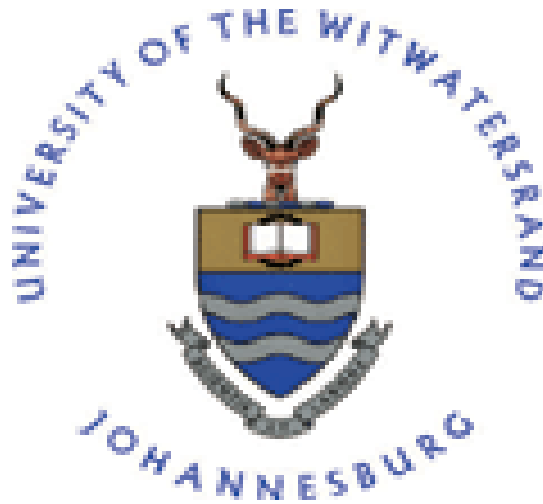




Title:

Evaluation of the deformation in the tailrace tunnel of the Ingula Pumped Storage Scheme

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Date

May 2018

Declaration

I declare that this research report is my own unaided work. It is being submitted for the Degree of Master of Science in Mining Engineering at the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination to any other University.

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ABSTRACT

In the design phase of a project it is imperative for engineers to have a solid understanding of the rock mass behaviour. Geotechnical investigations consisting of field and laboratory tests need to be conducted to determine the rock mass parameters in order to predict deformations, during the construction phase. Rock mass classification systems should also be employed during the geotechnical investigation to quantify the quality of the rock mass. Although the modulus of deformation of a rock mass is of critical importance for a geotechnical project, the use of in situ tests to determine the parameter is an extremely difficult, time consuming and expensive process. As a result empirical equations have been used by professionals to determine the modulus of deformation.

The implementation of a monitoring programme consisting of geotechnical instrumentation and mappings during the construction phase is also critical in the construction phase to validate design assumptions and long term deformation. Continuous data collection entailing geological mapping and the application of rock mass classification systems assists in reducing geological uncertainties, determining rock mass conditions of the underground excavation and determining parameters that influence the rock mass behaviour. Data from the monitoring program is used as input into numerical models to back analyse and determine parameters. Numerous models are run to reduce the uncertainty in the mechanical behaviour of the rock mass.

The main objective of this project is to assess how good numerical models are at determining the deformation in a rock mass as compared to in situ measured deformations, using the tailrace tunnel (TT) at the Ingula Pumped Storage Scheme as a case study. The comparative study was conducted by using the Phase2 elastic numerical analysis to determine stress, strains and deformation in five tunnel sections. Partial deformation readings of instrumentation necessitated the correction of in situ measured deformations using axisymmetric analysis. These results were then compared to the numerically modelled results. The effectiveness of empirical methods based on rock mass classification systems and back analysis methods at determining the modulus of deformation was also examined in the research project. Focusing on the empirical relationships of Serafim and Pereira (1983), Sonmez et al (2006), Hoek and Diederichs (2006), Hoek and Brown (1997) and the back analysis method developed by Kirsten (1976). Hazard Warning Levels were used to determine the stability of the TT.

Although the quality of the numerical analysis is based on the input rock mass parameters and in situ stress, results show the elastic numerical analysis was effective in the determination of deformation in the TT. Empirical and back analysis methods were also successful at determining the modulus of deformation, when compared to in situ tests.

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1. INTRODUCTION

1.1 Background

South Africa currently suffers from a shortage of electricity and has often experienced load shedding to compensate for this problem. In recent years, Eskom has constructed a number of power stations to remedy this situation. Ingula Pumped Storage Scheme (IPSS) is one of these stations and is Eskom's third hydroelectric power station. This project consists of a network of tunnels constructed in the Volksrust Formation and Normandien Formation of the Little Drakensberg.

Singh and Goel (2011) define the present day approach to the design of support system as the "design as you go". In the feasibility study, empirical correlations may be used to estimate rock parameters (Singh and Goel, 2011). Field tests are conducted to determine the rock mass properties at the design stage. During the construction stage, displacement field data is obtained on supported excavations and rock mass from instrumentation installed. Geological mapping is also done to reduce uncertainty regarding the behaviour of the rock mass. This data is then used in computer models for the back analysis of the models and parameters. Models are run numerous times to reduce any uncertainties in geological and engineering behaviour of the rock mass.

The purpose of this study is to evaluate how good empirical and numerical methods are in the estimation of the behaviour of the rock mass by comparing their predictions with actual field data, using IPSS as a case study. This study will look specifically at the deformations in the tailrace tunnel. During the construction of a project of this magnitude, it is important to monitor the rock mass behaviour in this confined stress environment. A very intensive monitoring program was implemented to monitor the stability and to validate assumed geotechnical design parameters in the caverns. Instrumentation such as load cells, convergence points and multiple point borehole extensometers (MPBX) were installed. Regular mapping was also conducted to report on the encountered geology and geological structures in the underground caverns. Numerical models were also employed to predict deformation in the underground caverns. Therefore the focus of this research is on the stresses and strains, in order to determine the modulus of deformation and stability in the tailrace tunnel.

1.1.1 Rock mass classification systems

Geotechnical investigations alone are not enough to determine parameters that influence the rock mass behaviour. Rock mass classification systems should also be applied for rock mass characterisation. Input parameters in rock mass classification systems include, joint conditions, groundwater conditions, field stresses, intact rock strength and drill core conditions. The use of both geotechnical investigations and rock mass classification systems give engineers meaningful information to describe the rock mass conditions encountered on respective site. Three rock mass classification systems were used to characterise the rock mass at the IPSS. These include the Rock Quality system (Q-system) (Barton, 2002), Rock Mass Rating (RMR) system (Bieniawski, 1989) and the Geological Strength Index (GSI) system (Marinos and Hoek, 2000). Information from these systems can be used to predict the response of a rock mass to an excavation.

1.1.2 Modulus of deformation

The International Society for Rock Mechanics defines the modulus of deformation as, the ratio of stress to corresponding strain during loading of a rock mass, including elastic and inelastic behaviour (Hoek and Diederichs, 2006). Modulus of deformation can be determined using in situ test, but these are very costly and time consuming. The tests include the Plate Jacking Test, the Plate Loading Test and the Radial Jacking Test.

Empirical relationships can be used to determine the modulus of deformation from rock mass classifications. These relationships have been derived from case histories on the basis of a best fit curve. Numerous relationships have been proposed with rock mass classification systems such as RMR (Serafim and Pereira, 1983; Sonmez et al., 2006), and GSI (Hoek and Diederichs, 2006 ; Hoek and Brown, 1997). It is important to note that all the empirical relationships assume the rock mass is isotropic. As an alternative, back analysis methods and Finite Element Models can also be used to determine the modulus of deformation.

1.2 Research objectives

The main objectives of this research are:

- To determine stresses and strains in the tailrace tunnel
- To analyse deformations and determine deformation modulus of the rock mass in the tailrace tunnel
- To compare actual deformations with predicted deformations

The scope of this research project covers the following:

- Review existing theories on rock mass characterization and rock mass modulus of deformation.
- Briefly describe the IPSS including in situ and laboratory tests carried out during the design phase.
- Describe the geology and support installed in the tailrace tunnel. Document the geotechnical parameters and deformations along the tailrace tunnel.
- Model the stress and strain in the tunnel. Compare modelled and in situ measured displacements
- Empirically determine the modulus of deformation from geotechnical parameters. Use in situ displacements and numerical model to back analyse modulus of deformation.
- Carry out stability analysis using in situ strain around the tunnel.

1.3 Research methods

The tailrace tunnel at the IPPS was used as a case study for deformation analysis. The following methodology has been applied during the study;

Stage 1: Literature review consisting of a background study of the following;

- Rock mass classification systems
- Estimation of modulus of deformation
- Numerical analysis
- Back analysis and stability monitoring

Stage 2: Data collection

The data will be collected from the IPSS site office in Ladysmith. This will consist of feasibility reports and other project related reports, engineering geological mapping, photographs, laboratory test results etc. The literature review will be used as the basis for the assumption of any outstanding data.

Stage 3: Determine modulus of deformation

Based on the geotechnical data collected, the modulus of deformation of the rock mass will be determined by empirical methods and back analysis. The empirical relationships of Serafim and Pereira (1983), Hoek and Diederichs (2006), Hoek and Brown (1997) as well as Sonmez et al. (2006) are to be employed to determine the rock mass modulus. In the back analysis method, the monitored displacement results will be used to calculate the rock mass modulus using the method developed by Kirsten (1976) involving finite element analysis.

Stage 4: Numerical analysis of deformation

The Phase2 Rocscience computer program will be used to analyse the deformation in the tailrace tunnel. Rock mass parameters from in situ and laboratory tests will be used to model elastic deformations. Modelled deformation will then be compared to the deformation in the tunnel.

Hazard warning levels will also be employed to evaluate the stability of the tunnel. The strain in the tunnel determined from tunnel deformations will be used to determine the hazard warning levels.

1.4 Organization of this research report

This research report consists of a total of five chapters. Each chapter starts with an introduction or background in order to give the context of the chapter, followed by the detailed information and discussions of each section and sub-section.

In Chapter 1, the research project is introduced and the objectives and scope of the project are set. A clear methodology, mapping how the research project will be executed is also defined.

The literature review is covered in Chapter 2. This chapter is subdivided into four sections namely, rock mass class classification systems, methods to determine modulus of deformation, numerical analysis and back analysis. A short description of each sub section is given below:

1. In the section on rock mass classification systems, the systems used at the IPSS are described. This is followed by a discussion on uses and limitations of the classification systems over the years.
2. Methods of determining the deformation modulus of a rock mass are described next. These include in situ tests and empirical relationships.
3. Back analysis is defined in this section. Methods of back analysis with the use of finite numerical methods are also described, looking specifically at the back analysis of excavation induced stress changes and modulus of deformation.
4. The Finite Element Method (FEM) is described in the numerical analysis section. Methodology, discontinuity representation in FEM and advanced modelling are covered as sub-sections. Phase2 is the computer program that will be used for data analysis. The abilities of the computer program are also defined.
5. The final section of the literature review illustrates the use of hazard warning levels.

The Ingula Pumped Storage Scheme is described in Chapter 3. This chapter covers the project setting, geology of the project, laboratory and in situ test results for the determination of geotechnical parameters. The rock mass properties and support in the tailrace tunnel are also discussed.

Data analysis is covered in Chapter 4, where the modulus of deformation is determined using both empirical and back analysis methods. Deformation modulus results using different methods are compared to find which method best describes the deformation in the tailrace tunnel. The Phase2, finite element model, is used to model the elastic deformation in the tailrace tunnel. Modelled deformation results are compared with in situ measured results. The stability of the tailrace tunnel is confirmed using hazard warning levels.

Chapter 5, the final chapter, covers the discussion of the results from the modulus of deformation calculations using empirical methods, back analysis and the numerical analysis. The research report ends with a conclusion and recommendations.

2. LITERATURE REVIEW

2.1 Introduction

During the feasibility and the design stage of an engineering project concerning underground excavations such as tunnels or underground caverns, it is essential to know the in situ rock mass conditions. Rock mass classification systems attempt to assess the suitability of the in situ rock mass for a given engineering project and consequently the selection of the parameters for such a classification is of importance. These parameters may include the rock mass type, orientation and conditions of discontinuities, magnitude and orientation of stress and hydrological conditions. In the design process the parameters will have different effects on the reaction of the underground excavation which will influence the excavation method and the engineering support system. A detailed geotechnical investigation is conducted to collect geological and engineering parameters of the in situ rock mass for the design stage. Rock mass classification systems combine these parameters to give a quantitative description of the in situ rock mass.

2.2 Rock mass classification systems

Bieniawski (1974) defines classification as the arrangement of objects into groups based on their relationship. Although rock mass classification systems form the basis of an empirical rock engineering design approach and are largely used in rock engineering practice throughout the world, these classification systems are not to be taken as a substitute for engineering design. They should be applied intelligently and used in conjunction with observational methods and analytical studies to formulate an overall design rationale compatible with the design objectives and site geology. When correctly applied, rock mass classifications can be powerful aids in design.

Bieniawski (1974) stated the aims of rock mass classification systems as:

1. Identify the most significant parameters influencing the behaviour of a rock mass.
2. Divide a particular rock mass formation into groups of similar behaviour, rock mass classes of varying quality.
3. Provide a basis for understanding the characteristics of each rock mass class.
4. Relate the experience of rock conditions at one site to the conditions encountered at others.
5. Derive quantitative data and guidelines for engineering design.
6. Provide a common basis for communication between engineers and geologists.

In many projects, the classification approach serves as a practical basis for the design of underground structures (Bieniawski, 1989). Three rock mass classification systems were used to describe the rock mass characteristics at the Ingula Pumped Storage Scheme, namely the Q-system (Barton, 2002), the rock mass rating (RMR) system (Bieniawski, 1989), and the geological strength index (GSI) system (Marinos and Hoek, 2000).

The Rock Quality Designation index (RQD) developed by Deere (1968) is an input into the Q and RMR systems and is defined as the percentage of intact core pieces longer than 100 mm in the total. This index provides a measureable estimate of borehole core quality. The core should be at least NW size (54.7 mm in diameter) and should be drilled with a double-tube core barrel. Figure 2-1 shows the calculation of RQD.

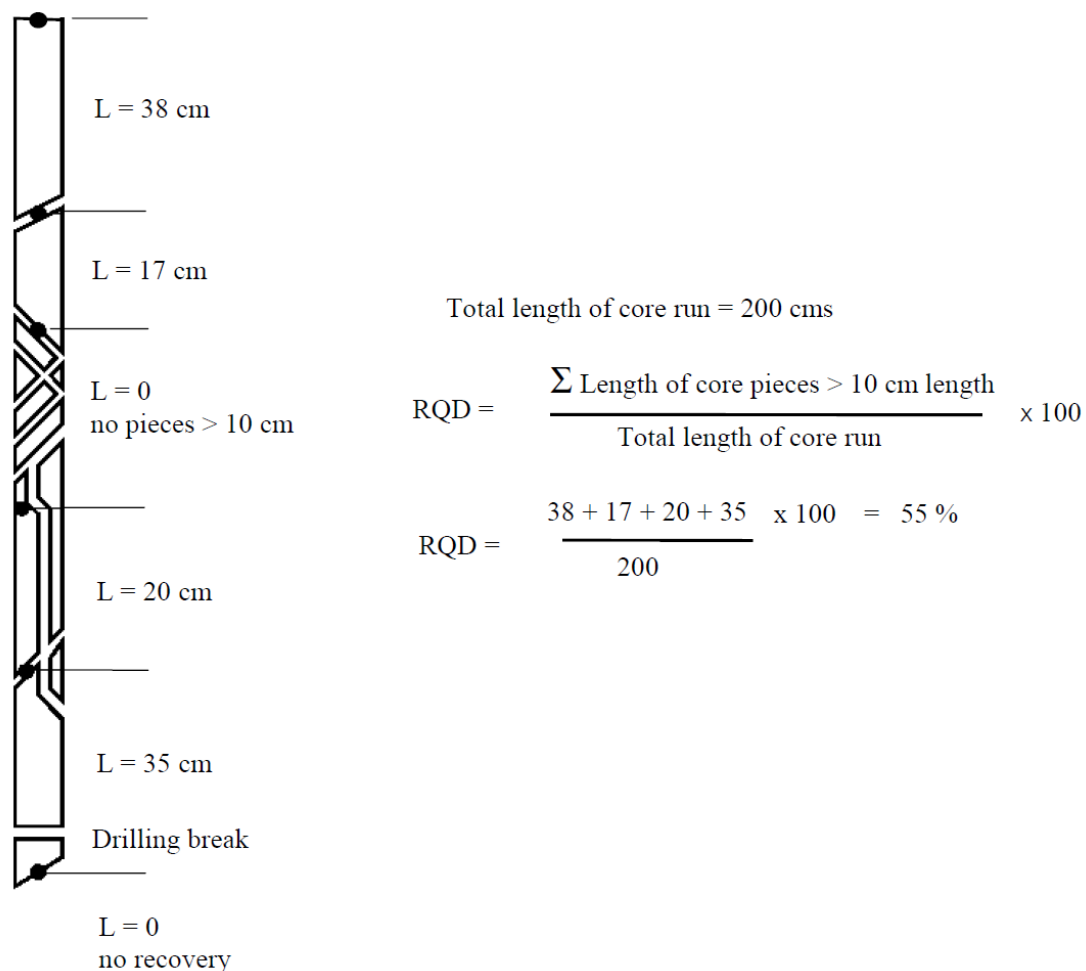


Figure 2-1: Procedure for measurement and calculation of RQD (Deere, 1968)

2.2.1 Q-system

The Q system of rock mass classification was developed by Barton, Lien and Lunde (1974) for the Norwegian Geotechnical Institute after the evaluation of two hundred underground excavation case histories. Rock mass characteristics and tunnel requirements can be determined from the system.

Six parameters are used to describe the rock mass quality namely:

RQD – Rock Quality Designation

J_n – joint set number

J_r – joint roughness number

J_a – joint alteration number

J_w – joint water reduction factor

SRF – Stress Reduction Factor

Input values for the six parameters are determined by visual examination of the rock outcrops, faces and borehole core and a value is then assigned using Table 2-1. The value of Q is calculated using the following equation:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad 2-1$$

The rock mass quality and conversion from actual Q-values to adjusted Q-values for design of wall support are shown on Table 2-2.

Barton (2002) explains the combination of the pairs of the parameters as follows. The first quotient (RQD/J_n) represents the rock mass structure and is a measure of the block size. This quotient is useful for distinguishing massive, rock-burst prone rock. The RQD is the rock quality designation which was discussed in the previous section. J_n defines the joint set number. Inexperienced users may describe a highly fractured rock mass to have few joint sets and a widely spaced jointed rock mass to consist of many joint sets. Cracks caused by drill and blast can also be mapped as a random joint set. This inexperience can make the joint set number not always a dependable gauge of the fracturing in the rock mass. Stereo-nets or joint rosettes should be used to define joint sets in the same location and therefore determine the joint set values for the whole project (Palmström and Broch, 2006).

The second quotient (J_r/J_a) represents the inter-block shear strength and describes the condition of the discontinuities. Joint wall roughness and joint wall characteristics such as the presence of fill material are also described by the quotient. *"This quotient is weighted in favour of rough, unaltered*

joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favourable to tunnel stability." (Brady and Brown, 2004).

Active stresses are represented by the third quotient (J_w/SRF), which is the relative effect of water, faulting, strength/stress ratio, squeezing or swelling. *"It has proved impossible to combine these two parameters in terms of inter-block effective stress, because paradoxically a high value of effective normal stress may sometimes signify less stable conditions than a low value, despite the higher shear strength."* (Brady and Brown, 2004). When determining the joint water reduction factor, the table does not give limits between medium, large and exceptionally high inflows. This may result in inaccurate characterization as each user will have to decide in each case. The stress reduction factor measures:

- A. Loosening load in the case weakness zones/faults intersecting the excavation
- B. Rock stresses in competent rock, and rock overstressing problems in brittle rocks
- C. Squeezing loads in plastic incompetent rocks
- D. Swelling loads from chemical reaction caused by water

Table 2-1: Description and parameter ratings of the Q-System from (Barton, 2002)

1. RQD (Rock Quality Designation)			RQD
A	Very poor	(> 27 joints per m^3)	0-25
B	Poor	(20-27 joints per m^3)	25-50
C	Fair	(13-19 joints per m^3)	50-75
D	Good	(8-12 joints per m^3)	75-90
E	Excellent	(0-7 joints per m^3)	90-100
Note: i) Where RQD is reported or measured as ≤ 10 (including 0) the value 10 is used to evaluate the Q-value ii) RQD-intervals of 5, i.e. 100, 95, 90, etc., are sufficiently accurate			

2. Joint Set Number		Jn
A	Massive, no or few joints 0.5-1.0	0.5-1
B	One joint set 2	2
C	One joint set plus random joints 3	3
D	Two joint sets 4	4
E	Two joint sets plus random joints 6	6
F	Three joint sets 9	9
G	Three joint sets plus random joints 12	12
H	Four or more joint sets, random heavily jointed "sugar cube", etc 15	15
J	Crushed rock, earth like 20	20
Note: i) For tunnel intersections, use 3 x Jn ii) For portals, use 2 x Jn		

3. Joint Roughness Number		Jr
a) Rock-wall contact, and b) Rock-wall contact before 10 cm of shear movement		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough, irregular, planar	1.5
F	Smooth, planar	1
G	Slickensided, planar	0.5
Note: i) Description refers to small scale features and intermediate scale features, in that order		
c) No rock-wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock-wall contact when sheared	1
Note: ii) Add 1 if the mean spacing of the relevant joint set is greater than 3 m (dependent on the size of the underground opening) iii) Jr = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented in the estimated sliding direction		

4. Joint Alteration Number		Φ_r Approx.	Ja
a) Rock-wall contact (no mineral fillings, only coatings)			
A	Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote.		0.75
B	Unaltered joint walls, surface staining only.	25-35°	1
C	Slightly altered joint walls. Non-softening mineral coatings; sandy particles, clay-free disintegrated rock, etc.	25-30°	2
D	Silty or sandy clay coatings, small clay fraction (non-softening).	20-25°	3
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
b) Rock-wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, etc.	25-30°	4
G	Strongly over-consolidated, non-softening, clay mineral fillings (continuous, but <5 mm thickness).	16-24°	6
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but <5 mm thickness).	12-16°	8
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but <5 mm thickness).	6-12°	8-12

	Value of J_a depends on percent of swelling clay-size particles.		
<i>c) No rock-wall contact when sheared (thick mineral fillings)</i>			
K	Zones or bands of disintegrated or crushed rock. Strongly over-consolidated	16-24°	6
L	Zones or bands of clay, disintegrated or crushed rock. Medium or low over-consolidation or softening fillings.	12-16°	8
M	Zones or bands of clay, disintegrated or crushed rock. Swelling clay. J_a depends on percent of swelling clay-size particles	6-12°	8-12
N	Thick continuous zones or bands of clay. Strongly over-consolidated.	12-16°	10
O	Thick, continuous zones or bands of clay. Medium to low over-consolidation	12-16°	13
P	Thick, continuous zones or bands with clay. Swelling clay. J_a depends on percent of swelling clay-size particles.	6-12°	13-20

5. Joint Water Reduction Factor		J_w
A	Dry excavations or minor inflow (humid or a few drips)	1.0
B	Medium inflow, occasional outwash of joint fillings (many drips/"rain")	0.66
C	Jet inflow or high pressure in competent rock with unfilled joints	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	0.33
E	Exceptionally high inflow or water pressure decaying with time. Causes outwash of material and perhaps cave in	0.2-0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay. Causes outwash of material and perhaps cave in	0.1-0.05
Note: i) Factors C to F are crude estimates. Increase J_w if the rock is drained or grouting is carried out ii) Special problems caused by ice formation are not considered		

6. Stress Reduction Factor		SRF
<i>a) Weak zones intersecting the underground opening, which may cause loosening of rock mass</i>		
A	Multiple occurrences of weak zones within a short section containing clay or chemically disintegrated, very loose surrounding rock (any depth), or long sections with incompetent(weak) rock (any depth). For squeezing, see 6L and 6M	10
B	Multiple shear zones within a short section in competent clay-free rock with loose surrounding rock (any depth)	7.5
C	Single weak zones with or without clay or chemical disintegrated rock (depth ≤ 50m)	5
D	Loose, open joints, heavily jointed or "sugar cube", etc. (any depth)	5
E	Single weak zones with or without clay or chemical disintegrated rock (depth > 50m)	2.5

