|---|---|---|
| Parameter | Best | Worst | Parameter | Best | Worst |
| Strength | 12 | 12 | RQD | 100 | 60 |
| RQD | 20 | 13 | Jn | 6 | 9 |
| Spacing | 15 | 20 | Jr | 3 | 1.5 |
| Condition | 20 | 0 | Ja | 2 | 6 |
| Groundwater | 15 | 10 | Jw | 1 | 1 |
| Adjustment | -5 | -10 | SRF | 2.5 | 5 |
| RMR value | 77 | 45 | Q value | 10 | 0.33 |

Table 3. Support recommendations by RMR and Q-System

| Support type | RMR | | | | Q-System | | | |
|---|---|---|---|---|---|---|---|---|
| | Best | | Worst | | Best | | Worst | |
| | Roof | Walls | Roof | Walls | Roof | Walls | Roof | Walls |
| Bolts | L=4m S=2.5m locally | None | L=5m S=1.5-2m | L=5m S=1.5-2m | L=4m S=1.5-2m | None | L=5m S=1.5m | L=4m S=1.5m |
| MH/FR | MH-occasional | None | MH | None | None | None | FR | FR |
| Shotcrete | 50mm (WR) | None | 50-100mm | 30mm | None | None | 90-120mm | 50-90mm |

L – length; S – spacing; MH - wire mesh; FR – fibre reinforced; WR - where required.

On the basis of the RMR and Q values in Table 2 and the support recommendations provided by Bieniawski (1989) and Barton and Grimstad (1994), support types were selected for the tunnel roof and walls in both best and worst ground conditions, and are presented in Table 3. An Excavation Support Ratio (ESR) of 1.8 was selected for Q-System. Since RMR-derived supports are only relevant to a 10 m diameter tunnel, the bolt lengths recommended by RMR were increased by approximately 30% to take into account the 13 m diameter of the case tunnel.

## 4. WEDGE STABILITY ANALYSIS

A tetrahedral rock wedge stability analysis was performed to examine whether the supports predicted by the empirical methods are adequate for the failure mechanisms that can be identified by the wedge analysis. For this purpose the ubiquitous joint method was adopted using UNWEDGE software code, which can analyse tetrahedral wedges formed by three intersecting joints and the free surface of the excavation, and allows identification of all possible tetrahedral wedges. The use of the ubiquitous joint method was justifiable because the classification of the rock mass using the two empirical methods also assumed that joints were ubiquitous.

The analysis showed that 19 different combinations of joints had the potential to form tetrahedral rock wedges with apex height greater than 1 m. Table 4 shows the joint set combination, location, maximum apex height and maximum weight of the 19 rock wedges. Factor of safety (FOS) against failure of each wedge was computed using joint shear strength parameters representative of the best and the worst joint surface conditions considered for RMR and Q-System. The Joint shear strength parameters (best: c=10 kPa, $\Phi=30^o$) and (worst: c=0 kPa, $\Phi=20^o$) were estimated based on the suggestions given by Barton & Grimstad (1994) and considering the potential for water saturation of the joints. Since the tunnel overburden is between 25 and 50 m, it was assumed that the wedges are subjected to gravity loading only with no clamping effect. FOS of unsupported rock wedges in the best and the worst ground conditions, F-B and F-W respectively, are given in Table 4.

Table 4. Results of wedge analysis using UNWEDGE

| Wedge # | Wedge characteristics | | | | FOS-Best Conditions | | | FOS-Worst Conditions | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Sets | Location | Apex(m) | Weight(kN) | F-B | F-BR | F-BQ | F-W | F-WR | F-WQ |
| 1 | 1, 2, 3 | Roof | 3.9 | 983 | 0.00 | 0.77 | 1.39 | 0.00 | 1.49 | 2.17 |
| 2 | 1, 3, 4 | Roof | 2.2 | 261 | 0.00 | 1.02 | 1.96 | 0.00 | 2.14 | 2.53 |
| 3 | 1, 2, 4 | Roof | 5.2 | 585 | 0.39 | 0.84 | 1.64 | 0.07 | 1.41 | 1.38 |
| 4 | 2, 4, 5 | Roof | 4.2 | 628 | 0.41 | 1.18 | 1.49 | 0.07 | 1.65 | 2.31 |
| 5 | 1, 3, 5 | Roof | 1.2 | 254 | 0.27 | 2.52 | 4.57 | 0.09 | 6.16 | 7.51 |
| 6 | 2, 3, 5 | R/wall | 6.2 | 9140 | 1.66 | 2.30 | 2.60 | 0.82 | 1.96 | 2.34 |
| 7 | 2, 3, 5 | L/wall | 6.0 | 7829 | 1.64 | 2.23 | 2.53 | 0.79 | 1.94 | 2.36 |
| 8 | 3, 4, 5 | R/wall | 5.2 | 7328 | 0.87 | 1.55 | 1.85 | 0.37 | 1.51 | 1.85 |
| 9 | 3, 4, 5 | L/wall | 5.1 | 6339 | 1.18 | 2.05 | 2.64 | 0.53 | 2.25 | 2.84 |
| 10 | 1, 3, 5 | R/wall | 3.9 | 4227 | 0.88 | 1.51 | 2.16 | 0.38 | 1.76 | 2.25 |
| 11 | 1, 3, 5 | L/wall | 3.9 | 4166 | 1.08 | 2.45 | 3.05 | 0.48 | 2.86 | 3.66 |
| 12 | 1, 3, 4 | L/wall | 3.6 | 1025 | 1.13 | 2.21 | 3.20 | 0.48 | 2.71 | 3.43 |
| 13 | 1, 3, 4 | R/wall | 3.6 | 962 | 1.83 | 2.71 | 3.23 | 0.73 | 2.26 | 2.82 |
| 14 | 1, 2, 3 | L/wall | 1.9 | 293 | 1.26 | 2.65 | 4.11 | 0.48 | 2.62 | 3.31 |
| 15 | 2, 4, 5 | R/wall | 2.6 | 278 | 1.02 | 1.58 | 1.57 | 0.38 | 1.39 | 1.88 |
| 16 | 2, 4, 5 | L/wall | 2.4 | 260 | 1.94 | 3.21 | 5.01 | 0.51 | 3.44 | 3.78 |
| 17 | 1, 2, 3 | R/wall | 1.6 | 196 | 1.14 | 2.44 | 2.55 | 0.17 | 1.88 | 3.24 |
| 18 | 1, 2, 4 | L/wall | 1.2 | 47 | 3.05 | 5.59 | 5.82 | 0.51 | 2.56 | 3.68 |
| 19 | 1, 2, 4 | L/wall | 1.1 | 36 | 2.25 | 2.29 | 3.15 | 0.17 | 1.30 | 1.30 |

Note: F-B = FOS for best ground conditions with no artificial support; F-W = FOS for worst ground conditions with no artificial support; F-BR = FOS for best ground with 4m long bolts in a 2.5m x 2.5m pattern (as per RMR); F-BQ = FOS for best ground with 5m long bolts in a 2.0m x 2.0m pattern (as per Q); F-WR = FOS for worst ground with 5m long bolts in a 1.5m x 2.0m pattern (as per RMR); F-WQ = FOS for worst ground with 5m long bolts in a 1.5m x 1.5m pattern (as per Q).

The analysis was extended to examine whether the empirically predicted support could stabilise the 19 theoretically possible rock wedges. Since the wedges may be present in the best or the worst ground conditions, the analysis considered both the best and worst joint shear strength scenarios mentioned earlier. Table 4 also shows the FOS assuming that only the bolts recommended by the two empirical methods were installed. The bolts considered are cement grouted type with 100% bond efficiency and an ultimate tensile strength of 180 kN installed normal to the rock face.

The effect of empirically recommended shotcrete layers was also analysed using UNWEDGE, which can compute punching shear capacity of shotcrete along the edge of a rock wedge. The results of the analysis (not provided here) showed that shotcrete increased the FOS of large rock wedges beyond the desired level. (A FOS of 1.5 and 2 for the walls and roof, respectively, were selected for long term stability.) The shotcrete layers would also provide the necessary support for the smaller rock blocks in between bolts and for fractured ground and would meet the selected FOS. The results of the analysis show that the supports predicted by the two empirical methods for the worst ground conditions are adequate to stabilise the theoretically possible tetrahedral rock wedges.

For the tunnel walls in the best rock mass, the two empirical methods recommend no support (Table 3). However, the analysis showed that there is potential for 14 different tetrahedral rock wedges in the tunnel walls (Table 4). Four of these wedges (# 8, 10, 11 & 15) have a FOS of less than or equal to one indicating potential instability in the best rock mass. Three more wedges (# 9, 12 & 17) have a FOS of less than 1.2, which is considered to be below the acceptable level.

For the tunnel roof in the best rock mass conditions, RMR recommends rock bolts plus mesh and 50 mm of shotcrete where required (mesh and shotcrete are probably for fractured ground, if present, and not for the entire roof). As can be seen from Table 4 there is potential for five rock wedges with zero or near zero FOS in the roof. First three of these wedges, with maximum possible weights of 983, 261 and 585 kN, will have a FOS of less than or equal to one, if RMR recommended rock bolting pattern for the best rock mass conditions is used (Table 4). With the same bolting pattern the fourth wedge with a maximum possible weight of 628 kN will have a FOS of only 1.18. The analysis showed that shotcrete would increase the FOS of these possible rock wedges to an acceptable level. However, RMR does not recommend shotcrete for large rock wedges in the roof.

With the Q-System recommendation for the tunnel roof in the best rock mass conditions the FOS for wedge nos. 1, 3 and 4 are below the acceptable level for long term stability of large rock blocks in the tunnel roof. Q-System does not recommend shotcrete for the roof in the best rock mass.

During excavation of the tunnel, several potentially unstable large rock blocks were identified within both the best and worst ground conditions described earlier. These blocks were temporarily stabilised using 6, 4 and 3 m long mechanically anchored and resin grouted rock bolts.

## 5. ASSESSMENT OF INTERNAL WATER PRESSURE EFFECT

Two important design considerations for water tunnels are hydraulic jacking and water leakage. Hydraulic jacking or uplift of the surrounding ground can occur if water pressures imposed within a rock mass are greater than the in situ compressive stresses in the rock mass (Benson, 1989).

In the case tunnel the internal water pressure along the centreline range from 0.45 to 0.82 MPa, and the gravity induced vertical stress at crown level (assuming rock mass density of 25 $kN/m^3$) range from 0.61 to 1.07 MPa depending on the overburden thickness. At any given point along the tunnel the internal water pressure is less than the confinement stresses due to overburden. However, since the natural groundwater pressure along the tunnel alignment is less than the water pressure inside the tunnel, water loss by seepage is possible through open interconnected joints. Further, the seepage may cause instability at the ground surface, particularly on the hill slope. This may be demonstrated by a simple two dimensional numerical model using UDEC software code.

For this purpose, and also to examine the type of support required to prevent the adverse effects of internal water pressure on the rock mass around the tunnel, a two dimensional numerical model was run using UDEC. The model was constructed assuming jointed rock from the natural ground surface, and ignoring the near surface soil profile. Two major joint sets (Sets 2 and 3), which are sub-parallel to the tunnel axis, were included in the model. The model assumed that joint spacing is 2 m and that joints are fully persistent with the best joint shear strength parameters considered earlier. Joint aperture size was varied to reflect the observed site conditions given in Table 1.

In situ stresses were assumed to be due to gravity only. The intact rock blocks were assumed to be elastically deformable, with intact rock bulk modulus of 30 GPa and shear modulus of 20 GPa based on the laboratory determined intact rock Young's modulus and Poisson's ratio mentioned earlier.

Estimated joint normal and shear stiffness values of 800 MPa/m and 100 MPa/m, respectively, were used, but were varied to investigate the sensitivity of the model and found that the higher the joint stiffness, the lower the maximum displacement of rock blocks.

Four sections across the tunnel representing different ground profiles and internal water heads were considered. Two cases were modelled for each section. Case 1: bolts installed as per empirical recommendations for the best ground conditions, and steady-state seepage was considered. Case 2: a fully impermeable liner installed covering the entire tunnel periphery.

The results of Case 1 modelling showed that seepage would occur through the rock mass in the four sections considered, even if the joint apertures were at their observed lowest range (0.25-0.5 mm). The Case 1 modelling also showed that, instability may occur at the ground surface (on the hill slope) when the overburden thickness is about 30 m (site average). The possible seepage paths, displacement and velocity vectors are shown in Figure 1 for a section where overburden thickness above crown is 32 m (vertical stress is 0.78 MPa) and internal water pressure is 0.6 MPa.

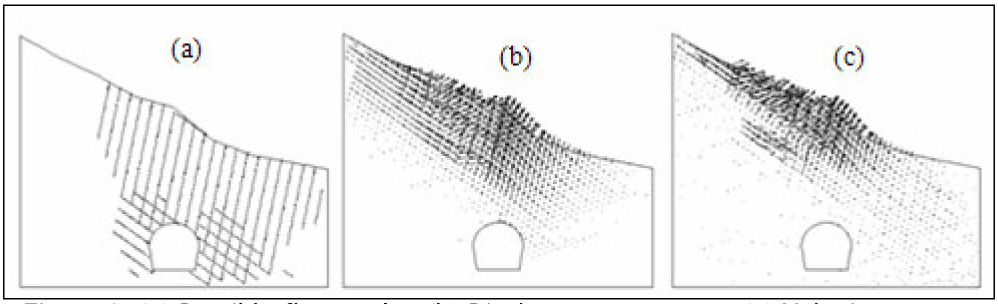

Figure 1. (a) Possible flow paths, (b) Displacement vectors, (c) Velocity vectors

Case 3, which assumed no flow through the tunnel periphery, showed insignificant movement at the ground surface. This demonstrates that an impermeable liner is required to prevent seepage from the tunnel and to minimise the risk of instability on the hill slope above the tunnel.

The empirical methods considered in this study claim that the predicted support measures represent permanent support and that the predictions take into account the purpose of the excavation. For instance, Q-System (Barton & Grimstad, 1994) recommends an ESR value of between 1.6 and 2.0 for predicting support requirements for hydropower tunnels. And, Bieniawski (1989) presents an example application of RMR to a shallow water tunnel with an overburden of between 15.3 and 61 m above crown level. This overburden thickness range is comparable to that of the case tunnel.

As mentioned earlier the empirical methods recommended rock bolts and shotcrete for the worst ground conditions intersected in the tunnel. For the best rock mass intersected in the tunnel no support is recommended for the walls and only bolts are recommended for the roof. Clearly, the empirically derived supports are unlikely to eliminate seepage, which could eventually lead to instability at the ground surface. Application of shotcrete along the entire tunnel periphery including the invert, additional to the empirical recommendations for the worst ground conditions, may be an option to control seepage. However, since shotcrete is known to have numerous incipient cracks formed by shrinkage, expansion and shear movement etc, it may not completely eliminate seepage and the risk of instability at the ground surface. It would perhaps delay the problem, as the pressures tend to build up slowly due to decreased flow through the shotcrete.

Considering the potential for leakage losses and hydraulic jacking, the tunnel was fully steel lined with the annulus between the tunnel and the lining filled with concrete. Rock bolting was used only as a temporary rock mass stabilisation measure. No shotcrete was used.

## 6. CONCLUSIONS

The wedge stability analysis showed that the rock support measure predicted by RMR and Q-System provide adequate safety factors against tetrahedral wedge failure mechanisms identified within the worst case rock mass conditions in the tunnel.

For walls in the best rock mass, the two empirical methods do not recommend any artificial support. In contrast, the wedge analysis showed that rock bolts could be warranted for walls in the best rock mass conditions.

Since the tunnel is shallow and located on a hill slope, seepage could occur through interconnected joints in the rock mass. Numerical modelling showed that even with the lowest joint apertures of the rock mass, seepage may occur and cause instability at the ground surface. These aspects are not considered in the two empirical methods.

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# Tetrahedral Rock Wedge Stability Under Empirically Derived Support

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## Abstract

*The support measures determined by RMR and Q rock mass classification methods were compared with a tetrahedral rock wedge stability analysis of the rock mass around two water conveyance tunnels. The study indicates that the support measures recommended by the two classification methods are generally adequate, but are insufficient to provide an adequate safety margin against some of the possible rock wedge failures under project specific conditions.*

## 1 Introduction

A common mode of instability in underground openings created in jointed rock formations is the failure of rock wedges formed by intersecting structural discontinuities in the rock and the free surface of the opening. Each wedge that fails will cause a reduction in the restraint and interlocking of the adjacent rock and this, in turn, will allow other blocks and wedges to unravel.

The design of support for eliminating rock wedge instability in a jointed rock mass may be undertaken by analytical methods such as a limit equilibrium stability analysis of tetrahedral rock wedges. This approach requires detailed information on the rock mass structure and the in situ stress conditions. The steps involved in the analytical approach are:

- Determination of orientation of major discontinuities.
- Identification of wedges with kinematic feasibility to move into the opening.
- Determination of shear strength of discontinuity surfaces.
- Estimation of the stress field around the opening.
- Assessment of likelihood of failure of the identified rock wedges.
- Determination of support required to reduce the risk of failure to an acceptable level.

Alternatively, an empirical approach using rock mass classification methods such as RMR (Bieniawski, 1979 and 1989) and Q (Barton et al., 1974; Barton and Grimstad, 1994) may be used for the design of support for openings created in jointed rock formations. These methods are sometimes preferred to analytical methods, particularly if detailed information required for analytical methods is lacking. The empirical methods require no analysis of any specific failure mechanisms, yet the support measures thus designed are considered to deal with all possible failure mechanisms in a jointed rock, including tetrahedral wedge failure.

There is, however, some uncertainty in this approach due to the lack of a scientific basis for identifying potential failure mechanisms, such as tetrahedral rock wedges, and to quantify the forces required to stabilise them. One approach to confirm whether the empirically derived support measures are sufficient to stabilise the potentially unstable rock blocks is to compare them with those derived by analytical methods.

This paper presents the results of the application of RMR and Q classification methods to two tunnels driven through jointed rock formations. The two methods were applied during the construction of the tunnel by rock mass mapping and testing of intact rock specimens for engineering parameters. The support measures recommended by the two empirical methods were subsequently compared with the results of a tetrahedral rock wedge analysis undertaken using the UNWEDGE software code (Rocscience, 2003).

## 2 Background

The two tunnels considered in this study are: (a) 493 m long, 11.3 m span horseshoe shaped Chiew Larn diversion (CLD) tunnel of the Chiew Larn hydropower project located in the Southern Province of Thailand,

and (b) 732 m long, 3.5 m wide and 3.5 m high D-shaped Huai Saphan Hin power (HSHP) tunnel of the Huai Saphan Hin hydropower project located in the eastern seaboard of Thailand.

The sedimentary rock formations in the two project sites belong to the Silurian to Carboniferous ages. The two tunnels were driven through jointed sedimentary rocks, mainly greywacke, by conventional drill and blast methods. During the excavation of the CLD and HSHP tunnels, Ratanasatayanont (1984) and Lasao (1986), respectively, studied the rock mass conditions intersected in the tunnels. Their studies consisted of detailed engineering geological mapping, testing of intact rock samples and classification of the rock mass according to RMR and Q systems.

In their studies, Ratanasatayanont and Lasao used two different approaches to apply the two classification methods. Ratanasatayanont divided the CLD tunnel into 47 sectors taking into account the number of discontinuity sets, discontinuity spacing and their surface characteristics as well as groundwater conditions. The two classification methods were applied to the most common ground conditions within each sector. Lasao divided the tunnel into five sectors based on the major discontinuities and groundwater conditions, and the two methods were applied to the most favourable (best) and most unfavourable (worst) rock mass conditions in each of the five sectors. They applied RMR (Bieniawski, 1979) and Q (Barton et al., 1974) versions, which were current at the time. For the present study, using the data presented by the two researchers, support measures were derived by applying the current versions of the two methods, RMR (Bieniawski, 1989) and Q (Barton and Grimstad, 1994).

# 3 Chiew Larn Diversion Tunnel

The 11.3 m span CLD tunnel was constructed for two purposes: (a) to temporarily divert the Klong Saeng River to facilitate the construction of the main dam; and (b) as an irrigation water supply tunnel for downstream users. To fulfil the second purpose, after the completion of the main dam construction, the tunnel was plugged and an irrigation outlet valve was provided in the plug. The plug is at approximately 105 m from the inlet and is located below the centreline of the main dam, which crosses the tunnel alignment. After filling the reservoir, the tunnel length upstream of the plug functions as a pressure tunnel under a hydraulic head equivalent to the reservoir level, which has a maximum elevation of 95 m RL (The tunnel invert level is approximately 10 m RL). In contrast the tunnel length downstream of the plug has an external water pressure equivalent to the groundwater level artificially elevated by the reservoir, and functions as a groundwater sink. During the excavation, groundwater flow into the tunnel was generally nil to low, except for some isolated areas of water flow, particularly during the wet season.

The average tunnel alignment is NW-SE with a 0.2% gradient towards SE. The tunnel overburden varies from 40 to 80 m with an average of about 60 m. The rock type present along the CLD tunnel is greywacke. The results of intact rock material testing, which included direct shear testing of saw-cut samples for basic friction angle ($\Phi_b$), conducted by Ratanasatayanont (1984) are summarised in Table 1.

**Table 1 Intact rock properties along the CLD tunnel**

| Property | Range | Mean | Std | # of tests |
|---|---|---|---|---|
| UCS (MPa) | 33-89 | 60 | 18 | 14 |
| E Modulus (GPa) | 20-90 | 55 | 17 | 14 |
| Joint wall strength (MPa) | - | 68 | 14 | 908 |
| Poisons Ratio | 0.13-0.29 | 0.20 | 0.05 | 14 |
| Density ($kN/m^3$) | - | 26.4 | - | - |
| Basic friction angle ($\Phi_b$) | - | $32^o$ | - | 28 |

Three discontinuity types were observed in the tunnel: joints, shears and bedding. Joints are the most common discontinuities. Shear zones of up to several centimetres in thickness are present. They usually contain crushed rock and strike more or less normal to the tunnel alignment with a dip of less than $45^o$. Bedding planes are rare. Several discontinuity (joint) sets were identified within the tunnel. In any given interval of the tunnel, with some exceptions, typically three joint sets are prominent with other sets present at random. The orientations of the prominent sets may change in different tunnel intervals. The exceptions are the fractured zones, where closely spaced four or more sets may be present. The presence of these joints creates a structural set up where rock blocks can theoretically be formed by several combinations of intersecting joints. Since the in situ stresses were low, movement of some of these blocks under gravity was possible. The orientations of the joint sets that contribute to tetrahedral wedge failure are listed in Table 2.

**Table 2 Average orientation of discontinuities in the CLD tunnel**

| Set # | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| Dip | 70 | 48 | 77 | 30 | 41 | 88 |
| Direction | 002 | 251 | 170 | 207 | 044 | 295 |

The discontinuity spacing, aperture size and their surface characteristics vary. The RQD along the tunnel is generally high. However, the presence of narrow fractured zones reduces the RQD value locally. The parameters concerning the discontinuities are shown in Figure 1.

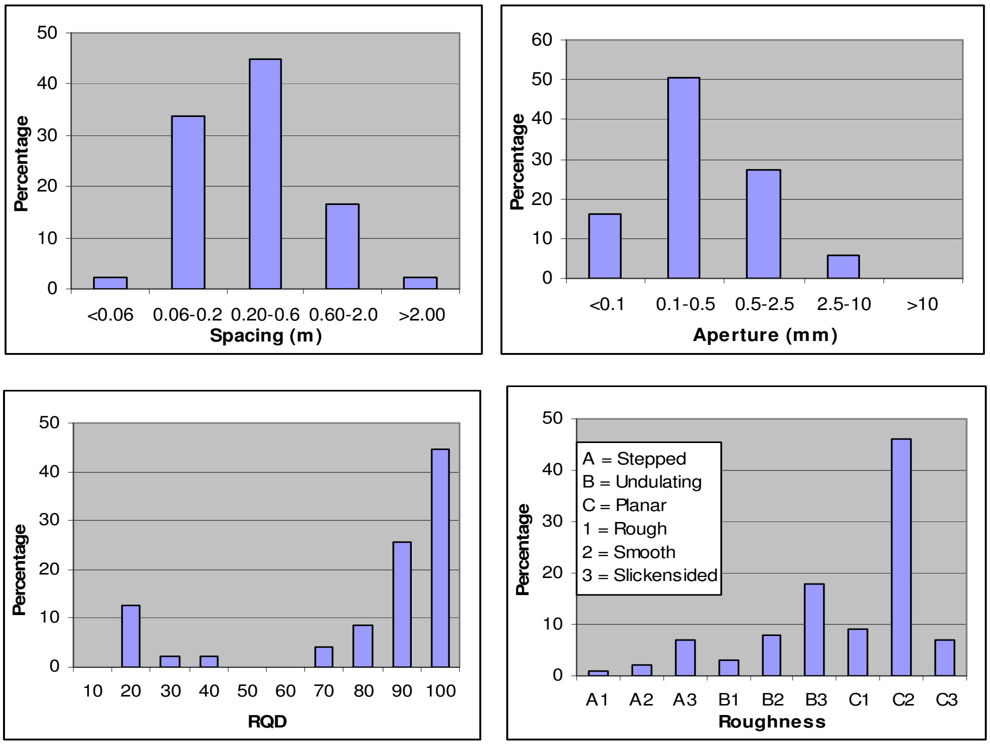


**Figure 1 Distribution of RQD, joint spacing, aperture size and roughness along the CLD tunnel**

## 3.1 RMR and Q derived support for the CLD tunnel

As mentioned earlier, the CLD tunnel was divided into 47 sectors. The sector lengths varied from 4 to 54 m, and the two methods were applied to each sector separately. Table 3 presents a summary of the ratings assigned to the input parameters of the two classification methods. Figure 2 presents the distribution of the ratings assigned to joint condition and groundwater condition parameters of the RMR method. Ratings for the other RMR parameters were based on the data presented in Figure 1. Figure 3 presents the distribution of ratings assigned to the Jn (joint set number), Jr (joint roughness number), Ja (joint alteration number) and Jw (joint water reduction factor) parameters of the Q system. Since the stress levels are low and favourable, and no major weak zones cut through or run parallel to the tunnel, for most of the tunnel, the stress reduction factor, SRF = 1. For the tunnel sectors affected by the zones of weakness, SRF values of 2.5 and 7.5 were used. An ESR (excavation support ratio) of 1.8 (for water tunnels) was used for support determination by Q. Tables 4 and 5 summarise the results of the application of RMR and Q, respectively.

**Table 3 RMR and Q input ratings for the CLD tunnel**

| RMR | | Q | |
|---|---|---|---|
| ***Parameter*** | Ratings range | ***Parameter*** | Ratings range |
| Strength | 7-12 | RQD | 15-100 |
| RQD | 3-20 | Jn | 2-20 |
| Joint spacing | 5-15 | Jr | 1.5-4 |
| Joint condition | 0-25 | Ja | 1-6 |
| Groundwater | 0-15 | Jw | 1-0.2 |
| Adjustment (-) | 0-12 | SRF | 1-7.5 |
| RMR value | 14-80 | Q value | 0.02-37.5 |

Table 4 shows that RMR classified 67% of the rock mass in the tunnel as good rock and 10%, 12% and 11% as fair, poor and very poor rock, respectively. For the tunnel roof and walls in poor and very poor classes of rock, RMR recommended rock bolts, mesh reinforced shotcrete and steel set support. For the roof and walls in fair and good classes of rock the recommended support measures were rock bolts and mesh reinforced shotcrete. No support was recommended for the tunnel walls in good rock. This means according to RMR no support is required for 67% of the tunnel walls.

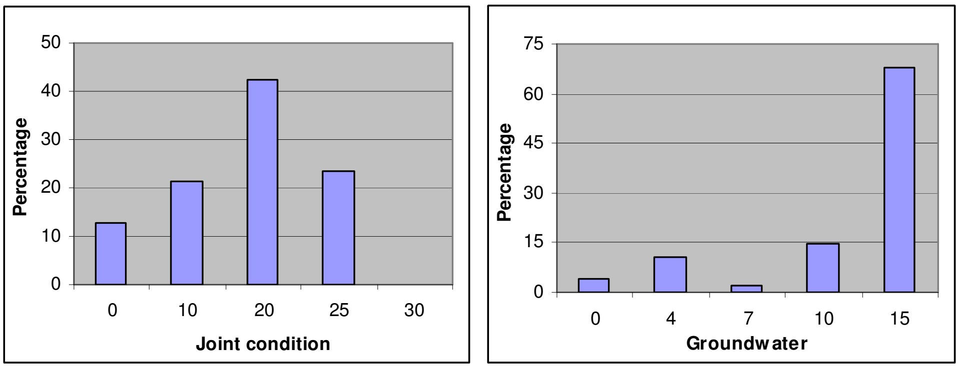


**Figure 2 Distribution of RMR's joint condition and groundwater ratings for the CLD tunnel**

**Table 4 RMR recommended support for the CLD tunnel**

| RMR value | 80-61 | 60-41 | 40-21 | <20 |
|---|---|---|---|---|
| Rock mass class | Good | Fair | Poor* | Very poor# |
| Amount in each class | 67% | 10% | 12% | 11% |
| *Roof support* | | | | |
| Bolts (m) | L=3 S=2.5 | L=4 S=1.5-2 | L=4-5 S=1-1.5 | L=5-6 S=1-1.5 |
| Shotcrete (mm) | 50 (mr)+ | 50-100 (mr) | 100-150 (mr) | 150-200 (mr) |
| Steel sets (m) | None | None | S=1.5* | S=0.75# |
| *Wall support* | | | | |
| Bolts (m) | None | As above | As above | As above |
| Shotcrete (mm) | None | 30 | 100 (mr) | 150-200 (mr) |

Note: L=length, S=spacing, mr=mesh reinforced; + If required, * Light to medium steel sets where required. # Medium to heavy steel sets with steel lagging & fore-poling, if required, and bolt & close invert.

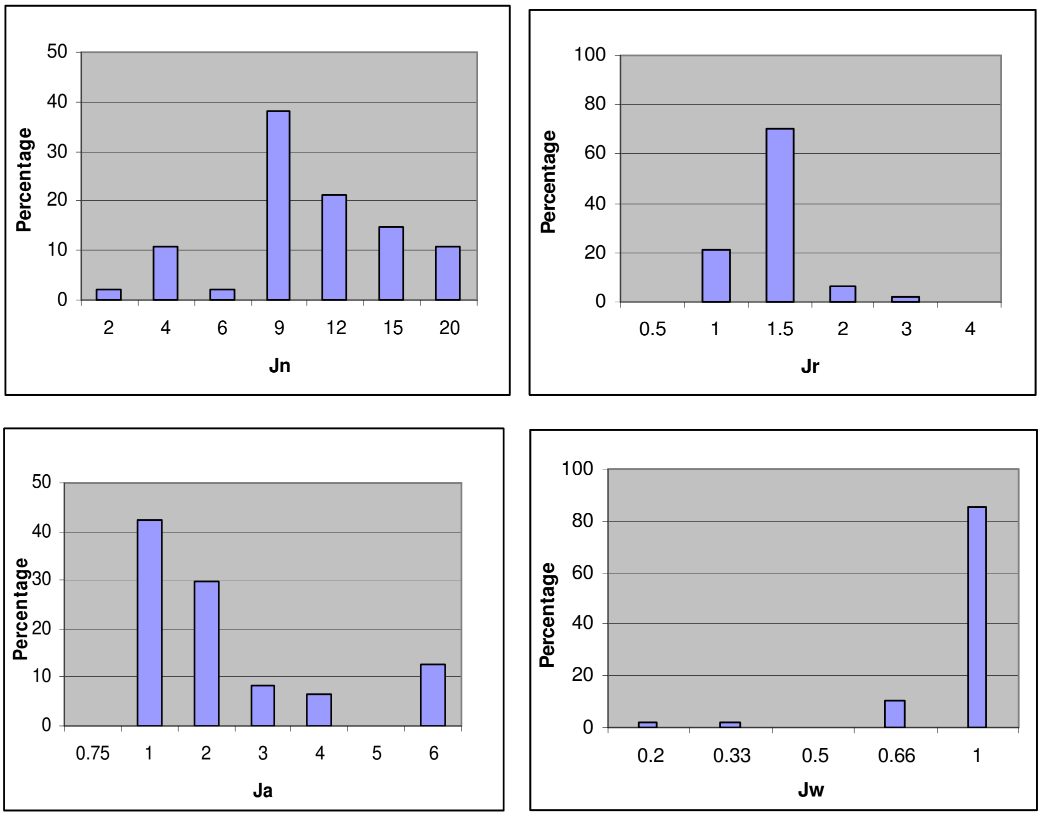


**Figure 3 Distribution of the ratings assigned to Jn, Jr, Ja and Jw (CLD tunnel)**

Table 5 shows that, Q classified 38% of the rock mass as good rock, 34% as fair rock, 5% as poor rock, 6% as very poor rock and 17% as extremely poor rock. Q recommended only spot bolting for the tunnel roof in good rock and systematic bolting plus 40 to 100 mm of shotcrete for the tunnel roof in fair rock. For the walls in both good rock and fair rock classes Q recommended no support. This means as per Q, 72% of the tunnel walls required no support (Table 5). Note that wall support is derived using $Q_{wall}$ ($Q_{wall} = 2.5Q$, when $0.1 < Q < 10$).