Note: i) Reduce these values of SRF by 25-50% if the weak zones only influence but do not intersect the underground opening				
<i>b) Competent, mainly massive rock, stress problems</i>		σ_c / σ_1	σ_θ / σ_c	SRF
F	Low stress, near surface, open joints	>200	<0.01	2.5
G	Medium stress, favourable stress condition	200-10	0.01-0.3	1
H	High stress, very tight structure. Usually favourable to stability. May also be unfavourable to stability dependent on the orientation of stresses compared to jointing/weakness planes*	10-5	0.3-0.4	0.5-2 2-5*
J	Moderate spalling and/or slabbing after > 1 hour in massive rock	5-3	0.5-0.65	5-50
K	Spalling or rock burst after a few minutes in massive rock	3-2	0.65-1	50-200
L	Heavy rock burst and immediate dynamic deformation in massive rock	<2	>1	200-400
Note: ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1 / \sigma_3 \leq 10$, reduce σ_c to $0.75 \sigma_c$. When $\sigma_1 / \sigma_3 > 10$, reduce σ_c to $0.5 \sigma_c$, where σ_c = unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_θ = maximum tangential stress (estimated from elastic theory) iii) When the depth of the crown below the surface is less than the span; suggest SRF increase from 2.5 to 5 for such cases (see F)				
<i>c) Squeezing rock: plastic deformation in incompetent rock under the influence of high pressure</i>			σ_θ / σ_c	SRF
M	Mild squeezing rock pressure		1-5	5-10
N	Heavy squeezing rock pressure		>5	10-20
Note: iv) Determination of squeezing rock conditions must be made according to relevant literature				
<i>d) Swelling rock: chemical swelling activity depending on the presence of water</i>				SRF
O	Mild swelling rock pressure			5-10
P	Heavy swelling rock pressure			10-15

7. Type of Excavation		ESR
A	Temporary mine openings, etc.	ca. 3-5
B	Vertical shafts*: i) circular sections ii) rectangular/square section * Dependant of purpose. May be lower than given values.	ca. 2.5 ca. 2.0
C	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks) water supply tunnels, pilot tunnels, drifts and headings for large openings.	1.6
D	Minor road and railway tunnels, surge chambers, access tunnels, sewage tunnels,	1.3

	etc.	
E	Power houses, storage rooms, water treatment plants, major road and railway tunnels, civil defence chambers, portals, intersections, etc.	1.0
F	Underground nuclear power stations, railways stations, sports and public facilities, factories, etc.	0.8
G	Very important caverns and underground openings with a long lifetime, ≈ 100 years, or without access for maintenance.	0.5

Table 2-2: Conversion from actual Q-values to adjusted Q-values for design of wall support (Grimstad and Barton, 1993)

In rock masses of good quality	$Q > 10$	Multiply Q-values by a factor of 5.
For rock masses of intermediate quality	$0.1 < Q < 10$	Multiply Q-values by a factor of 2.5. In cases of high rock stresses, use the actual Q-value.
For rock masses of poor quality	$Q < 0.1$	Use actual Q-value

Limitations and applications

Since the development of the Q system in 1974, for engineering classification of rock masses for the design of tunnel support, there have been numerous major developments of the system (Palmström and Broch, 2006). In 1977, the Q system was further developed for the estimation of support requirements for underground excavations. The Q system was also used for rock mass characterization. Grimstad and Barton (1993) updated Q system by adjusting the SRF values, new rock support methods, adjustment for narrow weakness zones and the Q estimated from seismic velocities. This update, further developed relations with the deformations in tunnels and caverns and the rock mass deformation modulus. These relations were updated in 1995 when the parameter Q_c was introduced. The parameter Q_c is the normalization of Q by a compressive strength different from 100 MPa ($Q_c = Q \times UCS / 100$). This parameter Q_c also correlated well with seismic velocities and deformation modulus. The Q_{TBM} system developed in 1999, is a method for predicting TBM single- shield and double-shield performance in jointed and faulted rock and for the estimation of support and rock reinforcement needs for TBM tunnels. Further developments of the Q-system were made by Barton in 2002, with some new correlations to assist in site characterization and tunnel design.

Over the past years, the Q system has been successfully applied in thousands of civil and mining engineering projects in many countries (Palmström and Broch, 2006). Apart from its comprehensive use in rock mass characterization and rock class determination, the Q system is linked to the Norwegian Method of Tunnelling (NMT) permanent tunnel support and reinforcement selection as

well as temporary support selection in the New Austrian Tunnelling Method (NATM). The logarithmic quality scale and some important parameters like joint set number (J_n), inter-block friction (J_r/J_a) and the evaluation of the stress/strength ratio (SRF) appear to give the Q system an advantage over the RMR system and the GSI system. Over-break amounts and subsequently shotcrete volumes are related to the J_n/J_r ratio when it is above or equal to 6. Q value is also linked to deformation and the deformation modulus and each are depth and/or stress dependent.

The Q-system is less useful for rock support descriptions during construction and is used more as an indicator and a source of useful information within the limitations of the system (Palmström and Broch, 2006). The accuracy of the Q-system is considered to be limited to good quality rock mass as there are limits to describe the RQD of the rock mass correctly. Grenon and Hadjigeorgiou (2003) stated the following about the RQD; *“RQD can often result in a sampling bias due to a preferential orientation distribution of discontinuities. A further limitation is that it cannot account for the size (length) of the considered discontinuities. Furthermore, RQD is insensitive when the total frequency is greater than $3m^{-1}$. Despite these limitations, RQD is used in its own rights and as an integral part of the more popular rock mass classification tools used in the mining industry (RMR and NGI). RQD is insensitive when the rock mass is moderately fractured. One has to keep in mind that RQD values are a function of the total frequency, which is highly sensitive to sampling line orientation”*. The correlation of RQD and the volumetric joint count is poor particularly where most of the pieces of core are around 0.1m long. While RQD is a practical parameter for core-logging, it is not adequate to provide sufficient description of a rock mass on its own (Bieniawski, 1984). The joint set number is not really a good indicator of jointing, as inexperienced users may give highly jointed rock masses a low joint set number and a high joint set number for widely spaced joint sets. Joint rosettes and stereo-nets do not represent the joint set number on the project as these show many joint sets than is present in one location. This has resulted in experienced users applying these for the whole project. As a result, Loset (1997) recommends that blasting cracks and very short joints from drill and blast excavations be treated as random joints. For active stress calculations, the limits for medium, large and exceptionally high water flows for the joint water are not given in the Q-system table, leaving each user to decide. This causes inaccuracy in the characterization of the rock mass.

As there are limits to describe the RQD of the rock mass correctly, good quality rock masses tend to require little or no support. Extremely poor to poor rock masses also tend to be difficult to describe. These rock masses occur in weakness zones, squeezing and swelling rocks and it is not reliable to depend only on the rock mass classification system for the support. For practical use Palmström and

Broch (2006) suggested that the support-diagram shown in Figure 2-2 is useful from the range of where Q is larger than 0.1 and smaller than 40. Stille and Palmström (2003) show that the Q -system is best used for jointed rock masses where instability is caused by block falls. Limitations occur for other types of underground rock conditions; this is shown on Table 2-3.

Table 2-3: The fitness of the Q -system in various types of ground behavior in tunnels (Stille and Palmström, 2003; Palmström and Broch, 2006)

Types of ground behaviour	Suitability (*)
Stable	2
Fall of block(s) or fragment(s)	1 - 2
Cave-in	3
Running ground	4
Buckling	3
Slabbing, spalling	2
Rock burst	3 - 4
Squeezing ground	3
Ravelling from slaking or friability	4
Swelling ground	3
Flowing ground	4
Water ingress	4
*) 1 Suitable; 2 Fair; 3 Poor; 4 Not applicable	

After undertaking mapping and classification of rock exposures by seventeen independent professionals in South Africa and Australia, Pells et al. (2017) noticed major discrepancies in the evaluated RQD values which gave rise to substantial errors in computed Q and RMR ratings and also in the Geological Strength Index (GSI) and Mining Rock Mass Rating (MRMR). These discrepancies prompted a review in the definition and origin of RQD. From the review it was seen that the greatest source of discrepancies in the RQD value originated from some engineers or geologists disregarding the “sound and hard” criterion in the RQD definition. Hencher (2014) also noticed this inconsistency in definition and intent where, in many circumstances, cored borehole data is unobtainable and RQD is estimated from exposed rock mass faces, or RADAR, or photographs. Additional long-recognised difficulties with the use and measurement of RQD include (Foster, 2015):

- measurements are not taken upon exposure in the core barrel splits, leading to incipient fractures opening up and lower RQD being logged than characteristic of the ground in situ,

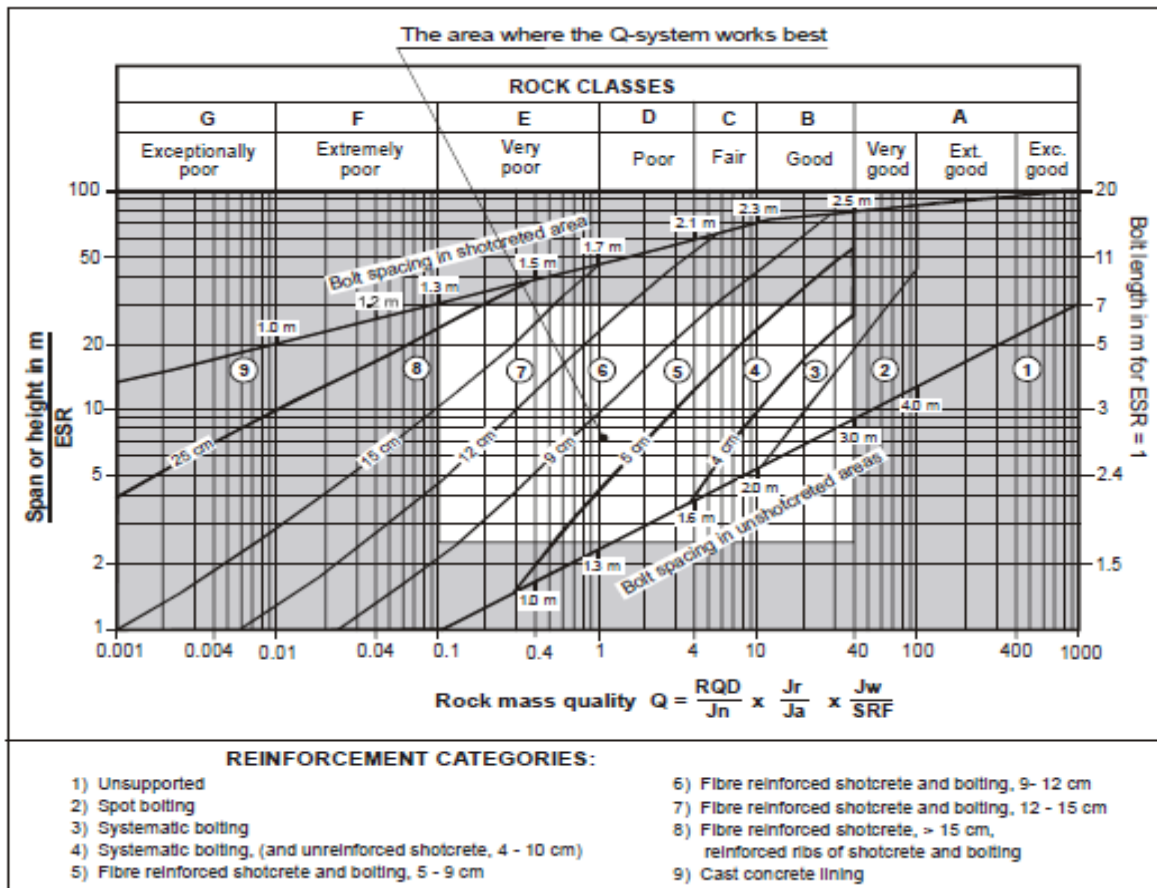


Figure 2-2: Limitations in the Q rock support diagram. Outside the unshaded area supplementary methods/evaluations/calculations should be applied (Palmström and Broch, 2006)

- standard practice is to maintain the original recommendation and measure RQD by core run, although Deere and Deere (1989) do propose logging by lithology as being suitable,
- under-sampling of defects where the geology is dominated by discontinuities near-parallel to the borehole,
- a misunderstanding with regards to the definition of ‘natural mechanical fractures’ within some rock types such as phyllite, shales and schists, and
- a misunderstanding in that when dealing with distinct emergent joints that have tensile strength should be disregarded when calculating RQD.

This has resulted in engineers and geologists adopting the relationship by Palmström (2005) between RQD and volumetric joint spacing (Jv):

$$\text{RQD} = 110 - 2.5J_v \quad (\text{for } J_v = 4 \text{ to } 44) \quad 2.2$$

or that of Priest and Hudson (1976) between defect frequency and RQD, namely:

$$\text{RQD} = 110.4 - 3.8/x \quad 2.3$$

where, x is the mean spacing of defects assuming an exponential distribution.

Based on an assessment of intrinsic restrictions of RQD, the inconsistent changes in definition, the increasing conception of RMR system and GSI system and wide multi-operator field experimentation, Pells et al. (2017) concluded that RQD should be done away with in rock mass classification, specifically regarding:

- 1 The RQD definition is not consistent worldwide, and in many countries the definition has changed from the original methodology and logic of its author, Don Deere.
- 2 Most dominant classification systems such as, RMR, Q, GSI and MRMR, require the estimation of RQD from rock exposures, for their application. Pells et al. (2017) has illustrated with the factual data that this process is filled with error and personal bias.
- 3 The intrinsic restrictions of RQD have already been acknowledged by the original authors of the RMR and MRMR systems, who have endorsed that it be substituted by fracture frequency.
- 4 It has been illustrated that GSI can be projected from Hoek's look-up chart as precisely as calculated from its components which include RQD.

2.2.2 Rock Mass Rating system

The Rock Mass Rating (RMR) system, also known as the Geomechanics Classification, was originally created by Bieniawski in 1972 and he continued to refine it until 1989 (Bieniawski, 1989). This system considers the following six parameters:

1. Uniaxial compressive strength of intact rock
2. RQD
3. Spacing of discontinuities
4. Condition of discontinuities
5. Ground water conditions
6. Orientation of discontinuities

These six parameters are given ratings as per the RMR table, shown on Table 2-4. The rock mass rating is calculated by adding the ratings given to the six parameters. The sum of the ratings given to each parameter are divided into rock mass classes from very good rock to very poor rock and each class has a recommended support as shown on Table 2-5. A positive rating is given to the first five

parameters and the sixth parameter is an adjustment that considers the effect of the discontinuities with respect to the excavation.

Application and limitations

The initial 1973 RMR system was only applicable to tunnelling, as merely forty-nine case studies were investigated (Bieniawski, 1989). With an increase of the number of case studies, the use of the system also extended to dam foundations, mining and slopes. When using this system, users need to be cognisant of the modifications that were made to the system.

The application of the RMR system mainly depends on observations and experts' experience which can result in extremely safe or unsafe conditions. Overdesign of the support system can also occur as a result of the conservative output of the RMR system. The in situ state of stress of the rock mass is only taken into account indirectly by the selection of support measures. At depths greater than 1000m, the consideration of the in situ stress becomes more important. The Modified Rock Mass Rating (MRMR) was developed to take in situ stress into account (Bieniawski, 1989).

Lowson and Bieniawski (2013) critically assessed the RMR based on an Engineer's approach to tunnel design. In their paper the authors dealt with five aspects of tunnel design namely:

1. new design charts for rock bolt, shotcrete and steel ribs support as a function of excavation span and rock mass quality,
2. deciding on tunnel shape and secondary liners,
3. RMR ratings graphs updated for practical applications,
4. refining prediction of *in situ* modulus of deformation based solely on rock mass quality , and
5. a new approach to handling conditions of squeezing ground and rock bursting.

Practical engineers require design charts and simple aids to practical design, with respect to modelling versus empirical assessments (Lowson and Bieniawski, 2013). Numerical modelling does not model the support effect for interlocking blocks well, unless a discontinuum model is used. Nevertheless, engineers have expressed enthusiasm in design rules for the evaluation of support requirements for both rock bolts and shotcrete, as they are often confronted with the predicament of a wide range of discontinuity parameters from laboratory tests that show tunnel stability on one hand and instability on the other.

Support guidelines based on RMR were originally determined based on a table for an excavation span of 10 metres (Bieniawski, 1989). Lawson and Bieniawski (2013) created easy to use charts to determine the preliminary support for excavations as a function of excavation spans.

Table 2-4: Rock Mass Rating (Bieniawski, 1989)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of		> 2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		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	Stickensided 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	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water pressure) (Major principal σ)	0	< 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
		Rating	15	10	7	4	0		
B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)									
Strike and dip orientations			Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable		
Ratings	Tunnels & mines	0	-2	-5	-10	-12			
	Foundations	0	-2	-7	-15	-25			
	Slopes	0	-5	-25	-50				
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS									
Rating			100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21		
Class number			I	II	III	IV	V		
Description			Very good rock	Good rock	Fair rock	Poor rock	Very poor rock		
D. MEANING OF ROCK CLASSES									
Class number			I	II	III	IV	V		
Average stand-up time			20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span		
Cohesion of rock mass (kPa)			> 400	300 - 400	200 - 300	100 - 200	< 100		
Friction angle of rock mass (deg)			> 45	35 - 45	25 - 35	15 - 25	< 15		
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions									
Discontinuity length (persistence)			< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m		
Rating			6	4	2	1	0		
Separation (aperture)			None	< 0.1 mm	0.1 - 1.0 mm	1 - 5 mm	> 5 mm		
Rating			6	5	4	1	0		
Roughness			Very rough	Rough	Slightly rough	Smooth	Stickensided		
Rating			6	5	3	1	0		
Infilling (gouge)			None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm		
Rating			6	4	2	2	0		
Weathering			Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed		
Ratings			6	5	3	1	0		
F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**									
Strike perpendicular to tunnel axis					Strike parallel to tunnel axis				
Drive with dip - Dip 45 - 90°			Drive with dip - Dip 20 - 45°		Dip 45 - 90°			Dip 20 - 45°	
Very favourable			Favourable		Very unfavourable			Fair	
Drive against dip - Dip 45-90°			Drive against dip - Dip 20-45°		Dip 0-20 - Irrespective of strike*				
Fair			Unfavourable		Fair				

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

** Modified after Wickham et al (1972).

Table 2-5: Guidelines for excavation and support of 10m span rock tunnels in accordance with the RMR system (Bieniawski 1989)

Rock mass	Excavation	Rock bolts (20mm diameter,- fully grouted)	Shotcrete	Steel Sets
I- Very good rock RMR:81-100	Full face, 3m advance	Generally no support required except spot bolting		
II-Good rock RMR:61-80	Full face , 1-1.5m advance. Complete support 20m from face	Locally, bolts in crown 3m long, spaced 2.5m with occasional wire mesh	50mm in crown where required	None
III-Fair rock RMR: 41-60	Top heading and bench 1.5-3mm advance in top heading Commence support after each blast Complete support 10m from face	Systematic bolts 4m long, spaced 1.5-2m in crown and walls with wire mesh in crown	50-100mm in crown and 30mm in sides	None
IV- Poor rock RMR:21-40	Top heading and bench 1-1.5mm advance in top heading Install support concurrently with excavation, 10m from face	Systematic bolts 4.5m long, spaced 1-1.5m in crown and walls with wire mesh	100-150mm in crown and 100mm in sides	Light to medium ribs spaced 1.5m where required.
V- Very poor rock RMR: <20	Multiple drifts 0.5-1.5mm advance in top heading Install support concurrently with excavation, shotcrete as soon as possible after blast	Systematic bolts 5-6m long, spaced 1-1.5m in crown and walls with wire mesh. Bolt invert	100-200mm in crown and 150mm in sides and 50mm on face	Medium to heavy spaced 0.75m with steel lagging and fore poling if required – close invert

Rock load on a support system is a function of rock mass condition and initial stress state (Lowson and Bieniawski, 2013). The design rock load where the rock mass is not overstressed and squeezing is not occurring, is expressed as:

$$\text{Design Rock Load: } P_r = \frac{100-RMR}{100} \cdot 10m \cdot \left(\frac{\text{span}}{10m}\right)^{1/2} \cdot \rho_r \cdot \gamma_r \quad 2-4$$

where γ_r is a partial factor and ρ_r is rock density. The resultant graphical relationship for $\gamma_r = 1.5$ and $\rho_r = 27\text{kN/m}^3$ is shown in Figure 2-3, where rock loading increases as RMR decreases with a

maximum excavation span of 10m. A modification factor, which compensates for thickness of any loosened zone, is applied for different sizes so maximum load is dependent on the square root of the span. The rock load is for gravity induced conditions where the rock mass is not overstressed. Figure 2-4 shows the relationship between the rock height and the span.

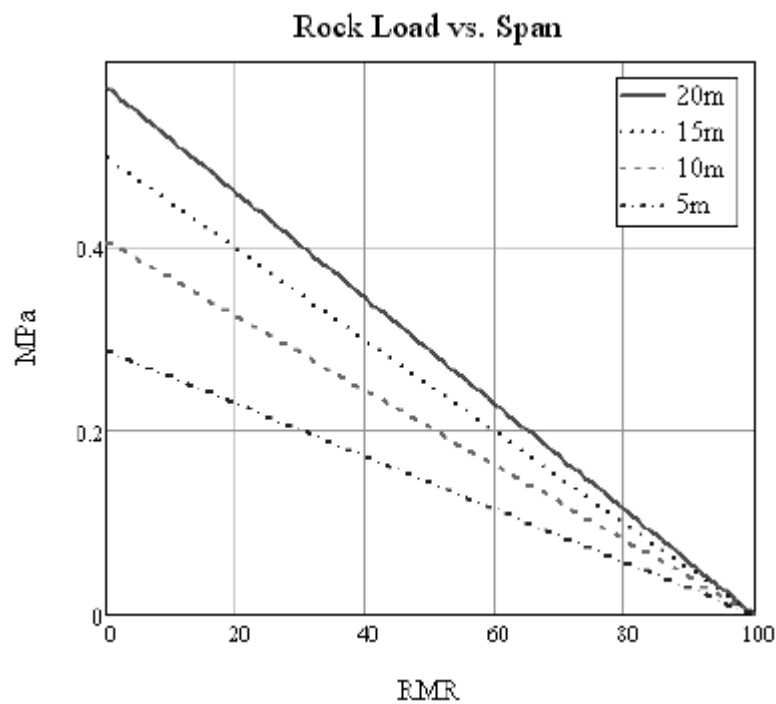


Figure 2-3: Rock load versus span (Lowson and Bieniawski, 2013)

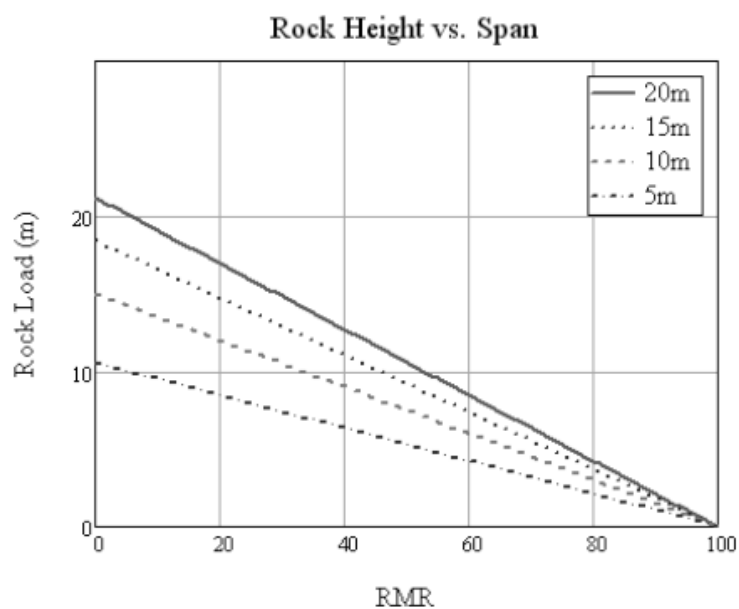


Figure 2-4: Height of supported rock versus span (Lowson and Bieniawski, 2013)

In some conditions, depending on the rock quality and tunnel span, tunnel support may not be necessary as is evident from Figure 2-5 (Lowson and Bieniawski, 2013).