**Table 5 Q recommended support for the CLD tunnel**

| Q value | 10-40 | 4-10 | 1-4 | 0.1-1 | 0.01-0.1 |
|---|---|---|---|---|---|
| Rock mass class | Good | Fair | Poor | Very Poor | Ext poor$^{+}$ |
| Amount in each class | 38% | 34% | 5% | 6% | 17% |
| *Roof support* | | | | | |
| Bolts (m) | Spot bolting | L=4 S=2-2.3 | L=4 S=1.7-2.2 | L=4 S=1.3-1.7 | L=4 S=1-1.3 |
| Shotcrete (mm) | None | 40-100 | 50-70 (Fr) | 70-130 (Fr) | 150-200 (Fr) |
| *Wall support* | | | | | |
| Bolts (m) | None | None | L=3 S=1.7-2 | L=3 S=1.3-1.7 | L=3 S=1-1.3 |
| Shotcrete (mm) | None | None | 40-100 | 50-120 (Fr) | 120-150 (Fr) |

Note: L=length, S=spacing, Fr=fibre reinforced, $^{+}$ Steel reinforced cast concrete arch also recommended.

## 3.2 Tetrahedral wedge stability analysis of the CLD tunnel

A tetrahedral rock wedge stability analysis was undertaken to examine the adequacy of support measures recommended by the two empirical methods to stabilise potentially unstable rock wedges formed by intersecting discontinuities (joints) in the rock. For this purpose the ubiquitous joint method was adopted using UNWEDGE software code, which can analyse tetrahedral wedges formed by three intersecting joints and the free surface of the excavation, and allows identification of all possible tetrahedral wedges.

Since the tunnel overburden is between 40 and 80 m, two stress scenarios were considered in the analysis. The first assumed that the wedges are subjected to gravity loading with no effect from the in situ stress field. The second included an inferred in situ stress field; the stress field was assumed to be due to the weight of the overlying rock with the horizontal to vertical stress ratio $k=\sigma_h/\sigma_v=1.5$.

### *3.2.1 Shear strength parameters of joints*

Since the wedges may be present in most favourable or most unfavourable ground conditions in the tunnel, three joint shear strength scenarios were considered in the wedge analysis. The first scenario shear strength parameters, representing a best case (with no clay filling in joints), were estimated by means of the shear strength relationship of Barton and Choubey (1977) given below:

$$\tau_p = \sigma_n \tan \{\Phi_b + \text{JRC} \ \text{Log}_{10}(\text{JCS}/\sigma_n)\} \tag{1}$$

where $\tau_p$ = shear strength; $\sigma_n$ = joint normal stress; $\Phi_b$ = basic friction angle; JRC = joint roughness coefficient; and JCS = joint compressive strength. From the data presented by Ratanasatayanont (1984) the following values were selected: JCS=60 MPa, JRC=2.5, $\Phi_b=32^o$, and $\sigma_n$=1 MPa (based on vertical stress due to gravity). Using these input values and the Mohr-Coulomb relationship $\tau_p = \sigma_n \tan \Phi + c$, joint shear strength parameters c=50 kPa and $\Phi=35^o$ were obtained for a best case (most favourable) joint conditions.

The second joint shear strength scenario was estimated using the frictional component only (c=0) relationship $\Phi=\tan^{-1}(Jr/Ja)$ suggested by Barton (2002). For this purpose the joint roughness (Jr) and joint alteration (Ja) parameters shown in Figure 3 were used. Figure 3 shows that the most common Jr value is 1.5 (70%) and the next common value is 1 (21%). The most common Ja value is 1.0 (43%) and the next common value is 2.0 (30%). Accordingly, the most common joint friction angle is $\Phi=\tan^{-1}(1.5/1.0)=56^o$. The other possible values are $\Phi=\tan^{-1}(1.5/2.0)=37^o$, $\Phi=\tan^{-1}(1.0/1.0)=45^o$, and $\Phi=\tan^{-1}(1.0/2.0)=27^o$. From these four friction angles, $\Phi=56^o$ was selected as a possible best case scenario, which also represent unfilled joints with no cohesion component (c=0).

For the filled joints, c=10 kPa and $\Phi=25^o$ were selected based on the data compiled by Barton (1974) and the suggestions given by Barton & Grimstad (1994) for filled joints, and also considering the potential for water saturation of the joints. The filled joints represent a worst case joint shear strength scenario in the rock mass. The lowest friction angle, $\Phi=27^o$, obtained by $\Phi=\tan^{-1}(Jr/Ja)$ formula indicates that the estimated worst case joint shear strength parameters for filled joints c=10 kPa and $\Phi=25^o$ are reasonable estimates.

### *3.2.2 UNWEDGE analysis*

The UNWEDGE analysis showed that several tetrahedral wedges are kinematically possible in the CLD tunnel roof and walls. The stability of the wedges with maximum apex heights greater than 1 m were analysed for the three shear strength scenarios with and without the effect of the stress field in the rock mass.

The empirically derived ground support measures were then included in the UNWEDGE model to examine their adequacy to stabilise the theoretically possible rock wedges. First the effect of rock bolts recommended by the two classification methods was analysed. The bolts considered were the cement grouted type with 100% bond efficiency and an ultimate tensile strength of 180 kN installed normal to the rock face. The effect of empirically derived shotcrete layers was also analysed using UNWEDGE, which can compute punching shear capacity of shotcrete along the edge of a rock wedge.

The results of the analysis show that the RMR recommended support measures for fair, poor and very poor classes of rock are sufficient to stabilise the theoretically possible rock wedges in the tunnel roof and walls. RMR recommended support for the good rock class is also sufficient for the possible tetrahedral rock wedges in the tunnel roof. Similarly, Q recommended support measures for poor, very poor and extremely poor classes of rock are sufficient for the theoretically possible rock wedges in the tunnel roof and walls. The Q recommendations for the tunnel roof in fair rock are also sufficient for the possible tetrahedral rock wedges. A factor of safety (FOS) of 1.5 and 2 for walls and roof, respectively, were selected for long term stability.

However, RMR classified 67% of the rock mass as good rock, and recommended no support for the tunnel walls. Similarly, Q classified 38% and 34% of the tunnel as good rock and fair rock respectively, and recommended no support for the walls in these two classes of rock (total of 72% of the tunnel).

The wedge analysis showed that there is potential for several different tetrahedral rock wedges in the tunnel walls. The analysis also showed that the stability (or the FOS) of these wedges is sensitive to both joint shear strength and the stress field around the tunnel. The stress field around the tunnel increases the FOS of deep narrow wedges by clamping the wedges in place. In the case of shallow flat wedges the stress field reduces the FOS by forcing the wedges out. It should be noted that UNWEDGE does not accurately model the wedge failure caused by the stress field around the tunnel. However, UNWEDGE allows identification of wedges with no restraining effect from the in situ stress field. For such wedges the FOS computed for gravity loading alone (unstressed state) may be considered applicable. In this study the rock wedges with sufficient clamping effect from the in situ stress field to provide an acceptable FOS were not analysed further. The flat wedges that have no stabilising effect from the stress field were further analysed to assess the effect of the changing groundwater conditions on their stability in the long term.

As mentioned earlier, after the completion of the main dam construction, the tunnel was plugged and converted to an irrigation outlet by providing a valve in the plug, approximately 105 m from the inlet. During the operation of the project the groundwater level around the tunnel length downstream of the plug is likely to be elevated by the reservoir. The elevated groundwater level around this interval of the tunnel could cause erosion of discontinuity infill materials and contribute to failure in the longer term. Hence the UNWEDGE analysis was extended to examine the effect of the elevated groundwater level on the stability of the theoretically possible rock wedges. The effect of the groundwater level was modelled using the elevation water pressure option in UNWEDGE. Several groundwater elevations were modelled ignoring the effect of the in situ stress field. The results of the analysis showed that when the groundwater level is elevated to about mid height of the tunnel the FOS of several rock wedges fall well below unity. When the groundwater table is about 8 m above the invert level, which is considered likely, the FOS of several rock wedges becomes zero or near zero indicating potential instability. For each wedge analysed the FOS for seven cases are presented in Table 6. Except wedge no. 1, the details presented in Table 6 are for wedges that have no stabilising effect from the in situ stress field. Wedge no. 1 illustrates the clamping effect of the in situ stress field on deep narrow wedges. The seven cases are explained at the bottom of Table 6.

**Table 6 Potentially sliding rock wedges in the CLD tunnel walls**

| Wedge | Sets | Wall | Apex (m) | Weight (kN) | FOS1 | FOS2 | FOS3 | FOS4 | FOS5 | FOS6 | FOS7 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 123 | Left | 3.7 | 517 | 2.70 | 2.71 | 1.40 | 0.83 | 1.73 | 1.33 | 0.53 |
| 2 | 125 | Left | 1.4 | 210 | 5.16 | 0.85 | 0.00 | 1.33 | 0.53 | 1.33 | 0.00 |
| 3 | 125 | Right | 1.4 | 242 | 5.38 | 0.84 | 0.00 | 1.42 | 0.52 | 1.70 | 0.00 |
| 4 | 126 | Left | 1.1 | 62 | 4.86 | 1.04 | 0.00 | 1.27 | 0.65 | 1.33 | 0.00 |
| 5 | 146 | Right | 1.9 | 177 | 4.90 | 1.24 | 0.00 | 1.15 | 0.78 | 0.78 | 0.00 |
| 6 | 245 | Left | 2.8 | 1494 | 3.08 | 0.96 | 0.00 | 1.18 | 0.61 | 2.57 | 0.00 |
| 7 | 245 | Left | 2.6 | 1581 | 3.42 | 0.96 | 0.00 | 1.06 | 0.64 | 1.70 | 0.00 |
| 8 | 256 | Left | 2.5 | 571 | 6.16 | 1.39 | 0.60 | 1.98 | 0.88 | 3.42 | 0.58 |
| 9 | 256 | Right | 2.3 | 606 | 5.33 | 1.36 | 0.61 | 1.57 | 0.86 | 2.28 | 0.73 |
| 10 | 356 | Right | 1.7 | 109 | 3.50 | 1.35 | 0.00 | 1.07 | 0.86 | 1.70 | 0.00 |
| 11 | 356 | Left | 1.2 | 54 | 8.13 | 1.39 | 0.00 | 1.82 | 0.87 | 0.89 | 0.00 |

FOS1 - c=50 kPa $\Phi=35^o$ $\sigma_h=\sigma_v=0$ $h_w=0$;
FOS2 - c=50 kPa $\Phi=35^o$ $\sigma_h=1.5\sigma_v$ $h_w=0$;
FOS3 - c=50 kPa $\Phi=35^o$ $\sigma_h=\sigma_v=0$ $h_w=7$ m;
FOS4 - c=10 kPa $\Phi=25^o$ $\sigma_h=\sigma_v=0$ $h_w=0$.
FOS5 - c=10 kPa $\Phi=25^o$ $\sigma_h=1.5\sigma_v$ $h_w=0$;
FOS6 – c=0 kPa $\Phi=56^o$ $\sigma_h=\sigma_v=0$ $h_w=0$;
FOS7 - c=0 kPa $\Phi=56^o$ $\sigma_h=\sigma_v=0$ $h_w=7$ m;

Since the analysis assumed that the joints are ubiquitous these wedges may or may not be present. If they are present in the tunnel walls, some form of support will be required to ensure their long term stability. As previously mentioned, RMR recommended no support for 67% of the tunnel walls and Q recommended no support for 72% of the tunnel walls.

During excavation several large potentially unstable rock blocks were identified in the roof and walls of the tunnel which were temporarily stabilised using 6, 4 and 3 m long mechanically anchored and resin grouted rock bolts and wire mesh. At the outlet portal area steel rib support was also installed. The tunnel was fully lined with a 700 mm nominal thickness in situ cast concrete liner. Among the issues considered in selecting the final support were fluctuating water levels during river diversion and increase in groundwater pressure around the tunnel after the reservoir filling. Both of these could cause erosion of discontinuity filling material, which in turn could lead to instability in the tunnel.

# 4 Huai Saphan Hin hydropower tunnel

The HSHP tunnel feeds water from the Hua Saphan Hin reservoir to a 12.2 MW powerhouse. The tunnel overburden varies between 35 m and 90 m, with 90% of the tunnel having more than 50m of overburden. The tunnel alignment is N55$^{o}$E with a 1% gradient. The rock formation intersected in the tunnel comprised a sequence of sub-vertical beds of greywacke and shale, striking more or less normal to the tunnel alignment. Greywacke is the main rock type, and shale is present in small amounts usually inter-bedded with greywacke. The cumulative thickness of shale in the tunnel is about 35 m (approximately 5% of the tunnel). The results of greywacke intact rock material tests are given in Table 7.

**Table 7 Intact rock properties along the HSHP Tunnel**

| Property | Range | Mean | Std | # of tests |
|---|---|---|---|---|
| UCS (MPa) | 76-141 | 104 | 27 | 8 |
| Point load index (MPa) | 64-163 | 111 | 26 | 12 |
| E Modulus (GPa) | 42-92 | 66 | 13 | 15 |
| Poisson's Ratio | 0.17-0.38 | 0.28 | 0.07 | 15 |
| Density (kN/m$^3$) | | 27.2 | | |

Four major geological discontinuity sets (1, 2, 3 and 4) and four minor sets (5, 6, 7 and 8) were identified by mapping. The general orientations of the discontinuity sets are given in Table 8. Three discontinuity types are present: joints, faults and bedding planes. Joints are the most common structural features. Of the 739 discontinuity measurements 649 are joints. Joint spacing varies from moderate (200-600 mm) to very close (20-60 mm). The joint aperture size ranges from tight (0.1-0.5 mm) to very wide (>10 mm). The joints were usually filled with silica and clayey materials. Faults are the next most common structural feature with displacement of 5 to 18 cm. The fault zones contain gouge and fault breccia and bounding surfaces are slickensided. Three subsets of faults, Crushed zones (CZ), Sheared zones (SZ) and Shattered zones (ShZ), are also present. They have fault characteristics, but their offset was not visible. CZ consists of zones of angular rock fragments and plastic clayey material. SZ represent closely spaced, sub-parallel and slickensided shear planes often coated with clay. ShZ are closely fractured and shattered rock zones consisting of angular rock fragments with minor amounts of clay. Generally the faults (including subsets) observed were less than 10 cm in thickness with occasionally up to 30 cm. A total of 72 faults were observed at regular intervals. Bedding planes are rare with only 18 measurements and present only along the contact between the two rock types. Their openings are usually filled with clay and surfaces are generally smooth. The faults (including CZ, SZ and ShZ) and beddings strike northwest (more or less normal to the tunnel axis) and dip 60 to 90$^{o}$ towards northeast or southwest. They represent discontinuity set 1.

**Table 8 Discontinuity orientation data (HSHP tunnel)**

| Set # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|---|
| Dip | 89 | 89 | 72 | 71 | 38 | 11 | 09 | 88 |
| Direction | 238 | 297 | 087 | 199 | 310 | 331 | 177 | 150 |

## 4.1 RMR and Q derived support for the HSHP tunnel

In order to apply the two rock mass classification systems, the HSHP tunnel was divided into five sectors based on the major discontinuities and groundwater conditions, and RMR and Q were applied to the most favourable (best) and most unfavourable (worst) rock mass conditions in each of the five sectors. The five sectors are: Sector 1: 011 to 150 m, Sector 2: 150 to 220 m, Sector 3: 220 to 320 m, Sector 4: 320 to 550 m and Sector 5: 550 to 733 m. The ratings assigned to the input parameters and the final RMR and Q values are shown in Table 9. The RMR and Q recommended support measures are shown in Table 10. An ESR of 1.8 (for water tunnels) was used with Q.

**Table 9 RMR and Q input ratings for the HSHP tunnel**

| RMR | | Q | |
|---|---|---|---|
| ***Parameter*** | Ratings range | ***Parameter*** | Ratings range |
| Strength | 7-12 | RQD | 32-99 |
| RQD | 8-20 | Jn | 9-15 |
| Joint spacing | 5-20 | Jr | 1-3 |
| Joint condition | 0-25 | Ja | 1-6 |
| Groundwater | 4-10 | Jw | 1-0.5 |
| Adjustment (-) | 0-12 | SRF | 2.5-10 |
| RMR value | 24-82 | Q value | 0.04-6.6 |

**Table 10 RMR and Q recommended support for the HSHP tunnel**

| Sector | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| *RMR* | | | | | |
| Best case RMR value | 82 | 70 | 64 | 87 | 82 |
| Bolts (m) | None | L=2 locally | L=2 locally | None | None |
| Shotcrete (mm) | None | 50 (mr) | 50 (mr) | None | None |
| Worst case RMR value | 34 | 27 | 24 | 43 | 41 |
| Bolts (m) | L=2 S=1-1.5 | L=2 S=1-1.5 | L=2 S=1-1.5 | L=2 S=1.5-2 | L=2 S=1.5-2 |
| Shotcrete (mm) | 100-150 (mr) | 100-150 (mr) | 100-150 (mr) | 50-100 (mr) | 50-100 (mr) |
| *Q* | | | | | |
| Best case Q value | 6.40 | 6.40 | 2.11 | 6.60 | 6.53 |
| | | No support recommended for the best case. | | | |
| Worst case Q value | 0.62 | 0.50 | 0.04 | 1.12 | 1.28 |
| Bolts (m) | L=2.7 S=1.5-1.7 | L=2.7 S=1.5-1.7 | L=2.7 S=1.5 | None | None |
| Shotcrete (mm) | 50 | 50 | 50-90 (fr) | None | None |

Note: L=length, S=spacing, mr=mesh reinforced; fr=fibre reinforced.

## 4.2 Tetrahedral wedge stability analysis of the HSHP tunnel

As for the CLD tunnel, two stress scenarios were considered in the tetrahedral rock wedge analysis. The first scenario assumed that the wedges are subjected to gravity loading only; the second scenario included an assumed in situ stress field corresponding to the weight of the overburden and $k=\sigma_h/\sigma_v=1.5$.

### *4.2.1 Shear strength parameters of joints*

In order to assess the sensitivity of wedge stability (wedge FOS) to the shear strength parameters of discontinuities, two shear strength scenarios were considered. Since the rock type and its joint filling materials are similar in description to those of the CLD tunnel discussed earlier, it was assumed that the same shear strength values c=50 kPa, $\Phi=35^o$ and c=10 kPa, $\Phi=25^o$ represent, respectively, the best case and the worst case ground conditions considered for classifying the rock mass using RMR and Q.

### *4.2.2 UNWEDGE analysis*

The results of the analysis performed using UNWEDGE confirmed that several rock wedges are kinematically possible in both roof and walls of the tunnel. The FOS of each wedge under the two joint shear strength scenarios with and without the effect of stress field, along with the joint set combination, location, maximum apex height and maximum weight of the rock wedges, are presented in Table 11.

**Table 11 Results of tetrahedral wedge analysis (HSHP tunnel)**

| Wedge | Sets | Location | Failure mode | Apex (m) | Weight (kN) | FOS1 | FOS2 | FOS3 | FOS4 |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 158 | Roof | Sliding | 2.0 | 164 | 0.80 | 1.58 | 0.17 | 1.08 |
| 2 | 168 | Roof | Sliding | 0.7 | 71 | 0.44 | 0.70 | 0.09 | 0.49 |
| 3 | 168 | Roof | Sliding | 0.8 | 96 | 4.86 | 0.74 | 0.98 | 0.51 |
| 4 | 246 | L/wall | Sliding | 0.9 | 30 | 2.38 | 0.99 | 0.54 | 0.65 |
| 5 | 246 | R/wall | Sliding | 0.7 | 15 | 4.12 | 0.88 | 0.83 | 0.83 |
| 6 | 247 | L/wall | Sliding | 1.0 | 29 | 2.96 | 1.18 | 0.66 | 0.77 |
| 7 | 258 | Roof | Sliding | 1.3 | 67 | 2.28 | 1.49 | 0.46 | 0.97 |
| 8 | 268 | Roof | Sliding | 0.7 | 84 | 0.80 | 0.67 | 0.17 | 0.45 |
| 9 | 268 | Roof | Falling | 0.8 | 105 | 4.24 | 0.69 | 0.86 | 0.46 |
| 10 | 358 | Roof | Falling | 1.6 | 105 | 0.00 | 1.38 | 0.00 | 0.89 |
| 11 | 368 | Roof | Sliding | 0.9 | 124 | 0.76 | 0.75 | 0.26 | 0.52 |
| 12 | 368 | Roof | Falling | 0.7 | 67 | 0.00 | 0.66 | 0.00 | 0.45 |
| 13 | 468 | Roof | Sliding | 0.8 | 80 | 5.32 | 0.99 | 1.06 | 0.65 |
| 14 | 568 | R/wall | Sliding | 1.5 | 522 | 3.02 | 1.11 | 1.02 | 0.79 |

FOS1 - c=50 kPa $\Phi=35^o$ $\sigma_h=\sigma_v=0$; FOS2 - c=50 kPa $\Phi=35^o$ $\sigma_h=1.5\sigma_v$; FOS3 - c=10 kPa $\Phi=25^o$ $\sigma_h=\sigma_v=0$;
FOS4 - c=10 kPa $\Phi=25^o$ $\sigma_h=1.5\sigma_v$.

The analysis showed that, the support measures recommended by RMR, for both best and worst case ground conditions considered, are sufficient for theoretically possible tetrahedral rock wedges in the tunnel.

The Q recommended support measures for the worst case ground conditions are also sufficient to stabilise the theoretically possible tetrahedral rock wedges in the tunnel. However, Q did not recommend any support for the best ground conditions considered in the study. As shown by the wedge analysis, several tetrahedral rock wedges are possible in the tunnel. The FOS of some of these wedges under the two joint shear strength scenarios are well below the acceptable level when the effect of in situ stress field around the tunnel is ignored. On the other hand there are shallow relatively flat wedges which may become unstable due to the effect of the in situ stress field even when the best case joint shear strength scenarios were assumed. If these wedges are present in the tunnel some form of restraint will be required to prevent failure.

During excavation of the HSHP tunnel, Lasao (1986) observed that structurally controlled failures were the main modes of instability, which included wedge failures due to intersecting discontinuities and slab failure from the roof due to flat dipping joints and loosening of the rock mass around weakness zones such as faults, which cut across the tunnel alignment. The results of the wedge analysis agree with these observations.

The actual support installed in the tunnel included steel ribs with steel/timber lagging and invert struts near the portals and at four intervals inside the tunnel in worst ground conditions (a 5 m interval each in Sectors 3 and 4, and a 4 m and an 8 interval in Sector 5). In other areas, rock bolts and wire mesh where necessary to provide safety during construction. After the completion of the excavation the tunnel, a 500 mm thick cast in place concrete lining was constructed, partly for hydraulic reasons. The final support (lining) installed was substantially greater than the support recommended by the two empirical methods.

# 5 Discussion

The wedge analysis assumed that the joints are ubiquitous and that the theoretically possible maximum size wedges would be formed. Therefore the wedge analysis may be considered conservative. However, it should be noted that only the tetrahedral (or pyramidal) wedges were analysed. Non-pyramidal blocks (and slabs) that may be formed by more than three intersecting joints were excluded, and the comparison is limited to the potential tetrahedral rock wedge instability under the selected joint shear strength and stress scenarios. As mentioned before, several potentially unstable rock wedges were identified in the case studies. Some roof failures occurred in the HSHP tunnel before the installation of support. However, both tunnels were supported with rock bolts, and occasional steel ribs in poor rock zones.

The limitations of the rock mass classification methods for tunnels subjected to changing loading conditions have been known and were discussed in detail by Speers (1992). Speers concluded that "the use of empirical support design methods such as the RMR-method and Q-method will lead to under designs …" and consequently recommended analytical approaches for such situations. For mining excavations with changing stress conditions Mikula and Lee (2003) showed that Q can be successfully applied by adjusting the SRF rating to reflect the expected future stress conditions. Similarly, the change in groundwater level subsequent to the construction of a tunnel may be accounted for by adjusting the Jw rating in Q (and ground water rating in RMR) to reflect the anticipated groundwater pressure. However, the artificially elevated groundwater head considered in the analysis of the CLD tunnel is not high enough to down grade the Jw rating. It falls within the minor inflow/pressure range and receives a Jw rating of 1. With RMR the inferred groundwater pressure lowers the overall rating by three. But this does not significantly change the recommended support.

For small diameter (i.e. 3.5 m) water tunnels, for which ESR = 1.6 to 2.0 and De = 1.75 to 2.2, the Q system recommends no support if Q > 1. However, as seen in the HSHP tunnel case tunnel, support could be warranted when the Q value is as high as 6. For temporary mine excavations such as ore development drives with a typical width of no more than 4 m, for which the recommended ESR value is around 2, the Q system would not recommend any support even when the computed Q value is marginally less than 1. Dependent on the site specific conditions this may have safety implications. In such situations, as mentioned by Mikula and Lee (2003), ESR should also be selected based on site specific ground conditions.

With RMR, some difficulties arise when support measures are designed for tunnels with diameters other than 10 m, such as the CLD and HSHP tunnel, because the RMR recommendations are for 10 m diameter tunnels. In this situation, other empirical guidelines should be used for selecting bolt lengths, shotcrete thickness and steel ribs etc.

The study shows that the two empirical methods should not be used on their own for support design without cross checking by means of other design tools.

# 6 Conclusions

Based on detailed geotechnical mapping of the rock mass and testing of intact rock samples, RMR and Q were applied to an 11.3 m diameter tunnel and a 3.5 m diameter tunnel driven by drill and blast methods through sedimentary rock formations, and support requirements were determined. The support recommendations were then compared with the results of a tetrahedral rock wedge stability analysis of the rock mass around the two tunnels. The study shows that while the empirically derived support measures are adequate for stabilising most of the potentially unstable rock wedges, in some instances, they are insufficient to provide adequate safety margin against some of the potential rock wedge failures, particularly under changing groundwater conditions.

In the case of the 11.3 m diameter diversion tunnel, under the groundwater conditions existed during the construction stage of the project, the empirically derived support measures were sufficient to stabilise the potentially unstable rock wedges identified by the tetrahedral wedge analysis. However, the wedge analysis showed that the subsequent change in groundwater level could cause previously stable rock blocks to become unstable, hence the empirical support recommendations become inadequate.

In the case of the 3.5 m diameter tunnel, the empirically derived support measurers were sufficient to stabilise the potentially unstable tetrahedral rock wedges in the worst case ground conditions. For the best scenario ground conditions, Q did not recommend any support. However, the wedge analysis showed that even under the best case joint shear strength parameters considered tetrahedral rock wedge instability is possible. During construction several structurally controlled failures occurred and the potential failures identified in advance were stabilised using rock bolts. Steel ribs were also used in the poor rock mass conditions, for which RMR recommended steel ribs, rock bolts and shotcrete and Q recommended rock bolts and shotcrete.

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# AN EVALUATION OF ROCK MASS CLASSIFICATION METHODS USED FOR TUNNEL SUPPORT DESIGN

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## Abstract

RMR and Q derived support measures for three tunnels were compared with those derived by rock wedge analysis. A numerical model was also run to assess the effect of internal water pressure in one of the tunnels. The study showed that although the empirical support recommendations are adequate in general, they do not meet some of the project specific requirements.

*Keywords*: tunnel; rock mass; RMR; Q; rock wedge.

## 1. Introduction

The most widely used rock mass classification methods for tunnel support design are RMR [1] and Q [2]. Over the years these methods have been revised to incorporate the experience gained subsequent to their initial introduction, RMR [3] and Q [4]. Notwithstanding the revisions, these methods still have limitations and room for improvements [5, 6, 7, 8, 9]. To identify their limitations and to suggest improvements, where possible, these methods can be evaluated by comparing their support recommendations with the support derived by other applicable methods.

This paper applies RMR and Q to three water conveyance tunnels using the data collected during construction. The support measures derived by the two methods are compared with the results of a tetrahedral rock wedge analysis undertaken using UNWEDGE [10]. The effect of internal water pressure on the rock mass around one of the tunnels was also assessed by a numerical analysis using UDEC [11].

## 2. The Tunnels Studied

The three case tunnels are: (a) 493 m long, 11.3 m span horseshoe-shaped Chiew Larn diversion (CLD) tunnel and (b) 240 m long, 13 m span horseshoe-shaped Chiew Larn power (CLP) tunnel of the Chiew Larn hydropower project in the Southern Province of Thailand, and (c) 732 m long, 3.5 m wide and 3.5 m high D-shaped Huai Saphan Hin power (HSHP) tunnel located on the eastern seaboard of Thailand. The three tunnels were driven by drill and blast methods.

The CLD tunnel, initially a river diversion tunnel for dam construction to create a reservoir, was converted to an irrigation tunnel by plugging it at approximately 105 m from the inlet and providing an outlet valve in the plug. During excavation, the tunnel was mostly dry, except for some isolated areas of water inflow during the wet season. The groundwater level around the tunnel length downstream of the plug was later elevated by the reservoir, which has a maximum elevation of 95 m RL (tunnel invert is at ~10 m RL). The average tunnel alignment is NW-SE with a 0.2% gradient. The tunnel overburden thickness varies from 40 to 80 m with an average of 60 m.

The CLP tunnel feeds three 80 MW power units through three steel penstocks. Located in a hill slope, it has an alignment of $140^{o}$ E and a plunge of $10^{o}$. Its overburden thickness varies from 25 to 50 m, with an average of 30 m. The tunnel is above the regional groundwater level and was mostly dry, with water only dripping in some places during the wet season.

The HSHP tunnel feeds a 12.2 MW powerhouse. Its overburden varies from 35 to 90 m, with 90% of the overburden greater than 50 m. The tunnel alignment is N55°E with a 1% gradient. It is located below the groundwater table and water inflow varied from nil to medium.

## 3. Rock Mass Conditions

During excavation, the rock masses in the CLD, CLP and HSHP tunnels were classified according to RMR and Q by Ratanasatayanont [12], Ranasooriya [13] and Lasao [14], respectively.

The rock type encountered in the three tunnels is greywacke, except for about 5% of the HSHP tunnel which intersected shale. The results of testing of greywacke intact rock materials from the three tunnels are summarised in Table 1, which also presents basic friction angle ($\Phi_b$) obtained by direct shear testing of saw-cut samples collected from the CLD tunnel.

Table 1 Intact rock properties along the three tunnels

| Property | CLD | CLP | HSHP |
|---|---|---|---|
| UCS (MPa) | 60 | 138 | 104 |
| E Modulus (GPa) | 55 | 51 | 66 |
| Poisson's Ratio | 0.20 | 0.23 | 0.28 |
| Density (kN/m$^3$) | 26.4 | 26.5 | 27.2 |
| Basic friction angle ($\Phi_b$) | 32° | - | - |

Several discontinuity (joint) sets were identified along the three tunnels. The presence of these joints creates a structural set up where potentially unstable rock blocks can be formed by several combinations of intersecting joints. Since the in situ stresses were low around the three tunnels, movement of these blocks under gravity was possible. The general orientations of the joint sets in the three tunnels that contribute to tetrahedral wedge failure are listed in Table 2.

Table 2 Orientation of discontinuities in the three tunnels

| Set | CLD | | CLP | | HSHP | |
|---|---|---|---|---|---|---|
| | Dip | Dir | Dip | Dir | Dip | Dir |
| 1 | 70 | 002 | 76 | 016 | 89 | 238 |
| 2 | 48 | 251 | 79 | 112 | 89 | 297 |
| 3 | 77 | 170 | 37 | 231 | 72 | 087 |
| 4 | 30 | 207 | 62 | 151 | 38 | 310 |
| 5 | 41 | 044 | 44 | 067 | 11 | 331 |
| 6 | 88 | 295 | - | - | 88 | 150 |

Typically three joint sets are prominent in any given interval of the three tunnels with other sets present at random. In the three tunnels joint persistence, spacing, aperture size and surface characteristics vary as summarised in Table 3.

Table 3 Joint surface characteristics of the three tunnels

| Property | CLD | CLP | HSHP |
|---|---|---|---|
| Persistence (m) | 3 to >20 | 3 to >20 | >3 |
| Aperture (mm) | 0.1 to 10 | 0.25 to 100 | 0.1 to >10 |
| Spacing (m) | 0.06 to >2 | 0.6 to >2 | 0.01 to >0.6 |
| Filling | Coated/clay | Coated/clay | Silica/clay |
| Roughness | Smooth to slickensided | Rough to slickensided | Rough to slickensided |
| Waviness | Undulating to planar | Undulating to planar | Undulating to planar |

## 4. Application of RMR and Q

The rock masses intersected in the three tunnels were classified according to RMR [3] and Q [4] using the data presented in the references cited earlier. Tables 4 summarises the RMR and Q ratings assigned to the rock masses in the three tunnels, and the results are presented in Table 5.

Table 4 RMR and Q input ratings for the three tunnels

| *Parameter* | CLD | CLP | HSHP |
|---|---|---|---|
| | | **RMR Ratings** | |
| Strength | 7-12 | 12 | 7-12 |
| RQD | 3-20 | 13-20 | 8-20 |
| Joint spacing | 5-15 | 15-20 | 5-20 |
| Joint condition | 0-25 | 0-20 | 0-25 |
| Groundwater | 0-15 | 15-10 | 4-10 |
| Adjustment (-) | 0-12 | 5-10 | 0-12 |
| RMR value | 14-80 | 45-77 | 24-82 |
| | | **Q Ratings** | |
| RQD | 15-100 | 60-100 | 32-99 |
| Jn | 2-20 | 6-9 | 9-15 |
| Jr | 1.5-4 | 1.5-3 | 1-3 |
| Ja | 1-6 | 2-6 | 1-6 |
| Jw | 1-0.2 | 1 | 1-0.5 |
| SRF | 1-7.5 | 2.5-5 | 2.5-10 |
| Q value | 0.02-37.5 | 0.33-10 | 0.4-6.6 |

For each class of rock (Table 5) in the three tunnels, support measures were derived according to RMR and Q using Table 6 and Figure 1, respectively. Since RMR support recommendations are for 10 m diameter tunnels only, the bolt lengths (L) were adjusted using the empirical formula: L=1.40+0.184a, where "a" is tunnel span. For deriving Q support, an ESR (excavation support ratio) of 1.8 (for water tunnels) was used; wall support for the CLD and CLP tunnels were derived using $Q_w$ ($Q_w$=5Q, when Q>10; and $Q_w$=2.5Q, when 0.1<Q<10).

Table 5 Approx volume of rock in each RMR and Q class

| RMR | VG[a] | Good | Fair | Poor | VP[b] |
|---|---|---|---|---|---|
| CLD | - | 67% | 10% | 12% | 11% |
| CLP | - | 70% | 30% | - | - |
| HSHP | 61% | 19% | 11% | 9% | - |
| **Q** | **Good** | **Fair** | **Poor** | **VP[b]** | **EP[c]** |
| CLD | 38% | 34% | 5% | 6% | 17% |
| CLP | - | 70% | 9% | 21% | - |
| HSHP | - | 69% | 22% | 6% | 3% |

[a] very good, [b] very poor, [c] extremely poor

Table 6 RMR recommended support for 10 m diameter tunnels [3]

| Rock Mass Class | Bolts: 20mm diameter fully grouted | Shotcrete | Steel sets |
|---|---|---|---|
| I - Very good rock<br>RMR: 100-81 | Generally no support required except for occasional spot bolting | | |
| II – Good rock<br>RMR: 80-61 | Locally bolts in crown, 3 m long, spaced 2.5 m, with occasional mesh | 50 mm in crown where required | None |
| III - Fair rock<br>RMR: 60-41 | Systematic bolts 4-5 m long, spaced 1-1.5m in crown & walls with mesh | 50-100 mm in crown & 30 mm in sides | None |
| IV - Poor rock<br>RMR: 40-21 | Systematic bolts 4 m long, spaced 1.5-2 m in crown & walls with wire mesh in crown | 100-150 mm in crown & 100 mm in sides | Light to medium ribs spaced 1.5 m where required. |
| V - Very poor rock<br>RMR: 20-0 | Systematic bolts 5-6 m long, spaced 1-1.5 m in crown & walls with wire mesh. Bolt invert | 150-200 mm in crown & 150 mm in sides & 50 mm in face | Medium to heavy ribs spaced 0.75 m; lagging & forepoling if required. Close invert |

## 5. Tetrahedral Wedge Stability Analysis

A tetrahedral rock wedge stability analysis was undertaken using UNWEDGE to examine the adequacy of empirically derived support for stabilising the potentially unstable rock wedges in the three tunnels. The support measures considered were rock bolts and shotcrete. The bolts are of the cement grouted type with 100% bond efficiency and an ultimate tensile strength of 180 kN installed normal to the rock face. For the CLD and HSHP tunnels, two stress scenarios were considered in the analysis. The first assumed that the wedges are subjected to gravity loading with no effect from the in situ stress field. The second included an inferred in situ stress field assumed to be due to the weight of the overburden with the horizontal to vertical stress ratio $k=\sigma_h/\sigma_v=1.5$. Since the average overburden is only 30 m, the rock wedges in the CLP tunnel were assumed to be subjected to gravity loading only. The factors of safety (FOS) used for long term stability of the walls and roof were 1.5 and 2.0, respectively.

### 5.1. *Shear strength parameters of joints*

The wedge analysis used upper bound joint shear strength parameters estimated based on the joint conditions observed in the three tunnels. For the CLD tunnel, joint shear strength parameters were estimated by the following formula [15]:

$$\tau_p = \sigma_n \tan \{\Phi_b + \mathrm{JRC}\ \mathrm{Log}_{10}(\mathrm{JCS}/\sigma_n)\} \quad (1)$$

where $\tau_p$=shear strength; $\sigma_n$=joint normal stress; $\Phi_b$=basic friction angle; JRC=joint roughness coefficient; and JCS=joint compressive strength.

From the available data [12] the values selected were: JCS=60 MPa, JRC=2.5, $\Phi_b=32^o$, and $\sigma_n=1$ MPa (gravity induced). These values and the Mohr-Coulomb equation $\tau_p=\sigma_n\tan\Phi+c$, returned c=50 kPa and $\Phi=35^o$.

These strength parameters were validated using the frictional component only (c=0) relationship $\Phi=\tan^{-1}(Jr/Ja)$ [16]. In the CLD tunnel, the most common Jr (joint roughness) values were 1.5 (70%) and 1.0 (21%). The most common Ja (joint alteration) values were 1.0 (43%) and 2.0 (30%). Accordingly, the most common joint friction angle is $\Phi=\tan^{-1}(1.5/1.0)=56^o$. The other possible values are $45^o$, $37^o$ and $27^o$. A stability analysis using these friction angles showed that c=50 kPa and $\Phi=35^o$ represent a set of upper bound values for the CLD tunnel.

In the CLP tunnel, within the best case rock mass conditions, the most common Jr and Ja values are 2.0 (70%) and 3.0 (70%), respectively. Accordingly, the most common joint friction angle is $33^o$. Taking this and clay coating and filling in joints into account, the best case joint shear strength parameters c=10kPa and $\Phi=30^o$ were assumed for the CLP tunnel.

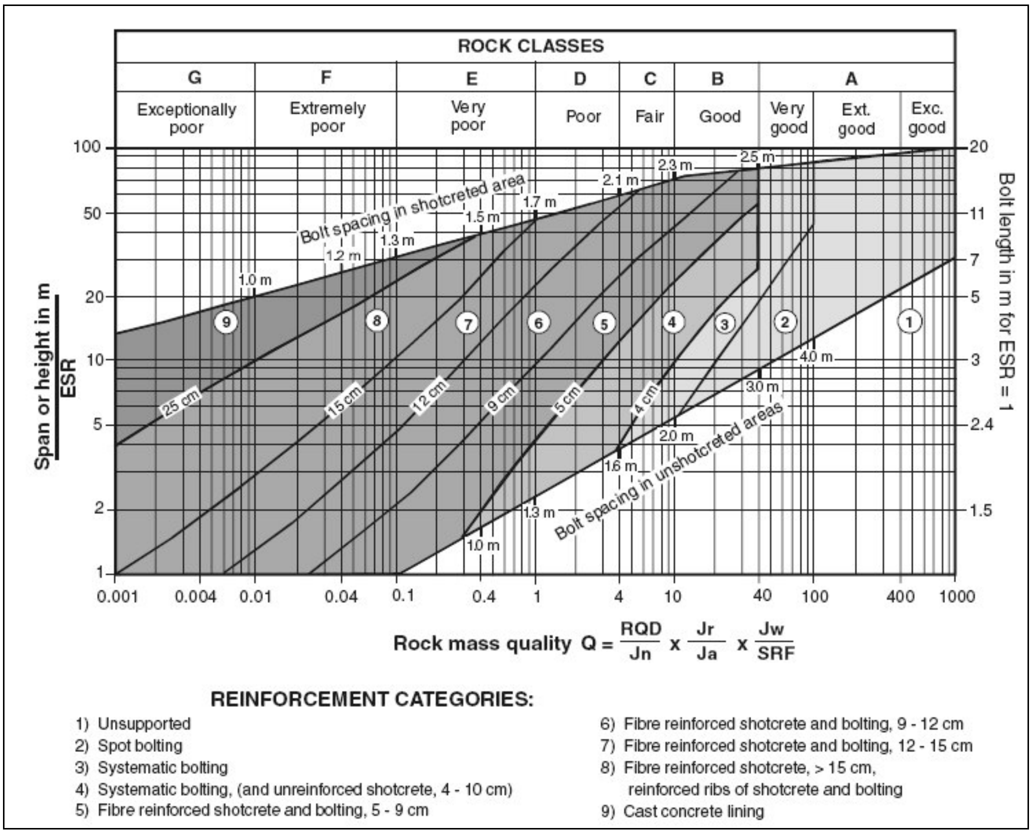


Figure 1 Q support chart after Barton and Grimstad [4]

In the best case rock mass conditions of the HSHP tunnel the most common Jr value is 3 (76%) and the other is 2 (14%). The most common Ja value is 2 (~100%). Accordingly, the most common joint friction angle in the best case rock mass is $\Phi=\tan^{-1}(3/2)=56^o$ and the next is $45^o$. These are comparable to those of the CLD tunnel. Hence, c=50 kPa and $\Phi=35^o$ were assumed to represent a set of upper bound joint shear strength parameters of the HSHP tunnel.

## 5.2. *Wedge analysis of the CLD tunnel*

The results of the analysis show that the RMR recommended support measures for fair to very poor classes of rock are sufficient to stabilise the possible rock wedges in the tunnel. Similarly, Q derived support measures for poor to extremely poor classes of rock are also sufficient. The Q recommendations for the roof in fair rock are also adequate for the tetrahedral rock wedges.

For the tunnel roof in good rock class, RMR recommended rock bolts plus mesh and 50 mm of shotcrete where required (shotcrete is probably for fractured ground, if present), and Q only recommended spot bolting. With these rock bolt systems, some roof wedges will have FOS values below acceptable levels if the effect of in situ stress field is ignored.

Further, RMR classified 67% of the rock mass as good rock (Table 5), and recommended no support for tunnel walls. Similarly, Q classified 38% and 34% of the tunnel as good and fair rock respectively, and recommended no support for walls in these classes of rock (72% of the tunnel).

The analysis showed that there is the potential for several tetrahedral rock wedges in walls, and that the stress field around the tunnel increases the FOS of deep narrow wedges by clamping them in place and reduces the FOS of shallow flat wedges by forcing them out. {Note that UNWEDGE does not accurately model the wedge failure caused by the stress field, but allows the identification of wedges with no restraining effect from the stress field. For such wedges, the FOS (FOS1 in Table 7) computed for gravity loading only may be considered applicable.} In this study the rock wedges in the CLD tunnel with sufficient clamping effect from the stress field to increase the FOS were not analysed further. The flat wedges that have no stabilising effect from the stress field (indicated by FOS2 in Table 7) were further analysed to assess the effect of the changing groundwater conditions, discussed in Section 2.

Several groundwater elevations were modelled using the "elevation water pressure" option in UNWEDGE, ignoring the effect of the stress field. The results of the analysis showed that when the groundwater level ($h_w$) is elevated to about mid height of the tunnel, the FOS of several rock wedges falls below unity. When the groundwater table is about 7 m above the invert level, which is likely, the FOS (FOS3 in Table 7) of several rock wedges becomes zero or near zero indicating potential instability (Table 7).

Table 7 Potentially unstable wedges in the CLD tunnel walls

| Sets | Wall | Apex (m) | Weight (kN) | FOS1 | FOS2 | FOS3 |
|---|---|---|---|---|---|---|
| 125 | L | 1.7 | 402 | 4.25 | 0.85 | 0.00 |
| 125 | R | 1.8 | 513 | 4.29 | 0.83 | 0.00 |
| 126 | L | 1.2 | 73 | 4.60 | 1.04 | 0.00 |
| 146 | R | 1.8 | 158 | 5.25 | 1.24 | 0.00 |
| 146 | L | 2.1 | 189 | 3.91 | 1.23 | 0.00 |
| 245 | L | 3.2 | 2336 | 2.69 | 0.97 | 0.00 |
| 245 | R | 3.1 | 2656 | 3.19 | 0.96 | 0.00 |
| 256 | L | 2.9 | 974 | 5.40 | 1.39 | 0.60 |
| 256 | R | 2.7 | 1151 | 4.51 | 1.35 | 0.61 |
| 356 | R | 1.7 | 113 | 3.50 | 1.35 | 0.00 |

FOS1: $\sigma_h=\sigma_v=0$ $h_w=0$; FOS2: $\sigma_h=1.5\sigma_v$ $h_w=0$
FOS3: $\sigma_h=\sigma_v=0$ $h_w=7$ m

## 5.3. *Wedge analysis of the CLP tunnel*

The analysis showed that RMR derived support for fair to very poor classes of rock and Q derived support for poor to very poor classes of rock are adequate to stabilise the possible rock wedges in the CLP tunnel.

For the tunnel walls in good rock mass class, RMR recommended no support. Similarly, for the walls in fair rock mass class Q recommended no support. However, there is potential for several tetrahedral rock wedges in walls with FOS (FOB in Table 8) below acceptable levels.

For the tunnel roof in good rock mass class, which represents almost 70% of the tunnel, RMR recommended rock bolts plus mesh and 50 mm of shotcrete where required (mesh and shotcrete are probably for fractured ground, if present). As can be seen from Table 8, there is potential for five rock wedges with zero or near zero FOS (FOB) in the roof. The first three of these wedges will have a FOS (FOR in Table 8) of less than or equal to one, if the RMR recommended rock bolting pattern is used. The analysis showed that shotcrete would increase the FOS of these possible rock wedges to an acceptable level. However, RMR does not recommend shotcrete for large rock wedges in the roof.

Table 8 Potentially unstable wedges in the CLP tunnel

| Sets | Location | Apex (m) | Weight (kN) | FOB | FOR | FOQ |
|---|---|---|---|---|---|---|
| 123 | Roof | 3.9 | 983 | 0.00 | 0.77 | 1.39 |
| 134 | Roof | 2.2 | 261 | 0.00 | 1.02 | 1.96 |
| 124 | Roof | 5.2 | 585 | 0.39 | 0.84 | 1.64 |
| 245 | Roof | 4.2 | 628 | 0.41 | 1.18 | 1.49 |
| 345 | R/wall | 5.2 | 7328 | 0.87 | n/a | n/a |
| 345 | L/wall | 5.1 | 6339 | 1.18 | n/a | n/a |
| 135 | R/wall | 3.9 | 4227 | 0.88 | n/a | n/a |
| 135 | L/wall | 3.9 | 4166 | 1.08 | n/a | n/a |
| 134 | L/wall | 3.6 | 1025 | 1.13 | n/a | n/a |
| 245 | R/wall | 2.6 | 278 | 1.02 | n/a | n/a |
| 123 | R/wall | 1.6 | 196 | 1.14 | n/a | n/a |

FOB: FOS with no support, FOR: FOS with RMR support, FOQ: FOS with Q support, n/a: not applicable.

With the support derived by Q for the tunnel roof in fair rock mass class the FOS (FOQ in Table 8) of some of the roof wedges is below the acceptable level for long term stability. Q did not recommend shotcrete for the roof in fair rock.

During excavation, several potentially unstable large rock blocks were identified within both the best and worst case ground conditions in the tunnel. These blocks were temporarily stabilised using 6, 4 and 3 m long mechanically and resin anchored rock bolts.

### 5.4. *Wedge analysis of the HSHP tunnel*

The analysis showed that RMR derived support for good to fair rock and Q derived support for very poor to extremely poor rock are sufficient for the tetrahedral rock wedges in the tunnel.

However, RMR did not recommend any support for very good rock (approximately 61%), and Q did nor recommend any support when Q>0.7. The analysis showed that the FOS (FOS1 in Table 9) of some of the tetrahedral rock wedges under the best case joint shear strength scenarios are below the acceptable level when the effect of the stress field around the tunnel is ignored. On the other hand, there are shallow, relatively flat wedges which may become unstable due to the effect of the stress field (FOS2). If these wedges are present in the tunnel, some form of restraint will be required to prevent failure.

Table 9 Potentially unstable wedges in the HSHP tunnel roof

| Sets | Apex (m) | Weight (kN) | FOS1 | FOS2 |
|---|---|---|---|---|
| 146 | 2.0 | 164 | 0.80 | 1.58 |
| 156 | 0.7 | 71 | 0.44 | 0.70 |
| 156 | 0.8 | 96 | 4.86 | 0.74 |
| 256 | 0.7 | 84 | 0.80 | 0.67 |
| 346 | 1.6 | 105 | 0.00 | 1.38 |
| 356 | 0.9 | 124 | 0.76 | 0.75 |
| 356 | 0.7 | 67 | 0.00 | 0.66 |

FOS1: $\sigma_h=\sigma_v=0$; FOS2: $\sigma_h=1.5\sigma_v$

During excavation, structurally controlled failure including wedge and slab instability occurred [14]. The results of the wedge analysis agree with these observations.

## 6. Internal Water Pressure (CLP tunnel)

Along the CLP tunnel, the internal water pressure is less than the confinement stresses due to overburden, hence, hydraulic jacking is unlikely. However, the natural groundwater level along the tunnel alignment is below the static water head and water loss from the tunnel by seepage through the rock mass is likely. This may cause instability at the ground surface, particularly on the hill slope. To assess this, a simplified 2D numerical (UDEC) model was run.

The model assumed two persistent joint sets, sub-parallel to the tunnel axis at 2 m spacing with the best case joint shear strength parameters considered earlier. Joint aperture size was varied to reflect the site conditions (Table 3).

In situ stresses were assumed to be due to gravity only. The intact rock blocks were assumed to be elastically deformable, with intact rock bulk modulus of 30 GPa and shear modulus of 20 GPa based on the laboratory determined intact rock

Young's modulus and Poisson's ratio (Table 1). Estimated joint normal and shear stiffness of 800 MPa/m and 100 MPa/m, respectively, were used, but were varied to investigate the sensitivity of the model. This study indicated, as expected, the higher the stiffness, the lower the displacement of rock blocks.

Two cases were modelled for several sections across the tunnel. Case 1: bolts were installed as per empirical recommendations with steady-state seepage from the tunnel. Case 2: an impermeable liner was installed covering the tunnel periphery.

Case 1 modelling indicated that seepage would occur through the rock mass even if the joint apertures were at the observed lowest range (0.25-0.5 mm). It was also evident that instability may occur at the ground surface (on the hill slope) when the overburden thickness is about 30 m (site average). The possible slope instability indicated by displacement vectors is shown in Figure 2 for a section with 32m thick overburden.

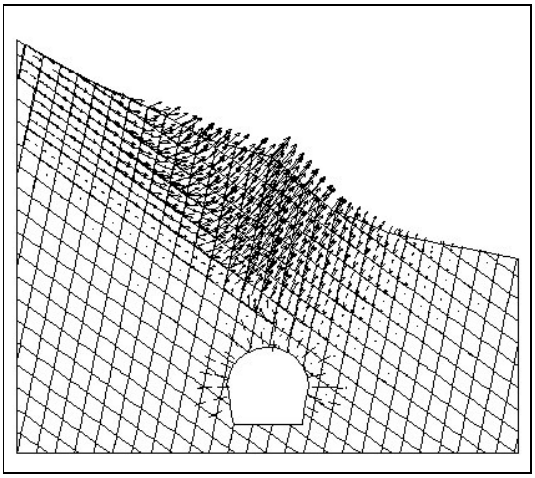

Fig.2 Displacement vectors showing ground movement

Case 2, which assumed no flow through the tunnel periphery, showed insignificant movement at the ground surface and demonstrated that an impermeable liner is required to prevent seepage losses from the tunnel and also to minimise the risk of instability on the slope above the tunnel.

The empirical methods recommended rock bolts and shotcrete for the worst case ground conditions intersected in the tunnel. For the tunnel roof in the best case rock mass conditions, only bolts are recommended and no support for walls. These empirically derived support measures are unlikely to eliminate seepage, which could eventually lead to instability at the ground surface. Even if shotcrete is applied along the entire tunnel periphery, additional to the empirical recommendations, due to the incipient cracks formed by shrinkage, expansion and shear movement etc., it may not completely eliminate seepage and the risk of instability at the ground surface. Considering the potential for water losses and ground instability on the hill slope the tunnel was fully steel lined. Rock bolting was used for construction safety.

## 7. Conclusions

The support measures derived using RMR and Q for three water conveyance tunnels were compared with the results of a tetrahedral rock wedge stability analysis. The study shows that while the empirically derived support measures are adequate for most of the potentially unstable rock wedges, in some instances, they are insufficient to provide adequate safety margins against some of the potential rock wedge failures.

Under the groundwater conditions that existed during construction of the 11.3 m diameter CLD tunnel, the empirical recommendations were adequate. However, changes in groundwater level could cause previously stable rock blocks to become unstable, hence empirical support recommendations become inadequate.

For the 13 m diameter CLP tunnel, the support derived by RMR and Q provide adequate safety factors against wedge failure mechanisms identified within the worst case rock mass conditions. For walls in the best case rock mass, the two empirical methods did not recommend any support. In contrast, the wedge analysis showed that rock bolts could be warranted for walls in the best case (RMR –good class, Q –fair class) rock mass conditions. Numerical modelling showed that, since the tunnel is shallow and located on a hill slope, seepage may occur from the tunnel and cause instability at the ground surface. These aspects are not considered in the two empirical methods.

In the 3.5 m diameter HSHP tunnel, the empirically derived support measures were sufficient for the potentially unstable rock wedges in the worst case ground conditions. For the best scenario (RMR – very good class, Q – good class) ground conditions, both RMR and Q did not recommend any support. The analysis showed that wedge instability is possible in the best case ground conditions. During construction, wedge failures occurred and the potential failures identified in advance were stabilised using rock bolts.

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# AN ASSESSMENT OF ROCK MASS CLASSIFICATION METHODS USED FOR TUNNEL SUPPORT DESIGN

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**ABSTRACT:** The support measures determined by RMR and Q rock mass classification methods for an access tunnel driven through jointed sedimentary rocks were compared with the support derived by limit equilibrium analyses of rock block stability and numerical simulation of the rock mass. The study indicates that the support measures recommended by the two classification methods are generally adequate, but under some of the conditions encountered in the tunnel, they do not necessarily meet the stability requirements.

*Keywords*: Rock mass; Classification; Tunnel; Support; Design

## 1. INTRODUCTION

Design of support for preventing instability in underground openings created in jointed rock formations can be undertaken by limit equilibrium analysis, numerical simulation or rock mass classification methods. The classification methods such as RMR [1] and Q [2] are sometimes preferred to other methods for the ease of their application at any stage of a project, even if detailed information on the rock mass is lacking. However, as with any other design tool, the rock mass classification methods also have limitations and, in some instances, the support designed by these methods may not be reliable [3, 4].

Under a given set of conditions, the reliability of support designed by classification methods may be assessed by comparing them with those derived by other applicable methods and also with the actual support installed. Such an assessment can be carried out efficiently during excavation of an underground opening by close observation and monitoring of the intersected ground conditions.

This paper presents an assessment of the support derived by RMR and Q for a tunnel driven through a jointed sedimentary rock formation. The two methods were applied during construction of the tunnel by project site personnel [5]. The support measures derived by the two methods were subsequently assessed by limit equilibrium analyses of rock block stability and numerical simulation of the rock mass behaviour around the tunnel. Further, the support measures derived by the classification methods were compared with the support installed.

## 2. THE CASE TUNNEL

The tunnel considered in this study is part of the Lam Ta Khong pumped storage project situated some 200 km northeast of Bangkok, Thailand. It is the main access route to the underground power station constructed between 1992 and 1997. The D-shape tunnel has a span of 6.8 m and a length of 1390 m. Its alignment is approximately 107$^{o}$ with a downward slope of 1:12 (V:H) towards the power station. The tunnel overburden varies from about 15 m at the entrance portal to approximately 350 m at the powerhouse end. The tunnel was driven by conventional drill and blast methods and instrumented for convergence monitoring [5].

The project consisting of several major tunnels was designed by the Electric Power Development Co Ltd (EPDC) of Japan for the Electricity Generating Authority of Thailand [5]. Throughout the project, standard tunnel support systems comprising rock bolt, shotcrete, welded wire mesh and steel sets were used and their performances were monitored by convergence monitoring at regular intervals.

The geological conditions and construction details of the project, as well as the results of monitoring of tunnel convergence and support performance, were reported by several researchers [6, 7, 8, 9, 10, 11, 12]. The details of the rock mass conditions intersected and the support measures installed in the case tunnel were presented by Sriwisead [5]. For this study only 885 m of the tunnel (from 180 m to 1065 m) was considered.

### 2.1. Ground Conditions Along the Tunnel

According to Sriwisead [5], from portal to 180 m, the tunnel was driven through talus material and highly weathered siltstone. Slightly weathered siltstone intersected in the tunnel interval from 180 to 220 m. From 220 m to the powerhouse end of the tunnel a fresh rock sequence consisting of sandy siltstone and sandstone is present.

Sandstone contains 2 to 20 mm thick micaceous seams and occasional 5 to 30 mm peat or lignite bearing layers. Sandstone beds are 200 to 1000 mm thick. Sandy siltstone is an inter-bedded rock consisting of 100 to 1500 mm thick beds of siltstone with occasional beds of sandstone.

At various stages of the project, intact rock material testing was conducted by the project owners and their consultants and the results were reported by Sriwisead [5] and Praphal [9]. A summary of the relevant results is presented in Table 1.

Table 1 Intact rock material properties

| Property | Sandstone | Siltstone |
|---|---|---|
| UCS (MPa) | 20-100 | 20-80 |
| E Modulus (GPa) | 25-33 | 20-22 |
| Poisons Ratio | 0.29 | 0.16 |
| Density (kN/m$^3$) | 25.5 | 25.9 |
| Tensile strength (MPa) | 7.6 | 9.3 |

By stereographic projection of discontinuity (joint) orientations measured in the tunnel, Sriwisead [5] identified five discontinuity sets in the rock mass. In any selected interval of the tunnel, typically two to three sets are present with others at random. The discontinuity orientations vary along the tunnel. The average orientations of the five sets are presented in Table 2.

The flat dipping bedding plane set (Set 1) with a dip range of 3 to 10$^o$ is the most prominent in the entire length of the tunnel. Bedding is well developed with a consistent orientation. Bedding planes are defined by thin alternating seams of mica and mud with smooth planar to slickensided planar surface characteristics [5].

Table 2 Average orientations of discontinuity sets

| Set # | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| Dip | 7 | 83 | 82 | 29 | 32 |
| Direction | 285 | 207 | 298 | 320 | 231 |

Sets 2 and 3 are major joint sets consistently present throughout the 885 m length of the tunnel. Their spacing varies from 500 to 2000 mm. They are continuous across the tunnel and are mostly tight and undulating to planar. In siltstone, they are sometimes slickensided with a 2 to 8 mm thick calcite infilling, and their surfaces are slightly weathered to fresh. Joint Sets 4 and 5 developed only in siltstone, and are mostly tight, slickensided and planar with calcite or gypsum infill material [5].

Set 1 has a significant effect on the stability of the tunnel as it forms rock slabs, particularly in the crown [5]. The sub-vertical joints (Sets 2 and 3) in combination with Set 1 were responsible for flat roofs, stepped over-breaks and block falls [6].

The in situ stress levels at relevant depths were determined by hydraulic fracturing tests conducted in boreholes. The tests indicated that the major principal stress is horizontal ($\sigma_H$) and normal to the tunnel axis [5, 11]. The intermediate principal stress is vertical ($\sigma_v$) and is slightly greater than the minor principal stress ($\sigma_h$). The $\sigma_H$:$\sigma_v$ ratio, k, is interpreted to be in the vicinity of 2. The tunnel was mostly dry, except for localized wet areas with water dripping along sub-vertical discontinuities, mostly in sandstone [5].

### 2.2. Rock Mass Behaviour During Excavation

Geberally, the rock mass behaviour was favorable for tunneling and excavation progressed steadily without major delays. However, some rock mass instability occurred during excavation. This was governed by the geological structure, in situ stress conditions, intact rock strength, and to a lesser extent by blast damage [5].

The main mode of instability reported during excavation of the tunnel was rock block and wedge failure due to the presence of discontinuities, particularly sub-horizontal bedding planes and near vertical joints with slickensided surfaces [5, 6]. Stress induced rock slabbing also occurred on side walls located between Sta. 570 and 880 m and formed in siltstone where shotcreting was initially limited only to the tunnel crown [5]. Minor slaking and swelling were also observed in siltstone when in contact with water.

2.3. The Support Measures Installed

For this project, tunnel support measures were designed using a rock mass classification system initially developed for dam engineering works by Tanaka [13] and Kikuchi et al [14] of the Central Research Institute of Electric Power Industry (CRIEPI) of Japan. EPDC modified the CRIEPI system and used a simplified version for underground excavations in sedimentary rocks in Northeast Thailand, and the modified system was found to give reasonable results in underground excavation design [15]. The system uses three rock mass parameters: weathering, hardness (expressed in terms of UCS) and joint spacing. During excavation of the case tunnel, RMR and Q were also applied for crosschecking and comparison [5].

The support measures installed in the 885 m tunnel length driven through rock were 2 m long rock bolts plus shotcrete with or without mesh reinforcement. The support quantities were based on the results of the application of the CRIEPI system to the rock mass intersected along the tunnel.