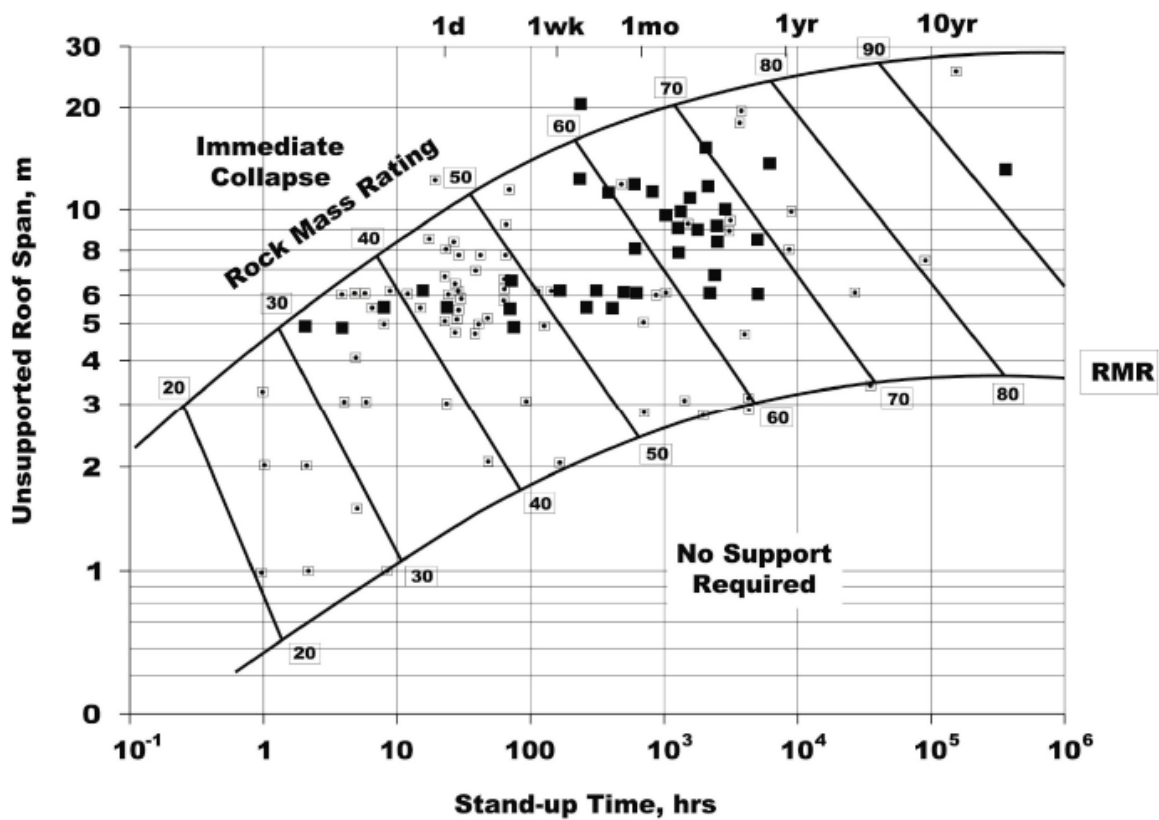


Figure 2-5: Stand up time of an unsupported tunnel span, as a function of rock mass quality RMR. (Lowson and Bieniawski, 2013)

Rockbolt spacing is solely considered as a function of RMR (Lowson and Bieniawski, 2013). Discontinuity frequency has to be reflected in the bolt spacing and the requirement for shotcrete to supply sufficient support between the bolts at the better rock end of the scale where the shotcrete cannot be perceived to work independently. Spot bolting is only expected to be necessary when RMR is equal to or larger than 85. For other RMR values rock bolt spacing is determined using the following functions:

$$S_b = 0.5m + 2.5m \cdot \frac{RMR-20}{65} \quad \text{if } 20 < RMR \leq 85 \quad \text{2-5}$$

$$S_b = 0.25m + \frac{(RMR-10)^{1.5}}{140} \cdot m \quad \text{if } 10 < RMR \leq 20 \quad \text{2-6}$$

$$S_b = 0.25m \quad \text{if } RMR \leq 10 \quad \text{2-7}$$

Where, S_b is the rock bolt spacing.

Rock bolt length varies with RMR and span (Lowson and Bieniawski, 2013). Function 2-8 was obtained, based on empirical guidelines applied in mining and numerical modelling studies results.

$$Span = \frac{(L_b + 2.5) \frac{RMR + 25}{52}}{3.6} \quad 2-8$$

where L_b is the embedded bolt length in metres. The rock bolt length chart is shown in Figure 2-6

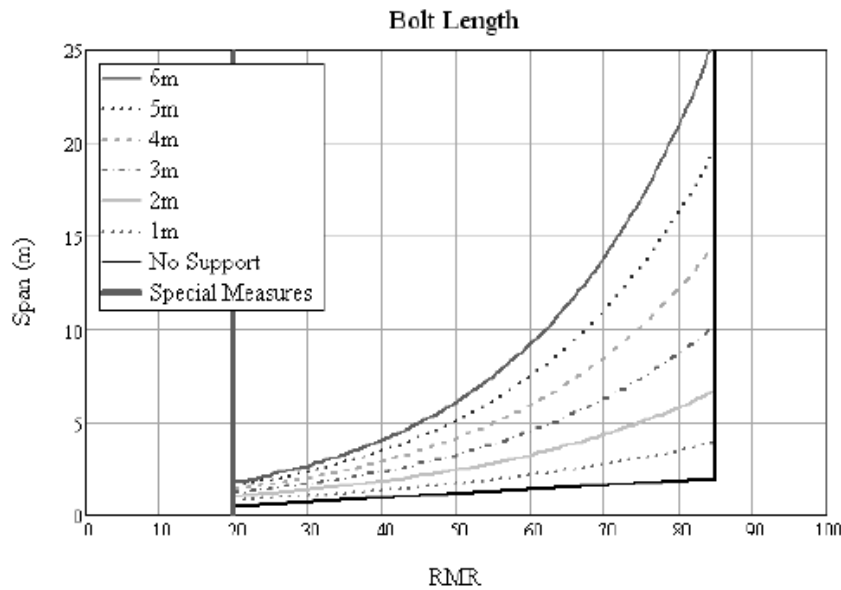


Figure 2-6: Rockbolt length chart (Lowson and Bieniawski, 2013)

The support capacity of pattern rock bolting is assumed to be determined by dividing the capacity of each bolt by the area it has to support (Lowson and Bieniawski, 2013). Rock bolt capacity decreases as RMR decreases due to the difficulty of obtaining an effective bond and the reducing efficiency of the bolts in mobilizing arching action in the rock mass. Equation 2-9 shows relationship developed to determine the rock bolt capacity. Figure 2-7 shows capacity for a typical 25mm rock bolt with an ultimate strength of 25 tons.

$$\text{Rock bolt capacity: } F_{bd} = \frac{F_b}{\gamma_b} \cdot \left(\frac{RMR}{85} \right)^{\frac{40}{RMR}} \quad 2-9$$

where, F_b is ultimate tensile capacity of bolt and γ_b is a partial factor.

The theory of the shotcrete acting as an arch in compression forms the basis of shotcrete design capacity. This type of support is described by the following formula:

$$\text{Support Pressure} = \text{Thickness} \times \text{design strength} / \text{Radius} \quad 2-10$$

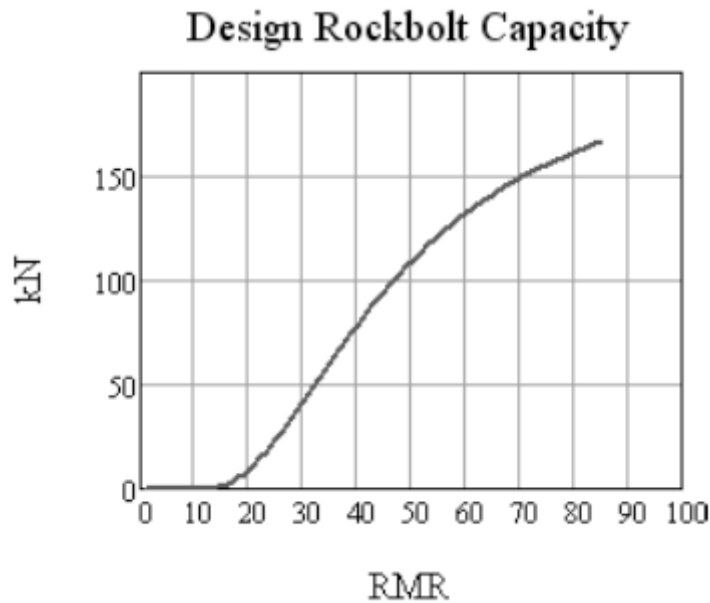


Figure 2-7: Design rock bolt capacity versus RMR, for: $F_b = 250\text{kN}$ and $\gamma_b = 1.5$ (Lowson and Bieniawski, 2013)

Only a thin layer of shotcrete is required for good quality rocks, where RMR is larger than 60 (Lowson and Bieniawski, 2013). Where the rock mass is of medium quality, RMR is between 30 and 60, the shotcrete can be thicker and is thought to be acting as an arch. The tunnel shape will be horseshoe with curved sidewalls. Multiple headings are required for RMR values of below 35 unless the tunnel is small. Equation 2-11 has been developed for the shotcrete design capacity, which shows the actual behaviour of shotcrete around a tunnel and the requirements placed on it. Figure 2-8 shows the design shotcrete strength chart.

$$f_{cd} = \frac{f_{ck}}{\gamma_s} \left[0.2 + 0.8 \cdot \left(\frac{RMR}{100} \right)^{3/2} \right] \quad 2-11$$

Where, f_{ck} is shotcrete cylinder strength and γ_s is a partial factor.

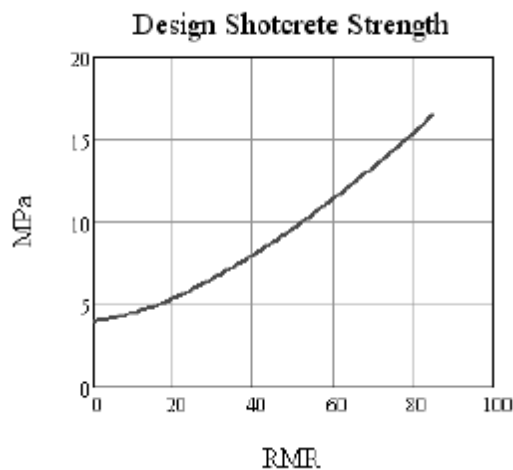


Figure 2-8: Design shotcrete strength for $f_{ck} = 30\text{ MPa}$, $\gamma_s = 1.5$ (Lowson and Bieniawski, 2013)

A chart relating span and RMR for a given thickness of shotcrete can be derived, using the above functions for the capacities of shotcrete and rock bolts.

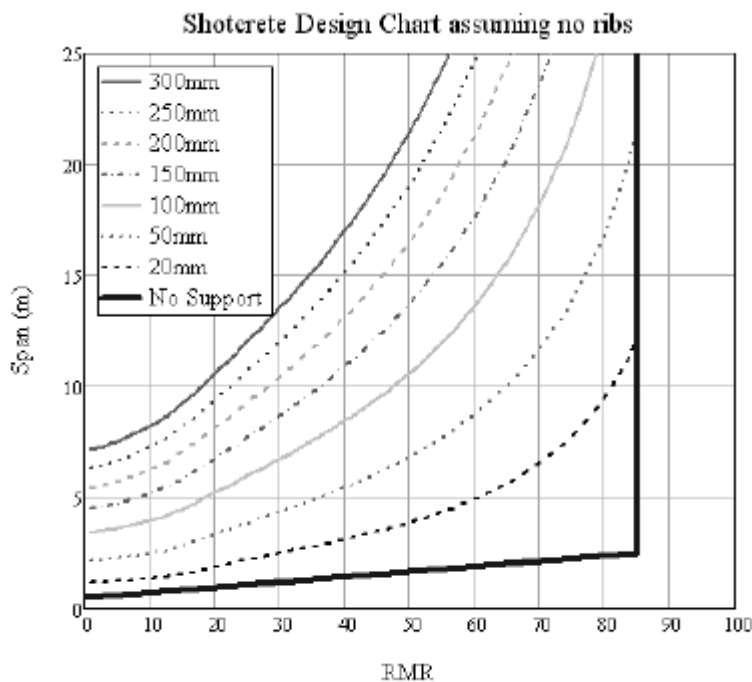


Figure 2-9: Shotcrete design chart (after Lowson and Bieniawski, 2013)

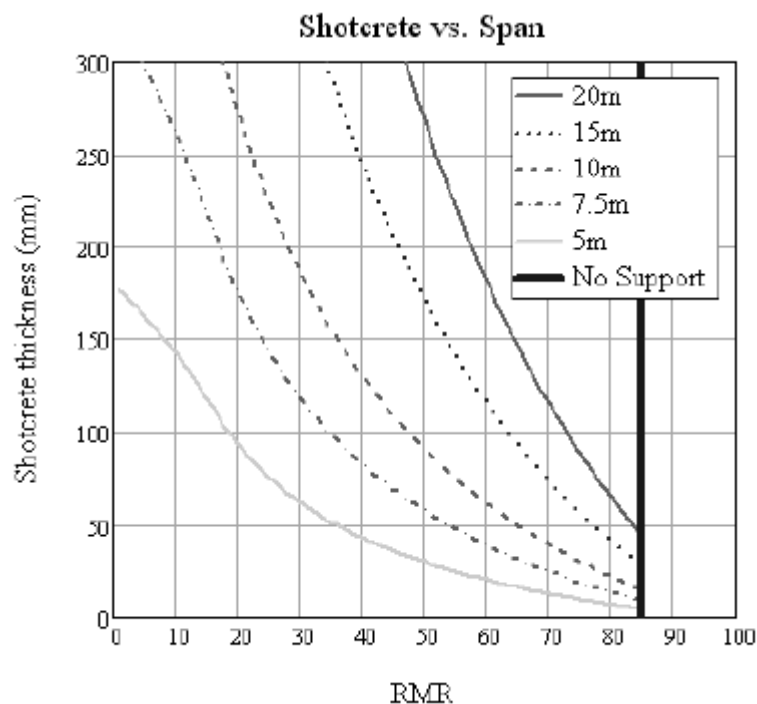


Figure 2-10: Shotcrete thickness for different tunnel spans (Lowson and Bieniawski, 2013)

Figures 2-9 and 2-10 are used for non-circular profiles where the shape of the tunnel above axis level is of a constant radius so that span is twice the radius.

Lowson (2012) developed the following equations for the capacity of blocked ribs:

$$\text{Elastic limit} \quad P_{el} = \frac{4 \cdot A_s \cdot I_s \cdot \sigma_y}{S_r \cdot r_i [4 \cdot I_s + A_s \cdot X \cdot r_i (1 - \cos(\theta))]} \quad 2-12$$

$$\text{Plastic limit} \quad P_{pl} = \frac{2 \cdot A_s \cdot S_{pl} \cdot \sigma_y}{S_r \cdot r_i [2 \cdot S_{pl} + A_s \cdot r_i (1 - \cos(\theta))]} \quad 2-13$$

Where, A_s , I_s , S_{pl} and X are section area, second moment of area, plastic section modulus, and section depth respectively, and S_r is rib spacing. The elastic and plastic limit relationships developed by Lowson (2012) yield the same result as the ultimate capacity for a small blocking angle which, is represented by the following formula:

$$P = A_s \cdot \sigma_y / r \quad 2-14$$

The capacity is reduced by the blocking angle as shown in Figure 2-11 for a typical steel rib arrangement consisting of 152 x 152 x 37kg/m UCS with 250MPa yield strength at 1m rib centres in a 10m tunnel (Lowson and Bieniawski, 2013). An increase in the blocking angle from zero to 15 degrees lessens the steel rib capacity by about 60%.

The thickness of shotcrete necessary to support the ground between the ribs is calculated by the following equation:

$$t = \sqrt{\frac{3 \cdot S_r^3 \cdot \rho_r \cdot \gamma_r \cdot \gamma_f}{4 \cdot f_{flex}}} \quad 2-15$$

Where, S_r is rib spacing, ρ_r is rock density, γ_r and γ_f are partial factors on loading and shotcrete flexural strength, and f_{flex} is the flexural tensile strength of the shotcrete. Figure 2-12 shows a chart of the shotcrete support versus the ribs.

Misconceptions have become evident over the years regarding the determination of the RMR parameter rating using the original 1989 chart (Bienawski, 2011). Several users of the system were unaware that the RMR parameter ratings in the original RMR rating table were average values for the ranges shown, and were not the minimum values. The use of the graphs is recommended, showing the continuous values of the ratings as illustrated in Charts A to C in Figure 2-13. With the use of the graphs, it is important to note that the minimum value for all ratings is zero so that the worst RMR rating is zero and not eight as some users may conclude with the use of the original RMR rating table. The use of RQD is also not recommended as RQD is difficult to determine at the tunnel face. Although Barton (2014) critiqued the RMR system for not considering joint set number, there is a correlation between the RQD and discontinuity spacing which can be used to determine the discontinuity density for RMR. RQD is determined using

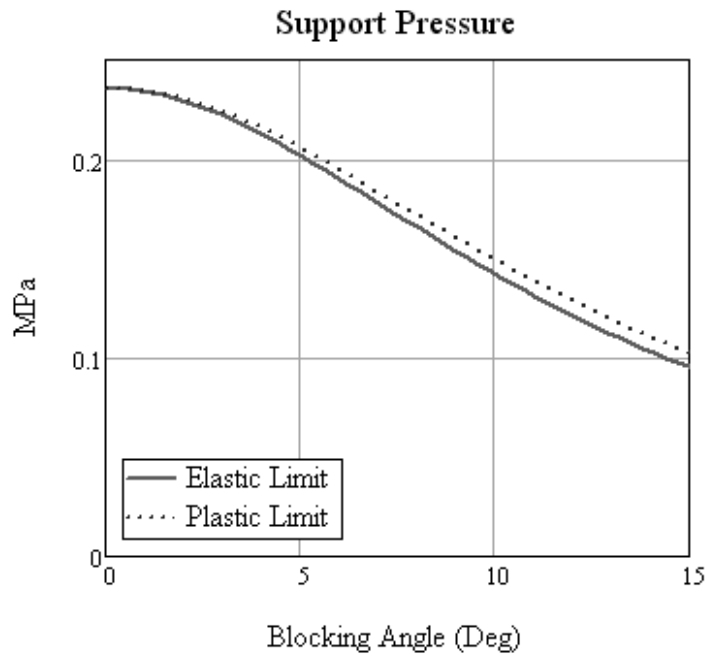


Figure 2-11: Rib Support vs. blocking angle (Lowson and Bieniawski, 2013)

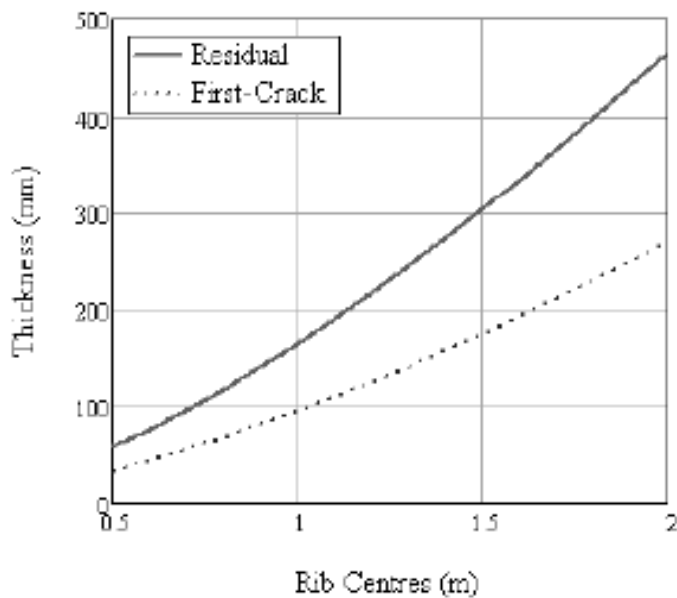


Figure 2-12: Shotcrete support between ribs (Lowson and Bieniawski, 2013)

borehole characterization and can be used to determine the discontinuity density when geological mappings are unavailable. On the other hand when geological mappings are available the discontinuity spacing can be used to determine the discontinuity density. This led to the use of discontinuity density for best practice as shown in Chart D.