From Sta. 180 to 230 m driven in slightly weathered siltstone, the bolt spacing used was 1.2 m and the mesh reinforced (MRF) shotcrete thickness was 150 mm. From Sta. 230 to 280 m in fresh siltstone, the bolt spacing was 1.5 m and the MRF shotcrete thickness was 100 mm. From Sta. 280 to 540 m in good quality sandstone, relatively less support quantities were used. Notable in this area was the absence of mesh reinforcement. In the interval between Sta. 370 to 417 m the shotcrete thickness was reduced to 70 mm and between Sta. 435 to 454 m, no rock bolts installed. Again between Sta. 945 to 1035 m in good quality sandstone, no mesh installed. In the remainder of the tunnel length studied the main rock type was siltstone, where rock bolts and MRF shotcrete were installed.

Apart from the above mentioned variations, the applied shotcrete thickness was 100 mm. Typically the number of bolts per section was 6 (mainly in the crown), but varied between 4 and 8. The bolt spacing varied from 1.2 to 2.5 m with a typical spacing range of 1.5 to 2 m. Shotcrete was applied to the entire 885 m length of the tunnel and 60% was initially reinforced with welded wire mesh. The bolt spacing, shotcrete thickness and mesh reinforcement along the tunnel are graphically presented in Figure 1. Note that zero bolt spacing in Figure 1 means no bolts were installed.

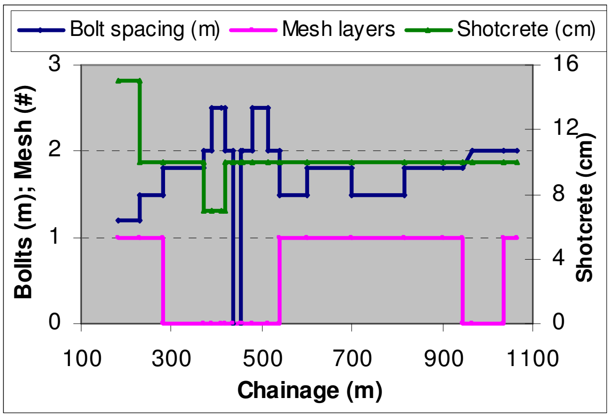


Figure 1 Support installed from 180 to 1065 m

2.4. Performance of the Installed Support

The support system used in the tunnel worked satisfactorily [9], except for some short intervals between Sta. 280 and 540 m where no mesh reinforcement was installed with shotcrete. In this area the shotcrete layer was damaged and additional support measures were installed to repair it. The damage was manifested by both longitudinal and transverse cracks in shotcrete, and was interpreted to be due to overloading of the support system. Notable shotcrete damage was observed in the following areas: Sta. 280 to 295 m, Sta. 340 to 374 m, Sta. 390 to 410 m, Sta. 462 to 469 m, Sta. 523 to 526 m and at Sta. 532 m [5].

The areas with significant shotcrete damage were re-spayed with an additional 100 mm layer of shotcrete. Depending on the severity of cracking, mesh reinforcement and additional rock bolts were also installed. The areas where additional support measures were installed are listed in Table 3.

Table 3 Areas where additional support installed

| Station (m) | Initial support | Additional support |
|---|---|---|
| 280-288 | B; S 70 mm of URF | S 100 mm URF |
| 288-291 | B; S 70 mm of URF | S 100 mm MRF |
| 291-295 | B; S 70 mm of URF | S 100 mm URF |
| 340-350 | B; S 100 mm URF | S 100 mm URF |
| 350-354 | B; S 100 mm URF | S 100 mm MRF |
| 354-374 | B; S 100 mm URF | S 100 mm URF |
| 390-416 | S 70 mm URF | B; S 100 mm URF |
| 462-469 | B; S 100 mm URF | S 100 mm MRF |
| 514-526 | SB; S 100 mm URF | B; S 100 mm MRF |

B=pattern bolts; SB=spot bolts; S=shotcrete; MRF=mesh reinforced; URF=un-reinforced

## 3. APPLICATION OF RMR AND Q

RMR and Q were developed and first introduced by Bieniawski [1] and Barton et al [2], respectively. Over the years these methods have been revised and updated, and their current versions, published in 1989 and 1994, are $RMR_{89}$ [16] and $Q_{94}$ [17].

### 3.1. Support Derivation by RMR and Q

During excavation of the case tunnel, $RMR_{89}$ and $Q_{74}$ were applied to each round of excavation within the 885 m length (from 180 to 1065 m) and the results were plotted on a tunnel map with a description of the observed conditions [5]. For this paper $RMR_{89}$ and $Q_{94}$ were used. The available data were reviewed and, where necessary, the ratings were downgraded to better reflect the poor rock conditions described in the tunnel map. The ranges of ratings assigned are presented in Table 4 and the final RMR and Q values along the tunnel are shown in Figure 2. The amount of rock mass falling into each relevant RMR and Q classes are shown in Figure 3.

Table 4 Ranges of RMR and Q ratings assigned

| RMR | | Q | |
|---|---|---|---|
| Parameter | Range | Parameter | Range |
| Strength | 2-7 | RQD | 60-98 |
| RQD | 13-20 | Jn | 6-12 |
| Joint spacing | 10-15 | Jr | 1-2 |
| Joint condition | 10-25 | Ja | 1-3 |
| Groundwater | 7-15 | Jw | 0.66-1 |
| Adjustment (-) | 5-10 | SRF | 1-5 |
| RMR value | 44-67 | Q value | 1.91-30 |

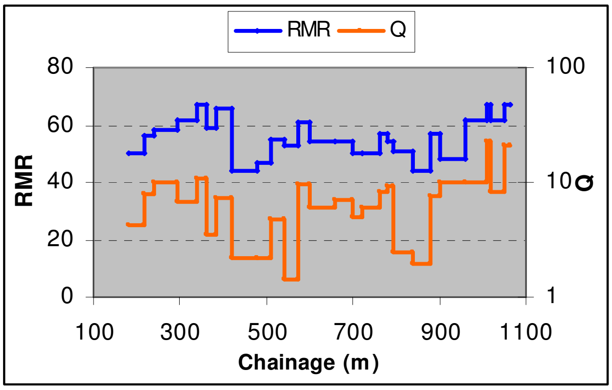


Figure 2 RMR and Q values along the tunnel

From Figure 3, it is evident that RMR classified 74% (655 m length) of the tunnel as fair rock and the remaining 26% (230 m) as good rock. Q classified 27% (238 m) as poor rock, 56% (497 m) as fair rock and the remaining 17% (150 m) as good rock. (Note that the descriptive terms poor, fair and good are used in both methods, but they do not necessarily mean identical rock mass conditions.)

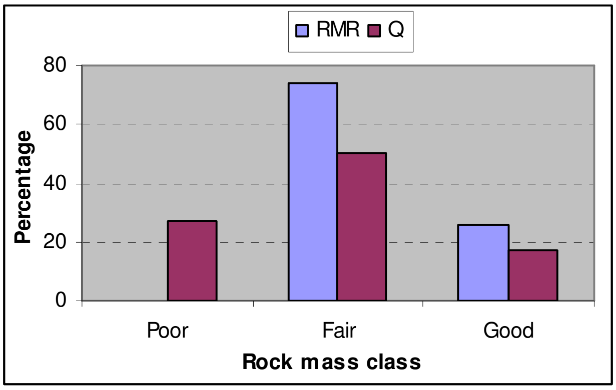


Figure 3 Amount of rock in each RMR and Q class

For each relevant class of rock along the tunnel, support measures were derived using $RMR_{89}$ and $Q_{94}$ versions. Table 5 presents the RMR and Q recommended support for the different rock mass classes in the tunnel. Since RMR recommendations given in the literature are for 10 m span tunnels only, the bolt lengths (L) were adjusted using the empirical formula: L=1.40+0.184a, where "a" is tunnel span. With Q, an Excavation Support Ratio (ESR) of 1.2 (for access tunnels) was used. Hence the equivalent dimension $D_e$=5.66.

Table 5 RMR and Q recommended support

| *RMR Class* | Good | Fair | Poor |
|---|---|---|---|
| Bolts (m) | L=2 S=2.5 | L=2 S=1.5-2 | NA |
| Shotcrete (mm) | 50 (mr) | 50-100 (mr) | NA |
| *Q Class* | Good | Fair | Poor |
| Bolts (m) | None | L=2.5 S=1.6-2 | L=2.5 S=1.7-2.1 |
| Shotcrete (mm) | None | None | 40-100* |

L=length, S=spacing, mr=mesh reinforced, NA=not applicable; *=fiber reinforce 10% of the tunnel where Q<2

## 4. ASSESSMENT OF RMR AND Q DERIVED SUPPORT MEASURES

To assess the adequacy of the RMR and Q derived support for stabilizing the potential rock instability in the tunnel, the following approaches were used:

- Suspended beam analysis,
- Tetrahedral wedge analysis,
- Numerical simulation, and
- Comparisons with the installed support.

### 4.1. Suspended Beam Analysis

The presence of sub-horizontal bedding planes and near vertical joints parallel to and normal to the tunnel axis creates a structural setup for rock beams to be formed in the roof. The support required for stabilizing the potentially unstable rock beams was determined by a conservative analysis using the suspended beam concept. The analysis assumed that the weight of the rock in the unstable zone is supported entirely by the force developed in the rock bolts anchored in the overlying solid rock. Ignoring the effect of shear and flexural strengths of the strata and the in situ stress field around the tunnel, the required bolt spacing was determined by

$$S = (T/\gamma\ D\ F)^{1/2} \qquad (1)$$

Where S = rock bolt spacing on both longitudinal and transverse directions, T = ultimate load capacity per rock bolt, D = thickness of unstable rock beam, $\gamma$ = unit weight of the rock, and F = required factor of safety against failure for long term roof stability, which is assumed to be 2.0 for this study.

As mentioned in Section 2.1, sandstone and sandy siltstone beds are up to 1 m and 1.5 m in thickness, respectively. Their respective unit weights are 25.5 and 25.9 $kN/m^3$. If 100% bond efficiency cement grouted bolts with an ultimate tensile strength of 180 kN were to be installed normal to the rock face, the required bolt spacing for 1.5 m thick sandy siltstone beds is 1.5 m and that for 1 m thick sandstone beds is 1.8 m.

It can be seen from Table 5 that the bolt spacing recommended by RMR for fair rock class and that by Q for fair and poor rock classes are comparable to the range of bolt spacing (1.5 to 1.8 m for the maximum bed thicknesses) determined by the beam analysis. For good rock RMR recommended a bolt spacing of 2.5 m. Since RMR also recommended 50 mm of mesh reinforced shotcrete, the combined support system is likely to be sufficient to control beam instability. In the case of Q, although beam instability is possible in good rock class, no support was recommended.

### 4.2. Tetrahedral Rock Wedge Stability Analysis

A tetrahedral rock wedge stability analysis was undertaken using UNWEDGE software code [18].

To compute wedge factors of safety (F) the shear strength parameters of the discontinuities were estimated considering their surface characteristics described earlier. The selected parameters were c=20kPa and $\Phi=30^o$. Two stress scenarios were considered. First the analysis assumed that the wedges are subjected to gravity loading only with no effect from the in situ stress field. It then included an in situ stress field assumed to be due to the weight of the overlying rock with k=2.

The analysis showed that several rock wedges are kinematically possible and most of them are stable under the joint shear strength and in situ stress conditions considered. The details of three roof wedges that have no significant stabilizing effect from the in situ stress field are listed in Table 4. The wedge factors of safety without and with the effect of the stress field are given as F1 and F2, respectively, when the overburden thickness is about 100 m.

Table 6 Kinematically possible rock wedges in roof

| Sets | Apex (m) | Weight (kN) | F1 | F2 |
|---|---|---|---|---|
| 123 | 1.1 | 73 | 0.63 | 0.80 |
| 234 | 1.3 | 53 | 0.98 | 1.09 |
| 235 | 0.6 | 13 | 1.32 | 0.83 |

The stability of these wedges was then examined under the support recommended by the two classification methods. The analysis showed that, RMR derived support for the entire 885 m length of the tunnel and Q derived support for poor and fair classes of rock are sufficient for stabilizing the possible rock wedges. However, Q recommended no support for good rock class, which represents approximately 17% of the tunnel length studied, where wedge instability was possible.

It should be noted that the analysis considered only the maximum size rock wedges that can be formed. The actual wedges may be smaller and may fall between the rock bolts installed in a standard pattern. To address this possibility, spot bolting and/or shotcreting may be required. However, the Q recommendation for fair rock is pattern bolting only and that for good rock class is no support. These two classes represent 73% of the tunnel.

### 4.3. Numerical Modeling

For jointed rock formations with Q values between 0.1 and 100, UDEC [19] is considered suitable for two dimensional simulations of the behaviour of rock mass around an underground opening [20]. In the case tunnel, the Q values ranged between 1.9 and 30, hence UDEC (Version 4.0) could be used for verifying the adequacy of the support derived by the two classification methods.

Four sections of the tunnel (Figure 4) resembling the conditions at Stations 410, 529, 670 and 830 m, reported by Sriwisead [5], were modeled. The section details are presented in Table 7. The simulation assumed k=2, the intact rock blocks are elastically deformable and the joints follow the Coulomb slip area contact failure model. Relevant intact rock material properties were selected from the data presented in Table 1. As in the case of wedge analysis the joint shear strength parameters were estimated to reflect their observed surface characteristics described earlier.

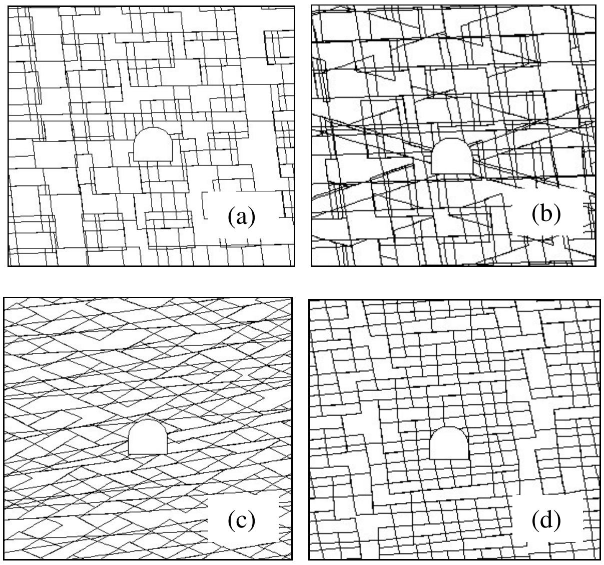


Figure 4 Discontinuity arrangements in modeled sections: (a) Sta. 410, (b) Sta. 529, (c) Sta. 670, and (d) Sta. 830

The following cases were modeled for each section:

- Case 1: unsupported tunnel.
- Case 2: with bolts and URF shotcrete support.
- Case 3: with bolts and MRF shotcrete support.

Table 7 UDEC model section details

| Station (m) | Depth (m) | Rock type | No. of joint sets |
|---|---|---|---|
| 410 | 80 | Sandstone | 2 |
| 529 | 110 | Sandstone | 3+ random |
| 670 | 160 | Siltstone | 3 |
| 830 | 210 | Siltstone | 2 |

The URF and MRF shotcrete were modeled using the shotcrete parameters listed in Table 8. The values listed for MRF shotcrete in Table 8 were assumed to represent the lower bound effect of wire mesh in shotcrete. The bolts included in the model were the cement grouted type as used in the limit equilibrium analyses discussed earlier.

Case 1 showed that rock block instability is possible both in the roof and walls, particularly when kinematically feasible blocks are present at the tunnel periphery (Figure 5). This is consistent with the observed rock mass behaviour in the tunnel. Further, Case 1 showed that tensile failure is also possible in tunnel walls when the in situ stress levels are relatively high (i.e. when the depth of tunnel is > 100 m). This is in agreement with the stress induced rock slabbing observed in the walls between Sta. 570 and 880 m.

Table 8 Shotcrete parameters

| Property | URF | MRF |
|---|---|---|
| Compressive strength *MPa* | 30 | 60 |
| Tensile strength *MPa* | 3 | 6 |
| Adhesive strength *MPa* | 0.5 | 0.5 |
| Elastic modulus *GPa* | 30 | 35 |

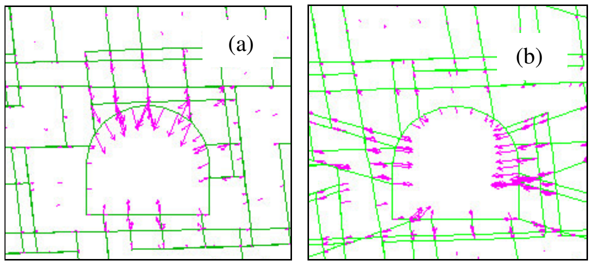


Figure 6 (a) Roof instability Sta. 410 m, (b) Wall instability Sta. 529 m

Case 2 indicated compressive failure and axial yielding of URF shotcrete under the conditions simulated in all four sections. As mentioned in Section 2.4 some damage occurred in the URF shotcrete layer installed from Sta. 280 to 540 m. Interestingly, the predicted failure zones in the tunnel periphery (Figure 6a) compare well with the actual locations of shotcrete damage reported by Sriwisead [5]. Case 3 showed that the extent of predicted shotcrete damage can be reduced significantly by using MRF shotcrete (Figure 6b). The simulation also showed that a marginal increase in the MRF shotcrete strength parameters above those listed in Table 8 would be sufficient to eliminate the predicted failure zone. Since no damage was reported in the areas supported with MRF shotcrete, it may be deduced that URF shotcrete is not the best option for the tunnel, although Q recommended URF shotcrete with rock bolts for poor class of rock, which represent 27% of the tunnel and no shotcrete for the remaining 73%.

### 4.4. Comparison with the Installed Support

The spacing of rock bolts installed along the tunnel and the RMR and Q recommended bolt spacing are shown in Figure 7 (zero spacing means no bolts). In general, the recommended and the actual bolt spacing may be considered comparable, except for some areas where RMR and Q recommended bolt spacing are greater than the bolt spacing used.

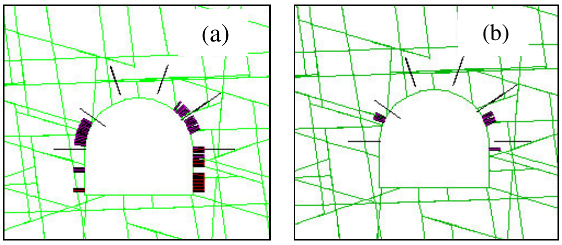


Figure 6 (a) Damage in URF shotcrete (b) Damage reduced in MRF shotcrete

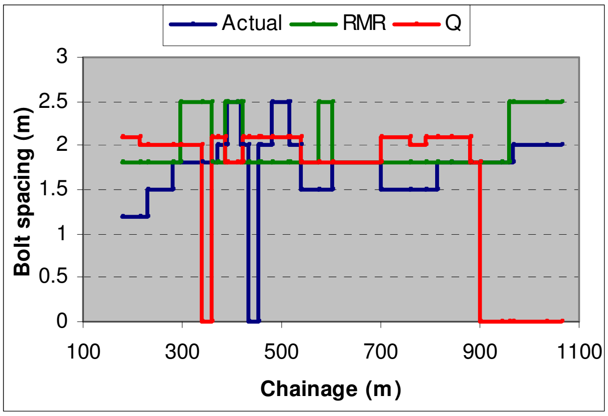


Figure 7 Actual and recommended bolt spacing

The applied shotcrete thickness was mostly 100 mm except for two areas with 150 mm and 70 mm thickness (Figure 8). As can be seen from Figure 8, the thickness of RMR recommended shotcrete layer was less than the actual shotcrete thickness. And in the case of Q shotcrete was recommended for only 27% of the tunnel.

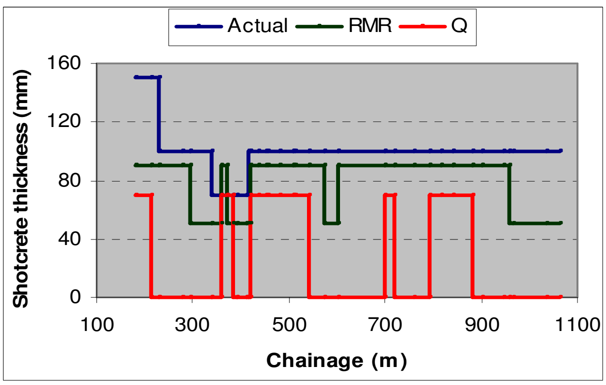


Figure 8 Actual and recommended shotcrete thickness

Figure 9 shows the extent of recommended and installed mesh/fiber reinforcement with shotcrete. RMR recommended mesh reinforcement for 77% of the tunnel, Q recommended fiber reinforcement for 10% of the tunnel (only when Q<2) and 60% of the tunnel was initially supported with mesh. As discussed in Section 2.4 the reported shotcrete damage was limited to the tunnel interval from 280 to 532 m, where no mesh reinforcement was used initially, but additional support was installed subsequently. This may mean that the RMR recommendation for mesh is more in line with the actual requirement for this tunnel. It should be noted that the RMR recommendations given in the literature are for 10 m diameter tunnels, and in this study these recommendations are assumed to be applicable to the 6.8 m diameter tunnel.

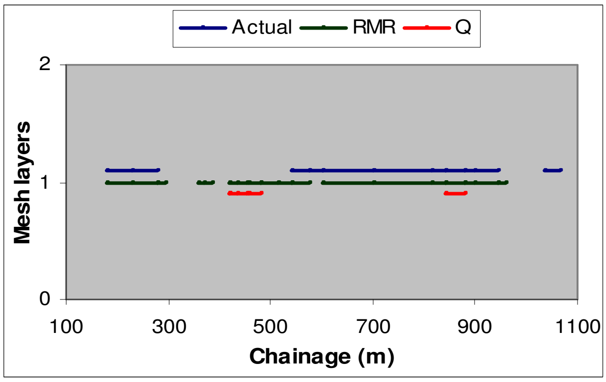


Figure 9 Actual and recommended mesh/fiber reinforcement

## 5. CONCLUSIONS

The tunnel support derived by RMR and Q rock mass classification methods were assessed by beam and wedge stability analyses, numerical simulation of the rock mass around the tunnel and by comparing them with the support installed.

The beam analysis, which ignored the in situ stress field, showed that RMR and Q derived support measures are adequate for stabilizing the potentially unstable rock beams in the tunnel.

The RMR derived support measures for the entire 885 m length of the tunnel and Q derived support measures for the poor rock class adequate for the potentially unstable tetrahedral rock wedges. Q recommended only pattern bolting for fair rock and no support for good rock. The pattern bolting is adequate for stabilizing the largest possible tetrahedral rock wedges in the tunnel. However, in fair class (and good class) of rock, shotcrete was warranted as there was the potential for small rock block instability in between the installed rock bolts.

The numerical simulation showed that the RMR derived support measures meet the numerically predicted support requirements. The simulation indicated that instead of the URF shotcrete recommended by Q, MRF (or fiber reinforced) shotcrete is a better option to reduce the risk of damage to shotcrete.

The study showed that the RMR recommended support types are in agreement with the support installed, which consisted of rock bolts, shotcrete and mesh reinforcement. There were some differences in the RMR recommended and installed bolt spacing and shotcrete thickness. But in general, they were comparable. Q recommended rock bolts for 83% of the tunnel, shotcrete for 27% and fiber reinforcement for only about 10%. Although the Q derived bolt pattern may be considered comparable to the installed bolt pattern, the Q recommended shotcrete and fiber/mesh reinforcement fall well short of the extent of shotcrete and mesh installed in the tunnel.

## ACKNOWLEDGEMENTS

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# Evaluation of Empirically Derived Support for an Access Tunnel: A Case Study

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**ABSTRACT**

The support measures determined by two empirical rock mass classification methods, Rock Mass Rating (RMR) and Tunnelling Quality Index (Q), for an access tunnel driven through jointed sedimentary rocks were compared with the actual support installed and the results of a numerical simulation undertaken using UDEC software package. The study indicated that the support measures recommended by the RMR method are in general agreement with the support installed, albeit some differences in the bolt spacing and shotcrete thickness used. In contrast, the Q recommended support measures, particularly shotcrete and fibre/mesh reinforcement, fall well short of the extent of shotcrete and mesh installed in the tunnel. The support performance monitoring and the results of numerical simulation showed that the installed support measures were required for the rock mass conditions present in the tunnel.

## 1 INTRODUCTION

This paper presents an evaluation of the support derived using two empirical methods, Rock Mass Rating (RMR) and Tunnelling Quality Index (Q), for an access tunnel to an underground power station driven through jointed sedimentary rocks. The RMR and Q methods used in this study were developed by Bieniawski (1973) and Barton et al. (1974), respectively, and were subsequently revised to enhance the reliability of their support predictions. Their current versions are $RMR_{89}$ (Bieniawski, 1989) and $Q_{94}$ (Barton & Grimstad, 1994). Despite the revisions, these methods have limitations some of which are discussed by Palmstrom & Broch (2006), Pells & Bertuzzi (2008) and Ranasooriya & Nikraz (2007, 2008).

A practical approach to identify the limitations of the empirical design methods and suggest improvements, where necessary and possible, is to compare their support predictions with those derived by other applicable methods and also with the performance of the support installed. The paper compares the support derived by the two methods with those installed in the tunnel, and evaluates their adequacy by two dimensional numerical simulation of the rock mass behaviour around the tunnel.

## 2 PROJECT BACKGROUND

The case tunnel considered is part of the Lam Ta Khong pumped storage project situated some 200 km northeast of Bangkok, Thailand. The major components of the project include an underground power station and several kilometers of tunnels and shafts. The D-shaped 6.8 m wide and 1390 m long case tunnel driven by drill and blast methods is the main access route to the underground power station. The tunnel overburden varies from 15 m at the entrance portal to about 350 m at the powerhouse end. The general tunnel alignment is $107^{o}$.

Throughout the project, standard support systems comprising rock bolt, shotcrete, wire mesh and steel sets were used and their performance was monitored. The geological conditions and construction details of the project, as well as the results of convergence and support performance monitoring, were reported by Jinye (1993), Sirikaew (1993), Praphal (1993), Tran (1994), Sriwisead (1996), Nitaramorn (1997), Gurung and Iwao (1998) and Phienwej (1999). The details of the rock mass conditions intersected and the support measures installed in the tunnel were presented by Sriwisead (1996). In this study the tunnel from 180 to 1065 m (885 m) was considered.

*2.1 Ground conditions along the tunnel*

The first 180 m of the tunnel encountered talus material and highly weathered rock and was excluded from this study. From 180 to 220 m a slightly weathered (SW) siltstone was present. A fresh rock sequence consisting of sandy siltstone and sandstone was intersected from 220 m to the far end of the tunnel. Sandstone beds are 200 to 1000 mm thick. Sandy siltstone is an inter-bedded rock consisting of 100 to 1500 mm thick beds of siltstone with occasional beds of sandstone. A summary of the results of relevant intact rock material testing reported by Sriwisead (1996) and Praphal (1993) is presented in Table 1.

Table 1 Intact rock material properties

| Property | Sandstone | Siltstone |
|---|---|---|
| UCS (MPa) | 20-100 | 20-80 |
| E Modulus (GPa) | 25-33 | 20-22 |
| Density (kN/m$^3$) | 25.5 | 25.9 |
| Tensile strength (MPa) | 7.6 | 9.3 |

Five discontinuity sets with the average orientations presented in Table 2 were identified in the rock mass. In any selected interval of the tunnel, typically two to three sets are present with others occurring at random. The flat dipping bedding plane set (Set 1) with slickensided to smooth planar surfaces is the most prominent. Joint Sets 2 and 3 with 500 to 2000 mm spacing are major sets present throughout the tunnel. They are continuous across the tunnel and are mostly tight and undulating to planar. In siltstone they are sometimes slickensided with up to 8 mm thick calcite infilling and their surfaces are slightly weathered to fresh. Joint Sets 4 and 5 developed only in siltstone are mostly tight, slickensided and planar with calcite or gypsum infill material (Sriwisead 1996). The sub-vertical joints (Sets 2 and 3) in combination with Set 1 were responsible for flat roofs, stepped over-breaks and block falls (Gurung and Iwao 1998).

Table 2 Average orientations of discontinuity sets

| Set # | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| Dip | 7 | 83 | 82 | 29 | 32 |
| Direction | 285 | 207 | 298 | 320 | 231 |

The major principal stress at relevant depths is horizontal ($\sigma_H$) and normal to the tunnel axis (Sriwisead 1996; Nitaramorn 1997). The intermediate principal stress is vertical ($\sigma_v$) and is slightly greater than the minor principal stress ($\sigma_h$). The $\sigma_H$:$\sigma_v$ ratio, k, is interpreted to be in the vicinity of 2. The tunnel was mostly dry, except for localized wet areas with water dripping along sub-vertical joints, mostly in sandstone (Sriwisead 1996).

*2.2 Rock mass behaviour during excavation*

The rock mass behaviour was favourable for tunnelling and excavation progressed steadily without major delays. However, some rock mass instability occurred during excavation, mainly in the form of rock block and wedge failure controlled by discontinuities, particularly bedding planes and near vertical joints (Sriwisead 1996; Gurung and Iwao 1998). Stress induced rock slabbing also occurred on side walls located between Sta. 570 and 880 m and formed in siltstone where shotcreting was initially limited only to the tunnel crown (Sriwisead 1996). Minor slaking and swelling were also observed in siltstone when in contact with water.

*2.3 The installed support*

The support measures installed in the 885 m tunnel length were 2 m long rock bolts plus shotcrete with or without mesh reinforcement. From Sta. 180 to 230 m driven mostly in SW siltstone, bolt spacing was 1.2 m and mesh reinforced (MRF) shotcrete thickness was 150 mm. From Sta. 230 to 280 m in fresh siltstone, bolt spacing was 1.5 m and MRF shotcrete thickness was 100 mm. From Sta. 280 to 540 m in good quality sandstone, relatively less support quantities were used. Notable in this area was the absence of mesh. In the interval from Sta. 370 to 417 m, shotcrete thickness was reduced to 70 mm and between Sta. 435 and 454 m, no rock bolts were installed. Again from Sta. 945 to 1035 m in good quality sandstone, no mesh was installed. In the last 30 m of the tunnel length studied, the main rock type was siltstone, where rock bolts and MRF shotcrete were installed.

Apart from the above mentioned variations, the applied shotcrete thickness was 100 mm. Typically the number of bolts per section was 6 (mainly in the crown), but varied between 4 and 8. The bolt spacing varied from 1.2 to 2.5 m with a typical spacing range of 1.5 to 2 m. Shotcrete was applied to the entire 885 m length and 60% of that was initially MRF.

*2.4 Performance of the installed support*

The support used in the tunnel worked satisfactorily (Sriwisead 1996), except for some short intervals between Sta. 280 and 540 m where shotcrete was not MRF. In this area the shotcrete layer was damaged and additional support measures were installed to repair it. The damage was manifested by both longitudinal and transverse cracks and was interpreted to be due to overloading of the support system. Notable damage was observed in the following areas: Sta. 280 to 295 m, Sta. 340 to 374 m, Sta. 390 to 410 m, Sta. 462 to 469 m, Sta. 523 to 526 m and at Sta. 532 m (Sriwisead 1996). The areas of significant damage were re-spayed with an additional 100 mm layer of shotcrete. Depending on the severity of damage, mesh reinforcement and additional rock bolts were also installed (Table 3).

## 3 APPLICATION OF RMR AND Q

During excavation of the case tunnel, $RMR_{89}$ and $Q_{74}$ were applied to each excavation round within the 885 m length (from 180 to 1065 m) and the results were plotted on a tunnel map with a description of the observed conditions (Sriwisead 1996). For this paper, $RMR_{89}$ and

$Q_{94}$ were used. The available data were reviewed and, where deemed appropriate, the ratings were downgraded to better reflect the poor rock conditions described in the tunnel map. The ranges of ratings assigned are presented in Table 4 and the spatial variation of the RMR and Q values along the tunnel are shown in Figure 1. A correlation of RMR versus Q values is presented in Figure 2. It can be seen from Figures 1 and 2 that the application of the two methods to the case tunnels was reasonably consistent.

Table 3 Areas where additional support installed

| Station (m) | Initial support | Additional support |
|---|---|---|
| 280-288 | B; S 70 mm of URF | S 100 mm URF |
| 288-291 | B; S 70 mm of URF | S 100 mm MRF |
| 291-295 | B; S 70 mm of URF | S 100 mm URF |
| 340-350 | B; S 100 mm URF | S 100 mm URF |
| 350-354 | B; S 100 mm URF | S 100 mm MRF |
| 354-374 | B; S 100 mm URF | S 100 mm URF |
| 390-416 | S 70 mm URF | B; S 100 mm URF |
| 462-469 | B; S 100 mm URF | S 100 mm MRF |
| 514-526 | SB; S 100 mm URF | B; S 100 mm MRF |

B=pattern bolts; SB=spot bolts; S=shotcrete; MRF=mesh reinforced; URF=un-reinforced

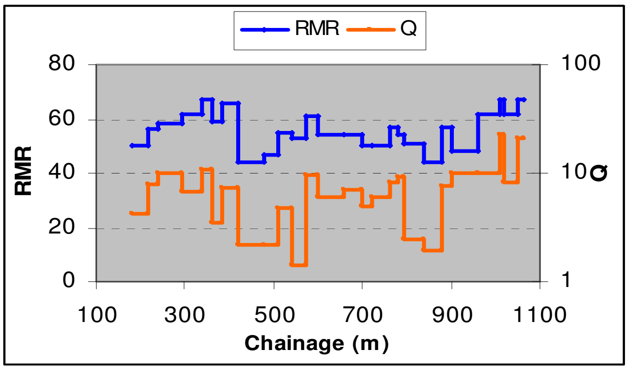


Figure 1 RMR and Q values along the tunnel

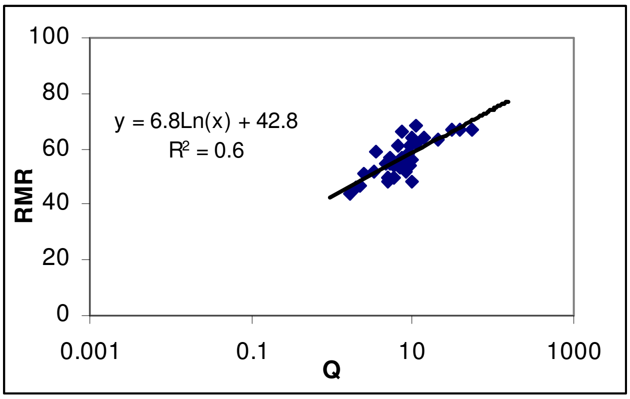


Figure 2 RMR versus Q values

Out of the total length of 885 m, RMR classified 74% (cumulative length of 655 m) as fair rock and the remaining 26% (230 m) as good rock. Q classified 27% (238 m) as poor rock, 56% (497 m) as fair rock and the remaining 17% (150 m) as good rock.

For each relevant class of rock along the tunnel, support measures were derived using $RMR_{89}$ and $Q_{94}$. Table 5 presents the support recommended for different classes of rock. Since RMR recommendations are for 10 m span tunnels only, the bolt lengths (L) were adjusted using the empirical formula: L=1.40+0.184a, where "a" is tunnel span. With Q, an Excavation Support Ratio (ESR) of 1.2 (for access tunnels) was used.

Table 4 Ranges of RMR and Q ratings assigned

| RMR | | Q | |
|---|---|---|---|
| Parameter | Range | Parameter | Range |
| Strength | 2-7 | RQD | 60-98 |
| RQD | 13-20 | Jn | 6-12 |
| Joint spacing | 10-15 | Jr | 1-2 |
| Joint condition | 10-25 | Ja | 1-3 |
| Groundwater | 7-15 | Jw | 0.66-1 |
| Adjustment (-) | 5-10 | SRF | 1-5 |
| RMR value | 44-67 | Q value | 1.91-30 |

L=length, S=spacing, mr=mesh reinforced, NA=not applicable; *=fibre reinforce 10% of the tunnel where Q<2

Table 5 RMR and Q recommended support

| *RMR Class* | Good | Fair | Poor |
|---|---|---|---|
| Bolts (m) | L=2 S=2.5 | L=2 S=1.5-2 | NA |
| Shotcrete (mm) | 50 (mr) | 50-100 (mr) | NA |
| *Q Class* | Good | Fair | Poor |
| Bolts (m) | None | L=2.5<br>S=1.6-2 | L=2.5<br>S=1.7-2.1 |
| Shotcrete (mm) | None | None | 40-100* |

L=length, S=spacing, mr=mesh reinforced, NA=not applicable; *=fiber reinforce 10% of the tunnel where Q<2

## 4 NUMERICAL MODELING

For jointed rocks with Q values between 0.1 and 100, UDEC (Itasca Consulting Group 2004) is considered suitable for two dimensional simulations of rock mass behaviour around underground openings (Barton 1996). In the case tunnel, the Q values ranged between 1.9 and 30, hence UDEC (Version 4.0) could be used for evaluating the support derived by the empirical methods.

Four sections of the tunnel (Figure 3) resembling the conditions at Stations 410, 529, 670 and 830 m, reported by Sriwisead (1996), were modelled. The section details are presented in Table 6.

Table 6 UDEC model section details

| Station (m) | Depth (m) | Rock type | No. of joint sets |
|---|---|---|---|
| 410 | 80 | Sandstone | 2 |
| 529 | 110 | Sandstone | 3+ random |
| 670 | 160 | Siltstone | 3 |
| 830 | 210 | Siltstone | 2 |

The simulation assumed k=2, the intact rock blocks are elastically deformable and the joints follow the Coulomb slip area contact failure model. Relevant intact rock material properties were selected from the data presented in Table 1. The joint surface strength parameters were estimated to reflect their observed conditions. Three cases were modelled for each section:

Case 1 unsupported tunnel; Case 2 with bolts and URF shotcrete; Case 3 with bolts and MRF shotcrete.

The URF and MRF shotcrete were modelled using the parameters listed in Table 7. The values listed for MRF shotcrete were assumed to represent the lower bound effect of wire mesh in shotcrete. The bolts included in the model were the 180 kN ultimate tensile strength full column cement grouted type.

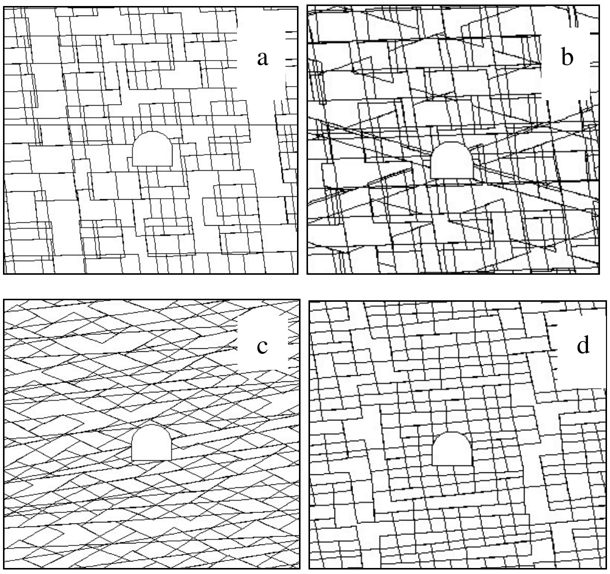


Figure 3 Discontinuity arrangements in modelled sections: (a) Sta. 410, (b) Sta. 529, (c) Sta. 670, and (d) Sta. 830

Case 1 showed that rock block instability is possible both in roof and walls when kinematically feasible blocks are present (Figure 4). This is consistent with the observed rock mass behaviour in the tunnel. Case 1 also showed that tensile failure is possible in tunnel walls when the in situ stress levels are relatively high (i.e. when the depth of tunnel is > 100 m). This is in agreement with the stress induced rock slabbing observed in the walls between Sta. 570 and 880 m.

Table 7 Shotcrete parameters

| Property | URF | MRF |
|---|---|---|
| Compressive strength *MPa* | 30 | 60 |
| Tensile strength *MPa* | 3 | 6 |
| Adhesive strength *MPa* | 0.5 | 0.5 |
| Elastic modulus *GPa* | 30 | 35 |

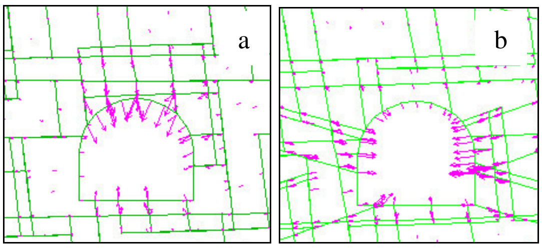


Figure 4 (a) Roof instability Sta.410, (b) Wall instability Sta.529

Case 2 indicated compressive failure and axial yielding of URF shotcrete under the conditions simulated in all four sections. As mentioned in Section 2.4 some damage occurred in the URF shotcrete layer installed from Sta. 280 to 540 m. Interestingly, the predicted failure zones in the tunnel periphery (Figure 5a) compare well with the actual locations of shotcrete damage reported by Sriwisead (1996). Case 3 showed that the extent of predicted shotcrete damage can be reduced significantly by using MRF shotcrete (Figure 5b). The simulation also showed that a marginal increase in the MRF shotcrete strength parameters above those listed in Table 8 would be sufficient to eliminate the predicted failure zone. Since no damage was reported in the areas supported with MRF shotcrete, it may be deduced that MRF shotcrete was the best option for the tunnel.

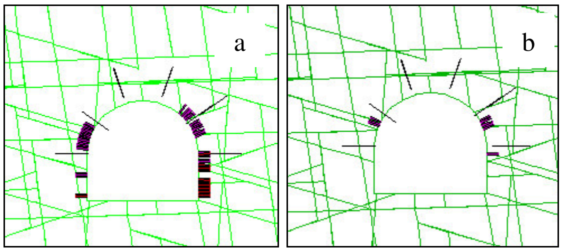


Figure 5 (a) Damage in URF shotcrete (b) Damage reduced in MRF shotcrete

## 5 COMPARISON WITH INSTALLED SUPPORT

The spacing of rock bolts installed along the tunnel and the RMR and Q recommended bolt spacing are shown in Figure 6 (zero spacing means no bolts). In general, the recommended and the actual bolt spacing may be considered comparable, except for some areas where RMR and Q recommended bolt spacing greater than that being used.

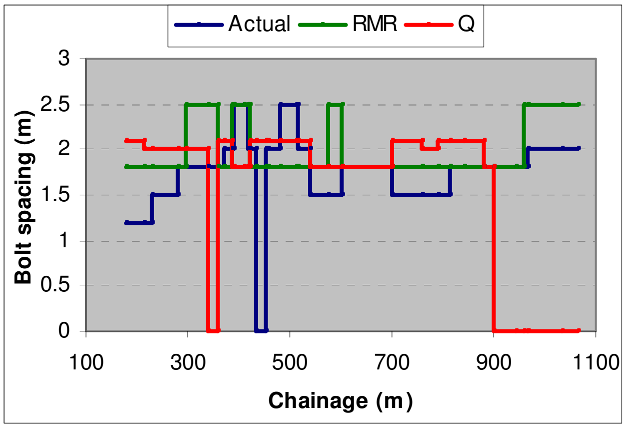


Figure 6 Actual and recommended bolt spacing

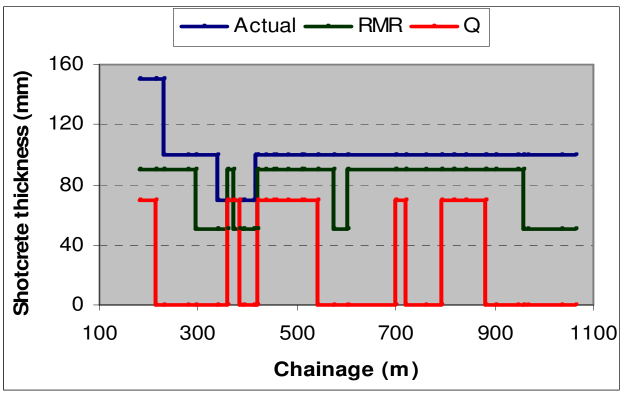


Figure 7 Actual and recommended shotcrete thickness

The applied shotcrete thickness was 100 mm except for two areas of 150 and 70 mm (Figure 7). The thickness of the RMR recommended shotcrete layer was less than the actual shotcrete thickness. In the case of Q, shotcrete was recommended for only 27% of the tunnel.

RMR recommended mesh reinforcement for 77% of the tunnel, Q recommended fibre reinforcement for 10% (only when Q<2) and 60% of the tunnel was initially supported with mesh. As discussed in Section 2.4, the shotcrete damage was limited to the tunnel interval from 280 to 532 m, where no mesh was used initially. Subsequently additional support was installed. This means that the RMR recommendation for mesh is more in line with the actual requirement for this tunnel. Note that the RMR recommendations given in the literature are for 10 m diameter tunnels, and in this study these recommendations, except for bolt lengths, were assumed to be applicable to the 6.8 m diameter tunnel.

## 6 CONCLUSIONS

The tunnel support derived by RMR and Q rock mass classification methods were assessed by numerical simulation of the rock mass around the tunnel and by comparing them with the support installed.

The simulation showed that RMR derived support measures meet the numerically predicted support requirements and that instead of the un-reinforced shotcrete recommended by Q, mesh reinforced (or fibre reinforced) shotcrete is a better option to reduce the risk of damage to shotcrete.

The study showed that the RMR recommended support types are in agreement with the support installed which consisted of rock bolts, shotcrete and mesh reinforcement. There were some differences in the RMR recommended and installed bolt spacing and shotcrete thicknesses, but in general, they were comparable. Q recommended rock bolts for 83% of the tunnel, shotcrete for 27% and fibre reinforcement for only about 10%. Although the Q derived bolt pattern may be considered comparable to the bolt pattern used, the Q recommended shotcrete and fibre/mesh reinforcement fall well short of the extent of shotcrete and mesh installed in the tunnel.

## ACKNOWLEDGEMENTS

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# Predicted and actual conditions of the Namroud project diversion tunnel

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ABSTRACT: The 5.5 m wide 740 m long Namroud project diversion tunnel located in the Tehran province of Iran was driven through sedimentary rocks comprising limestone, marlstone and limy shale. The initial design of primary support for the tunnel was based on the data collected from a limited program of site investigation and the application of rock mass classification approach. During excavation the rock mass conditions encountered were different to those predicted at the project design stage, and as a result the excavation methods and the tunnel support systems had to be changed. Despite the changes the tunnel was constructed on time and within budget. This paper presents (a) the predictions made at the design stage, (b) the results of the classification of rock mass intersected during construction, and (c) the actual excavation and the support methods used.

## 1 INTRODUCTION

The Namroud water resources development project is currently being built in Firouz Kouh, Tehran province, Iran. The project comprises a rock fill embankment dam with a clay core. The dam has a crest length of 652 m, a maximum height of 82 m and a crest width of 12 m. The project also comprises a 5.5 m diameter 740 m long horseshoe shaped diversion tunnel constructed for two purposes: (a) to temporarily divert the Namroud River to facilitate the construction of the dam; and (b) as a bottom outlet during operation of the project to provide drinking and irrigation water for downstream users. The tunnel located in the left abutment has an overburden of between 30 and 90 m and was driven through weak sedimentary rocks comprising limestone, marlstone and limy shale.

At the design stage of the project, based on the data collected by a program of site investigation mainly involving exploration drilling and logging, the primary tunnel support requirements and excavation methods were determined empirically by applying one of the most widely used rock mass classification methods, Tunnelling Quality Index (Q) of Barton et al. (1974) and Barton & Grimstad (1994). During excavation of the tunnel the rock mass conditions encountered were different to those predicted at the design stage, and as a result the excavation methods and the tunnel support systems had to be changed. Despite the changes made the tunnel was constructed on time and within budget. This paper presents (a) the predictions made at the design stage, (b) the results of the classification of rock mass encountered during construction, and (c) the actual excavation methods and the support systems used.

## 2 SITE GEOLOGY AND GEOTECHNICAL CONDITIONS

The regional geology of the project area is characterised by sedimentary rock formations that have been subjected to a series of folding and faulting. Several major geological structures are present in the general area of the project including the Seleh Bon syncline, the Nachoostan anticline, the Namroud and the Masha Fasham faults, and the Barijan, Alborz, Garmsar, Namroud and Firoozkooh thrusts. The site is located on the southern limb of the Seleh Bon syncline.

To the downstream of the dam axis is a region of several sedimentary rock units which have been subjected to different tectonic events. Along the dam axis and to the upstream is a group of soft sedimentary rocks of Karaj Formation. The tectonic activities in the region caused several splay faults and shears and as a result some of the rock units along the tunnel alignment are shattered and sheared. In general the project area is overlain by recent sediments and most of the dam foundation is located on alluvial deposits. The Namroud riverbed is composed of an alluvium deposit and therefore the dam included a foundation cut-off wall to reduce the potential for water loss through the foundation.

The main rock types along the tunnel alignment comprise limestone, marlstone and limy shale, most of which are tectonically disturbed. A summary log of rock types intersected in the tunnel is given in Table 1.

Several geological discontinuity sets are present along the tunnel alignment. These include bedding planes, joints and some shears. The vast majority of the discontinuities are filled with calcite infill material while some are either clean or coated with oxide material. The discontinuity orientation, spacing and surface characteristics vary along the tunnel. The general orientation and spacing of bedding and joint sets in tuff marl are given in Table 2.

Of particular concern was tuff marl rocks of Karaj formation which are highly shattered and weak due to tectonic activity. In the tunnel length from chainage 80 m to 120 m, the weakness of tuff marl was further exasperated by the presence of a minor shear zone in this area.

As per the ISRM suggested methods, the intact rock materials intersected in the tunnel can be described as weak with a typical UCS range of 5 to 11 MPa. The tunnel is located below the groundwater table and was wet during excavation.

Table 1. Summary log of rock types in the tunnel.

| Chainage (m) | Rock type |
|---|---|
| 000 – 145 | Tuff marl |
| 145 – 244 | Limy shale/shaly limestone |
| 244 – 304 | Tuff marl |
| 304 – 504 | Tuffmarl/shaly limestone/limy shale |
| 504 – 585 | Marly limestone |
| 585 – 670 | Sand marlstone |
| 670 – 693 | Limestone |
| 693 – 728 | Sandy marlstone |
| 728 – 740 | Limestone |

## 3 EXCAVATION METHODS AND PRIMARY SUPPORT MEASURES USED

In the original design the proposed excavation method was drilling and blasting and the proposed primary support measures comprised a 1 m x 1 m pattern of rock bolts, 100 mm of mesh reinforced shotcrete and lattice girders made of three 25 mm diameter steel bars. In accordance with this design the entire tunnel was to be pattern bolted, part of it tunnel was to be shotcreted with wire mesh reinforcement, and lattice girders installed for almost half its length. As this design was based on the data collected from a limited program of site investigation involving exploration drilling, surface mapping and rock sample testing, it needed revision and updating based on more detailed information available at the construction stage.

Table 2. Orientation of discontinuity sets.

| | | Spacing (% in each range) | | | |
|---|---|---|---|---|---|
| Set. | Dip/direc | >2.0m | 0.6-2.0m | 0.2-0.6m | 0.06-0.2m |
| B1 | 70/310 | 38 | 17 | 27 | 18 |
| J1 | 83/211 | 9 | 18 | 55 | 18 |
| J2 | 73/269 | 10 | 38 | 38 | 14 |
| J3 | 60/070 | 37 | 40 | 12 | 11 |

Subsequent to the commencement of construction it was observed that the proposed drill and blast excavation method was not the best option for some of the rocks intersected because blasting, even when well controlled, caused unnecessary rock mass damage and instability in the tunnel. After considering the available options, jackhammer and drum-cutter techniques were used for the excavation of approximately 340 m of the 740 m long tunnel. These techniques reduced the rock mass damage.

At a very early stage of excavation, in light of the additional information collected from direct observation of the rock mass, the original support design was reviewed and it was found that the proposed support, rock bolting in particular, was not appropriate especially for areas where rock mass was weaker than expected.  It was therefore decided to rely on surface support, i.e. mesh reinforced shotcrete and light steel ribs etc. A comparison of the proposed excavation and support methods with the actual methods used is presented in Figure 1.

As can be seen from Figure 1, rock bolting was not used in this tunnel. Mesh reinforced shotcrete was the most common support system over a total tunnel length of 406 m. In the weaker rock zones, a total length of approximately 200 m, light steel ribs and shotcrete with or without mesh, locally made steel shield and concrete, mass concrete and pre-bolting (forepoling) followed by mesh reinforced shotcrete were used. The steel ribs were connected by welding 25 mm steel bars parallel to the tunnel axis and shotcreted with or without mesh. Despite the fact that the initial design required pattern bolting of the entire tunnel, approximately 100 m of the tunnel from the downstream portal was unsupported.

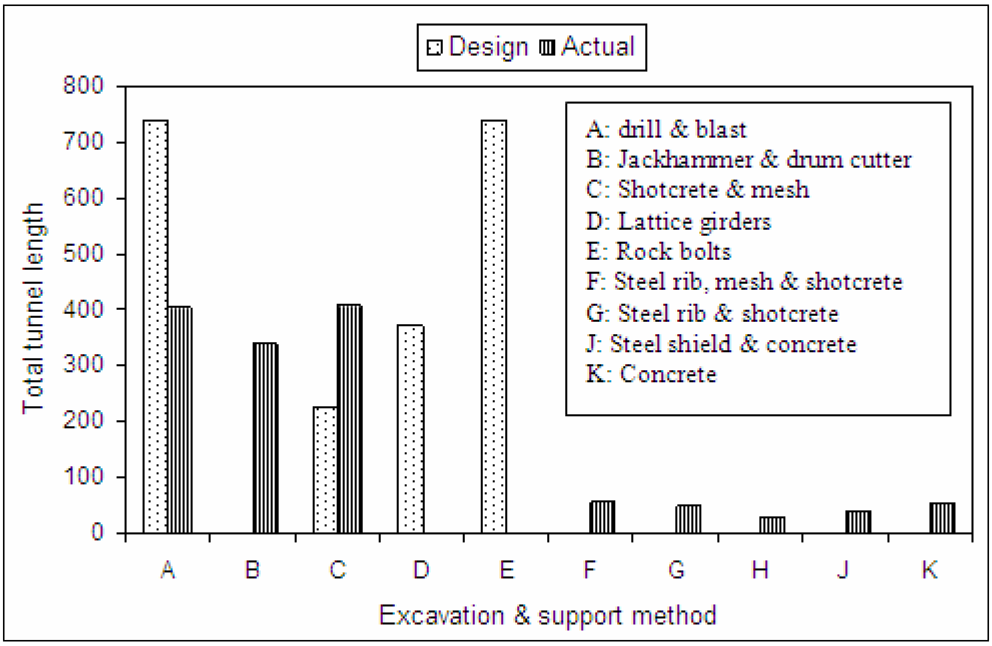


Figure 1. Comparison of the initially proposed excavation and support methods with those actual used.

Of particular interest was an approximately 30 m long zone of very weak rock supported using an on-site assembled steel shield made of a steel sheet and a frame. Here the tunnel advance was reduced to about 0.75 m per round and a shield was installed immediately after the excavation and the space between the shield and the tunnel periphery was back filled with concrete. Though this support system significantly reduced the safety risks arising from tunnel collapse during support installation, it was found to be less efficient in terms of rate of advance and the time required for concrete curing. In the remainder of this weak zone pre-bolting was used to pre-stabilise the rock mass. For this purpose 3 m long 25 mm diameter rebar bolts were installed in the crown in the direction of tunnel advance. The bolts were spaced at 300 mm and were inclined $10^{o}$ to the horizontal. The excavation round was increased to 1.5 m and after the excavation 150 to 200 mm of mesh reinforced shotcrete was applied. This system worked satisfactorily and the construction completed within budget and time schedule. Other weak zones were supported with light steel sets, mesh and shotcrete as required.

The tunnel was fully concrete lined mainly as it is to be converted to a bottom outlet after the dam construction. The primary support measures were, therefore, kept to the required minimum.

## 4 APPLICATION OF THE CLASSIFICATION METHODS DURING CONSTRUCTION

During the construction of the tunnel the Rock Mass Rating (RMR) system of Bieniawski (1989) was applied to the entire length by detailed mapping of the exposed rock mass conditions. Based on the rock type and its condition, the tunnel was divided into 11 geotechnical sectors (domains) so that different conditions in each sector could be accounted for in classifying the rock mass according to the RMR system. For comparison purposes, the Q index of Barton & Grimstad (1994) was also applied by indirect means using the *RMR-Q* linear correlation (Equation 1) proposed by Bieniawski (1976, 1989, 1993).

$$RMR = 9 \ln Q + 44 \tag{1}$$

The above equation was used because, due to time constraints, ratings for the Q input parameters were not determined during excavation. It is acknowledged that the correlation given by Equation 1 may not necessarily be applicable to the rock mass conditions in the tunnel. Further, as noted by Palmstrom (2009), this correlation is a very crude approximation involving an inaccuracy of ±50% or more. Nevertheless, for the present study it was assumed that the equation would be accurate enough for the purpose of comparing the support measures predicted by the two classification systems. Table 3 presents a summary of the RMR and Q values and the relevant rock mass classes representing the 11 tunnel sectors.

As expected, the RMR system classified the majority (66%) of the rock mass into the very poor class (the lowest in the RMR rating scale) and the remainder (33%) into the poor class. When the RMR values were transformed into Q values using Equation 1, the corresponding Q rock mass classes were extremely poor (66%) and very poor (33%). In general the rating values of any given RMR class would not directly transform into a single rock mass class of the Q system because the latter has nine classes against the five in the former. However, due to the relatively narrow range of RMR values obtained for this tunnel each class falls into a single Q class when transformed by Equation 1. It should be noted that RMR classified the rock mass as very poor and poor, the two lowest classes in this system, and Q classified it as extremely poor and very poor, the second and third lowest classes of the Q system. It would, therefore, be expected that heavier support would be required for the tunnel.

Using the RMR and Q values in Table 3, for comparison with those installed in the tunnel, support requirements were determined according to the two classification systems. Although the RMR support measures recommended by Bieniawski (1989, 1993) are for 10 m wide tunnels it was assumed that they are applicable to the 5.5 m wide Namroud diversion tunnel except for bolt lengths which needed adjustment to match its width. This may be justified because the previous RMR versions (Bieniawski 1974, 1975) recommended the same support measures for 5 to 12 m wide tunnels. With the Q system an excavation support ratio (*ESR*) of 1.8 (for water tunnels) was used as suggested by Barton & Grimstad (1994). This gives a *De* value of 3.0 for the 5.5 m diameter tunnel. The relevant Q support categories therefore are 5 and 6 for very poor and extremely poor classes of rock respectively. The relevant RMR and Q support measures are presented in Table 4. (Note that these are for permanent support.)

As can be seen from Table 4 both RMR and Q recommended rock bolts and shotcrete with mesh or fibre reinforcement. The RMR system also recommended steel ribs for the very poor rock class and also for the poor class, if required. Additionally, it also recommended forepoling (pre-bolting) for the very poor class, if required. The steel sets and forepoling recommendations comply with some of the support methods used in the weaker rock zones.

Table 3. Summary of RMR and Q values.

| Tunnel interval (m) | RMR value | RMR class | Q value | Q class |
|---|---|---|---|---|
| 000 – 060 | 17 | Very poor | 0.05 | Extremely poor |
| 060 – 145 | 18 | Very poor | 0.06 | Extremely poor |
| 145 – 244 | 20 | Very poor | 0.07 | Extremely poor |
| 244 – 304 | 24 | Poor | 0.11 | Very poor |
| 304 – 504 | 18 | Very poor | 0.06 | Extremely poor |
| 504 – 540 | 26 | Poor | 0.14 | Very poor |
| 540 – 585 | 17 | Very poor | 0.05 | Extremely poor |
| 585 – 670 | 35 | Poor | 0.37 | Very poor |
| 670 – 693 | 37 | Poor | 0.46 | Very poor |
| 693 – 728 | 40 | Poor | 0.64 | Very poor |
| 728 – 740 | 33 | Poor | 0.29 | Very poor |

Table 4. RMR and Q derived support measures.

| RMR class | RMR derived support | Q class | Q derived support |
|---|---|---|---|
| Poor (33%) | Bolts at 1-1.5 m spacing with mesh. Shotcrete 100-150 mm in crown & 100 mm in walls. Light ribs spaced at 1.5 m where required | Very poor (33%) | Bolts at 1.3-1.5 m spacing. Fibre reinforced shotcrete 50-90 mm. |
| Very poor (66%) | Bolts at 1-1.5 m spacing with mesh. Shotcrete 150-200 mm in crown, 150 mm in walls & 50 mm on face. Medium to heavy ribs spaced at 0.75 m with steel lagging and forepoling if required, close invert | Extremely poor (66%) | Bolts at 1.2-1.3 m spacing. Fibre reinforced shotcrete 90-120 mm. |

As can be seen from Figure 1, rock bolting was not used as in the original design. Mesh reinforced shotcrete was used for a total tunnel length of 406 m. Weaker rock zones in an approximate total length of 200 m were supported with light steel ribs and shotcrete with or without mesh, steel shield and concrete, mass concrete and pre-bolting. As stated earlier, although some of the primary support measures used in the weaker zones are comparable to those recommended by the RMR method, they significantly differ from those recommended by the Q system. For the 100 m unsupported length of the tunnel (from ~ Ch. 640 to 740 m), the RMR values ranged from 33 to 40 and the corresponding Q values ranged from 0.29 to 0.64 (Table 3). According to the two methods this length required pattern bolts plus mesh or fibre reinforced shotcrete.

Since the RMR and Q recommendations are for permanent support, direct comparison with the primary support could be open to conjecture. Nevertheless, in this instance heavier primary support measures were required for 200 m of the tunnel than those recommended by the two methods.

## 5 CONCLUSIONS

The rock mass conditions encountered during construction of the 740 m long Namroud diversion tunnel were different to those predicted based on the data collected for project design. Due to the weakness of some of the sedimentary rocks intersected in the tunnel, the conventional drilling and blasting excavation method was found to be problematic for part of the tunnel as blasting caused unnecessary rock mass damage. Jackhammer and drum cutter techniques were adopted for just over 300 m thus significantly reducing unnecessary damage to the rock mass.

During the early stages of excavation, it was observed that instead of the initially proposed primary support system mainly consisting of pattern bolting, other support systems could lead to better safety performance in weaker rock zones. Mesh reinforced shotcrete was the most common support system used with a total tunnel length of 406 m. In 200 m of weaker rock zones light steel ribs and shotcrete with or without mesh, steel shield and concrete, mass concrete and pre-bolting plus mesh reinforced shotcrete were used. Approximately 100 m of the tunnel was unsupported despite the fact that the initial design required pattern bolting of the entire tunnel.

The rock mass conditions exposed in the tunnel were mapped and the RMR system was applied directly to the as-excavated rock mass conditions. The Q system was also applied indirectly by converting the RMR values by means of a published *RMR-Q* correlation. Both methods recommended rock bolting and shotcrete with mesh or fibre reinforcement for permanent support. While mesh reinforced shotcrete was used, rock bolts were not. In weaker rock zones the RMR derived support generally agreed with the installed heavier support. The Q derived support measures differ except for the mesh (or fibre) reinforced shotcrete, which is only part of the heavier support installed in the weaker rock zones. The primary support measures installed in approximately 200 m of the tunnel were heavier than those recommended by Q for permanent support.

For the 100 m unsupported tunnel length both RMR and Q recommended rock bolts and mesh or fibre reinforced shotcrete.