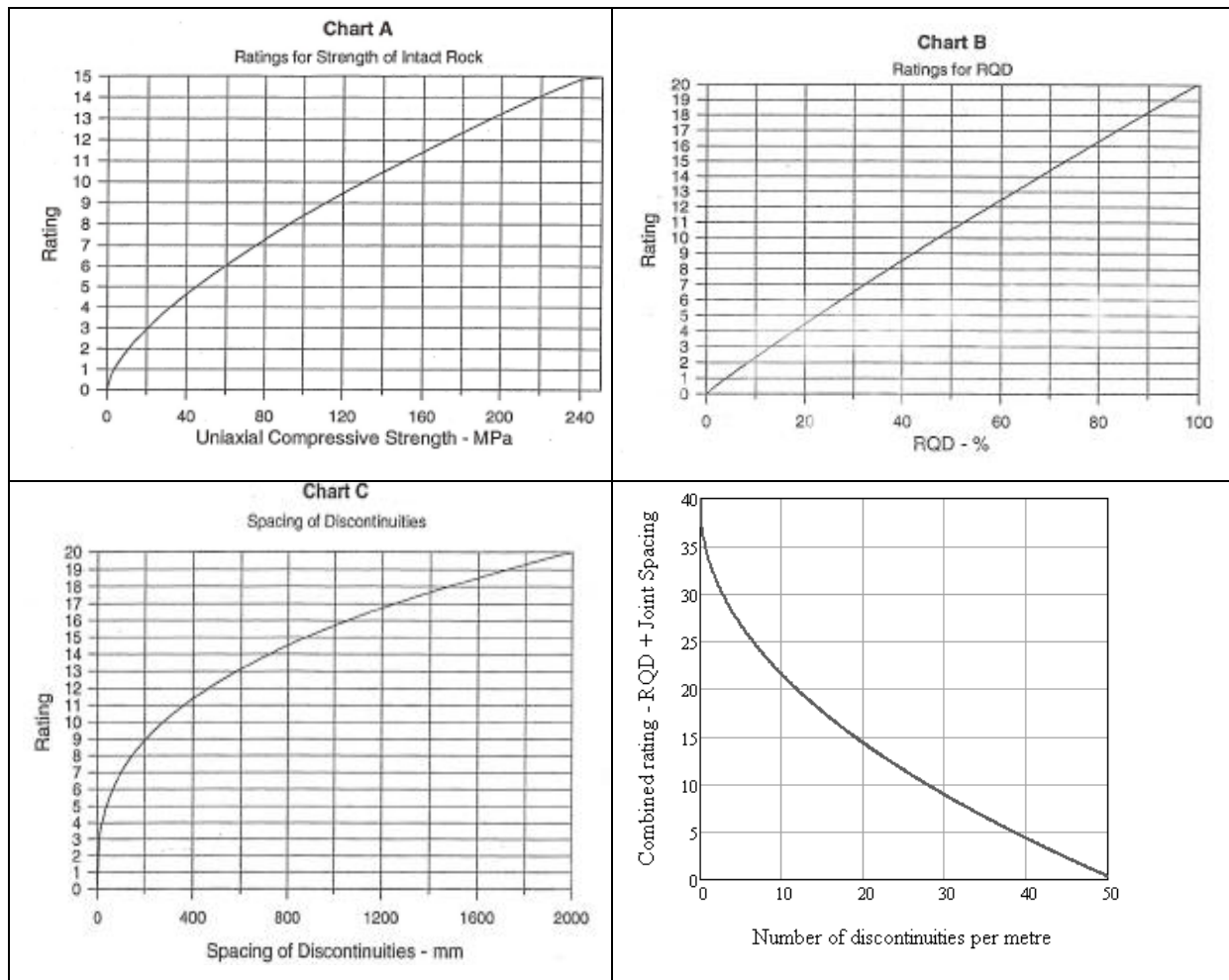


Figure 2-13: Chart A to C - Rating charts for RMR parameters (Bieniawski, 1989); Chart D – combined rating of the discontinuity density parameters (Lowson and Bieniawski, 2013)

Although some authors such as Pells and Bertuzzi (2007) are of the idea that classification systems should not be used as the main design tool of primary support, the use of both the RMR and Q system for underground excavations has been supported by both Barton and Bieniawski (2008). Moreover, Goodman (2007) in honour of Riedmuller went on to criticize the assignment of parameters of the two systems stating, *"...Engineers seem to be relying on generalised correlations of rock behaviour with rock mass ratings by Bieniawski's RMR and Barton's Q; yet the assignment of parameters is too schematic, the collection of data from exposures might not be adequately representative, and the assigned parameters are neither independent nor directly connected with rock behaviour."* After the collection of a number of case studies, shown in Figure 2-14, Bieniawski (1984) published a correlation between RMR and Q system:

$$\text{RMR} = 9 \ln Q + 44$$

2-16

The misconception that the RMR system could not be applied to weak rocks arose when Hoek et al. (1995) dealt with weak rock masses and concluded that the system did not work for weak to very weak rocks (Bieniawski, 2011). To overcome this Hoek developed the GSI system.

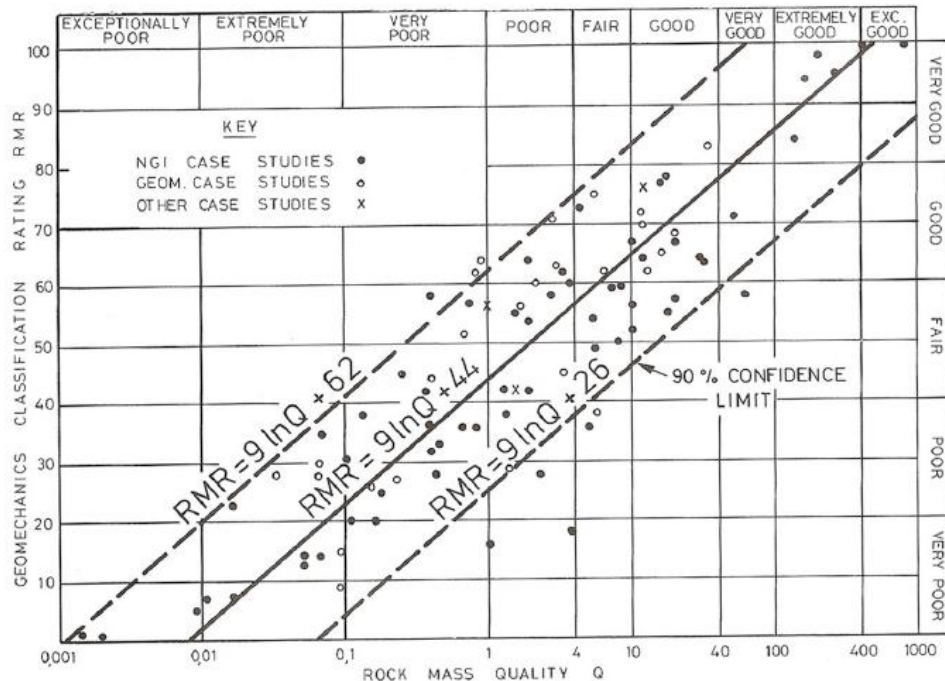


Figure 2-14: Correlation between RMR and Q (Bieniawski, 1984)

2.2.3 GSI system

After experiencing difficulties applying the RMR system to very poor quality rock mass, engineers and geologists had shown interest in a quick, easy to use reliable classification system (Singh and Goel, 2011). To overcome this Hoek and Brown (1997) developed the Geological Strength Index (GSI). This classification system is based on visual inspection of the rock mass and the condition of discontinuity surfaces of the rock mass. The GSI system focuses on the geological structure, the rock mass and estimates rock mass properties rather than rock mass reinforcement and support (Marinos et al., 2005). Hoek continued to develop the system and its correlation with the Hoek-Brown failure criterion but the system still only worked for a good rock mass. After dealing with difficult material Marinos and Hoek (2000) further developed the GSI to include poor rock mass. Figure 2-15 shows a simple chart devised by Marinos et al. (2005) to determine the GSI. Marinos and Hoek (2000) continued to work on the system so it could also be applied on to heterogeneous rocks.

GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS						
From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.						
STRUCTURE		SURFACE CONDITIONS				
		VERY GOOD Very rough, fresh unweathered surfaces				
		GOOD Rough, slightly weathered, iron stained surfaces				
		FAIR Smooth, moderately weathered and altered surfaces				
		POOR Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments				
		VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings				
		DECREASING SURFACE QUALITY →				
	INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities					
	BLOCKY - well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets					
	VERY BLOCKY- interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets					
	BLOCKY/DISTURBED/SEAMY - folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity					
	DISINTEGRATED - poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces					
	LAMINATED/SHEARED - Lack of blockiness due to close spacing of weak schistosity or shear planes					
		DECREASING INTERLOCKING OF ROCK PIECES ↓				

Figure 2-15: Geological strength Index for jointed Rock mass (Marinos et al., 2005)

Applications and limitations

The GSI system functions as an estimate of rock mass properties and was never intended to replace the RMR and Q-system (Marinos et al., 2005). This easy to use and geologically sound system combines two fundamental parameters of the geological process namely the joint condition and the rock mass structure. A GSI value is given to the rock mass upon inspection. This value can be used as an input in numerous empirical equations and used to determine the mechanical properties of a rock mass, especially the compressive strength of the rock mass (σ_{cm}) and its deformation modulus (E).

Marinos et al. (2005) discuss the applications and the limitations of application of the GSI system in their paper. In summary the GSI system is based on the assumption that the rock mass behaves as an isotropic mass. This system can therefore not be applied to an anisotropic rock mass. An example of an anisotropic rock mass is undisturbed slate, which should not be given a GSI value based on the GSI charts. However, when failure of such a rock mass is not controlled by its anisotropy both the Hoek-Brown criterion and the GSI charts can be used with caution. GSI values can also not be assigned to rock masses that have few discontinuities which are spaced at the same magnitude or larger than the excavation itself. This typically occurs at depths of 1000 or more metres or in strong hard rock. If the rock mass is highly weathered to an extent that the discontinuities in the rock mass are not visible the GSI index cannot be used and the rock mass must be assessed as a soil.

Bieniawski (2011) believes that the misconception that the RMR system was not applicable for weak rock masses led to the development of the GSI system. Although the developers of the GSI system indicated that the system was an index of rock mass classification and not a replacement of the Q and RMR systems, users have found it easier to use this easy to apply system. This has led to the inaccurate results obtained being used as 'reliable' inputs for numerical modelling. According to Bieniawski, the GSI is a useful method if correctly used, but he believes users should be aware of the index's limitations when estimating rock mass properties and when using available software without checking its approximations and assumptions.

In an interview Barton (2011) also expressed his concerns about the GSI system stating, " *My concern remains to actively include stress and water rather than 'externalise' them when classifying or characterizing. Of more concern to me these days is the absurdity of the algebraic equations linked to GSI (which is only RMR minus 5?) anyway. I do not believe, nor ever will believe, that one can look*

at a picture and 'classify' a rock mass. The children's-method of diagram recognition is entirely inappropriate to the challenges of describing the anisotropic water-bearing medium that we call rock masses. It is time to question this wide-spread method, and the absurdly complex algebra 'links' to parameters, that are not actually empirically based."

It is important to note that the only claimed function of the GSI was to estimate the rock mass strength using the Hoek-Brown criterion; especially for very weak rock masses. Brown (2008) reviewed the Hoek-Brown criterion, clarifying that the criterion was intended for peak rock mass strength estimations in preliminary studies of underground excavations in rock engineering projects. Three aspects deserving special caution were noted:

1. The Hoek-Brown criterion should not be used in rock masses where there are only one or two sets of joints.
2. The criterion may be confidently applied to a limited range of GSI values: care must be taken when dealing with brittle fractures in strong, massive rocks with high GSI values of over 75, and similarly, special care must be taken at *low values of GSI of below about 30, and low σ_c cases* as well as tectonically disturbed, heterogeneous rocks. Experienced professionals should be tasked with the estimation of GSI values in such cases as the Hoek-Brown criterion may not apply for very *weak rocks with $\sigma_c < 15$ MPa* since then the index a can be greater than its maximum value of 0.666 and can approach one, a value that is usually taken to apply to soil.
3. The disturbance factor D should be applied to the disturbed zone of a few metres only, instead of applying it to the entire rock mass which is subject to substantial errors.

2.3 Modulus of deformation

The deformation modulus of a rock mass is a significant input parameter in any deformation behaviour investigation of a rock mass (Hoek and Diederichs, 2006). The International Society for Rock Mechanics (ISRM) defines the deformation modulus as, the relation of stress to corresponding strain during loading of a rock mass, including elastic and inelastic behaviour (Palmström and Singh, 2001). Laboratory tests, in situ loading tests and empirical equations are the commonly used methods to estimate the deformation modulus. However, laboratory tests cannot yield reliable deformation modulus estimates as the sample sizes are small and limited due to the size of the equipment. In situ tests can be used to determine the modulus of deformation using the Plate Jacking Test (PJT), the Plate Loading Test (PLT) and the Radial Jacking Test, shown in Figure 2-16. These tests are expensive and not easy to conduct. It is also noted that it is very challenging to

depend on one in situ test as different results may be achieved in fairly uniform and good quality rock mass conditions. In situ tests are only conducted on site where accurate measures of deformation are crucial for the design and safe operation of an engineering structure.

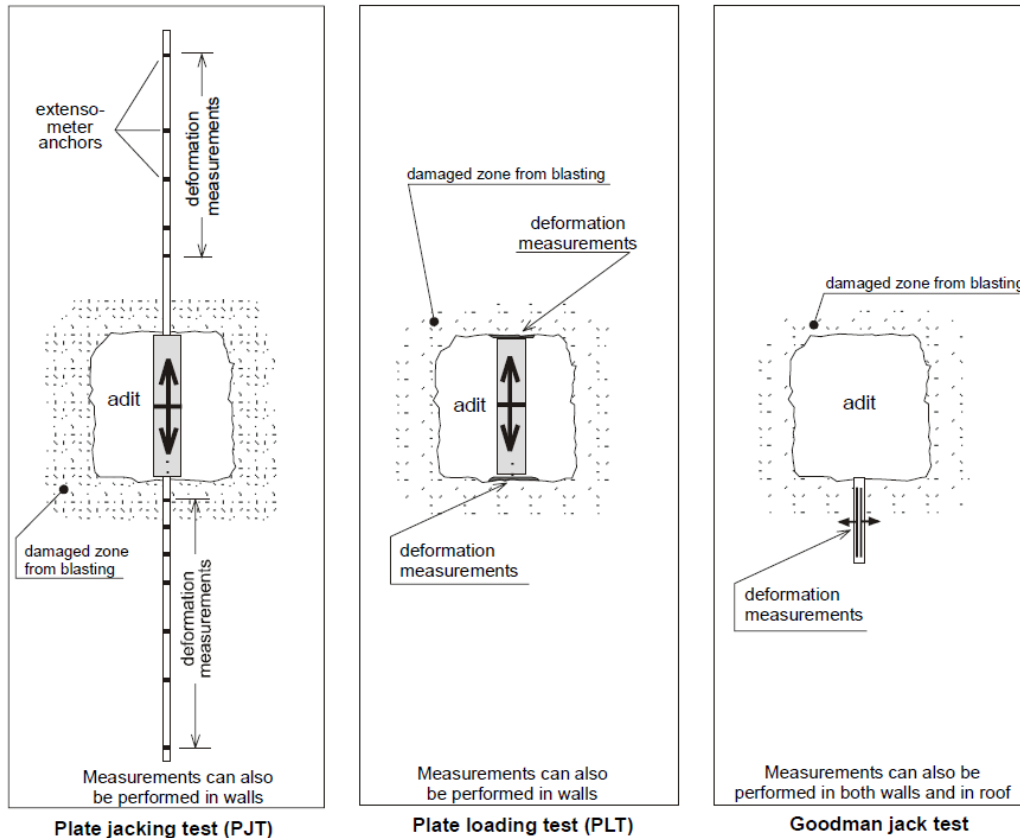


Figure 2-16: Principles of three main methods for in situ deformation measurement (Palmström and Singh, 2001)

2.3.1 Empirical methods of determining modulus of deformation

As a result of complexity and expense, the deformation modulus is often estimated from observed rock mass parameters (Palmström and Singh, 2001). Empirical relationships for estimating the value of an isotropic rock mass deformation modulus on the basis of classification schemes have been proposed by authors using rock mass classifications, such as the Rock Mass Rating (RMR), the Geological Strength Index (GSI), the Tunnelling Quality Index (Q) and the Rock Mass Index (RMI). Parameters which describe the quality of rock masses such as the Weathering Degree (WD) and Rock Mass Quality Designation (RQD) have also been used by other authors. The first empirical equation to determine the modulus of deformation was developed by Bieniawski (1978) using the RMR classification system. This was followed by other empirical equations such as, Serafim and Pereira

(1983), Nicholson and Bieniawski (1990), Mitri et al. (1994), Hoek and Brown (1997), Palmström and Singh (2001), Hoek and Diederichs (2006).

Statistical methods, such as regression analysis were generally used to derive the existing empirical equations. The reliability of these estimation equations is also dependent on the quality and the quantity of data used in the statistical analysis. As a result, the confidence in the projected value can be reduced due to large discrepancies in the predicted deformation modulus value caused by the use of different empirical equations. It should be noted that all empirical equations are open to additional development as a result of new data.

In a study conducted by Shen et al. (2012), the prediction performance of existing empirical equations using the RMR and GSI classification systems was evaluated. The high quality data from Bieniawski (1978), and Stephen and Banks (1989) were used. Shen et al. (2012) shows empirical equations developed by different authors using different classification systems as shown in Table 2-6. From the comparative analysis of the existing empirical equations it can be seen that the equations proposed by Read et al. (1999) and Hoek and Diederichs (2006) give the best predictions for the RMR and GSI category respectively. It is to be noted that the intact rock mass deformation modulus (E_i) is not included in both these equations. The empirical equations of Serafim and Pereira (1983), Hoek and Diederichs (2006), Sonmez et al. (2004), Hoek and Diederichs (2006) where E_i is considered, and Hoek and Brown (1997) will be used in this research project.

Bieniawski (1978) proposed the first correlation between RMR and rock deformation modulus as:

$$E_m(GPa) = 2 \cdot RMR - 100 \quad \mathbf{2-17}$$

The correlation was established from the results of extensive in situ rock deformability tests conducted, cautiously monitored and analysed.

Table 2-6: Empirical equations using RMR and GSI for predicting modulus of deformation.

(Shen et al., 2012)

Input Parameters	Empirical equations	
Group 1 RMR	Bieniawski (1978) Serafim and Pereira (1983) Mehrotra (1992) Read et al. (1999)	$E_m = 2RMR - 100, RMR > 50$ $E_m = 10^{(RMR-10)/40}$ $E_m = 10^{(RMR-\frac{20}{3})}$ $E_m = 0.1 \left(\frac{RMR}{10}\right)^3$
Group 2 RMR and E_i	Nicholson and Bieniawski (1990) Mitri et al. (1994) Sonmez et al. (2006)	$E_m = 0.01E_i(0.0028 RMR^2 + 0.9e^{\frac{RMR}{22.21}})$ $E_m = E_i[0.5 \left(1 - \left(\cos\left(\frac{\pi RMR}{100}\right)\right)\right)]$ $E_m = E_i 10^{((RMR-100)(100-RMR))/(4000 \exp(-\frac{RMR}{100}))}$
Group 3 GSI and D	Hoek et al. (2002) Hoek and Diederichs (2006)	$E_m = (1 - 0.5D)10^{\left(\frac{GSI-10}{40}\right)}, \sigma_{CI} > 100 \text{ MPa}$ $E_m(\text{MPa}) = 10^5 \left(\frac{1-0.5D}{1+e^{\left(\frac{75+25D-GSI}{11}\right)}}\right)$
Group 4 GSI, D and E_i	Sonmez et al. (2004) Hoek and Diederichs (2006)	$E_m = E_i(s^a)^{0.4}, s = \exp\left(\frac{GSI-100}{(9-3D)}\right), a = 0.5 + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right)$ $E_m = E_i\left(0.02 + \frac{1-0.5D}{1+e^{\left(\frac{60+15D-GSI}{11}\right)}}\right)$
Group 5 GSI, D and σ_{ci}	Hoek and Brown (1997) Hoek et al. (2002) Beiki et al. (2010)	$E_m = \sqrt{\frac{\sigma_{ci}}{100}} 10^{\left(\frac{GSI-10}{40}\right)}$ $E_m = (1 - 0.5D) \sqrt{\frac{\sigma_{ci}}{100}} 10^{\left(\frac{GSI-10}{40}\right)}, \sigma_{ci} \leq 100 \text{ MPa}$ $E_m = \tan\left(\sqrt{1.56 + (\ln(GSI))^2}\right) \sqrt[3]{\sigma_{ci}}$

Later, Serafim and Pereira (1983) used in situ data that they had collected to derive their popular empirical equation:

$$E_m \text{ (GPa)} = 10^{\frac{(RMR-10)}{40}} \quad 2-18$$

Both equations and their comparison are shown in Figure 2-17. From the graph it can be seen that the Bieniawski (1978) equation is suitable when RMR is larger than 50 whereas the Serafim and Pereira (1983) is suitable for the full range of RMR less than and equal to 50.

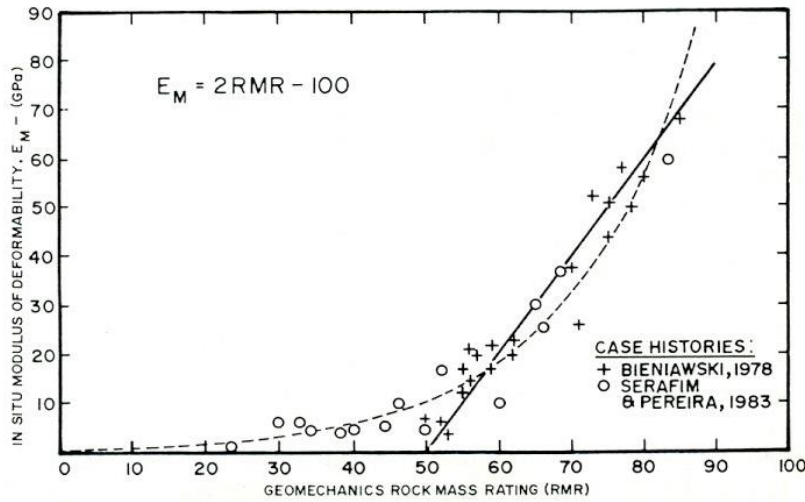


Figure 2-17: Correlation between RMR and modulus of deformation of rock masses E_m (Bieniawski, 1989)

Palmström and Singh (2001) recommended the equations shown in Figure 2-17 as they represent the best fit to experimental data and indicate a realistic approach. The use of two arrays of RMR has the benefit that modulus values are not underestimated at the lower array nor overestimated at the higher array.

The Serafim and Pereira (1983) proposed empirical relationship worked well for better quality rocks, however predicted high deformation modulus values for poor quality rock mass. To resolve this problem, Hoek and Brown (1997) amended the Serafim and Pereira (1983) empirical relationship as follows:

$$E_m(\text{GPa}) = \sqrt{\frac{\sigma_{ci}}{100}} 10^{\left(\frac{GSI-10}{40}\right)} \quad 2-19$$

The modification was based on practical observations and back analysis of excavation behaviour in poor quality rock masses. It should be noted that the RMR was replaced with GSI in the equation and that E_m is reduced progressively as the value of σ_{ci} falls below 100. The decrease is based upon the rational that the deformation of better quality rock masses is controlled by the joints while, the deformation of the intact rock pieces affects the overall deformation process for poorer quality rock masses (Hoek and Brown, 1997).

Finally, Hoek and Diederichs (2006) used data from a large database and came up with their empirical equation. Relationships between the RMR and the GSI suggested by Hoek and Brown (1997) were applied to analyse the data. These relationships are as follows:

Pre-1990, $GSI = RMR_{76}$

Post 1990, $GSI = RMR_{89} - 5$

The in situ measurement from the data from China and Taiwan was plotted against GSI as shown in Figure 2-18.

Hoek and Diederichs (2006) made the following assumptions during the analysis of the data:

1. This data set represents the best collection of quality field data which is likely to be accessible to any researcher. Scatter in the data indicates inherent scatter in the values of GSI, intact rock properties, rock mass modulus E_m , and the effects of disturbance due to blasting and/or stress relief.
2. The upper limit of the data set shows the rock mass modulus of confined and intact rock such as that which would be present around a tunnel at depth.
3. The disturbance or damage that decreases the rock mass modulus from this upper limit can be denoted by the disturbance factor D introduced by Hoek et al. (2002).
4. The maximum rock mass modulus corresponds to the deformation modulus of massive rock which is characterised by a GSI value of 90–100.
5. The modulus estimates discussed by Hoek and Diederichs (2006) apply to isotropic rock masses.
6. In order to constrain the increase of modulus with increasing GSI, a sigmoid function is used as a basis for this analysis. This S shaped function has the following general formula:

$$y = c + \left(\frac{a}{1 + e^{\left(\frac{x - x_0}{b} \right)}} \right)$$

2-20

where, a , b and c are constants.

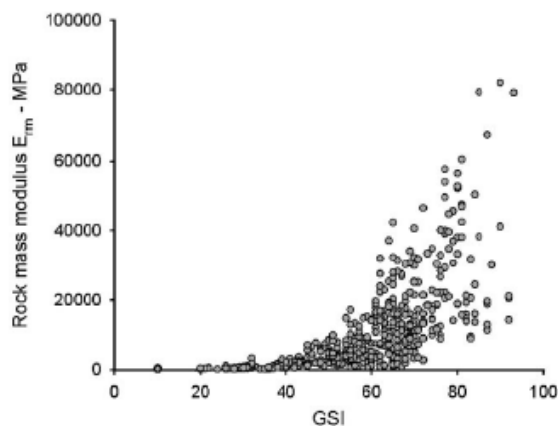


Figure 2-18: Measured rock mass modulus of deformation against GSI for Chinese and Taiwanese data (Hoek and Diederichs, 2006)

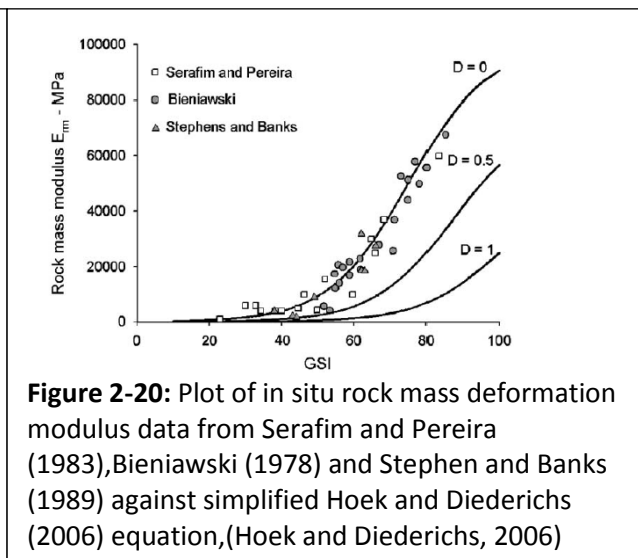
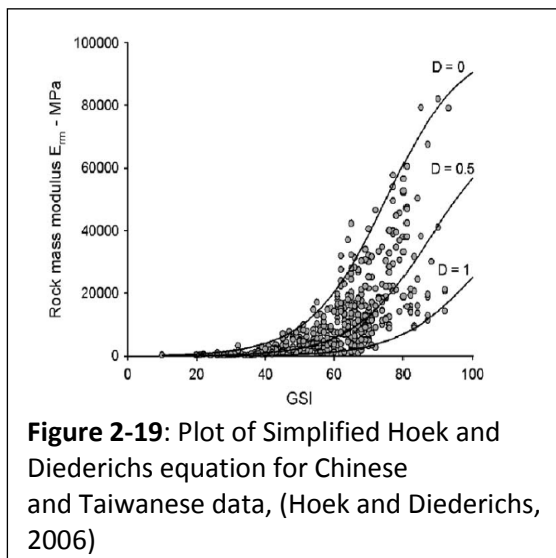
Curve fitting software was used to input the general equation used in the assumption to the data from China and Taiwan. GSI and disturbance factor D were fitted into the equation to replace the a and b constants. The GSI and the disturbance factor were also adjusted to an average curve which fitted more than ninety percent of the data plotted. Hoek and Diederichs (2006) came up with the following best fit equation:

$$E_m(\text{MPa}) = 10^5 \left(\frac{1-0.5D}{1+e^{\left(\frac{75+25D-GSI}{11}\right)}} \right) \quad 2-21$$

Curves were plotted for different disturbance factors and were superimposed onto the China and Taiwan data as shown on Figure 2-19. The measured field data reported by Serafim and Pereira (1983), Bieniawski (1978) and Stephens and Banks (1989) were used to ensure that the best fit equation is accurate by plotting them together. This is shown on Figure 2-20. From the plot it can be seen that a disturbance factor of zero best fits the measured field data. The China and Taiwan data also included information on the geology and uniaxial compressive strength of the rock mass which allowed Hoek and Diederichs (2006) to further analyse the data. With the application of the modulus ratio proposed by Deere (1968), the detailed analysis of the data resulted in the following equation:

$$E_m = E_i \left(0.02 + \frac{1-0.5D}{1+e^{\left(\frac{60+15D-GSI}{11}\right)}} \right) \quad 2-22$$

The main role of the GSI index is only to provide input for the Hoek-Brown criterion and its role in determining the deformation modulus is secondary and should be left to the RMR (Bieniawski, 2011). The introduction of the disturbance factor (D) is unnecessary when using the RMR system. The practice of sometimes substituting GSI in the place of the RMR in the Bieniawski (1978) and Serafim and Pereira (1983) equations is not correct as there is no relationship between the GSI index and the RMR classification system, except in extremely poor rock conditions.



The use of GSI is not strongly recommended as it introduces empiricism in rock mass classification. Although the Shen et al. (2012) analysis showed that the Hoek and Diederichs (2006) empirical equation yields the best results for the estimation of the deformation modulus the use of RMR is however recommended. In a review, Stille and Palmström (2003) warned as follows, “*Visual determination of GSI parameters represents the return to quality descriptions instead of advancing quantitative input data as in RMR, Q and Rmi systems. GSI was found mainly useful for weaker rock masses with RMR<20. As GSI is used for estimating input parameters (strength), it is only an empirical relation and has nothing to do with rock engineering classification.*”

Sonmez et al. (2006) noticed a limitation in empirical relationships which do not consider the deformation modulus of intact rocks. The empirical relationships produce extremely high modulus of deformation values for good quality rock plus the softer pieces. For such cases, the modulus of deformation obtained is much higher than the elastic modulus of the intact rock. To curb this, it was assumed that when GSI is equal to 100, the ratios of rock mass modulus and intact rock elastic modulus should be equal to each other. The worldwide known data used by Nicholson and Bieniawski (1990) and three additional data derived values were used by Sonmez et al. (2006) to develop an empirical relationship. The empirical relationship derived is as follows:

$$E_m = E_i 10^{((RMR-100)(100-RMR))/(4000 \exp(-\frac{RMR}{100}))} \quad 2-23$$

This equation performs better than the Nicholson and Bieniawski (1990) equation.

2.4 Back analysis and modelling

The use of numerical methods for the analysis of geotechnical problems has increased over the years. These numerical methods enable the consideration of intricate geomechanics and geological characteristics in the design of structures such as tunnels, underground caverns and foundations of structures, slopes etc., and the monitoring of the stability of the structures during construction. Consequently, these numerical analyses ensure that rationale design of engineering structures is achieved (Sakurai, 1981).

Numerical model predictions do not always agree with the actual behaviour of the structure (Sakurai, 1997). Differences between numerical model predictions and the actual behaviour of the structure are often caused by many uncertainties involved in the modelling of complex geological formations with complex geotechnical characteristics. Initial stresses also cause difficulties in numerical analyses. To overcome this, the observational method which was suggested by Terzaghi

and Peck (1948) is employed. Field measurements are conducted during construction, allowing the design parameters for the original design to be evaluated and adapted if required. The method can also be utilized for monitoring the stability of the structures.

2.4.1 Back analysis

Back analysis is an essential tool in the observational method for assessing design parameters (Sakurai, 1997). In back analysis, measured values such as displacements, strains, stresses and pressures are the input data while the output data are material parameters such as, load, initial states of stresses, permeability and even boundary conditions. Forward analysis is the opposite of back analysis, which is commonly used in the design of structures, where material parameters such as, load, initial states of stresses are inputs and displacements, strains, stresses and pressures are outputs.

With the use of numerical models in the observational method, Sakurai and Akutagawa (1995) emphasized that the back analysis of geotechnical engineering problems should be able to recognize the mechanical constraints and the mechanical model. However, the definition of the mechanical model to represent their behaviour can be extremely difficult due to the complex mechanical characteristics of materials.

In forward analysis, the numerical model is usually assumed or given. The values for mechanical parameters of the model are assumed using the most appropriate exploration, such as laboratory or in situ tests (Sakurai, 1997). Displacements, strains and stresses can then be calculated after all the mechanical parameters are known. The results of the analysis give a unique solution which confirms the input and output values.

The opposite happens in back analysis; displacements, strains, stresses and pressures from field measurements are obtained first, and the mechanical parameters are then determined, by back analysis by assuming a numerical model (Sakurai, 1997). The results of the mechanical parameters determined by back analysis are dependent on the assumed numerical model used in back analysis. For example, if we assume an elastic model, then Young's modulus can be determined, but this is not the case when a rigid-plastic model is assumed, though the same values of measured results are used. This indicates that the back analysis results are a matter of assumption of the numerical

models representing the behaviour of materials. The uniqueness of the solution of a back analysis cannot be confirmed between the input and output results.

Back analysis can therefore not be simply seen as a reverse calculation of a forward analysis. Its concept should not be the same as forward analysis in such a way that back analysis can recognize the mechanical parameters for the model and also provide a model (Sakurai, 1997). Figure 2-21 shows a comparison between the procedures of forward analysis and back analysis.

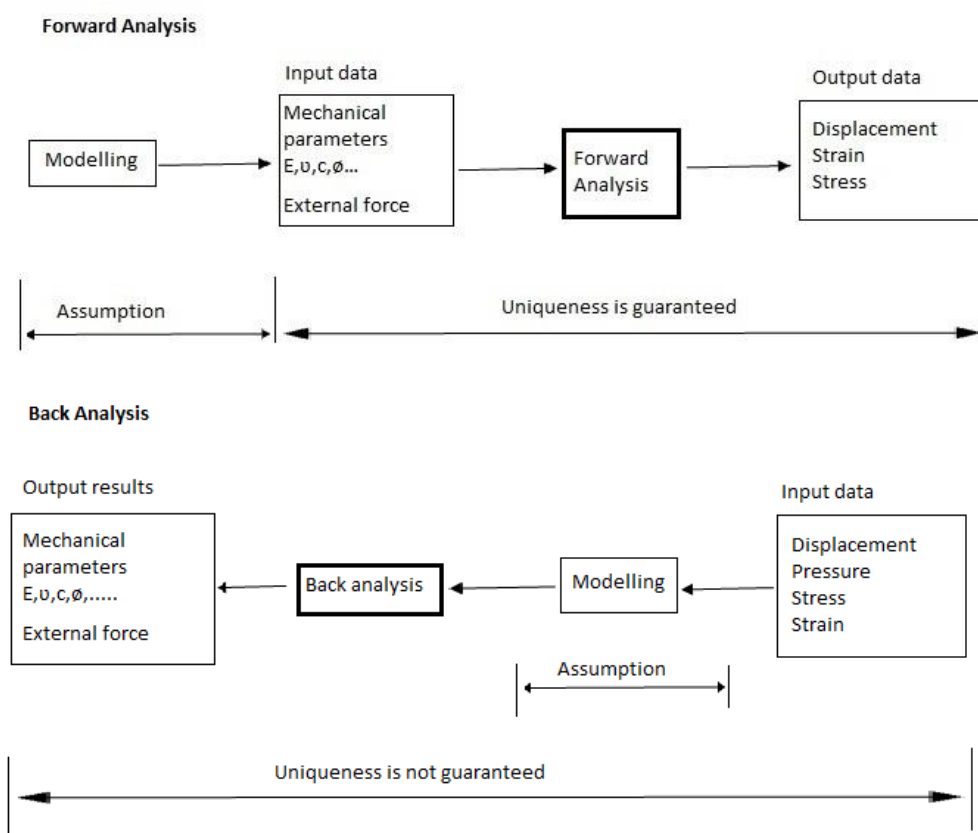


Figure 2-21: Comparison between the procedures of forward analysis and back analysis (Sakurai, 1997)

2.4.2 Back analysis of excavation induced stress changes

For the design and construction of underground excavations, it is imperative to know the state of the in situ stress, that is, the magnitude and the orientation of the principal stress. Conventional in situ tests, such as flat jack and over-coring are often very expensive (Zou and Kaiser, 1990). In 1990, Zou and Kaiser developed a method for the determination of the in situ stress in a volume of rock

mass that is representative for an underground excavation. This is achieved by comparing stress change measurements with analytical or numerical modelled stress change predictions. These stress change measurements are determined by installing stress change cells, such as CSIRO-HI-cells, ahead of the advancing face in the undisturbed rock mass.

The method assumes the rock mass behaves in an isotropic, homogeneous, linear elastic manner and is limited to a two-dimensional stress state (Zou and Kaiser, 1990). Principal stresses (σ_1, σ_3) and the inclination (β), the angle from σ_1 to the x axis of an orthogonal coordinate system with the origin in the centre as seen on Figure 2-22, describe the in situ stress state. A virgin stress field before any excavation is denoted by $S_0 = (\sigma_3, \sigma_1, \beta)$ or $(\sigma_0, K\sigma_0, \beta)$. The comparison is made on two stress invariants namely the centre (C) and the radius (R) of the stress circle. These two invariants are described in terms of three stress change components as follows:

$$2C = \Delta\sigma_x + \Delta\sigma_y \quad 2-24$$

where, σ_x and σ_y are the principal stresses in the x and y direction respectively.

$$2R = \sqrt{(\Delta\sigma_x - \Delta\sigma_y)^2 + 4\Delta\tau_{xy}^2} \quad 2-25$$

where, τ_{xy} is the shear stress. The quality of the agreement between predicted and the measured stress can be assessed by comparing the two stress change functions. Two errors are defined namely;

- a. the difference between the centres of the stress change circles

$$e_c = 2 (C_p - C_m) \quad 2-26$$

- b. and the difference between the radii of the stress change circles

$$e_R = 2 (R_p - R_m) \quad 2-27$$

where, the subscripts p and m stand for predicted and measured respectively.

If the predicted stress field in the model is equal to the real stress field, the predicted and the measured stress changes must match meaning the differences of the centres and the radii must be zero. However, this is not always the case, due to errors in the model. A fit may be achieved at one point but, this does not imply that the boundary stresses are correctly modelled. Hence, a best fit solution is found by considering measurements at a number of locations. The error function is monotonic and has one minimum which requires measurements from at least two points to obtain the stress field. Addition of more information from instrumentation increases the accuracy of this approach. The best-fit solution is obtained minimizing the following total error function:

$$e_t = \sum e_{Ci}^2 + \sum e_{Ri}^2 \quad 2-28$$

Where e_C and e_R are the differences between the measured and predicted centres and radii of the stress change circles; $i=1,2,\dots,n$ and n is the number of cells (Zou and Kaiser, 1990).

In order to determine the three stress parameters of the in situ field $(\sigma_0, K\sigma_0, \beta)$, the analysis is carried out by normalizing the stress field σ_0 . This implies that the numerical model needs to be run only once for a set K and β values (Kaiser et al., 1990)

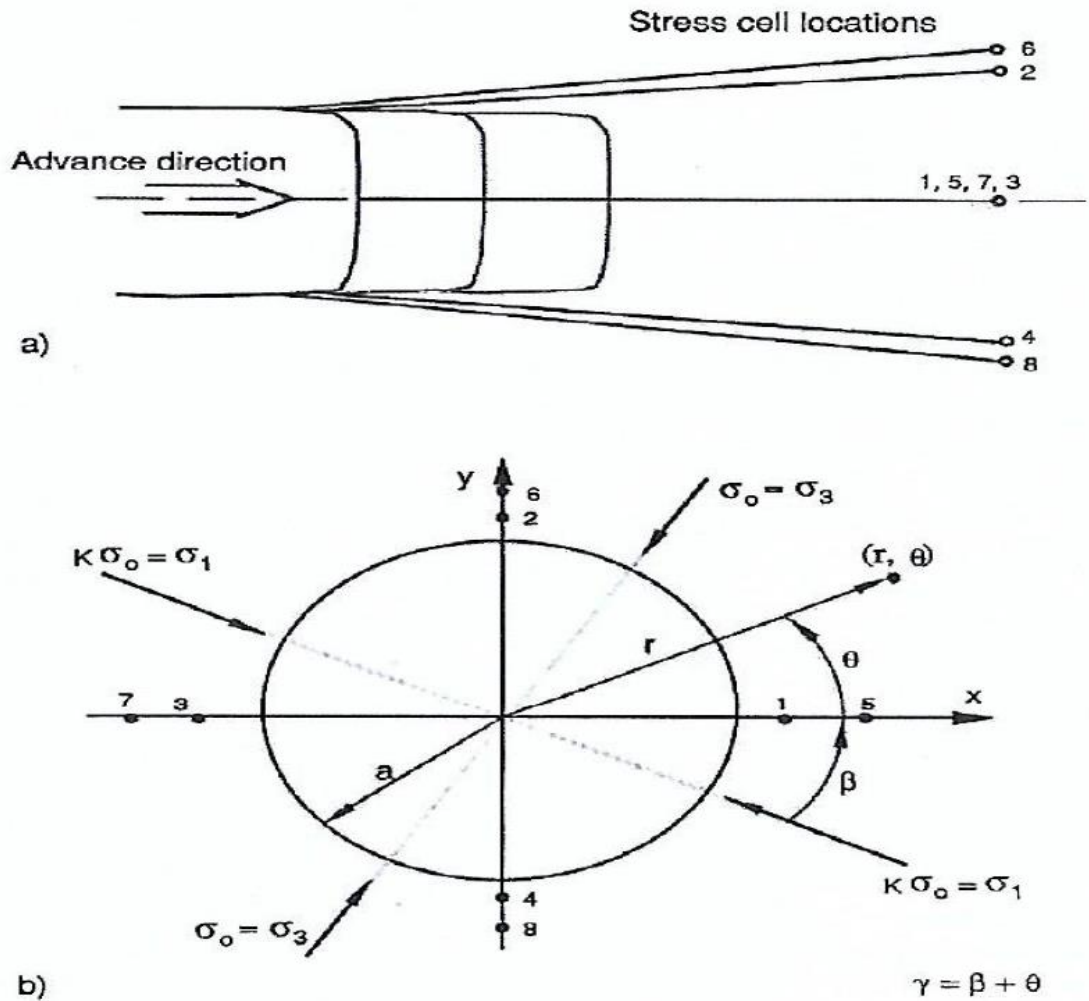


Figure 2-22: Circular opening with inclined, non-uniform stress field and eight stress change monitoring locations (1-8): a) longitudinal section; b) cross-section at cell location (Zou and Kaiser, 1990)

Zou and Kaiser (1990) identified the advantages of the error fitting method as:

1. Being easily applied to more complex rock mass models ;
2. provides an opportunity to update stress information during excavation of underground opening;
3. an inaccurate measurement can easily be spotted, by comparing the error functions for individual measurements, provided the measurements are correct and

4. relatively cheap as compared to the other stress measurement methods.

2.4.3 Back analysis of modulus of deformation

The importance of being able to effectively design, analyse and control rock engineering problems has increased over the years. This resulted in the substance for design, analysis and control of rock engineering problems being initiated in constitutive stress- strain relationships. Limited in situ tests and small scale laboratory tests are not adequate means to assess rock mass characteristics in these relationships. To overcome this, Kirsten (1976) developed a closed form theory for the determination of the large scale elastic moduli of in situ rock masses from simple observations of excavation induced displacements.

The theory is an affordable and reliable indirect assessment of the rock mass behaviour over a large extent; developed for a two-dimensional classical elasticity problem. The founding concept of the theory is that induced stresses and displacements are related by the in situ elastic moduli. Both these quantities diminish over increasing distances from the disturbance and are, as a result affected by the elastic moduli of a very large domain of material. Therefore, the elastic moduli of the rock for an extensive area are represented in the displacements which occur in the rock mass (Kirsten, 1976).

The theoretical derivation considers a two-dimensional plane strained section in which the x y axes are as shown in Figure 2-23 (Kirsten, 1976).

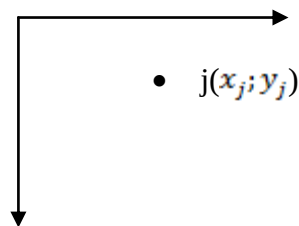


Figure 2-23: Cartesian coordinates in a two-dimensional plane strained section (Kirsten, 1976)

Let j denote a bench mark at a point with coordinates (x_j, y_j) . Also let a and b denote elastic constants, expressed as follows in terms of Young's modulus E and Poisson's ratio ν (Kirsten, 1976).

$$a = (\nu^2 - 1)/E \quad 2-29$$

$$b = \nu(1 + \nu)/E \quad 2-30$$

The resulting stress-strain relationship, from Hooke's law is as follows (Kirsten, 1976):

$$\varepsilon_x = -\delta u / \delta x \quad 2-31$$

$$\varepsilon_y = -\delta v / \delta y \quad 2-32$$

Thus,

$$u_j = a \int \sigma_x dx + b \int \sigma_y dy + r y_j \quad 2-33$$

$$v_j = a \int \sigma_y dy + b \int \sigma_x dx + R x_j \quad 2-34$$

Alternatively,

$$u_j = a p_j + b q_j + r_j \quad 2-35$$

$$v_j = a P_j + b Q_j + R_j \quad 2-36$$

Where,

$$\left. \begin{aligned} p_j &= \int \sigma_x dx \\ q_j &= \int \sigma_y dx \\ r_j &= r(y_j) \\ P_j &= \int \sigma_y dy \\ Q_j &= \int \sigma_x dy \\ R_j &= R(x_j) \end{aligned} \right\} \quad 2-37$$

These relationships do not contain any elastic constants and are therefore characteristic influence coefficients for bench mark j (Kirsten, 1976). By running a finite element model for a specific choice for the a and b constraints, a set of values may be acquired for u_j and v_j . If two more models are run each with a different set of values for a and b , adequate information would be acquired to calculate p_j, q_j, r_j, P_j, Q_j and R_j with the use of simultaneous equations.

Let A and B represent the required in situ elastic constraints and U_j and V_j the corresponding field monitored deformations at bench mark j (Kirsten, 1976). Having found functions 2-37, applicable to the bench mark j as described above, allows the determination of the constants A and B from the following simultaneous equations:

$$p_j A + q_j B = U_j - r_j \quad 2-38$$

$$P_j A + Q_j B = V_j - R_j \quad 2-39$$

A and B may be found as follows if only horizontal deformations are monitored, say at bench mark j and k. Kirsten (1976) derived the following simultaneous equation to determine A and B:

$$p_j A + q_j B = U_j - r_j \quad 2-40$$

$$p_k A + q_k B = U_k - r_k \quad 2-41$$

Alternatively, if only vertical deformations are monitored in the field, the simultaneous equation to determine A and B would be as follows (Kirsten, 1976):

$$P_j A + Q_j B = V_j - R_j \quad 2-42$$

$$P_k A + Q_k B = V_k - R_k \quad 2-43$$

Eventually, Kirsten (1976) found the elastic moduli from equations 2-40 to 2-43 as follows:

$$E = (2B - A)/(B - A)^2 \quad 2-44$$

$$v = B/(B - A) \quad 2-45$$

It should be noticed that functions 2-35 and 2-36 do not suggest that the displacements u and v are linearly related to the elastic moduli (Kirsten, 1976). Taking the first derivative of the displacements with respect to E and v can verify this.

The following assumptions made by Kirsten (1976) apply to the rock mass of the above derivation:

- I. a continuum,
- II. intact,
- III. homogeneous,
- IV. isotropic,
- V. linearly elastic.

The given expressions apply to problems which are assumed to be:

- VI. two dimensional,
- VII. free of body forces,
- VIII. having no boundaries on which conditions of stress and displacement are mixed, and
- IX. simple or multiply connected bodies on separate boundaries in which the resultants of forces are zero (Kirsten, 1976).

Through practical use, the applicability of the assumptions on the medium (i) to (v) are justifiable (Kirsten, 1976).

The derivations from the back analysis are exact in cases of an ideal elastic medium and of a stratified inhomogeneous material (Kirsten, 1976). Although it is inaccurate to an insignificant degree in the cases where Young's modulus varies as a damped exponential function, remarkable results have been obtained from test applications of the technique to these cases. Displacements in continued excavation can be predicted with the use of the back analysis technique. Results from the predicted displacements can then be used to give warning of unstable conditions. The technique can also be used as a field measuring device.