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# RELIABILITY OF THE LINEAR CORRELATION OF ROCK MASS RATING (RMR) AND TUNNELLING QUALITY INDEX (Q)

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## ABSTRACT

With the advent of the RMR and Q classification methods for underground excavation support design, a linear correlation between the two methods was suggested by linear regression analysis of the data obtained from several case studies. The data used in deriving the relationship was widely scattered and the range of values covered by the 90% confidence limits demonstrated that the relationship had very little practical value. In subsequent publications, the 90% confidence limits were omitted when referring to the relationship. Consequently, some practitioners in the field of rock engineering assumed that this relationship, expressed as a semi logarithmic equation, is universally applicable for transforming the ratings assigned by one system to the ratings of the other. This assumption is erroneous and deserves scrutiny. This paper reviews some of the relevant published information and illustrates that there is no sound scientific basis to assume a universally applicable linear relationship between the two.

## 1 INTRODUCTION

Of the several rock mass classification methods developed for underground excavation support design applications, only RMR and Q, introduced by Bieniawski (1973) and Barton et al. (1974), respectively, have stood out. Over the years, these methods have been revised and updated, and the current versions are $RMR_{89}$ (Bieniawski, 1989) and $Q_{94}$ (Barton and Grimstad, 1994). Their main applications are in the prediction of support requirements, stable unsupported spans and stand-up times of underground excavations, particularly during the planning stage of projects. Others include estimations of modulus of rock mass deformation and rock mass strength, which are input parameters for elegant design tools such as numerical modelling.

Both RMR and Q methods are based on six parameters considered to represent the behaviour of rock masses. The primary aim of these classification systems is to divide the rock mass into distinct classes of similar characteristics that are easily identified by visual observation or by simple tests. Since both methods aim at the same objective, a correlation may be expected between the two. If a true correlation exists, then it should be possible to obtain ratings for one system by transforming the ratings determined for the other. This would save time and effort if both systems are to be applied for the design of an excavation project.

A correlation based on linear regression analysis of *RMR* and *Q* values was first presented by Bieniawski (1976). Since then several other researchers have also presented somewhat different correlations based on regression analysis of *RMR* and *Q* values obtained from tunnelling and mining projects in different parts of the world. While these correlations may be valid for the rock mass conditions from which they were derived, they may not necessarily be applicable to other rock mass conditions. Despite the fact that different correlations can be obtained from different rock mass conditions, there is a tendency among some practitioners of rock engineering to overly rely on the first correlation published in 1976 and transform ratings between the two systems. In recent years this injudicious tendency has found its way into the underground mining sector in Western Australia where RMR and Q methods are often used for excavation support design.

This paper presents a brief overview of the evolution of the RMR and Q methods and their existing correlations, and a discussion on the differences of the two. In light of the existing correlations and the scattering of the data used in deriving them, the paper illustrates that there is no sound scientific basis to assume a universally applicable linear relationship between RMR and Q, as alluded to by some publications.

## 2 THE RMR SYSTEM

The RMR system evolved through several versions (Bieniawski, 1973, 1974, 1975, 1976, 1979 and 1989). It is an index of rock mass competency based on six parameters:

- Intact rock strength (*IRS*)
- Rock quality designation (*RQD*)
- Joint (discontinuity) spacing (*JS*)
- Joint surface condition (*JC*)
- Groundwater condition (*GW*)
- Rating adjustment (*RA*) for discontinuity orientation

Each of the six classification parameters is given five separate ranges of rating values. Guidelines on the selection of ratings based on the observed or measured conditions in a rock mass are provided in the system. The sum of the ratings assigned to the six parameters is defined as the *RMR* value, which linearly varies from 0 to 100.

From 1973 to 1989 the ratings scales and some of the parameters used in the RMR system have changed as listed in Table 1. In the 1973 version, eight parameters were used and from 1974 onwards these were reduced to six by combining joint separation, continuity and weathering parameters of the first version to create the joint condition parameter, *JC*. From 1974 to 1975 the maximum ratings given to *JC* and *IRS* were increased by 10 and 5 points, respectively.

In the 1973 and 1974 versions, the *RA* was given a positive rating ranging from 0 for the most unfavourable orientation to 15 for the most favourable orientation. From 1975 onwards this parameter was given a negative rating from 0 for the most favourable orientation to -12 for the most unfavourable orientation. From 1975 to 1976 the rating scales were not changed, but the rock mass class boundaries for support selection were. In the 1979 version, the maximum rating for the *JS* term was reduced by 10 points and the influence of both *JC* and *GW* was increased by 5 rating points each. In the 1989 version ($RMR_{89}$), the rating ranges did not change, but the assessment of sub-horizontal discontinuities (joints) was changed from unfavourable to fair for the stability of tunnels. This results in a difference of 5 rating points in the RMR value.

The *RMR* value of a given rock mass is related to five rock mass classes and each class in turn is related to permanent support measures and construction procedures presented in a tabulated form for 10 m wide horseshoe shaped tunnels with a vertical stress of less than 25 MPa excavated by drill and blast methods. The method also provides an unsupported span versus stand-up time chart, which may be used to estimate the stand-up time and the maximum stable unsupported span for a given *RMR* value.

| Parameter | 1973 | 1974 | 1975 | 1976 | 1979 | 1989 |
|---|---|---|---|---|---|---|
| Intact rock strength (*IRS*) | 0 - 10 | 0 - 10 | 0 - 15 | 0 - 15 | 0 - 15 | 0 - 15 |
| *RQD* | 3 - 16 | 3 - 20 | 3 - 20 | 3 - 20 | 3 - 20 | 3 - 20 |
| Joint spacing (*JS*) | 5 - 30 | 5 - 30 | 5 - 30 | 5 - 30 | 5 - 20 | 5 - 20 |
| Separation of joints | 1 - 5 | | | | | |
| Continuity of joints | 0 - 5 | | | | | |
| Weathering | 1 - 9 | | | | | |
| Condition of joints (*JC*) | - | 0 - 15 | 0 - 25 | 0 - 25 | 0 - 30 | 0 - 30 |
| Groundwater (*GW*) | 2 - 10 | 2 - 10 | 0 - 10 | 0 - 10 | 0 -15 | 0 - 15 |
| Rating adjustment (*RA*) | 3 - 15 | 3 - 15 | 0 - (-12) | 0 - (-12) | 0 - (-12) | 0 - (-12) |

Table 1 Rating allocations in different versions of the RMR system

## 3 THE Q SYSTEM

The Q system, developed by Barton and co-workers (Barton et al., 1974, 1975, 1977, 1980 and Barton, 1976), also uses six parameters considered to represent the behaviour of rock masses:

- Rock quality designation (*RQD*)
- Joint (discontinuity) set number (*Jn*)
- Joint roughness number (*Jr*)
- Joint alteration number (*Ja*)
- Water reduction factor (*Jw*)
- Stress reduction factor (*SRF*)

In the Q system the *RQD* is used as determined by core logging or scanline mapping, without allocating a system specific rating. *RQD* intervals of 5 are considered to be accurate enough and, if *RQD* is $\leq 10$, a nominal value of 10 is used. The recommended rating values for the other five parameters and guidelines for their selection are provided in the system. Once the numerical ratings are assigned to the six parameters, the *Q* value is calculated using the equation:

$$Q = (RQD/Jn)(Jr/Ja)(Jw/SRF) \qquad (1)$$

The *Q* value is related to support requirements through an "equivalent dimension", *De*, which is defined as:

$$De = (Span,\ diameter\ or\ height)/ESR \qquad (2)$$

where *ESR*, excavation support ratio, is a dimensionless function of the purpose of the opening. A list of recommended *ESR* values is provided in the system. The Q system provides a support chart with a *Q* value as its abscissa and *De* as its ordinate. By plotting the *Q-De* pair on the chart, the support requirements for excavations can be determined.

For nearly 20 years the system remained unchanged from its original version proposed in 1974 which consisted of 38 support categories plus a no support "zone". In 1993, the system was revised and updated (Grimstad and Barton, 1993; Barton and Grimstad, 1994) to incorporate the experience and technological advances subsequent to its initial introduction. In the updated version, the original classification parameters have not changed and their rating ranges also remain largely unchanged, except for changes in the *SRF* term to accommodate rock slabbing and bursting. The 1993 version also provided a revised support chart and reduced the number of support categories to nine. The revised chart has simplified the support selection process and is more user-friendly compared to the earlier version.

## 4 THE EXISTING CORRELATIONS BETWEEN RMR AND Q

The first correlation between the two methods (Equation 3) proposed by Bieniawski (1976) was based on a linear regression analysis of 111 sets of *RMR* and *Q* values from Scandinavian, South African, North American, European and Australian case histories.

$$RMR = 9\ ln\ Q + 44 \qquad (3)$$

By adding Indian case histories compiled by Jethwa et al. (1982), Bieniawski (1989) supplemented the database used for Equation 3. When the *RMR-Q* relationship given by this equation was first published, Bieniawski (1976) provided the 90% confidence limits (Equation 3a) which would contain 90% of the data used.

$$RMR = 9\ ln\ Q + 44 \pm 18 \qquad (3a)$$

The range of values represented by the 90% confidence limits given by Equation 3a covers almost two RMR ground classes, and as a result Equation 3 was of little practical value. In subsequent publications (Bieniawski, 1979, 1989, 1993; Barton, 1995; Barton and Bieniawski, 2008) the 90% confidence limits were omitted when referring to Equation 3. Consequently, some practitioners of rock engineering assumed that this equation is universally applicable for transforming the ratings of one system to that of the other. This assumption appears to be flawed for two reasons. Firstly, the data used in deriving the equation are widely scattered. Secondly, subsequent to its establishment, several different correlations between *RMR* and *Q* were derived by others as given in Table 2 and the data used by them are also scattered.

## 4.1 DATA SCATTERING AND RELIABILITY

Obviously, wide scattering of the data used for the first correlation (Equation 3) can be seen from Figure 1, reproduced after Bieniawski (1989), which plots the data used in 1976 and the data from Jethwa et al. (1982). According to the data in Figure 1, when the $Q$ value is 1.1 (poor rock), the corresponding $RMR$ value can range from < 20 (very poor rock) to > 61 (good rock), while Equation 3 transforms it to a $RMR$ value of 45 (fair rock). Further, when $Q < 0.008$ and $Q > 500$ Equation 3 returns RMR values which are outside the range defined in the system. In other words, if $Q < 0.008$, $RMR < 0$ and if $Q > 500$, $RMR > 100$. Palmstrom (2009) noted that this correlation is a very crude approximation, involving an inaccuracy of ±50% or more.

| Correlation | Source | Equation No. |
|---|---|---|
| $RMR = 9\ ln\ Q + 44$ | Bieniawski (1976) | 3 |
| $RMR = 5.9\ ln\ Q + 43 = 13.5\ log\ Q + 43$ | Rutledge and Preston (1978) | 4 |
| $RMR = 5\ ln\ Q + 60.8$ (from in situ data) | Cameron-Clarke & Budavari (1981) | 5 |
| $RMR = 4.6\ ln\ Q + 55.5$ (from bore core data) | Cameron-Clarke & Budavari (1981) | 6 |
| $RMR = 5.4\ ln\ Q + 55.2 = 12.5\ log\ Q + 55.2$ | Moreno Tallon (1982) | 7 |
| $RMR = 10.5\ ln\ Q + 41.8$ | Abad et al. (1983) | 8 |
| $RMR = 7.5\ ln\ Q + 42$ | Baczynski (1983) | 9 |
| $RMR = 5.3\ ln\ Q + 50.81 = 12.11\ log\ Q + 50.81$ | Udd and Wang (1985) | 10 |
| $RMR = 6.3\ ln\ Q + 41.6$ | Kaiser et al. (1986) | 11 |
| $RMR = 8.7\ ln\ Q + 38 \pm 18$ (probability theory)[a] | Kaiser et al. (1986) | 12 |
| $RMR = 6.8\ ln\ Q + 42$[b] | Sheorey (1993) | 13 |
| $RMR = 43.89 - 9.19\ ln\ Q$ | Celada Thamames (1983) | 14 |
| $RMR = 10\ ln\ Q + 39$ | Choquet & Charette (1988) | 15 |
| $RMR = 10.3\ ln\ Q + 49.3$ (when $Q \leq 1$, $SRF = 1$)[c] | Rawlings et al. (1995) | 16 |
| $RMR = 6.2\ ln\ Q + 49.2$ (when $Q > 1$, $SRF = 1$)[c] | Rawlings et al. (1995) | 17 |
| $RMR = 6.6\ ln\ Q + 53$ (when $Q \leq 0.65$)[c] | Rawlings et al. (1995) | 18 |
| $RMR = 5.7\ ln\ Q + 54.1$ (when $Q > 0.65$)[c] | Rawlings et al. (1995) | 19 |
| $RMR = 7\ ln\ Q + 36$ | Tugrul (1998) | 20 |
| $RMR = 4.2\ ln\ Q + 50.6$ | Asgari (2001) | 21 |
| $RMR = 5.97\ ln\ Q + 49.5$ | Sunwoo & Hwang (2001) | 22 |
| $RMR = 4.7\ ln\ Q + 56.8$ | Kumar et al. (2004) | 23 |
| $RMR = 8.3\ ln\ Q + 42.5$ (with $SRF = 1$) | Kumar et al. (2004) | 24 |
| $RMR = 6.4\ ln\ Q + 49.6$ (with revised $SRF$ values) | Kumar et al. (2004) | 25 |
| $RMR = 3.7\ ln\ Q + 53.1$ | Sari & Pasamehmetoglu (2004) | 26 |

Table 2 Correlations between $RMR$ and $Q$

[a] assuming $RMR$ and $ln\ Q$ are normal variates and satisfy the central limit theory of probability; [b] derived from the data presented by Sheorey (1993); [c] from bore core data

A review of the information available from the relevant publications shows scattering of the data used in deriving the other equations listed in Table 2

Rutledge and Preston (1978) derived Equation 4 in Table 2 using the data obtained from nine tunnel headings in New Zealand, noting that "*There is considerable scatter in the results*". On the two relationships (Equations 5 and 6) obtained using bore core data and in situ observations in South African tunnels, Cameron-Clarke and Budavari (1981) stated the following: "*The scatter of points about the regression lines is greater for the in situ values than for the bore core values. In both cases, however, it is probably too great to indicate any meaningful correlation between the two classification systems.*" The linear relationship (Equation 7) presented by Moreno Tallon (1982) used rock mass data from four tunnel headings in Spain. Although not specifically mentioned, scattering of the data is evident from the fact that four separate equations were obtained by separately analysing the data collected from the four headings. The four equations are similar but not identical to each other or to the equation derived by combining all the data from the four headings.

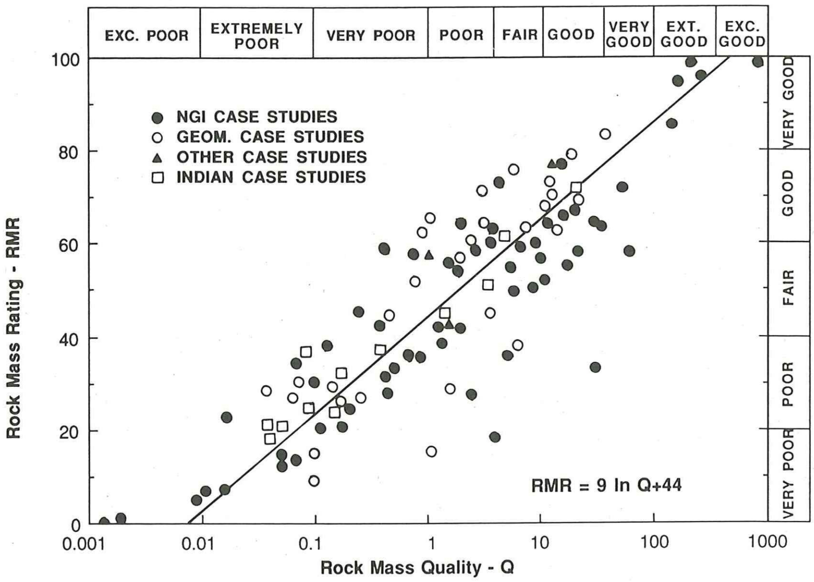


Figure 1 Correlation between *RMR* and *Q* (after Bieniawski, 1989)

Rawlings et al. (1995) analysed *RMR* and *Q* values assigned to bore core data from a geological formation comprising volcanic rocks. Two sets of Q values were considered: the first assumed *SRF*=1 and the second used the *SRF* values recommended in the Q system. By correlating the two sets of *Q* values with the relevant *RMR* values, Rawlings et al. obtained four separate relationships (Equations 16 to 19, inclusive), and suggested that the bilinear relationship given by Equations 16 and 17 for the un-factored (meaning *SRF*=1) *Q* values fitted well for the data used. Although Rawlings et al. (1995) did not provide the data used in the analysis, the apparent need to derive two different formulae from each of the two data sets indicates scattering of the data used.

Equation 22 in Table 2 was derived by Sunwoo and Hwang (2001) using approximately 300 data sets from widely different geological environments representing sedimentary, igneous and metamorphic rocks in Korea. The data used in this case are statistically significant in both numbers and range of rock mass conditions. Yet again, there is a wide scattering of the data.

From observations in major tunnelling projects in the Himalayas, India, Kumar et al. (2004) found that the *SRF* values provided in the Q system are not applicable to overstressed moderately jointed rocks that are subject to rock slabbing and bursting, and proposed a revised set of *SRF* values for those rock stress problems. The proposed *SRF* values range from 1.5 to 3.0, which are significantly smaller than the range of values (5 to 400) given by Barton and Grimstad (1994) for slabbing and bursting in competent rocks. Kumar et al. (2004) then presented three *RMR-Q* relationships. The first (Equation 23) assumed *SRF*=1, the second (Equation 24) used the revised *SRF* values, and the third (Equation 25) used the *SRF* values recommended in the Q system; all three relationships are different to Equation 3 proposed by Bieniawski (1976, 1989).

Wide scattering of the data used for deriving correlations can be seen clearly from the *RMR-Q* plots provided by Abad et al. (1983), Kaiser et al. (1986) and Sheorey (1993). The possibility of obtaining different correlations for different rock mass conditions and wide scattering of the data used for deriving them means that the linking of the two methods by a single formula and conversion of the ratings between them could lead to significant errors.

### 4.2 CHOICE OF THE INDEPENDENT VARIABLE AND METHOD OF ANALYSIS

The relationships listed in Table 2 are based on least square linear regression analysis of *RMR* and *Q* values with *Q* as the independent variable (abscissa of the *RMR*-*Q* plot as in Figure 1). Kaiser et al. (1986) pointed out that the correlations developed using linear regression analysis should be viewed with caution because the results depend on the choice of the dependent variable. From the linear regression analysis of the data collected from the Wolverine West Tunnel in Canada, they derived two relationships; the first (Equation 11) used *Q* as the independent variable, and the second (Equation 27) used *RMR* as the independent variable.

$$RMR = 6.3 \ln Q + 41.6 \qquad (11)$$

$$\ln Q = 0.087\, RMR - 2.28 \qquad (27)$$

Kaiser et al. demonstrated that despite the fact that the two relationships were derived using the same data set, they do not lead to the same result. For example, the first equation would predict an *RMR* value of 40 from a *Q* value of 0.8, while in turn the second equation would predict a *Q* value of 3.35 from a *RMR* value of 40. This clearly demonstrates the weakness of the conventional least square linear regression analysis. To overcome this weakness, Kaiser et al. (1986) used a probabilistic approach to determine a unique relationship, assuming that *RMR* and *ln Q* are normal variants and satisfy the central limit theorem of probability theory. Despite the use of a probabilistic approach, Kaiser et al. (1986) observed wide scattering of the data and therefore proposed two equations representing 90% confidence limits within which 90% of the data used for their study fall. However, Kaiser et al. noted that the range of values represented by the two equations is of little practical value as the range covers almost two RMR ground classes, as in the case of 90% confidence limits of Bieniawski (1976).

## 5 DIFFERENCES IN THE TWO RATING SYSTEMS

The presence of several different correlations and wide scattering of the data used in deriving them may be attributed to the fact that, the two methods have significantly different assessments of some of the rock mass parameters as discussed below:

- Intact rock strength (*IRS*) is a factor in the *SRF* term of the Q system, only if the excavation stability is affected by the in situ stress field. In contrast *IRS* is always included in the *RMR* value. If *IRS* changes while all the other parameters remain virtually the same, several *RMR* values are possible for a single *Q* value.
- In situ stress field is not accounted for in the RMR system in classifying a rock mass. In the Q system it is a factor in the *SRF* term if excavation instability is stress driven. Thus for a rock mass with a given *RMR* value, several different *Q* values are possible depending on the *SRF* value used. As pointed out by Baczynski (1983), the *RMR* versus *Q* correlations are stress-dependent. The relationship will be significantly altered if different *SRF* values are assumed in the determination of the *Q* rating.
- Joint spacing (*JS*) is a key parameter in the RMR system; the closer the *JS* the lower the *RMR* value and the wider the *JS* the higher the *RMR* value. This is not so in the Q system. As pointed out by Milne et al. (1998), if three or more joint sets are present and the joints are widely spaced, it is difficult to get the Q system to reflect the competent nature of a rock mass. For widely spaced jointing, *Jn* in the Q system appears to unduly reduce the resulting *Q* value (Milne et al., 1998). Thus for a single *Q* value several *RMR* values are possible depending on *JS*.
- *RQD* is used in both methods, and is a function of joint spacing *JS*, albeit it does not fully represent its true nature. In addition to *RQD*, as mentioned above, *JS* is also a key parameter in the RMR method. In the Q system, although the number of joint sets is taken into account, their spacing is not considered directly. This means the joint spacing is counted twice in the RMR method, while Q system uses it indirectly only once.
- In the RMR method joint orientation is accounted for directly through *RA* by allocating a rating between 0 and -12. In the Q system this is considered implicitly, but what is meant by adversely oriented discontinuities is not defined and the selection of the most critical discontinuity set is user dependent. In any case no rating is given to *RA* in the Q system. Thus for a given *Q* value different *RMR* values are possible depending on the orientation of the excavation relative to the discontinuity set orientation.
- Rating scale: The rating scales in the RMR method have been changed several times as shown in Table 1, while Q remained unchanged for nearly 20 years until 1993. For a rock mass with a given *Q* value, different *RMR* values can be obtained depending on the RMR version used. Since 1993, the *SRF* parameter of the Q system has been given a rating scale of 1 to 400 for competent rock with rock stress problems. As mentioned earlier, depending on the *SRF* value used, different *Q* values can be obtained for a rock mass with a given

*RMR* value. With the 1 to 400 range of *SRF* values, the difference in the *Q* value can be more than two orders of magnitude. By setting the *SRF* value to 1 in deriving the *Q* values, this problem may be overcome if the *SRF* term represents only stress. But, it is not strictly a stress factor. It also represents weakness zones, which are rock mass parameters.

From the foregoing it is clear that there is unlikely to be a universally applicable single formula for linking *RMR* and *Q* values. Any relationship will be specific to the rock mass from which the data were obtained, the potential failure mode assumed in deriving the *Q* values and the orientation of the excavation considered for the *RMR* values. It is also noteworthy that the data used for deriving the *RMR* and *Q* correlations listed in Table 2 were obtained by applying different versions of the RMR system. For instance, the correlation given in Equation 3 was obtained using the pre 1976 version(s) of RMR, while the subsequent correlations may be based on either pre or post 1976 versions. Since different versions of the RMR method use somewhat different ranges of ratings, it is important to state which version is being used when correlating the *RMR* and *Q* values. The lumping of the ratings assigned using different RMR versions to compare and correlate them with the *Q* values has a very limited scientific basis.

Sheorey (1993), Goel et al. (1996) and Kumar et al. (2004) attempted to reduce data scattering and obtain better correlations using truncated versions of the RMR and Q methods. They defined $RMR_{mod}$ (also called *RCR* – rock condition rating) as *RMR* without *IRS* and *RA* and $Q_{mod}$ (also denoted as *N*) as *Q* with *SRF*=1. By regression analysis of the truncated versions of the two methods, Sheorey (1993), Goel et al. (1996) and Kumar et al. (2004) obtained the relationships given by Equations 28, 29 and 30, respectively.

$$RCR = 9.5 \ln N + 31 \qquad (28)$$

$$RCR = 8.0 \ln N + 30 \qquad (29)$$

$$RCR = 8.0 \ln N + 42.7 \qquad (30)$$

Despite the relatively high correlation coefficients ($r^2$=0.87, 0.92 and 0.88, respectively) of these equations, the relevant data plots show that the data are still scattered around the regression lines of the three equations. For instance, according to the data provided by Goel et al. (1996) when *N* is 3, the corresponding *RCR* can be between 25 and 45. Further, Sari and Pasamehmetoglu (2004) found that regression analysis of *RCR* and *N* values does not always yield high correlation coefficients. Their *RMR-Q* correlation (Equation 26) with $r^2$=0.86 is better than their *RCR-N* correlation given as Equation 31 with $r^2$=0.65, showing a distinction from the three equations given above.

$$RCR = 1.7 \ln N + 51.5 \qquad (31)$$

Based on their analysis, Sari and Pasamehmetoglu stated that the correlation between *RCR* and *N* cannot be generalised.

## 7 CONCLUSIONS

The rock mass classification systems, such as RMR and Q, despite their limitations, will continue to be used for underground excavation support design, particularly in the early stages of projects as they provide a useful means of transferring previous experience to new projects. Since both methods have limitations, it is advisable to apply both simultaneously if they are to be used as a design tool. However, it is not implied that by applying both methods all possible rock mass problems can be adequately dealt with.

The review of the published RMR and Q correlations showed that everyone is different and the data used in deriving them are often widely scattered. The main reasons for this are the differences in the parameters and the rating methods used, and the manner in which the final RMR and Q values are computed.

It is clear from the available information that a different relationship can be obtained for each case study and that each relationship is applicable only to that particular rock mass and project conditions from which the relationship was obtained. Even for the same rock mass, if the data used are widely scattered, such relationships are of very little practical value and their use for transforming the ratings between the two methods could lead to errors. Further, Kaiser et al. (1986) showed that the results of correlations depend on the choice of the dependent variable. From the foregoing, it is apparent that there is no sound scientific basis to assume a universally applicable linear relationship between the two systems.

When both methods are to be applied to a project, which is desirable, each should always be applied independent of the other, without attempting to convert the ratings of one method to that of the other using the relationships published in the literature. Such relationships, bearing in mind their obvious limitations, may be used as a crude guide for checking the general accuracy of the ratings derived by the two systems.

## 8 DISCLAIMER

The views expressed in this paper are those of the two authors and not necessarily of their respective employers.

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APPENDIX B

UDEC Analysis of CLH tunnel

UDEC Analysis of CLH tunnel
Sample data file

```
title
Chiew Larn Tunnel Station 100 (File CH100F.dat)
;Rock bolts and shotcrete as per Q-System, internal water pressure applied
;
round 0.01
set ovtol 0.1
set delc off
bl 0,0 0,70 80,70 80,0
;
;Tunnel outline
cr 34.5,5 45.5,5
cr 45.5,5 46,8.5
cr 46,8.5 46.5,11.5
cr 34.5,5 34,8.5
cr 34,8.5 33.5,11.5
arc 40,11.5 46.5,11.5 180 12
;
jset 79,0 100,0 0,0 3,0 0,0
jset -37,0 100,0 0,0 3,0
;
;Create topograpy
table 100 0,70 10,71 17,73 20,72 30,68 40,65 50,61 60,56 70,50 80,46
crack table 100
delete range above table 100
;pl hold bl
;
change jmat=2 range angle 78,80
change jmat=2 range angle -38,-36
change jmat=1 range angle -38,-36 xrange 43.2,44.6 yrange=16,17.2
;
prop mat=1 den=2500 bu=30e9 sh=20e9
prop jmat=1 jkn=40e9 jks=20e9 jcoh=1e10 jfr=50 jten=1e10
prop jmat=2 jkn=8e9 jks=1e9 jcoh=0 jfr=20 jten=0
prop jmat=2 jperm=3e8 azero=1e-2 ares=1e-3
prop jmat=1 jperm=3e8 azero=1e-2 ares=1e-3
;
gen edge=3
;
;Apply boundary conditions
boun xvel=0 yvel=0 range xr -1,1
boun xvel=0 yvel=0 range xr 79,80
bound xvel=0 yvel=0 range yr -1,1
set gravity 0,-9.81
;pl hold bl zon bou ycon
;
damp auto
step 2000
```

```
;
;pl hold bl disp ye
;pl hold bl str yel
reset disp
;
;Excavate tunnel
table 200 34.5,5 34,8.5 33.5,11.5 34.37,14.75 35.4,16.1 36.75,17.1 40,18 43.25,17.1
44.6,16.1 45.63,14.75 46.5,11.5 46,8.5 45,5 34.5,5
delete range inside table 200
pl bl hold dis ye
window 20,60 0,40
;
;Apply shotcrete
struct generate fang=0 theta=360 np=150 mat=5 thi=.01 xc=40 yc=11.5
prop mat=5 st_dens=2500 st_prat=0.15 st_ymod=21e9 st_ycom=4e6 st_yield=2e6
st_yresid=1e6
prop mat=5 if_kn=2e9 if_ks=1e9 if_fric=45 if_ten=1e6 if_coh=1e6
;
;------------------------------
;FISH routine for reinforcement
def setup
;Variables for reinforce
  xCentre = 40.0       ; x-coord of tunnel centre
  yCentre = 11.5       ; y-coord of tunnel centre
  theta1  = -30.0      ; starting angle for cables
  theta2  = 210.0      ; ending angle for reinforcemnt
  radius1 = 6.5        ; radius of tunnel
  radius2 = 11.5       ; ending radius for remote end of reinforcemnt
  nReinfs = 18         ; number of reinforcemnt
end
;
def place_reinf
;Place reinforce elements along a given arc of tunnel.
;
  ;calculate angle increment between successive reinforcemnt
  theta1 = degrad * theta1
  theta2 = degrad * theta2
  _angInc = (theta2 - theta1) / float(nReinfs - 1)
  _ang = theta1
;
  ;get endpoint coordinates
  loop ii (1, nReinfs)
    _x1 = radius1 * cos(_ang) + xCentre
    _y1 = radius1 * sin(_ang) + yCentre
    _x2 = radius2 * cos(_ang) + xCentre
    _y2 = radius2 * sin(_ang) + yCentre

    ;place reinforcemnt
    command
      reinf 10 _x1 _y1  _x2 _y2
```

```
    endcommand
    _ang = _ang + _angInc
  endloop
;
end
;
setup
;
prop mat=10 r_astiff=1e8 r_sstiff=1e8 r_leng=1 r_uaxial=1e6 r_ushear=1e6
;Apply rock bolts
place_reinf
;-------------------
solve
;step 2000
pl bl hold disp ye struc iw reinf lm
reset disp
;-------------------
;
set caprat 10
set flow steady
;set dtflow=0.5 voltol=1e-4 maxmech=100
;
boun imper range xr 0,80 yr -1,1
boun imper range xr -0.1,0.1 yr 0,70
boun imper range xr 79,81 yr 0,70
fluid dens=1000 bulkw=2000e6
ini sat=0 range outside table 200
pfix pp 0.56e6 range inside table 200
step 10
;
pl blo hold vflow iw
solve
window
pl blo hold vflow iw
```