2.5 Numerical analysis

With recent technology advances, computer programs have been used to analyse the deformation in tunnels. Jing (2003) identifies the main task of numerical modelling as being able to characterize the rock mass, "material conceptualization", as well as the interaction of the rock mass and the engineering structure in the modelling process for design. This ensures that the consequences of the construction process have also been characterized. There are two main categories of numerical models namely, continuum models and discontinuum models.

In continuum models the material throughout the rock mass is assumed to be continuous (Jing, 2003). There are several methods that fall in the continuum model category, namely, Boundary Element Method (BEM), Finite Element Method (FEM) and the Finite Difference Method (FDM). BEM are commonly known as integral methods, in which only the boundaries of a problem are discretized into zones. Phase2, FLAC, FLAC3D, ANSYS, and ABAQUS are some examples of FDM and FEM.

Discontinuities are included in the model geometry from the start of the calculation for discontinuum models (Jing, 2003). Continuum behaviour is treated as a special case. Discontinuum models are also known as Discrete Element Models. UDEC and its 3D equivalent, 3DEC, are the two widely-used discrete element models.

2.5.1 Finite Element Method in numerical analysis

The FEM was developed in the late 1960s to early 1970s to deal with rock mechanics problems. A major advantage of the FEM is that it out performs the conventional FDM, as it was the first numerical method with enough flexibility for dealing with material heterogeneity, non-linear deformability (mainly plasticity), intricate boundary conditions, in situ stresses and gravity (Jing and Hudson, 2002). The method gained popularity as it was developed at a time where the conventional FDM could not meet these requirements for rock engineering problems. Besides out performing other numerical models, the FEM has some challenges representing discontinuities. Challenges arise from the general continuum assumption in FEM formulations require discontinuity elements to have no spacing. Therefore, block rotations, complete detachment and large scale discontinuity opening cannot be captured. Special elements can be used to represent discontinuities and small scale spacing can be simulated by a reduction of the normal stiffness of the discontinuities.

Methodology

Jing (2003) identified three stages that are essential to complete an FEM analysis: domain discretization, local estimation and assemblage and solution of the global matrix equation. The domain discretization involves the definition of the problem domain surrounding the excavation and dividing the domain into an assemblage of discrete interacting elements. The assemblage of finite elements in the domain is called a mesh. A cross section of an underground opening generated in an infinite body subjected to initial stresses is shown in Figure 2-24 a. A statically determinate problem is shown in Figure 2-24 b where the boundary and the problem domain, suitable supports and conditions have been well defined. The domain has been divided into triangular elements and a representation is shown in Figure 2-24 c.

The problem is to determine the state of total stress and the excavation-induced displacements in the mesh. Initially, a selection of a set of functions which define the displacement components within a finite element in terms of the nodal displacement is made. Interpolation from nodal values is used to obtain the displacements [u] at any point within the element. The following is a suitable interpolation formula:

$$u^e = \sum_{i=1}^M N_i u_i \quad 2-46$$

Where M is the order of the elements, and N is the shape function or interpolation function defined in inherent coordinates in order to use Gaussian quadrature integration (Brady and Brown, 2004; Jing, 2003).

The element stiffness matrix $[K^e]$ which is applied to the elasticity problems is represented in the equation below.

$$[K^e] = \int_V ([B])^T [D] [B] dV \quad 2-47$$

Where, matrix $[B]$ is the geometry matrix, matrix $[D]$ is the elasticity matrix and V is the volume of the element. The relationship between the nodal displacement and the elemental strain is used to determine these two matrixes (Jing, 2003).

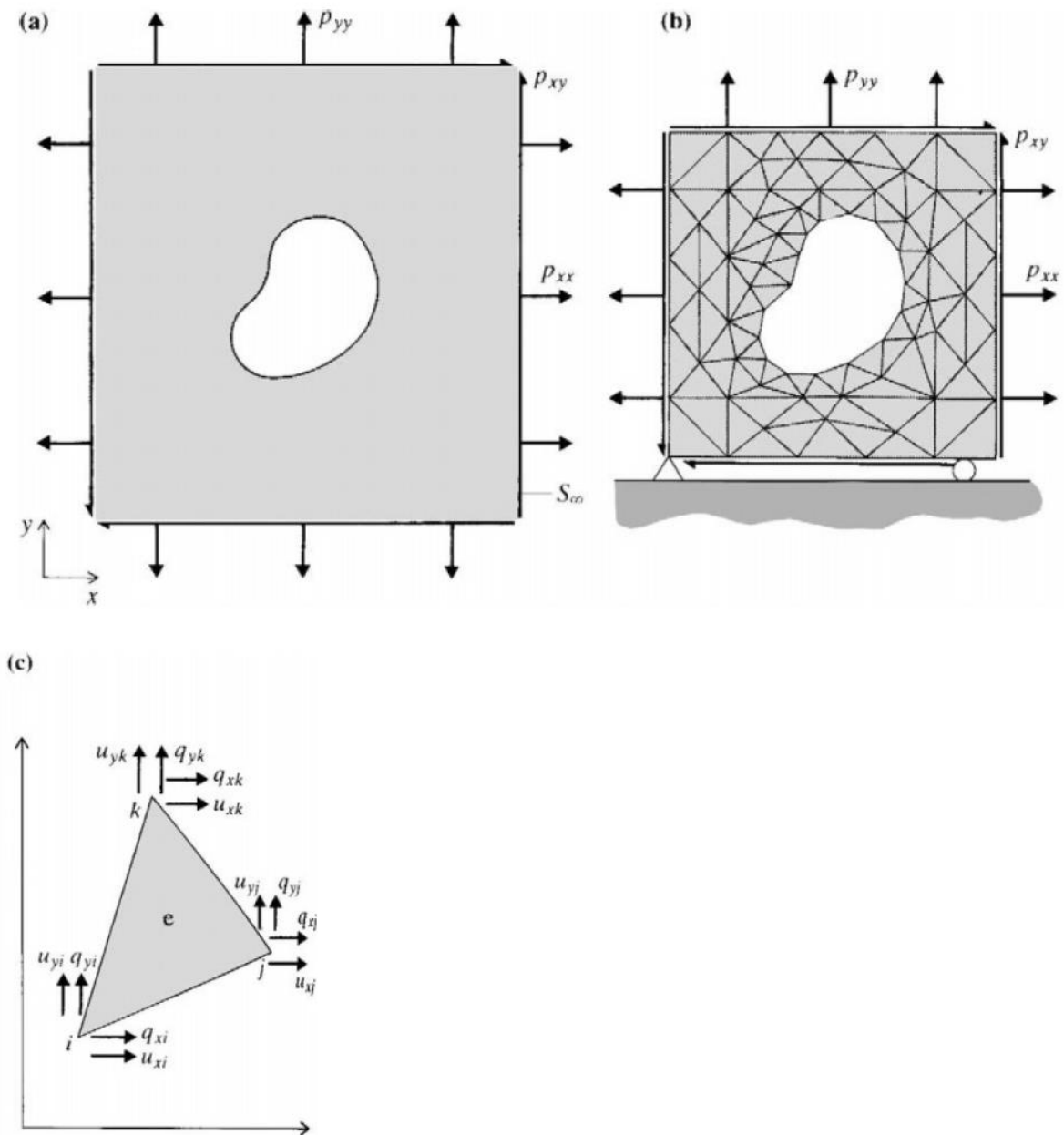


Figure 2-24: Development of a finite element model of a continuum problem and specification of element geometry and loading for a constant strain, triangular finite element (Brady and Brown, 2004)

The formula, $Ku = F$, represents the global stiffness matrix, which is banded and symmetric. The global stiffness matrix can also be expressed in the equation below:

$$\sum_{i=1}^N [K_{ij}^e] \{u_j^e\} = \sum_{i=1}^N \{f_i^e\} \quad 2-48$$

Where, vector u_j^e is the nodal value vector of displacements and vector f_i^e is the vector for body forces and boundary conditions respectively. The equation is used to simultaneously determine the displacements of all nodes in the mesh, from which the stresses and strains can be assessed (Jing, 2003).

Discontinuity representation in FEM

The well-known four-node line element of zero thickness also known as the “Goodman Joint Element” was proposed by Goodman et al. (1968) for fracture analysis with FEM. The development incorporated the “Goodman Joint Element” into the FEM analysis with the assumption that the contact stresses and relative displacements along and across the rock discontinuities are of zero thickness as shown in Figure 2-25. A linear relationship is followed by the model with constant normal K_n and shear stiffness K_s . The model’s local equilibrium equation is expressed as:

$$k^G u^G = f^G \quad 2-49$$

Where, the matrix k^G is a symmetric matrix with its entries defined by the shear and normal stiffness. The vector u^G is the nodal displacement vector for the four nodes defining the joint element.

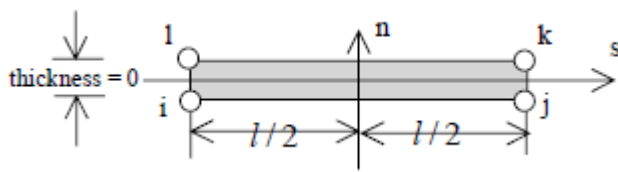


Figure 2-25: Fracture elements in FEM by Goodman et al. (1968) (Jing 2003)

The Goodman joint element has been applied to FEM codes based on continuum assumptions that sliding, large-scale opening, and complete detachment of elements is not allowed. However, this has caused numerical problems and the Generalized FEM and Enriched FEM approaches were developed using special algorithms. In the Generalized FEM, a shortened discontinuous form function used to simulate the fractures and treat the continuum bodies, fractured bodies and grouping of discrete blocks in an integrated form, and is a natural bridge between the continuum and discrete depictions.

The Enriched FEM is applied to the analysis of tunnel stability with fractures simulated as displacement joints (Jing and Hudson, 2002).

Tunnel advance modelling in FEM

The longitudinal displacement profile (LDP) is the graphical representation of radial displacement that occurs along the unsupported tunnel axis away from the face. This relationship is shown in Figure 2-26 where the longitudinal profile of closure is plotted against the distance from the face of the tunnel. LDP is used for design purposes to define the appropriate time for support installation or when optimizing the support installation with a specific displacement capacity.

Relationships for LDP ahead, at and behind the tunnel face, in terms of the tunnel radius, the plastic zone radius around the tunnel as well as the present and maximum tunnel wall displacement are provided by Vlachopoulos and Diederichs (2009). A simple plane-strain FEM numerical analysis can be used to calculate the maximum tunnel wall displacement (u_{max}) and the maximum radius of the plastic zone (R_p).

According to Vlachopoulos and Diederichs (2009), in order to determine the LDP, Panet (1995) derived the following equation based on plastic analysis;

$$u^* = \frac{u_R}{u_{max}} = \frac{1}{4} + \frac{3}{4} \left(1 - \left(\frac{3}{3+4X^*}\right)^2\right), \quad X^* = X/R_T \quad 2-50$$

where, u_R is the radial displacement and u_{max} is the maximum short term radial displacement distant from face. This formula is only used behind the face where the value of X is positive.

Chem et al. (1998) also proposed a 'best fit' relationship based on actual measured data (Vlachopoulos and Diederichs, 2009):

$$u^* = \frac{u_R}{u_{max}} = \left(1 + e^{\frac{X^*}{1.10}}\right)^{-1.7} \quad 2-51$$

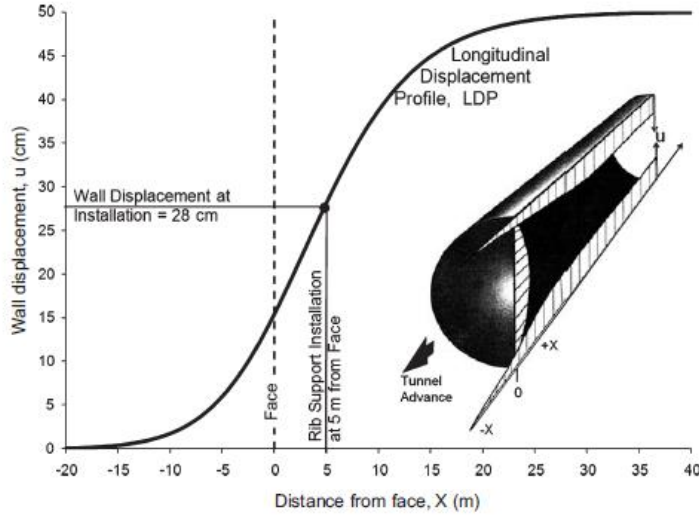


Figure 2-26: Use of the longitudinal displacement profile, (LDP), to relate support installation location to nominal wall displacement for use in convergence–confinement ground/support reaction analysis (Vlachopoulos and Diederichs, 2009)

Studies have shown that these two relationships are applicable to plastic analysis only if the plastic zone radius does not exceed 2 tunnel radii and as long as that the yielding zone in the tunnel face does not interrelate with the developing yield zone around the tunnel walls. It is however possible for the plastic yielding zone to exceed 2 tunnel radii. In order to account for the increase in the yielding shape of the longitudinal displacement profile the term plastic zone radius was used. The plastic zone radius is expressed as $R^* = R_p / R_T$, where R_p is plastic zone radius and R_T is tunnel radius. Vlachopoulos and Diederichs (2009) proposed a new set of best fit relationships based on the analysis using Phase2 in plane-strain cross section and axisymmetric models. These relationships are shown in following equations:

$$u_0^* = \frac{u_0}{u_{max}} = \frac{1}{3} e^{-0.15R^*} \quad 2-52$$

For $X^* \leq 0$ (in rock mass);

$$u_0^* = \frac{u_0}{u_{max}} = u_0^* \cdot e^{X^*} \quad 2-53$$

For $X^* \geq 0$ (in tunnel);

$$u_0^* = 1 - (1 - u_0^*) \cdot e^{\frac{3X^*}{2R^*}} \quad 2-54$$

The above relationships can be used to relate displacement and the position along the tunnel of the LDP. For FEM numerical analysis, the maximum tunnel wall displacement (u_{max}) and plastic zone radius (R_p) need to be calculated prior to the sequenced analysis. The sequencing of the plane-

strain analysis can be completed through a core replacement technique or by gradually reducing the tunnel boundary tractions from in situ to zero in phases. (Vlachopoulos and Diederichs, 2009).

2.5.2 Computer Program

The Phase 2 Rocscience computer program was used to analyse the tunnel deformations. Phase 2 is a 2-dimensional, elasto-plastic finite element method (FEM) program used for designing underground or surface excavations and their support systems. The windows based program is used in the analysis of complex problems in rock and soil mechanics such as, stress analysis, tunnel design, slope stability, support design and groundwater seepage analysis etc. Complex models can easily and quickly be analysed using this program (Rocscience, 2014).

Version 7 of the software has a number of constitutive models built in, including models for joints and materials and support types. Axisymmetric conditions or plane-strain conditions are used to analyse 3-dimensional problems, although the input is 2-dimensional. The software also includes the “Goodman joint element” and joint network functionality. This functionality allows the specification of 2-dimensional joint sets as input in the model. The simplicity of the software allows a number of probabilistic and deterministic algorithms obtainable for joint network generation.

Numerous parameters have to be computed in the software to model the deformation, such as material properties, in situ stress conditions and support types. Material properties include unconfined compressive strength of intact rock (σ_{ci}), Geological strength index (GSI), Hoek-Brown constant (m_i), Poisson’s ratio (ν), Young’s Modulus of intact rock (E_i) and the density of the rock mass. Field stress conditions in the Phase2 can either be gravity or constant stress. Gravity field stress is usually used for surface or at shallow depths and the areas where there is a topographical effect on the stress magnitudes and directions. The gravity stress field also varies linearly with depth from an elevation defined by the user.

2.6 Stability Monitoring using Hazard Warning Level

Monitoring the stability of the structure is recommended during and after construction. The stability of a structure can be evaluated by comparing the measurements with allowable values. The structure is stable if the allowable values are larger than all the monitoring instrumentation measured values. These allowable values are known as the hazard warning levels. Even when

measured values are lower than hazard warning levels, engineers need to consider what happens after a certain period of time. If the measured values are larger than the hazard warning level after a period of time, engineers will have to stabilize the structure (Sakurai, 1997).

Sakurai (1981) proposed that critical strain is used to define the hazard warning levels, by assessing displacements results in the sidewalls and the crown from extensometer and inclinometer measurements.

The critical strain of in situ rock masses is related to that of intact rocks by the equation below:

$$\varepsilon_{\theta R} = \frac{\sigma_{cR}}{E_R} = \frac{m\sigma_c}{nE} = \left(\frac{m}{n}\right)\varepsilon_{\theta} \quad 2-55$$

Where, m and n are reduction factors of the uniaxial strength and Young's modulus, respectively. The values of m and n reduction factors were determined using in situ and laboratory tests. Due to the existence of discontinuities, both the uniaxial strength and Young's modulus decrease. Sakurai (1983) found the ratio of the two reduction factors to be between 1.0 and 3.0 depending on the rock type. Although the uniaxial strength and Young's modulus of the intact rocks largely differ from those of in situ rock masses, the critical strain of the in situ rocks are in the same order as the intact rocks.

Displacement measurements were conducted in order to validate if the hazard warning level are applicable as described above. The strain in the rock mass around the tunnel was determined from tunnel displacements using the following formulae (Sakurai, 1997):

$$\text{For crown : } \varepsilon_{\theta} = \frac{u_c}{a} \quad 2-56$$

$$\text{For sidewalls : } \varepsilon_r = \frac{u_1 - u_2}{L} \quad 2-57$$

Where,

u_c is the crown displacement,

u_1 and u_2 are tunnel displacements measured at point 1 and 2,

a is the tunnel radius and

L is the length between the measured points 1 and 2.

The strains calculated using the above equations were plotted relative to the uniaxial strength of the rock mass shown in Figure 2-27 and Figure 2-28. Figure 2-27 shows the relationship between the strain and the uniaxial compressive strength from displacements in the crown and Figure 2-28 shows the same relationship from displacements in the sidewalls. The dotted lines show the lower and upper limits for the critical strain acquired from laboratory tests (Sakurai, 1981). The types of

problems faced during the excavation of the tunnels are shown in the numbers next to the data points. Data points with no numbers show that there were no problems during the construction of the tunnel. The forms of problems are categorised as follows: “

1. complications in maintaining tunnel face;
2. cracking or failure of the shotcrete;
3. buckling of steel ribs;
4. breaking of rock bolts;
5. fall-in of roof;
6. swelling at invert; and
7. miscellaneous.”

From the Figure 2-27 and Figure 2-28, it is seen that when strains are less than the lower limit of the critical strains, the tunnels could be excavated with no problems. When strains reach the upper limit of critical strain, there are many difficulties experienced during the construction of the tunnel.

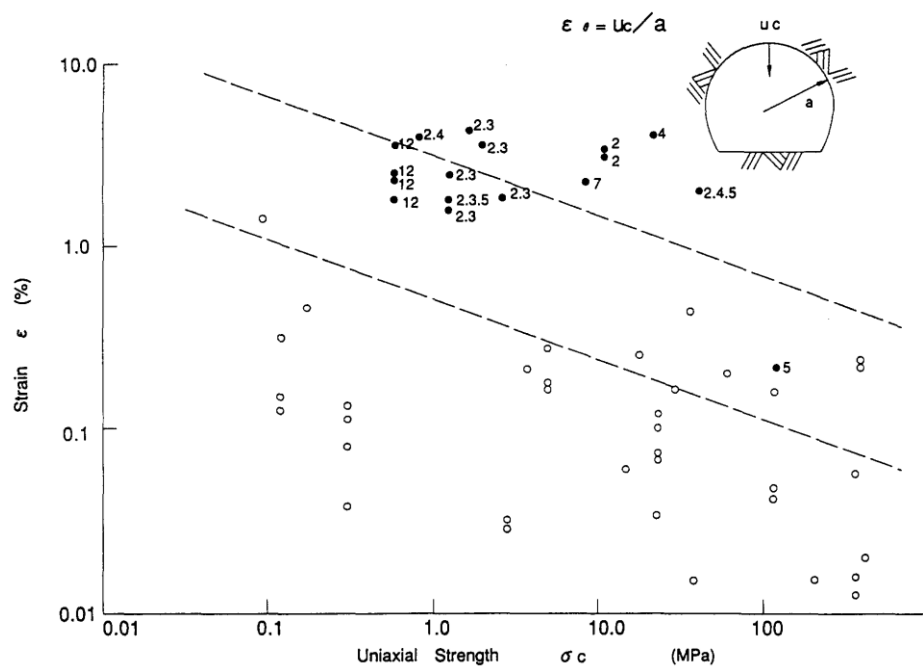
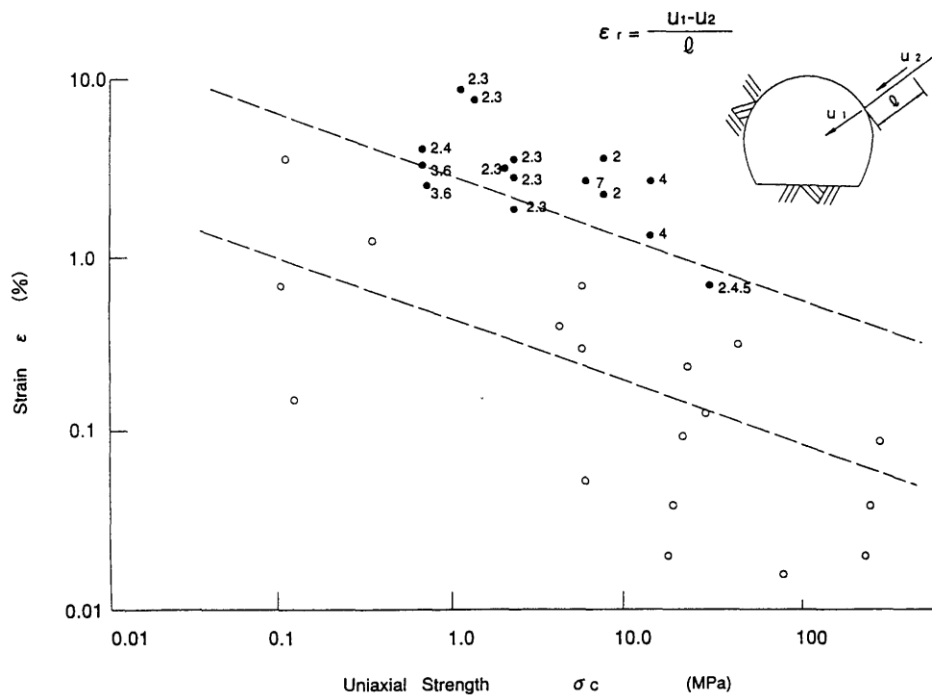


Figure 2-27: Relationship between the measured strain (obtained from crown displacement) and hazard warning levels (Sakurai, 1997)



3. CASE STUDY: INGULA PUMPED STORAGE SCHEME (IPSS)

3.1 Project description

The Ingula Pumped Storage Scheme (IPSS) is situated 55km from Ladysmith (23km northeast of Van Reenen) within the Drakensberg range, on the border between the Free State and KwaZulu-Natal provinces as seen in Figure 3-1 (Keyter et al., 2008). The scheme consists of two dams, Bedford dam and Braamhoek dam, which are interconnected by a tunnel system with reversible pump-turbine units. Upon completion it will be Eskom's third pumped storage scheme with an output of 1332MW, mostly used during peak-demand periods. The station has been fully operational since the end of 2016.

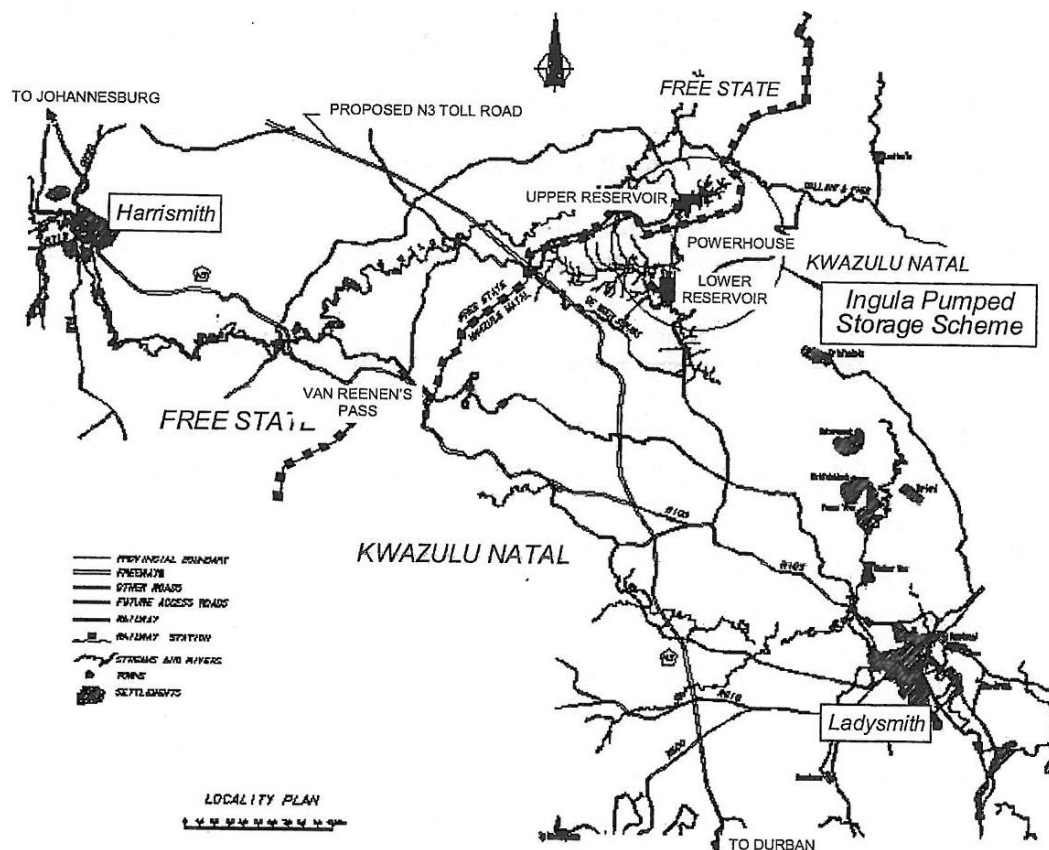


Figure 3-1: Location of the Ingula Pumped Storage Scheme (Keyter et al., 2008)

3.1.1 Project layout features

The scheme layout is shown in Figure 3-2 with the underground works comprising of:

- The main access tunnel (MAT), exploratory tunnel and surge chamber access tunnel
- Twin low-pressure headrace tunnels (LPHT) from the intake structure at the upper reservoir to two surge shafts (SS)/ surge risers (SR) near the escarpment edge
- Twin inclined high-pressure shafts (IHPS) and tunnels leading from the surge shafts/ surge risers to the machine hall (MH)
- Four busbar tunnels (BT) connecting the transformer hall to the machine hall
- Four draft tubes (DT) leading to two tailrace surge chambers
- A dewatering shaft linking the main dewatering sump to a drainage and ventilation adit above the powerhouse
- A fresh air inlet shaft (FAIS) and a smoke extraction shaft linking the main powerhouse caverns to ground surface
- Two tailrace tunnels (TT) from the surge chambers combining into a single tailrace tunnel connecting to the outlet structure at the lower reservoir
- Other miscellaneous tunnels, galleries and adits at the level of the powerhouse complex and the drainage and ventilation adits above the powerhouse

The powerhouse complex is 187m long, 26m wide and 54m high. This will be the largest span of underground cavern in Africa in mud rock. It accommodates four Francis turbines which will each generate 333 MW.

The distance between the Bedford dam and Braamhoek dam, is approximately 6km with a difference in elevation of 470km. The upper reservoir, Bedford dam, is a concrete-faced rock fill embankment dam, 41m high, with a total capacity of 22.4-million cubic metres and an active water storage volume of 19.2-million cubic metres. This reservoir is located on the headwater tributary to the Wilge River, which flows into the Vaal River system. The 39m high lower dam is a roller-compacted concrete (RCC) dam, with a total capacity of 26.3-million cubic metres and an active storage volume of 21.9 million cubic metres. This reservoir is located on a headwater tributary of the Klip River, which flows into the Tugela River System (Keyter et al., 2008).

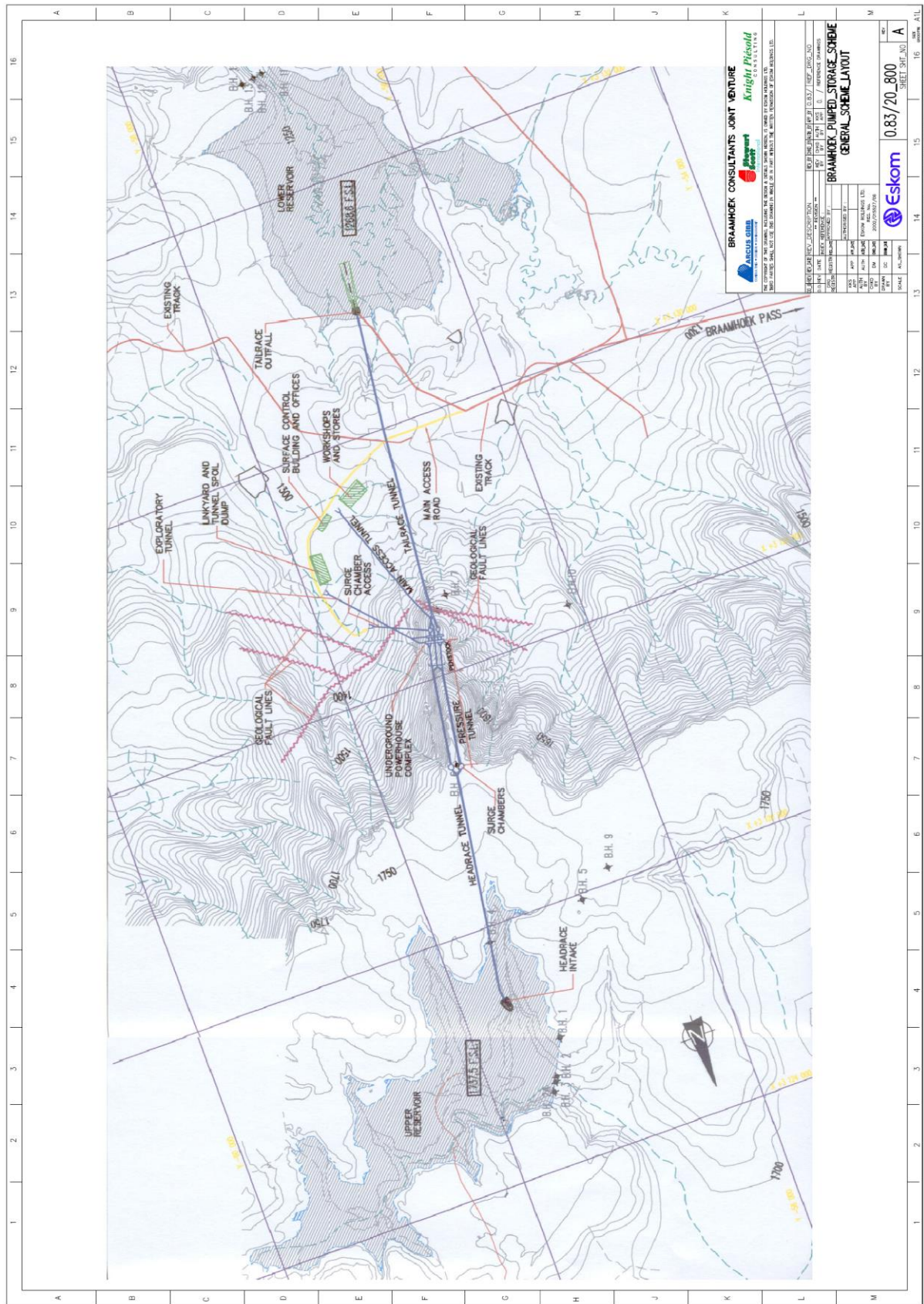


Figure 3-2: Layout scheme of the Ingula Pumped Storage Scheme (BCJV, 2007)

3.2 Geology of the Ingula Pumped Storage Scheme

3.2.1 Geotechnical investigation

Partridge, Maud and Associates (PMA, 1999) carried out the initial geotechnical investigation. This investigation comprised of 11 boreholes drilled from surface, 4 of which are relevant to the underground works (BH4, BH6, BH7 and BH8) with BH8 a vertical hole near the position of the powerhouse caverns.

Braamhoek Consultants Joint Venture (BCJV, 2007) completed the main phase of surface geotechnical for the underground works in 2005, which included the drilling of 17 boreholes (BH101 to BH117), including 4 boreholes drilled into the area of the powerhouse complex using drill rigs airlifted onto the mountain ridge in a position directly above the powerhouse. In 2006, four additional surface boreholes were drilled along the MAT alignment (BH208 to BH211). A further four boreholes (ET301 to ET304) were drilled underground from the end of the exploration tunnel (ET), into the area of the powerhouse. A short vertical borehole (BH401) was drilled in 2008 at the collar position of the FAIS to the underground powerhouse. Three more boreholes (BH501 to BH503) were drilled in 2008 along the LPHT and IHPS alignment, for further hydrofracture test work in finalizing the detail design of the waterway linings in the upper part of the headrace.

Core samples taken from the drill rig were extracted from the borehole, wrapped in plastic cling wrap and tin foil before being waxed into cut-to-length PVC pipes of the appropriate size to prevent air slaking of samples. The core samples were then transported by road to rock testing laboratories in Gauteng, South Africa.

3.2.2 Regional geology

The Ingula Pumped Storage Scheme is constructed in sedimentary units of the Karoo Supergroup of Middle to Upper Permian age (260-250 million years) which has been intruded by dolerites belonging to the Karoo Dolerite Suite belonging to the Lower Jurassic (183 million years). Figure 3-3 shows the stratigraphy of the Karoo Supergroup. The powerhouse and all access tunnels are constructed in the Volksrust Formation mudstones of the Eccu Group which are overlain by Normandien Formation siltstones and sandstones of the lower Beaufort Group (Keyter et al., 2008).

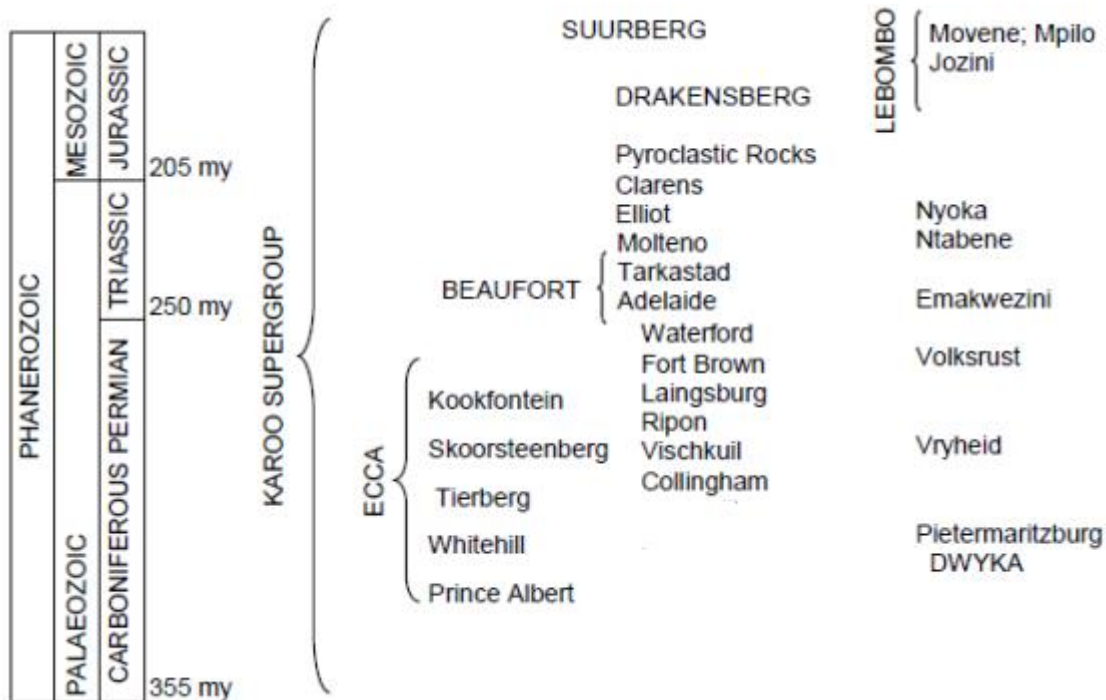


Figure 3-3: The stratigraphy of the Karoo Supergroup (Keyter et al., 2008)

The rocks of the Volkstrust formation in this project comprise interbedded siltstones or mudstones and carbonaceous mudstones with a thickness of up to 200m. Two main dolerite sills are also found on the project. The upper, 17m thick, sill is found at approximately 8m below the upper contact of the Volkstrust Formation. Located at about 30m above the machine hall is the lower dolerite sill with a thickness of approximately 40m. Along the margins of the main dolerite sills, prominent light brown zones of almost massive shale or indurated mudstone are often found. A number of south-east/ north-west dipping dolerite dykes with a thickness of 1-8m are found in the area. Fresh dolerite is often highly jointed, especially near its contact with sedimentary rocks (Keyter et al., 2008).

Experiencing rapid disintegration is typical of many Karoo mudrocks. However, the mudrocks above the powerhouse caverns, in which most of the access tunnels were being constructed, appear less susceptible to rapid disintegration and swelling, with oven-dried specimens exhibiting free swells on prolonged immersion in water of less than 1.5 to 2% (low potential expansiveness). This is probably due to induration effects of the two dolerite sills above the powerhouse caverns (Keyter et al., 2008).

A bulk of the stratigraphic succession at the project site is made up by the Normandien Formation of the Beaufort Group, with almost the entire escarpment face comprising these rocks. The Frankfort Member is the basal unit of the Normandien Formation which is approximately 130m thick, including an aggregate of about 35m of dolerite sills and comprises a succession of horizontally interbedded siltstone and sandstone layers, including frequent carbonaceous lenses, and often containing abundant mica along bedding planes. An approximately 280 m thick, series of strata dominated by mudrocks, some markedly carbonaceous is above the Frankfort Member. This unit contains two thin dolerite sills near its base and numerous horizons of sandstone (Keyter et al., 2008).

A prominent sandstone horizon known as the Rooinek Member forms the upper edge of the escarpment (Keyter et al., 2008). The sandstone is generally pale grey-white to olive in colour, and mainly comprised of quartz and quartzose rock fragments, with feldspar, a little mica, and a few clay clasts forming the remainder.

A number of dolerite dykes pass through the site. The prominent direction of the dykes is south-east or north-west. Size of the dykes vary from 1 to 8m in width and some wider with the larger dykes taking the form of sills. Fresh dolerite is often highly jointed especially close to the contacts with sedimentary rocks (Keyter et al., 2008).

Figure 3-4 shows a geological long-section of the project. The positions of most of the boreholes drilled during exploration can also be seen.

3.2.3 Faulting and seismicity

Numerous shear beddings have been identified in boreholes and in excavations. These shear beddings are characterised by an open rough, undulating contact on one side and a polished or smooth, planar wall contact on the other side with a thin calcite infilling. The open bedding planes with very little infill in places are reduced when the infill is pinched and swells. On a macro scale the shears roll gently and have been offset where intersected by fault planes. These beds have also been noticed to be stepped up or down between more prominent joints (Keyter et al., 2008).

Predominately E-W or ESE-WNW trending faults are found in the project area. Two other groups of small displacement faults striking NW-SE and SE-SW are also present. A normal fault was intersected

in the access tunnels close to the powerhouse. The fault is striking NE-SW (dip/dip direction 52°/342°) and is dipping towards the powerhouse; a few of these fault planes intersect the far eastern side of the transformer hall (Keyter et al., 2008).

Although the Tugela Fault is located 50km to the south of the site, there is no known seismically active fault present in the immediate project area. In this region the Tugela fault follows the boundary between the Kaapvaal Craton and the Namaqua Province (Keyter et al., 2008).

A study on the probabilistic seismic hazard analysis of the project site showed a maximum regional earthquake magnitude of 6.5 ± 0.38 , with an annual probability of occurrence of maximum horizontal peak ground acceleration (PGA) of 0.11g (Kijko et al., 2005). This earthquake will probably occur 50km or less away from the project site, estimated at approximately 1.3×10^{-5} per annum. The vertical-to-horizontal ground acceleration ratio was taken after Ambraseys (1995) as 0.6. Mean PGA values associated with the 1 in 200, 500, 1000 and 10 000 year return periods are given in the Table 3-1 below.

Table 3-1: List of Mean PGA Values for Different Return Periods (Kijko et al., 2005)

Return Period (years)	PGA (g)
200	0.01
500	0.018
1000	0.025
10 000	0.075

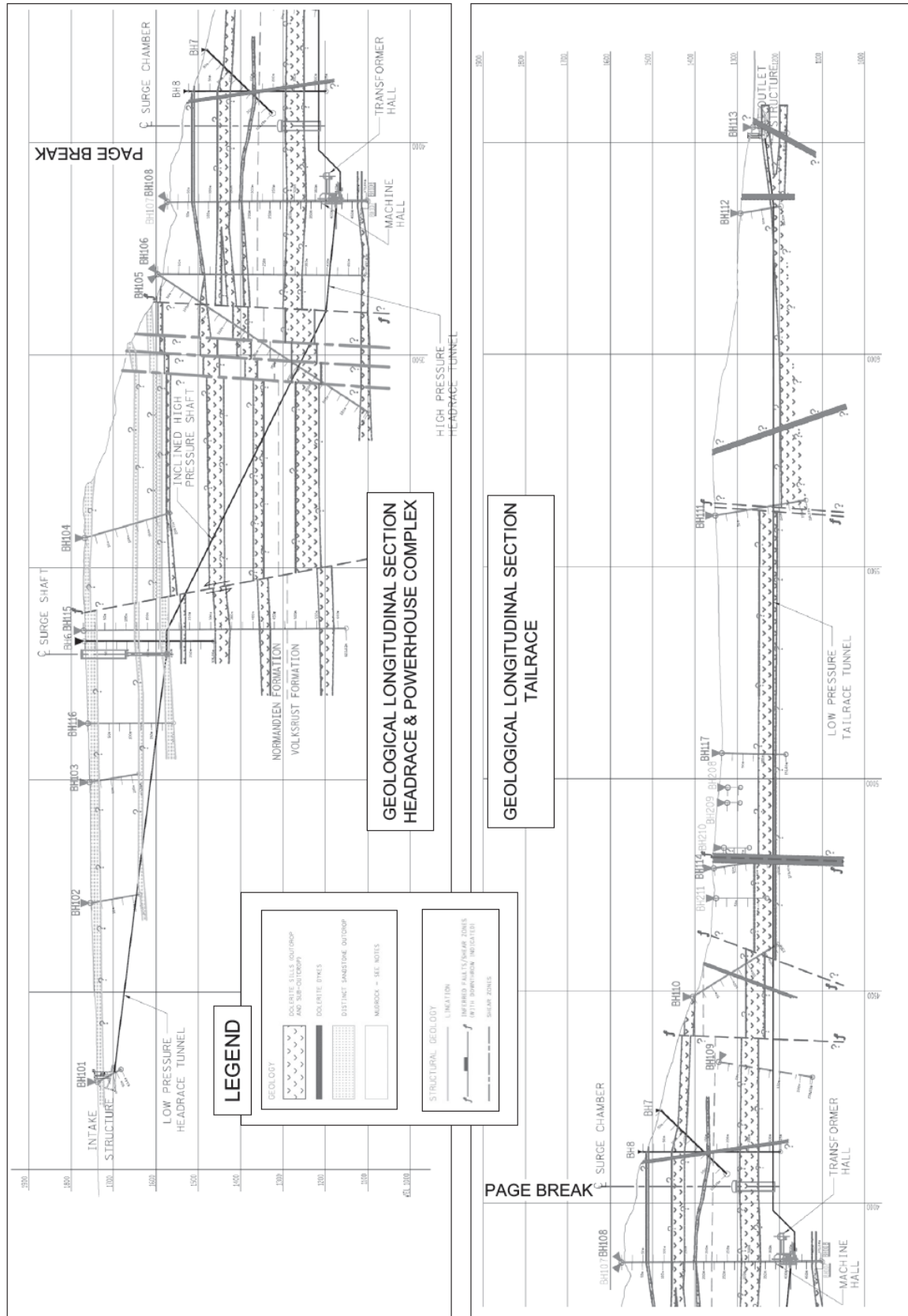


Figure 3- 4: Geological Long-section for the Scheme (Keyter et al., 2008)

3.3 Rock mass condition at the Ingula Pumped Storage Scheme

3.3.1 Rock mass classification

Q-system (Barton, 2002), RMR (Bieniawski, 1976) and GSI classifications (Marinos and Hoek, 2000) were carried out in the powerhouse. Table 3-2 and Table 3-3 represent the Q rating and the RMR_{89} in different lithological units.

Table 3-2: Summary of Q Ratings (Keyter et al., 2008).

Lithological Unit	RQD	RATINGS					Q (typical range)	Rock Class
		I_n	I_r	I_a	I_w	SRF		
Mudstone	95	95	1 – 1.5	1 – 2	1	1	8.9 (3.2 – 19)	Fair (poor-good)
Dolerite	80	6 - 12	1.5 - 3	2 - 4	1	1	8.4 (2.5 – 20)	Fair

Table 3- 3: Summary of RMR_{89} Classifications (Keyter et al., 2008)

Lithological unit	RATINGS						Q (typical range)	Rock Class
	σ_{ci}	RQD	Joint Spacing	Joint Condition	Groundwater	Joint Orientation		
Mudstone	7	20	20	20	15	-5	77 (60-85)	Good
Dolerite	15	17	11	25	25	-12	71 (50-70)	Good

3.3.2 Rock mass modulus

The rock mass modulus at the IPSS was determined using empirical methods and in situ tests. In situ tests include Goodman Jack Testing, wireline geophysics and plate bearing testing. Results for the rock modulus will be discussed in this section.

Empirical Method

The rock mass modulus in the mudstone was derived using the empirical method by Hoek and Diederichs (2006) for GSI ranging from 60 to 85. For undisturbed rock mass with a disturbance factor of zero, the method yielded rock mass modulus values ranging from 9.9 to 17.6 GPa. A rock mass modulus ranging between 3.2-9.1 GPa was calculated for the blast damaged zone, using a D factor of 0.8 (Keyter et al., 2008).

Goodman Jack Testing

The Goodman Jack Test was carried out in borehole ET 301 in accordance with ISRM (1996). In situ modulus of deformation values were then calculated using the methods described by Heuze (1984) and Heuze and Amadei (1985). An average in situ modulus of deformation of 15.8GPa was yielded from the Goodman Jack between 1155 and 1200 masl, which was 80% of that derived from laboratory testing of intact rock core in the mudstone. The presence of a dolerite sill resulted in higher in situ deformation modulus values being computed between 1130 and 1155 masl (Keyter et al., 2008).