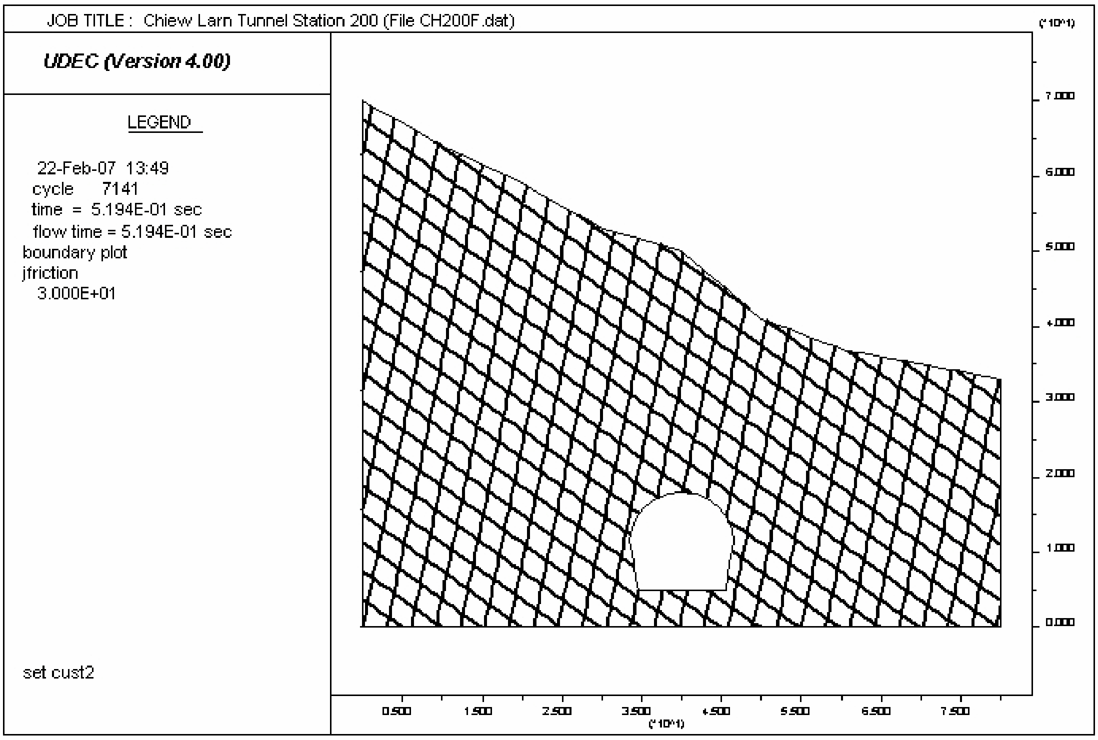



Figure B1 Discontinuity block model for tunnel section at Sta. 200 m

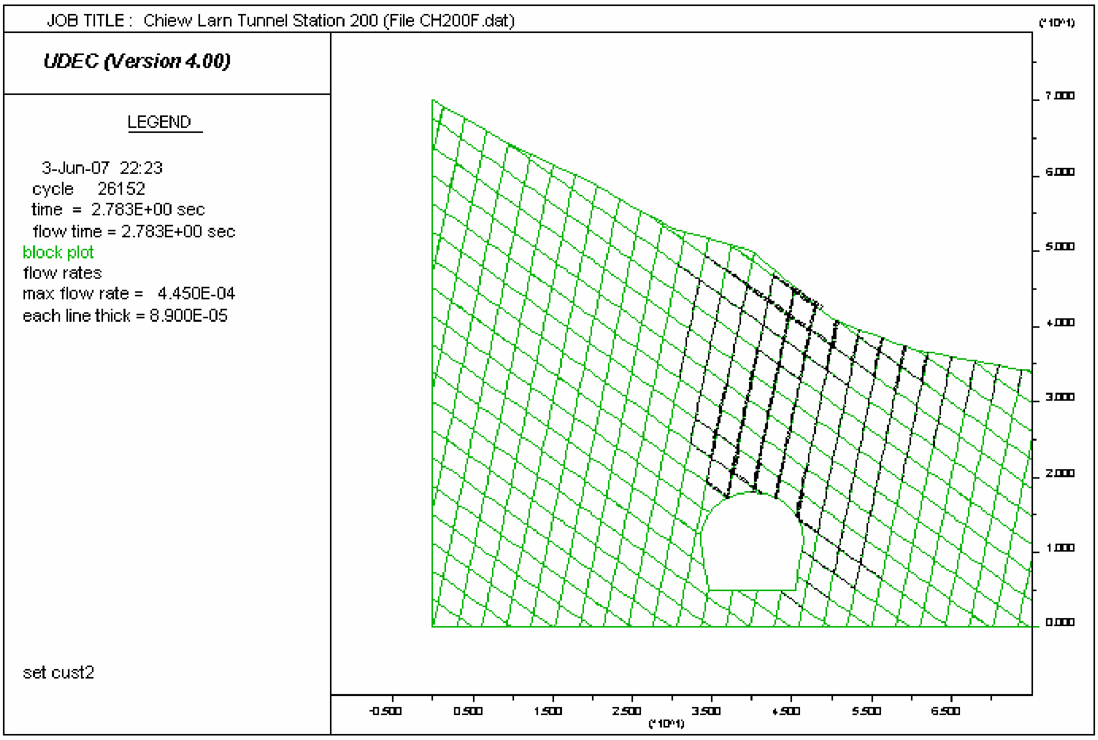



Figure B2 Potential seepage paths for tunnel section at Sta. 200 m

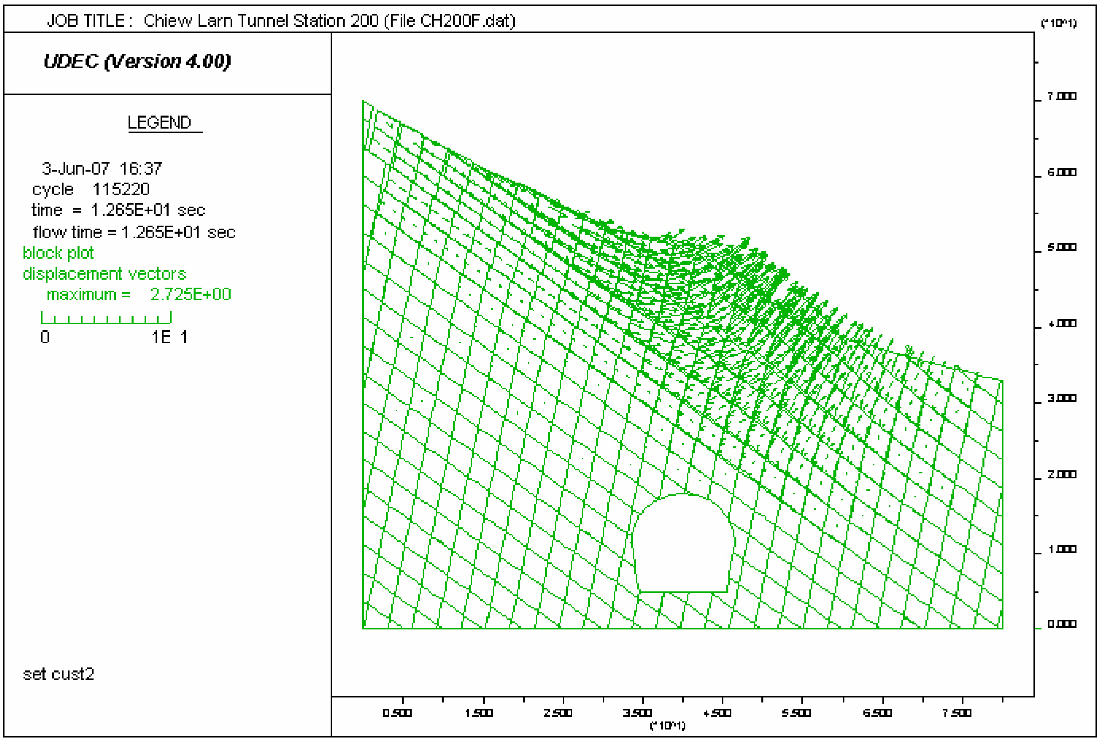



Figure B3 Displacement at ground surface due to seepage induced weakening

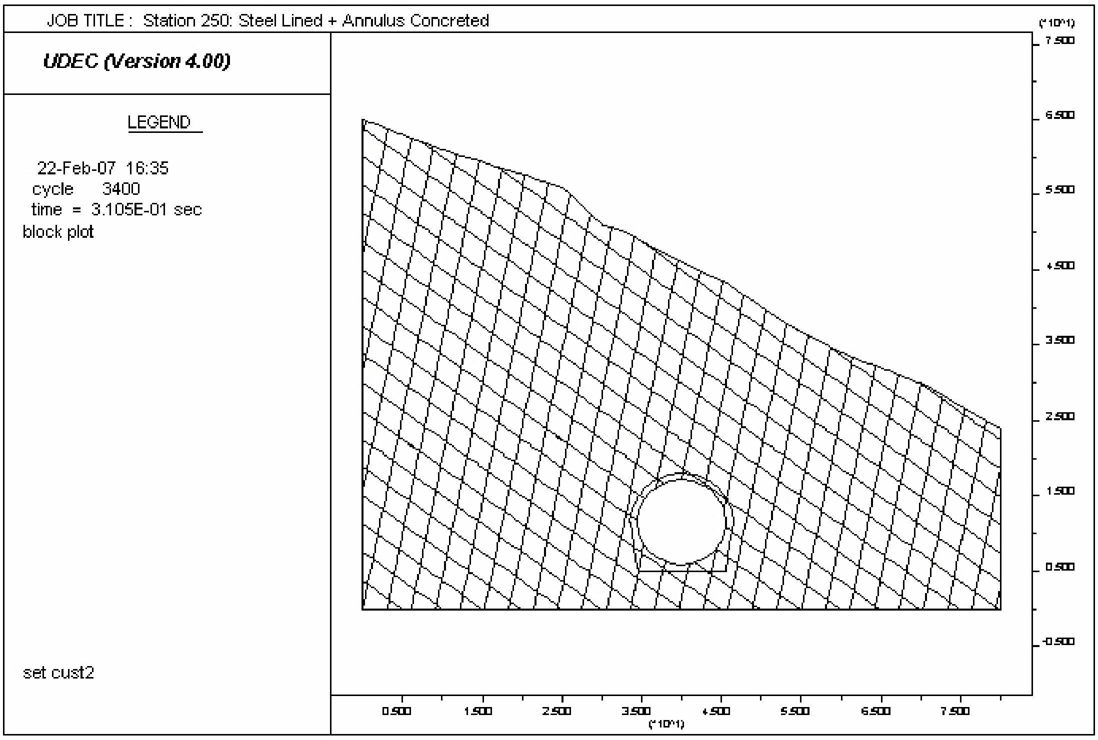



Figure B4 Model used for fully lined tunnel

# APPENDIX C

## UDEC analysis for LTKPA tunnel

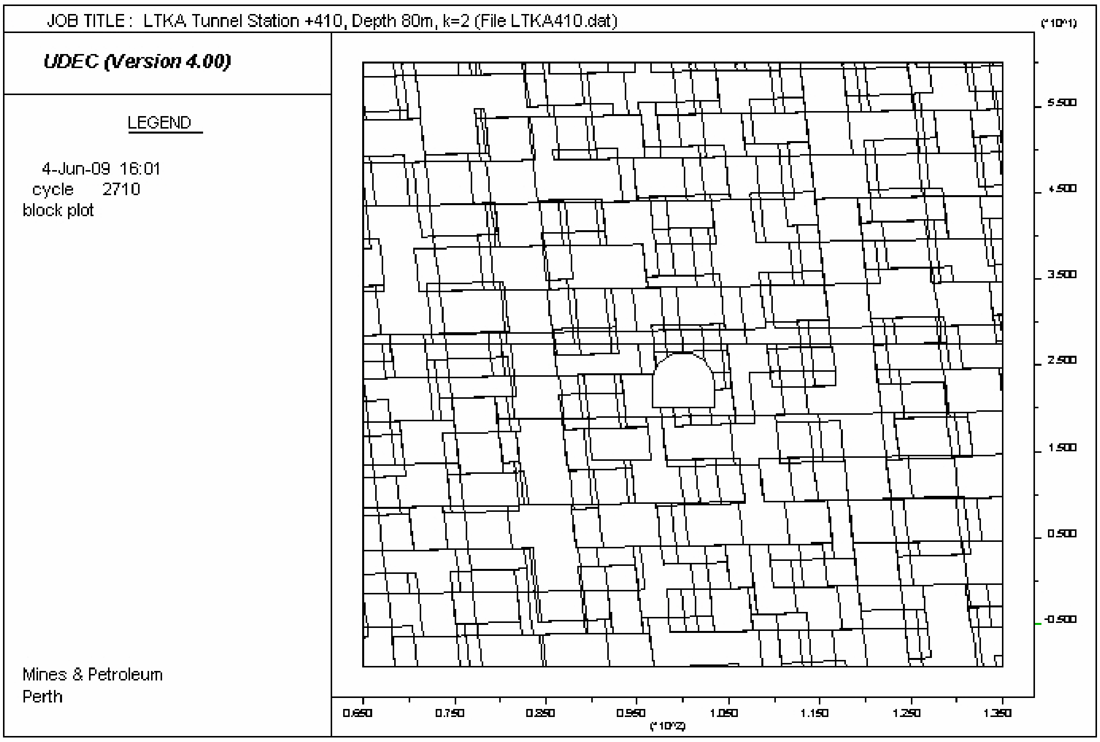



Figure C1 Discontinuity block model for tunnel section at Sta. 410 m

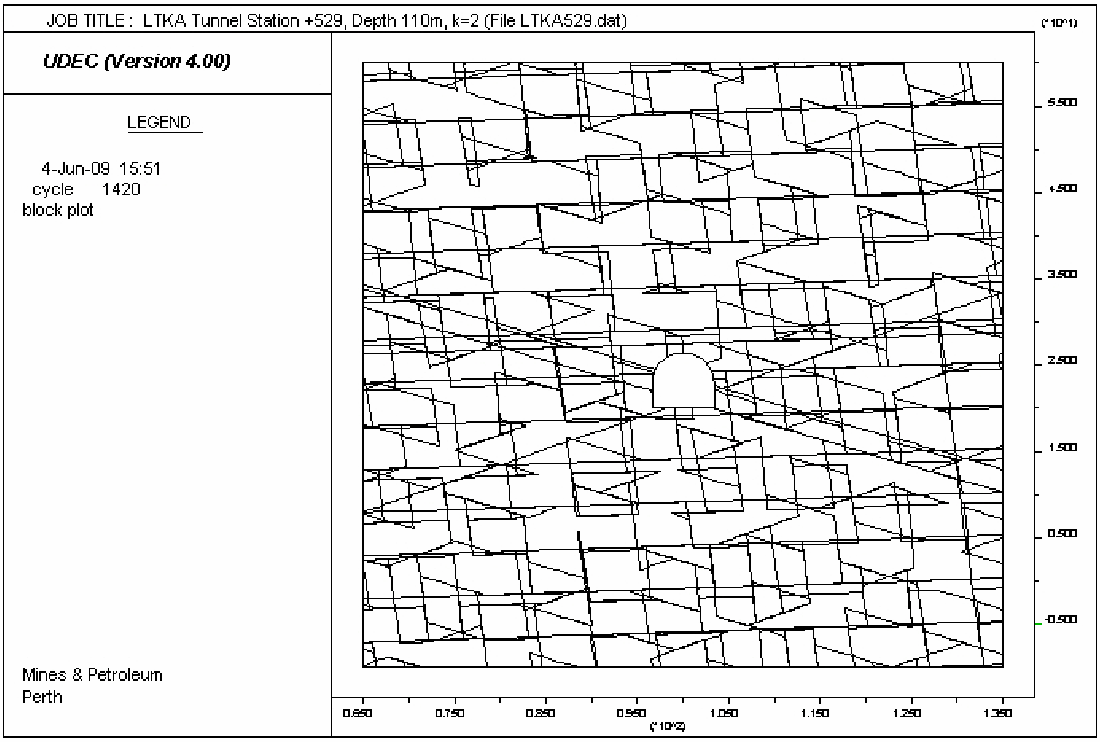



Figure C2 Discontinuity block model for tunnel section at Sta. 529 m

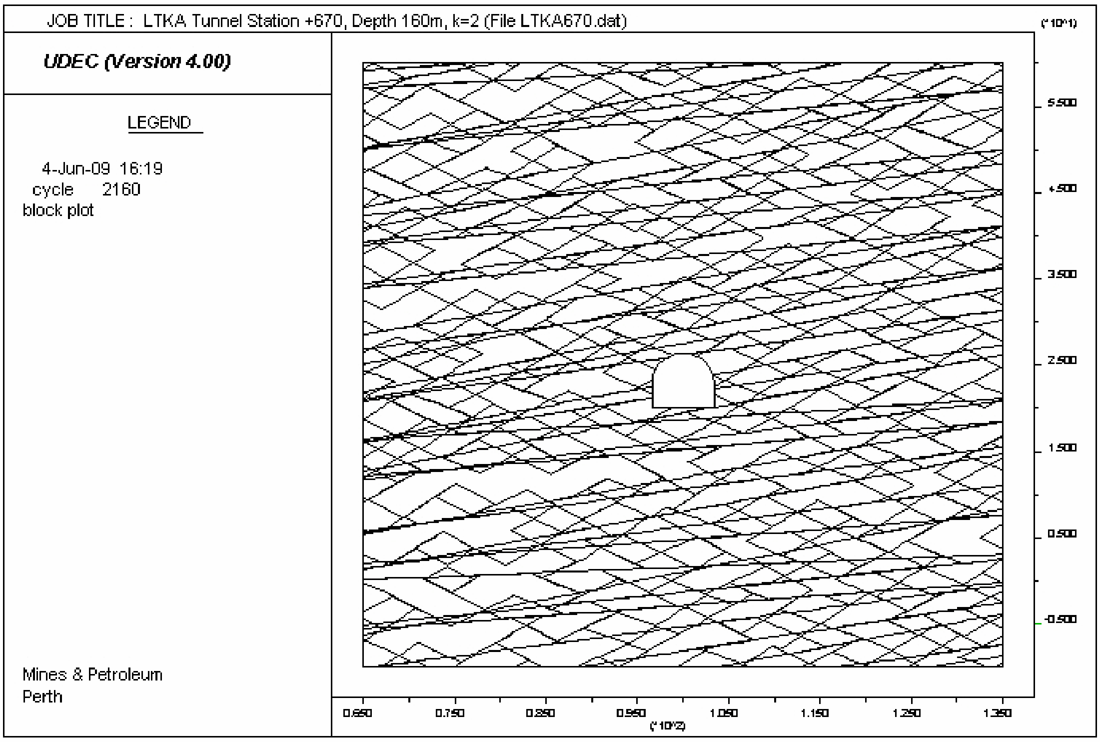



Figure C3 Discontinuity block model for tunnel section at Sta. 670 m

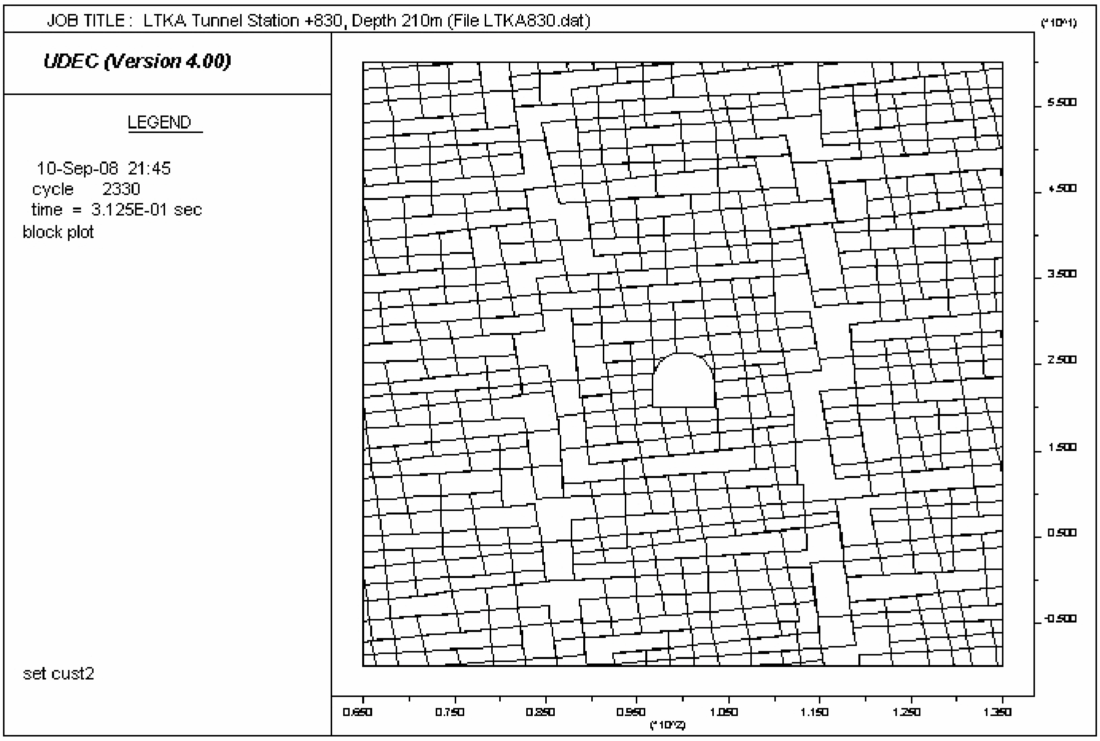



Figure C4 Discontinuity block model for tunnel section at Sta. 830 m

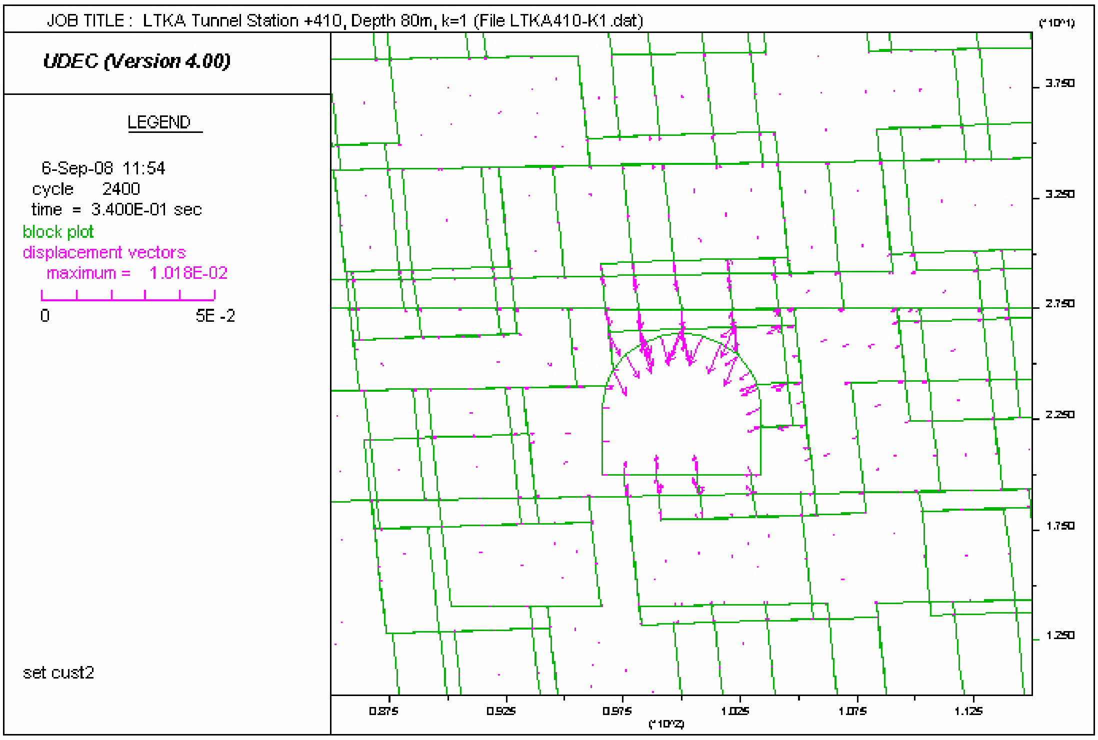



Figure C5 Displacement vectors at Sta. 410 m with no support installed

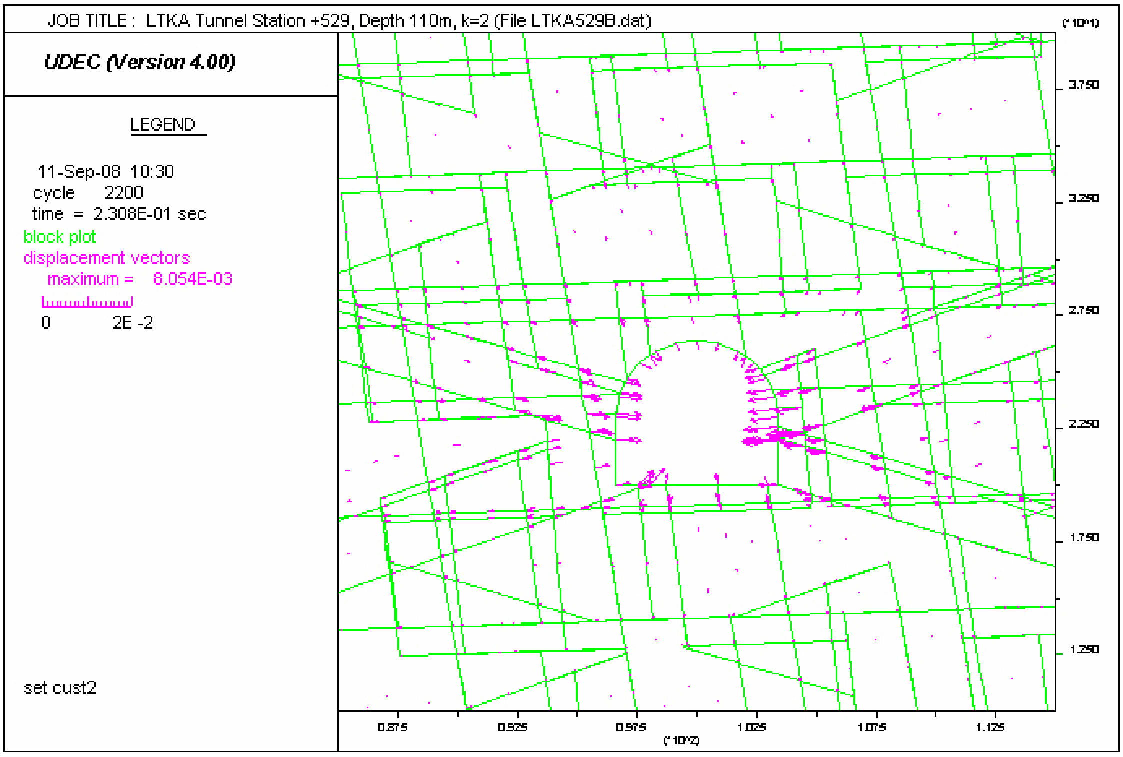



Figure C6 Displacement vectors at Sta. 529 m with no support installed

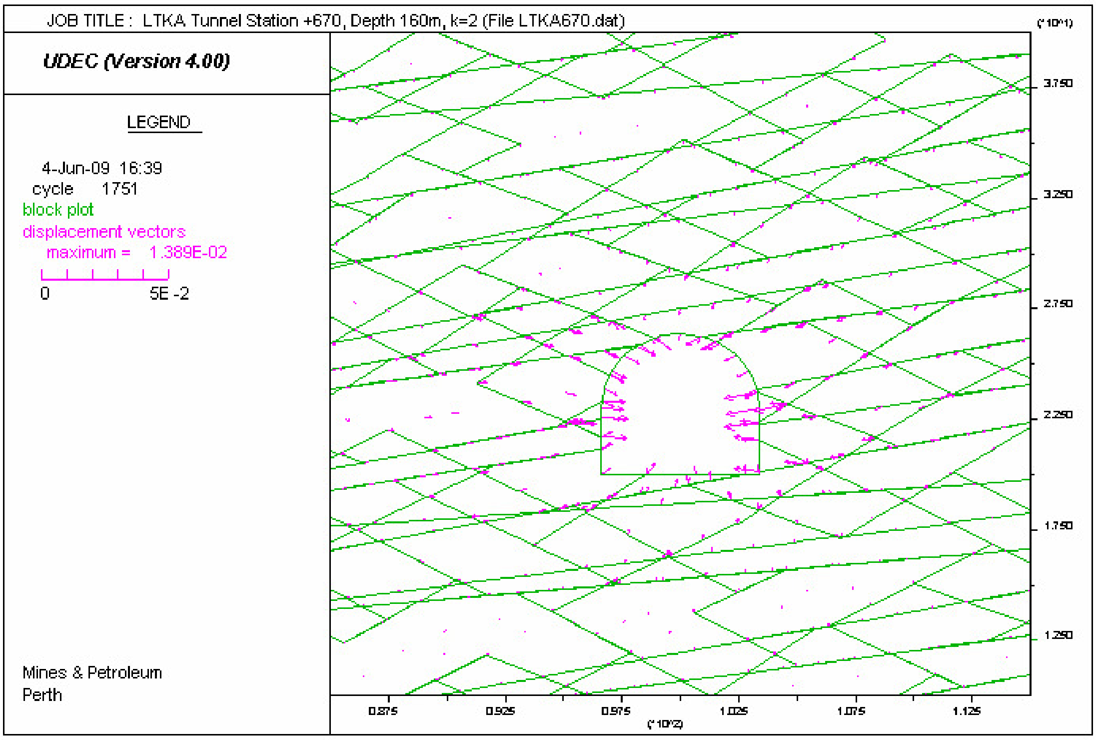



Figure C7 Displacement vectors at Sta. 670 m with no support installed

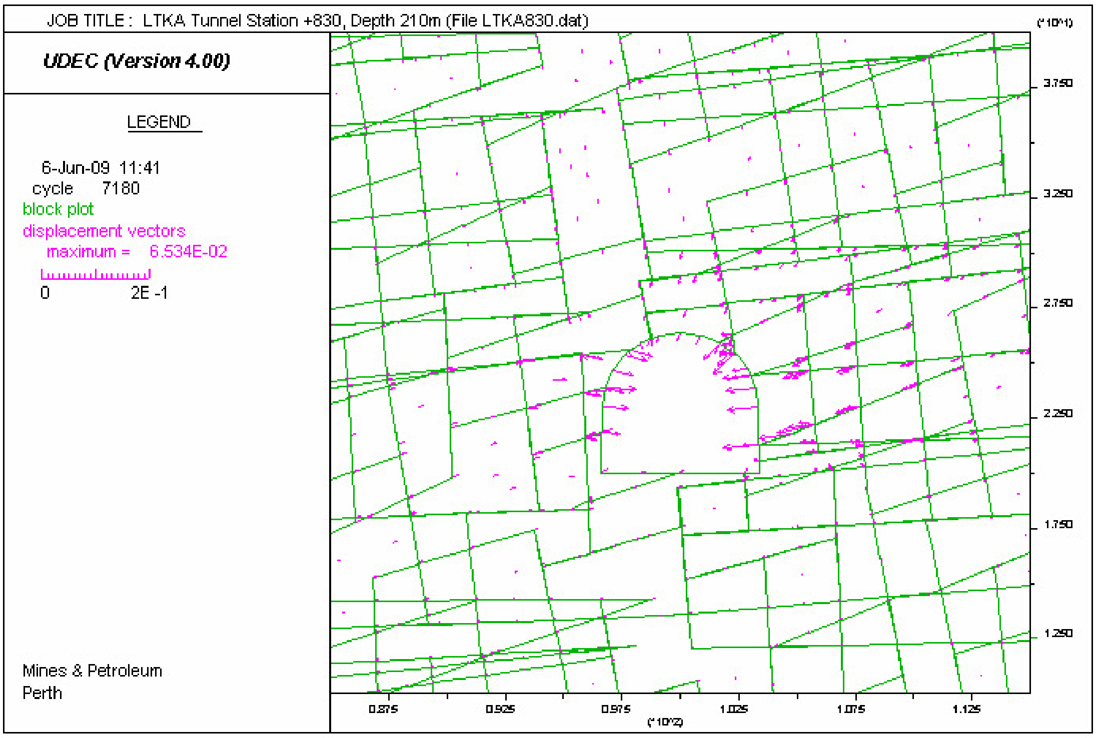



Figure C8 Displacement vectors at Sta. 830 m with no support installed

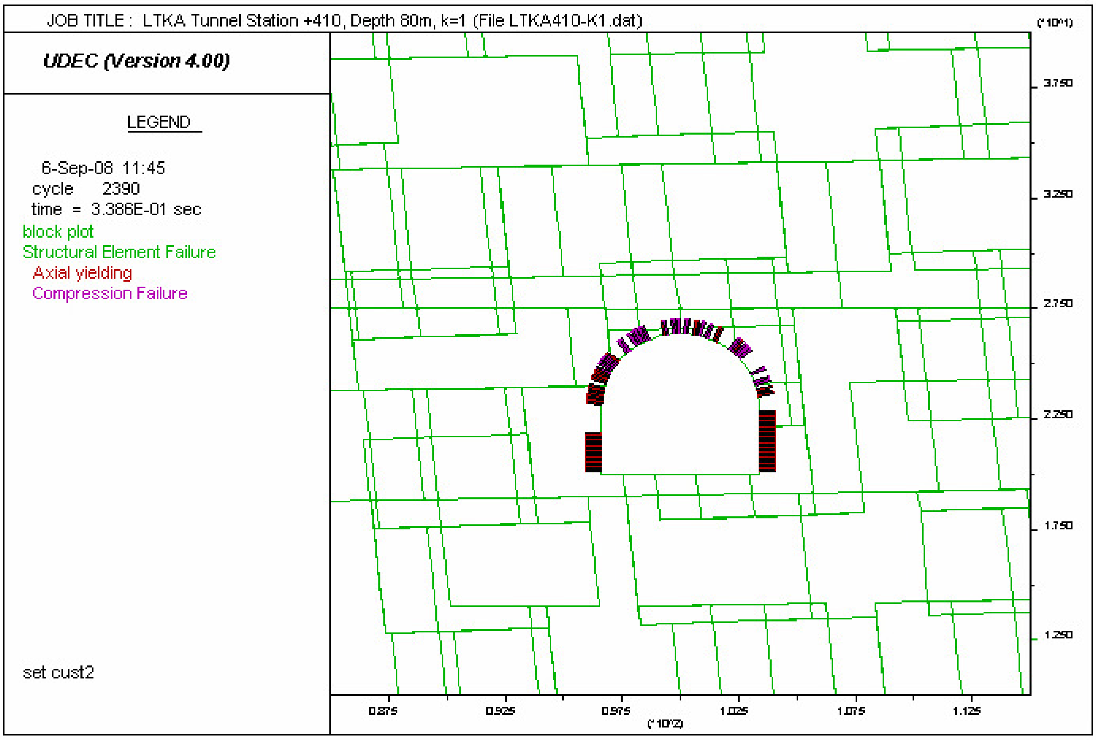



Figure C9 Failure of URF shotcrete at Sta. 410 m when k=1 (rock bolts not shown)

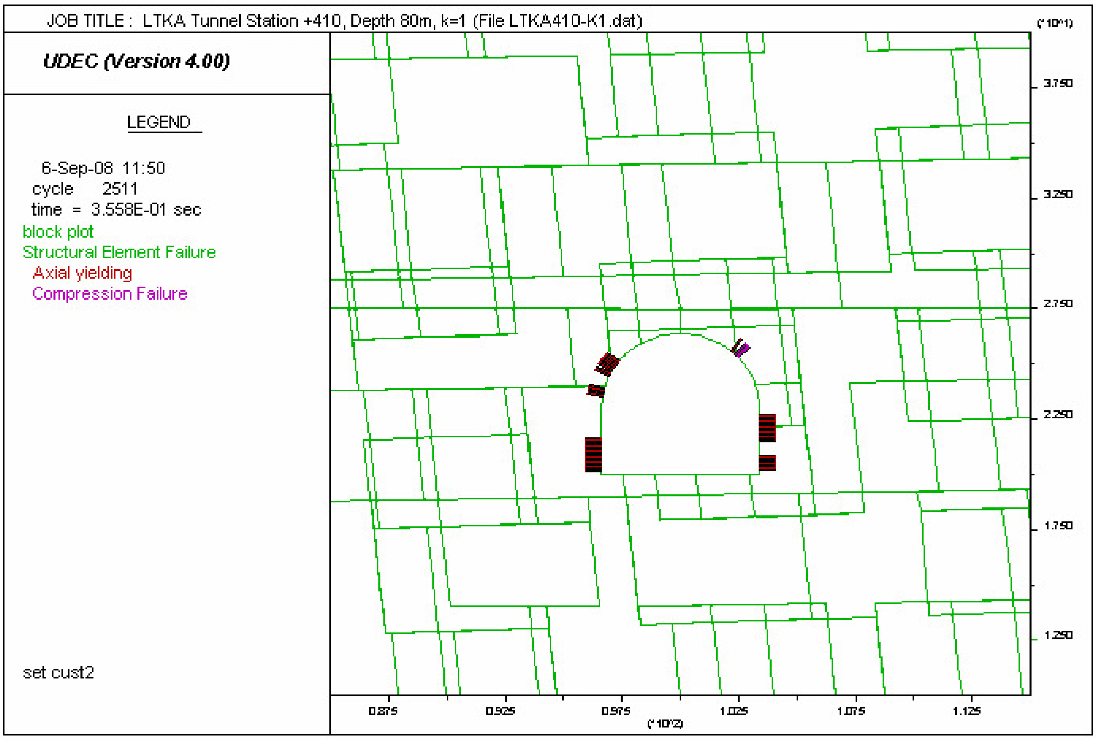



Figure C10 Reduction in failure at Sta. 410 m when mesh reinforcement was added

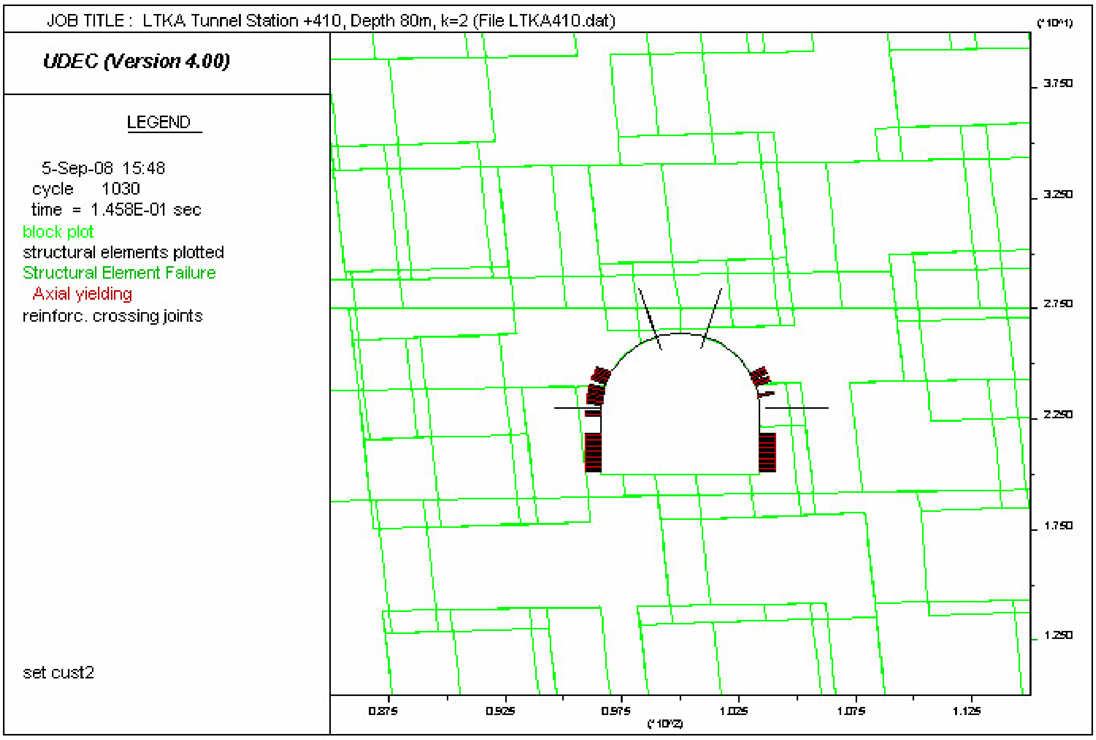



Figure C11 Failure of URF shotcrete at Sta. 410 m when k=2 (rock bolts installed)

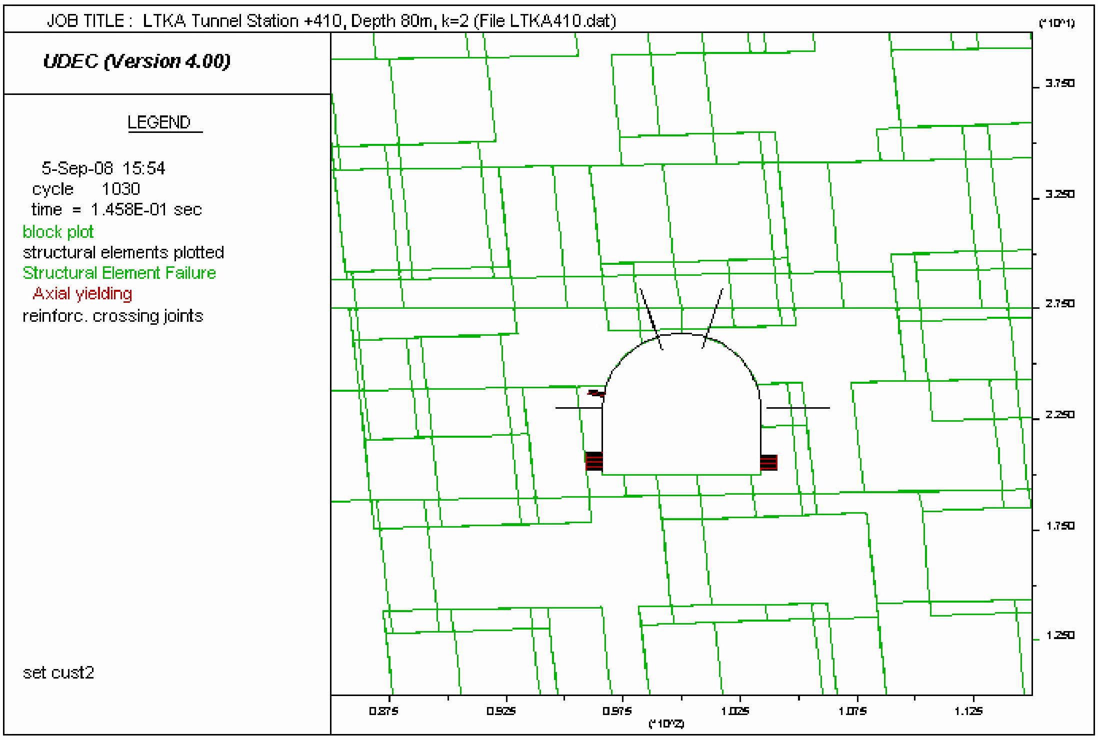



Figure C12 Reduction in failure at Sta. 410 m when mesh reinforcement was added

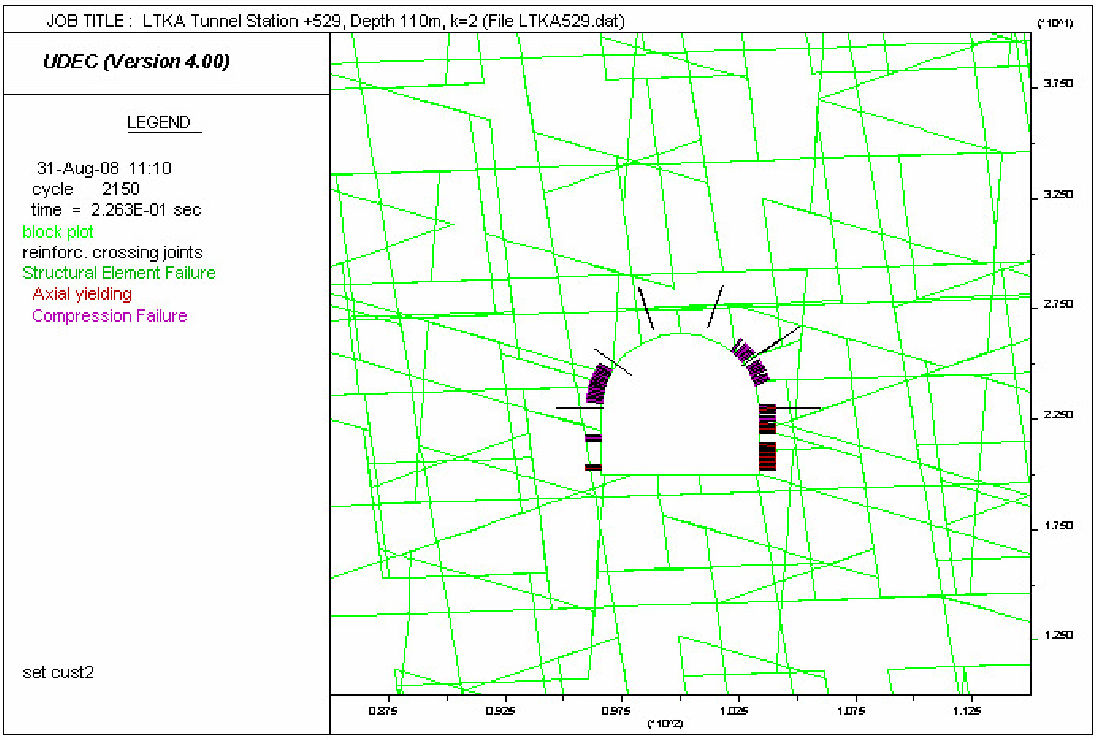



Figure C13 Failure of URF shotcrete at Sta. 529 m when k=2 (rock bolts installed)

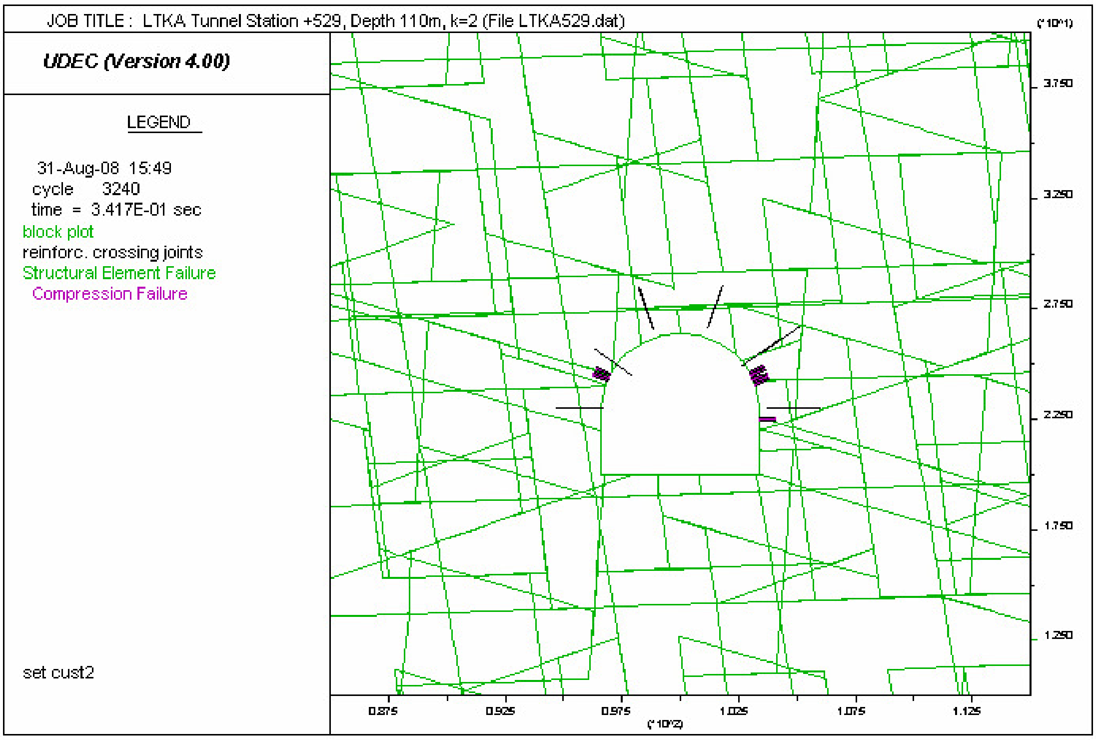



Figure C14 Reduction in failure at Sta. 529 m when mesh reinforcement was added

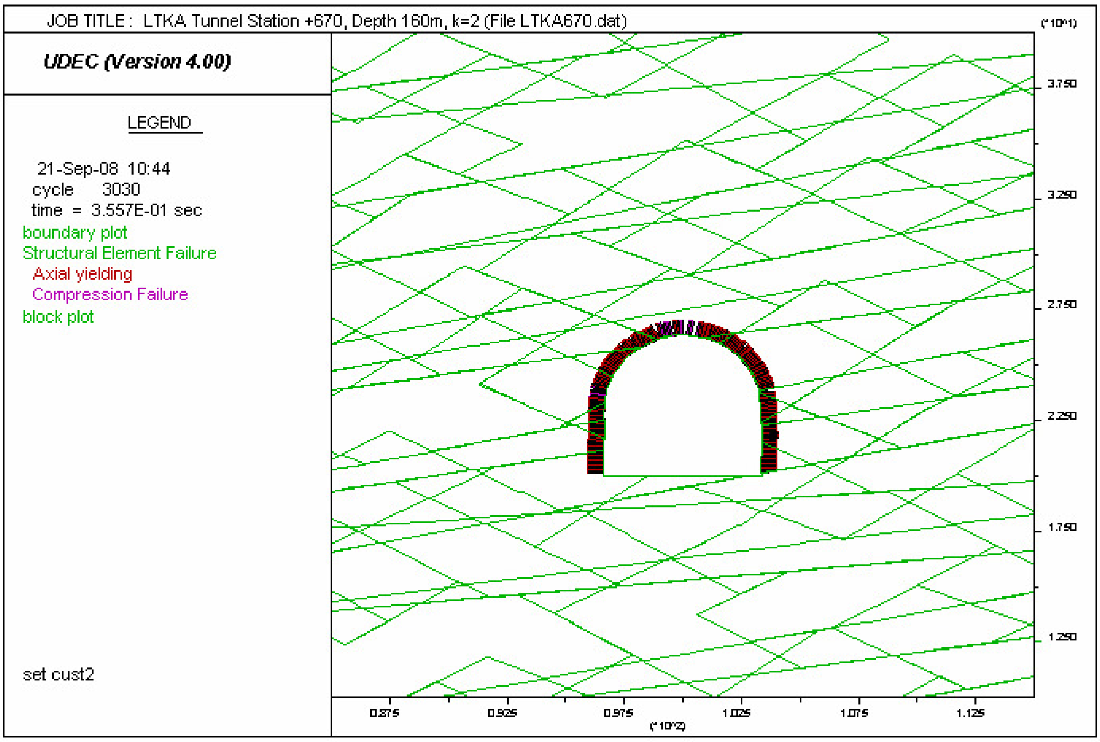



Figure C15 Failure of URF shotcrete at Sta. 670 m when k=2 (rock bolts installed)

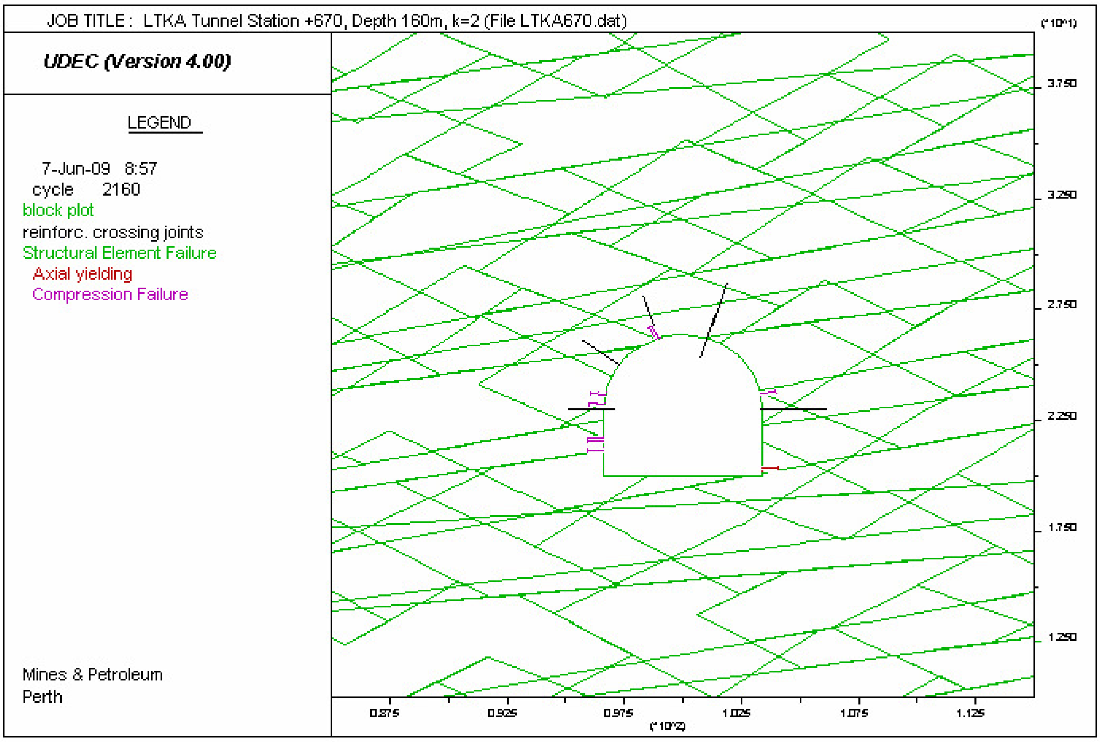



Figure C16 Reduction in failure at Sta. 670 m when mesh reinforcement was added

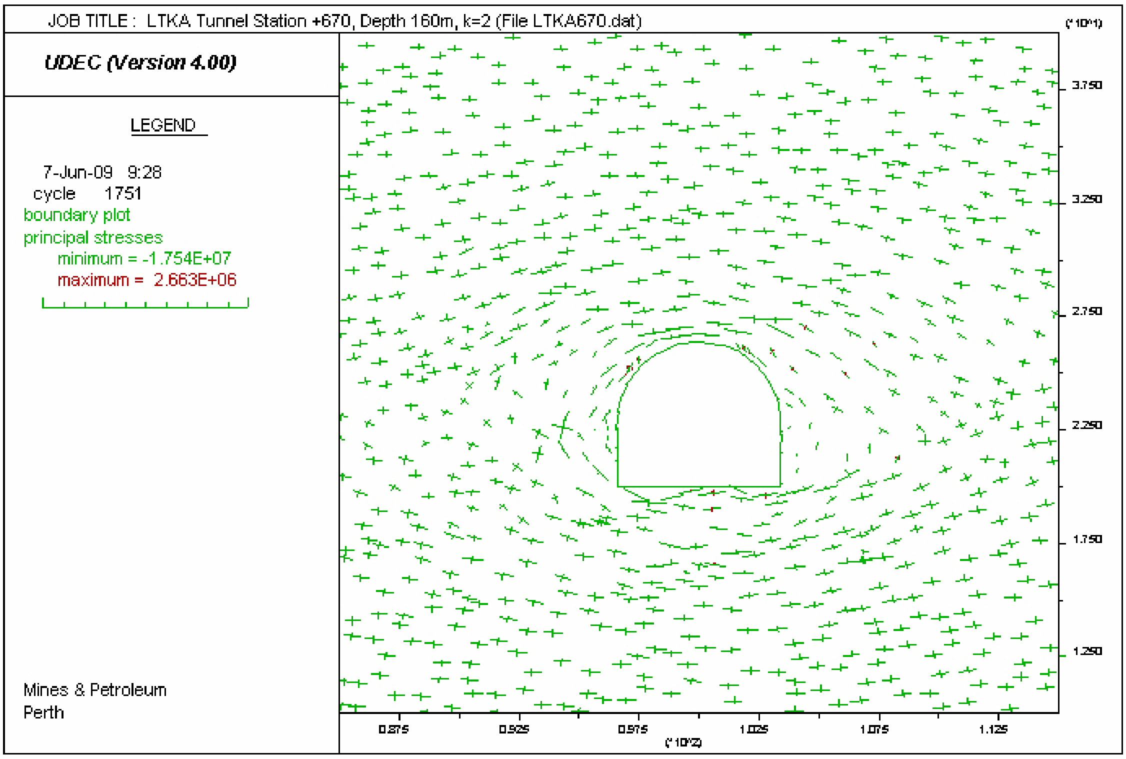
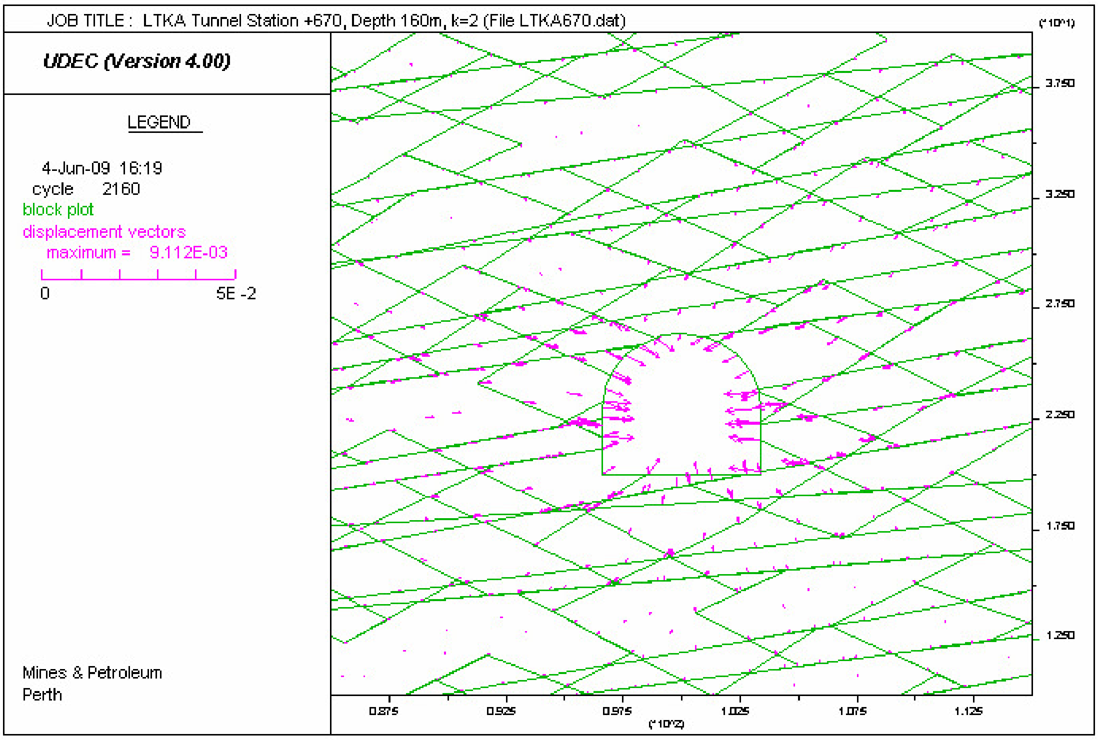

JOB TITLE : LTKA Tunnel Station +670, Depth 160m, k=2 (File LTKA670.dat)
UDEC (Version 4.00)
LEGEND
7-Jun-09 9:28
cycle 1751
boundary plot
principal stresses
minimum = -1.754E+07
maximum = 2.663E+06
Mines & Petroleum
Perth

Figure 6.20


JOB TITLE : LTKA Tunnel Station +670, Depth 160m, k=2 (File LTKA670.dat)
UDEC (Version 4.00)
LEGEND
4-Jun-09 16:19
cycle 2160
block plot
displacement vectors
maximum = 9.112E-03
0
5E -2
Mines & Petroleum
Perth

UDEC analysis for LTKPA tunnel
Sample data file

```
title
LTKA Tunnel Station +670, Depth 160m, k=2 (File LTKA670.dat)
set random
;
;Rock Density 2600
;
round 0.05
;set edge 0.003
;set ovtol 0.05
;set delc off
bl 65,-10 65,60 135,60 135,-10
;
;pl bl hold
;Tunnel outline
cr 96.6,20 103.4,20
cr 103.4,20 103.4,23
cr 96.6,23 96.6,20
arc 100,23 103.4,23 180 12
;pl bl hold
;
jset 7,5 100,0 0,0 2,0.5 65,-10
jset -25,5 10,2 0,0 3,1
jset 30,5 10,2 0,0 3,1
;
jdel
;
change jmat=2
;
prop mat=1 den=2600 bu=9.8e9 sh=8.6e9
prop jmat=1 jkn=40e9 jks=20e9 jcoh=1e10 jfr=50 jten=1e10
prop jmat=2 jkn=8e9 jks=1e9 jcoh=20e3 jfr=30 jten=10
;
gen edge=3
;
; Apply in situ stress
set gravity 0, -9.81
insitu stress -9.69e6 0 -4.846e6 ygrad 5.5e4 0 2.5e4 szz -4.846e6 zgrad 0 5.5e4
;
;Apply boundary conditions
bound stress -9.69e6 0 -4.846e6 ygrad 5.5e4 0 2.5e4 range 64,66 -10,60
bound stress -9.69e6 0 -4.846e6 ygrad 5.5e4 0 2.5e4 range 134,136 -10,60
bound stress 0 0 -3.06e6 range 65,135 59,61
boun xvel=0 yvel=0 range xr 64,66
boun xvel=0 yvel=0 range xr 134,136
bound xvel=0 yvel=0 range yr -11,-9
;
;
```

```
damp auto
solve elastic
reset disp
;
table 10 96.6,20 96.6,23 96.715,23.88 97.055,24.7 97.595,25.4 98.3,25.94
99.12,26.28 100,26.4 100.879,26.28 101.7,25.94 102.404,25.4 102.944,24.7
103.284,23.88 103.4,23 103.4,20 96.6,20
;
delete range inside table 10
;
;------------------------------
;Apply shotcrete
struct generate fang=-40 theta=260 np=150 mat=5 thi=0.1 xc=100 yc=23
prop mat=5 st_dens=2500 st_prat=0.15 st_ymod=35e9 st_ycom=60e6 st_yield=40e6
st_yresid=20e6
prop mat=5 if_kn=2e9 if_ks=1e9 if_fric=45 if_ten=1e6 if_coh=1e6
;
;------------------------------
;FISH routine for reinforcement
def setup
;Variables for reinforce
  xCentre = 100.0        ; x-coord of tunnel centre
  yCentre = 23.0         ; y-coord of tunnel centre
  theta1  = 0            ; starting angle for cables
  theta2  = 180.0        ; ending angle for reinforcemnt
  radius1 = 3.4          ; radius of tunnel
  radius2 = 5.4          ; ending radius for remote end of reinforcemnt
  nReinfs = 6            ; number of reinforcemnt
end
;
def place_reinf
;Place reinforce elements along a given arc of tunnel.
;
  ;calculate angle increment between successive reinforcemnt
  theta1 = degrad * theta1
  theta2 = degrad * theta2
  _angInc = (theta2 - theta1) / float(nReinfs - 1)
  _ang = theta1
;
  ;get endpoint coordinates
  loop ii (1, nReinfs)
    _x1 = radius1 * cos(_ang) + xCentre
    _y1 = radius1 * sin(_ang) + yCentre
    _x2 = radius2 * cos(_ang) + xCentre
    _y2 = radius2 * sin(_ang) + yCentre

    ;place reinforcemnt
    command
      reinf 10 _x1 _y1  _x2 _y2
    endcommand
```

```
    _ang = _ang + _angInc
  endloop
;
end
;
setup
;
;Apply rock bolts
prop mat=10 r_astiff=1e8 r_sstiff=1e8 r_leng=1 r_uaxial=1e6 r_ushear=1e6
place_reinf
;-------------------
;
;pl bl hold reinf iw
hist ydis 100,26.5
hist xdis 103.5,23
hist xdis 96.5,23
;
solve
pl his 1 2 3 hold
pl bl hold disp ye
;
win 85,115 10,40
pl hold bou reinf iw stru afail
ret
;
```