Wireline Geophysics

An average dynamic modulus of elasticity of 32.4 GPa was derived from using down-hole geophysics in the mudstone. When the dynamic modulus is compared to the true modulus obtained from the Goodman Jack test a ratio of 0.6 is attained. A lower ratio of 0.5 is attained when compared to the static modulus values of the laboratory tests (Keyter et al., 2008).

Plate Bearing Testing

Plate bearing tests were conducted at the machine hall crown level (Keyter et al., 2008). Four tests were conducted in accordance with ISRM (1981) and Unal (1997), two set up vertically and two set up horizontally. Each test involves five cycles of equal loading increment to a maximum load of 12 MPa, with loads applied using hydraulic jacks and 400mm diameter stiff plates acting on opposite walls of the adit. Plate bearing tests yielded rock mass modulus results ranging from 5 to 21 GPa during reloading cycles and from 6 to 19 GPa during unloading cycles. These results are representative of the undisturbed rock mass and correlate well with the Goodman Jack test work described above.

Back Analysis from Convergence Monitoring Data

Keyter et al. (2008) describe how the back analysis of convergence monitoring data from an array of multiple borehole extensometers was conducted to estimate the rock mass modulus for the mudstone unit in the construction access tunnel at the crown level of the machine hall.

The instrumented tunnel section was modelled in Phase 2. Open bedding located around the spring line as well as two shear beddings, one 0.5m below the tunnel crown and the other just above the invert were present in the tunnel section. These were modelled as discrete planes. The rock mass between the bedding planes was modelled as a Hoek-Brown material using mean values for σ_{ci} and E_i . A disturbance factor of 0.8 was used in the Phase2 model (Keyter et al., 2008).

Results of the analysis demonstrated that to emulate convergence monitoring data from the construction access tunnel, a GSI of 60 to 70, with a rock mass value in the order of 10 to 14 GPa is required (Keyter et al., 2008).

3.3.3 In situ Stress

It was initially assumed that the horizontal: vertical ratio of 2.5 determined for the Drakensberg Pumped Storage Scheme would apply for the IPSS (Hoek, 2007). Golder Associates (2006) conducted in situ measurements using hydrofracturing in five boreholes, at three topographically and lithologically different sites as follows:

- Boreholes BH115 and BH116 are above the escarpment slope. The topography is relatively gentle here. Borehole BH115 is located at the surge shafts. The escarpment slope has a series of prominent sandstone beds from ground surface to approximately 200m depth.
- Boreholes BH106 and BH108 are on the escarpment slope, immediately above the powerhouse complex. Streams have cut into the slope and may have an influence on the ground stresses. Borehole BH106 is located close to a shear zone, which was intersected in borehole BH105.
- BH117 is in the tailrace area, below the escarpment slope. The topography is relatively gentle here. Prominent dolerite sills occur below the escarpment and are present above and below the powerhouse complex.

The hydrofracture tests yielded low minimum stress in sedimentary rocks and higher minimum stresses in the dolerite sill. The data as a whole had a mean ratio of horizontal stress: vertical stress (σ_{min}/σ_v) of 1.0 with a range from 0.4 to 4.4; the higher being in the dolerite.

The vertical ground stress was not measured during hydraulic fracturing tests and is derived from the unit weight of the rock mass and the depth to the test point. Two equations are used to determine the intermediate stress from the rock material's tensile strength and /or test parameters (Golder Associates, 2006).

A review of the hydrofracture data suggested that the value of the minimum stress in the sedimentary rocks would be depth related. Due to few data points being available at shallow depth; projection of the regression line suggested that minimum stress would be zero between ground surface and 100m. Further hydraulic fracture tests were conducted in three boreholes between the intake channel and the surge shafts. Hydrofracturing tests in BH 106 and BH 108 were validated using overcoring tests conducted in a test adit of a construction tunnel near the powerhouse. Major stress differences were shown between the stress data derived from hydrofracture test (Keyter et al., 2008).

Hydrojacking tests were undertaken in BH116 to further examine the minimum stress data. Results show that above 200m the rate of stress reduction towards the ground is reduced significantly. Figure 3-5 shows three regression lines were apparent from jacking pressures:

- Ground surface to 200m depth: $\sigma_j(\text{MPa}) = 0.01 \times \text{depth (m)}$
- Below 200m depth: $\sigma_j(\text{MPa}) = 0.021 \times \text{depth (m)} - 2.7$

where, σ_j is the jacking pressure

Due to few data points, the upper regression line is only a projection.

Rock mass above the main dolerite sill

Eighteen hydrofracture tests were conducted in the sedimentary rocks above the dolerite sill. The sandstones and/or siltstones yielded a σ_{min}/σ_v ratio of 0.5, which was lower than that of the mudstone of 0.8 to 1.4. The σ_{min} value would therefore be different in the mudstones and sandstones/siltstones. The sandstones / siltstones had steeply dipping hydrofractures, but then tend to be shallow in the mudstones (Golder Associates, 2006).

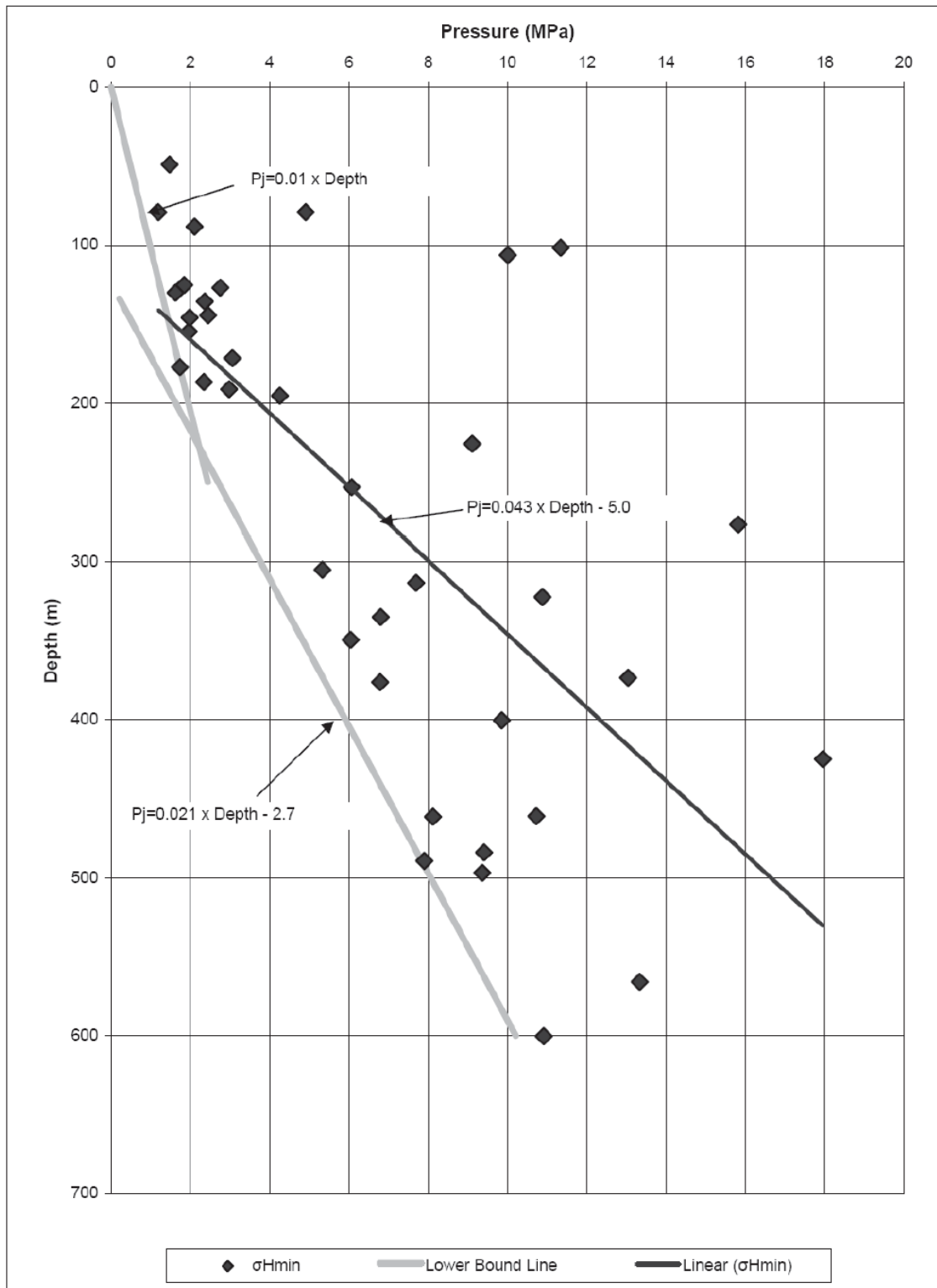


Figure 3-5: Jacking Pressure vs. Depth (Keyter et al., 2008)

Mudstones below the main dolerite sill

The σ_{min}/σ_v ratio ranged from 0.5 to 0.9, with a mean of 0.7, in eleven tests (Keyter et al., 2008). Two tests located in a zone with horizontal bedding gave higher ratios of 1.2 and 1.6. It is assumed the higher horizontal stress is also due to the dolerite intrusion at the same level of the shear zone. The σ_{min} values show no relationship with the depth of the main dolerite sill; which also suggests that σ_{min} is not influenced by proximity of the sill. Two predominant orientations are shown for the σ_{min} , which are similar to those of faults and dykes. This suggests that minimum horizontal stress and maximum horizontal stress have similar magnitudes.

In situ stress in the dolerite sill

A large dolerite sill crosses above the powerhouse and intersects the downstream surge shafts, exploration tunnel and tailrace tunnel. Four hydrofracture tests were performed in the 33m to 41m dolerite sill. The σ_{min} value ranged from 12.0 MPa to 12.8 MPa even at test depth range of 101.7 to 565.8m. This suggests that the sill locks in the minimum stresses at approximately 12 MPa (Keyter et al., 2008).

3.4 Rock mass conditions and support along the tailrace tunnel

Rock mass conditions in the tunnel were estimated during the planning stage of investigations using surface and subsurface investigations. Five boreholes (BH110 to BH112, BH114 and BH117) were drilled into the tailrace tunnel during the surface geotechnical explorations by Braamhoek Consultants Joint Venture (BCJV). A summary of the drilled borehole depths and borehole inclination is shown in Table 3-4. During excavation some of the rock mass conditions were found to be different from those in the planning stage. The tailrace tunnel has already been excavated to completion.

Excavation of the tailrace tunnel commenced in March 2009 and was completed in April 2011. The whole tailrace tunnel from chainages 0.000m to 2347.77m was excavated by adopting the conventional drill and blast methodology. Support is installed after scaling and mucking the face before the next blast. Shotcrete and rock bolts were used to support the tailrace tunnel. Drilling and blasting happened from the two ends of the tunnel until there was a breakthrough in the middle of the tunnel.

Table 3-4: Summary of boreholes drilled along the tailrace tunnel (Golder Associates, 2006)

Borehole No	Borehole depth (m)	Borehole inclination (°)
BH 110	235m	55°
BH 111	216.57m	80°
BH 112	96.33m	80°
BH 114	173.42m	80°
BH 117	151.49m	90°

Actual rock mass conditions encountered during excavation were used to determine the support that was installed in the tunnel. Three types of rock mass classification systems were used on site namely the Q-system (Barton, 2002) the rock mass rating (RMR) system (Bieniawski, 1989). The support class was determined using the Q-system and the geological strength index (GSI) system (Marinos and Hoek, 2000). For the project, the support classes were adjusted to suit the project. This adjustment is shown in Table 3-5. A summary of the rock mass classification and support classes is given in Table 3.6 as well as the geology encountered in the tunnel.

Table 3-5: IPSS adjustment of Q values for support classes (Keyter et al., 2008)

Support Class	Q values	IPSS adjusted Q values
A	1000 – 40	1000 – 10
B	40 – 10	10 - 4
C	10 - 4	4 – 1
D	4 – 1	1 – 0.4
E	1 – 0.4	0.4 – 0.1

Table 3-6: Rock mass support class and geology of tailrace tunnel

Chainage (m)		Rock Quality class	Support Class	Geology
From	To			
0000.0	0002.50	Poor	C	Mudstone/ Dolerite
0002.50	0009.60	Fair	B	Mudstone/ Dolerite
0009.60	0020.00	Poor	C	Mudstone/ Dolerite
0020.00	0054.55	Poor	C	Indurated Mudstone/ Dolerite

0054.55	0086.60	Fair	B	Indurated Mudstone/Dolerite
0086.60	0099.70	Fair	B	Dolerite
0099.70	0124.00	Fair	B	Indurated Mudstone/Dolerite
0124.00	0158.16	Fair	B	Mudstone/ Dolerite
0158.16	0165.50	Poor	C	Indurated Mudstone/Mudstone/Dolerite
0165.50	0212.00	Fair	B	Indurated Mudstone/Mudstone/Dolerite
0212.00	0247.30	Fair/Poor	B/C	Indurated Mudstone/Mudstone/Dolerite
0247.30	0352.76	Good/Fair	A/B	Indurated Mudstone/Dolerite
0352.76	0363.00	Poor	C	Indurated Mudstone/Dolerite
0363.00	0378.40	Fair	B	Indurated Mudstone/Dolerite
0378.40	0405.50	Poor/Fair	C/B	Indurated Mudstone/Mudstone/Dolerite
0405.50	0483.00	Fair/Poor	B/C	Indurated Mudstone/Mudstone
0483.00	0696.60	Poor	C	Mudstone
0696.60	1025.40	Poor/Fair	C/B	Mudstone
1025.40	1107.00	Poor	C	Mudstone
1107.00	1227.50	Poor	C	Indurated Mudstone/Mudstone
1227.50	1328.10	Poor	C	Dolerite
1328.10	1412.50	Poor	C	Indurated Mudstone/Dolerite
1412.50	1542.13	Poor	C	Dolerite
1542.13	1604.50	Poor /Very Poor	C/D	Indurated Mudstone/Dolerite
1604.60	1960.60	Poor	C	Dolerite
1960.60	1983.50	Poor	C	Indurated Mudstone/Mudstone
1983.50	2026.17	Poor	C	Indurated Mudstone/Mudstone/Dolerite
2026.17	2045.52	Poor	C	Indurated Mudstone/Mudstone
2045.52	2081.80	Poor	C	Indurated Mudstone/Mudstone/Dolerite
2081.80	2090.15	Poor	C	Indurated Mudstone/Mudstone

2118.10	2345.77	Poor	C	Mudstone
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The entire tailrace tunnel is located within the mudstone of the Volkrust Formation with a dolerite sill intercepting the excavation in places. The thickness of the dolerite sill is approximately 40m thick. Indurated mudstone is encountered at the dolerite and mudstone contact zones. Tailrace tunnel comprises of 37% mudstone, 26% dolerite and 37% mixed geology, in mixed geology the face of the tunnel consists of more than one geology i.e. indurated mudstone, mudstone and dolerite. The different geology encountered from mapping the tunnel can also be seen in Table 3-6.

Mappings of the tunnel were done by geologists after mucking and scaling of the tunnel face. Figure 3-6 to 3-9 show a typical profile found along the tailrace tunnel. Face mappings of the tunnel describe the different lithology as follows:

- Medium strong, thickly bedded, dark grey, fine grained, MUDSTONE (Volkrust Formation). Fresh, medium widely jointed, rough planar, very tight joints. Rock mass is generally massive. No water seepage observed.
- Strong, massive, dark, greenish to grey, DOLERITE (Karoo Suite). Slightly weathered, closely to moderately widely spaced, irregular planar, undulated and partly open joints. Rock mass is generally massive to blocky. No water seepage observed.
- Strong, thickly bedded, light grey, fine grained, INDURATED MUDSTONE (Volkrust Formation) fresh to slightly weathered, closely to moderately spaced, smooth to slightly rough, planar and tight joints. Rock mass is massive to blocky. No water seepage observed.

A number of features parallel to the bedding and faults have been identified within the mudstones through boreholes and mappings of the tailrace tunnel. Mappings have identified a number of shear beddings typically 10 to 25mm thick with a calcite infill in the mudstones. The bedding planes are mostly dipping towards the north. There are numerous small displacement faults that intercept the tunnel striking north east/south west and north west/south east. A half a metre thick hydrothermal vein is observed between the chainages 1007.00m to 1146.00m.



Figure 3-6: The face of the tailrace tunnel at chainage 165.60m

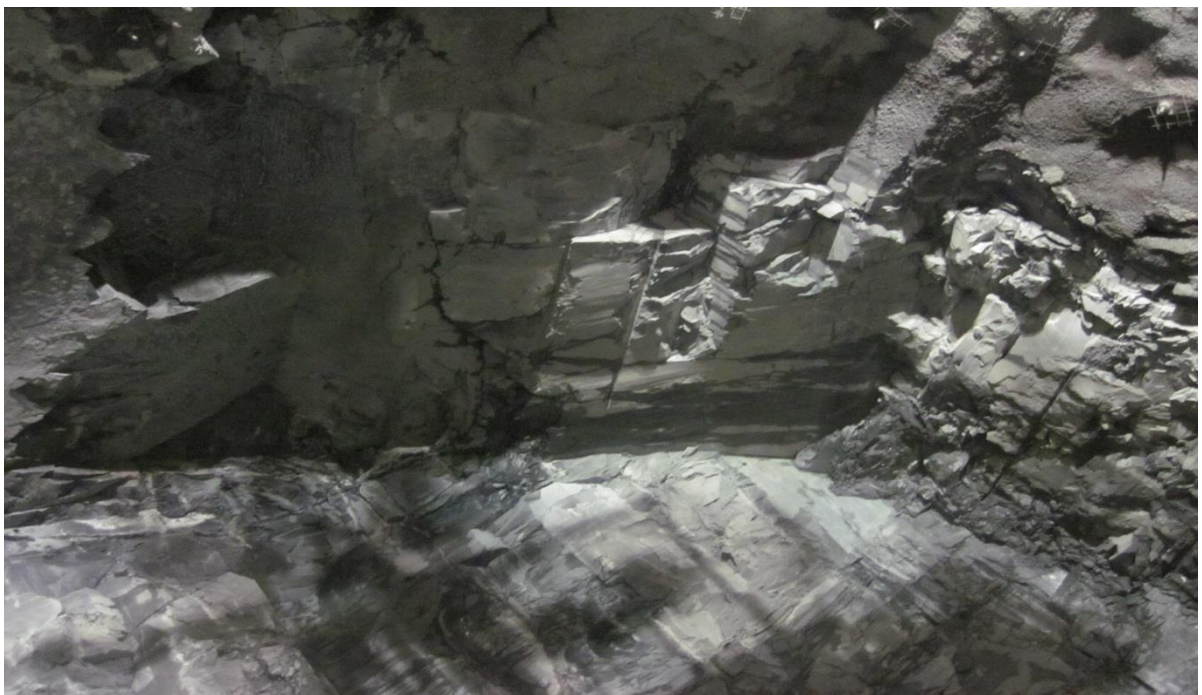


Figure 3-7: Tailrace tunnel crown at chainage 165.60m

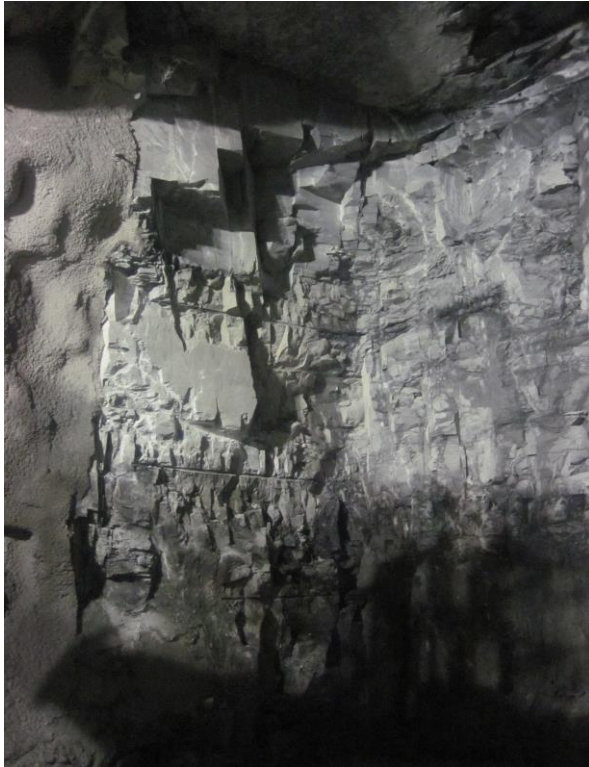


Figure 3-8: Left sidewall of the tailrace tunnel at chainage 165.60m



Figure 3-9: Right sidewall of the tailrace tunnel at chainage 165.60m

4. Deformation analysis

4.1 General

In the initial stage of the tunnel design, numerical models are developed to predict how a rock mass will respond to stress after tunnel excavation. These models are based on geotechnical parameters acquired during the feasibility stage investigations. The main concern of this study is to compare the model predicted deformations in the tailrace tunnel (TT) to the in situ measured deformations and determine the modulus of deformation in the rock mass. Differences between the predicted deformation and monitored deformation necessitate the review of the numerical models used for design. Numerous studies including geotechnical investigations, hydraulic fracture investigations and in situ tests were conducted for the design of the Ingula Pumped Storage Scheme (IPSS) caverns. An extensive geotechnical monitoring and instrumentation programme was put into place during the construction of the IPSS to validate design assumptions and monitor deformations. Data from the investigations and the geotechnical monitoring will be made use of in this research report.

Methods and approaches described in Chapter 2 are used to analyse the data. A finite element model is used to numerically analyse the deformation in the TT. Empirical relationships and back analysis are employed to calculate the modulus of deformation. The empirical relationships of Serafim and Pereira (1983), Hoek and Diederichs (2006), Hoek and Brown (1997) as well as Sonmez et al. (2006) are employed to determine the rock mass modulus. In the back analysis method, the monitored displacement results are used to calculate the rock mass modulus using the method developed by Kirsten (1976) involving finite element analysis. The back analysed deformation modulus values are then compared to the empirically computed deformation modulus values to determine which empirical relationship best estimates the modulus of deformation in the TT. Five points in the tailrace tunnel are modelled, three points in mudstone and two points in dolerite.

4.2 Input data collection

4.2.1 General

Geotechnical investigations and in situ tests were undertaken to determine the rock engineering aspects of the IPSS. The geology of the site proved to be structurally complex with weak mudstones intruded by dolerite sills with associated faulting and sheared bedding. Geotechnical reports, test

results, tunnel logs, photographs, instrumentation data from the field, drawings, feasibility reports and other project related reports were used in this research.

Tunnel logs

Mapping of the tunnel face was conducted by a geologist after each excavation advance and mucking. The mapping included a representation of geological structures, rock mass description, description of the main discontinuities and rock mass classification. Classification systems used are the Q-system (Barton, 2002), the rock mass rating (RMR) system (Bieniawski, 1989), and the geological strength index (GSI) system (Marinos and Hoek, 2000). Figure 4-1 shows a typical face map at chainage 165.52 m.

The deformation data was chosen from five sections in the TT. Tunnel sections were selected based on rock type and the information available in the particular section. Three of the sections are in mudstone at chainages 520.40m, 872.70 m and 2259.50 m. Two sections are in dolerite at chainages 1718.70m and 1937.80 m.

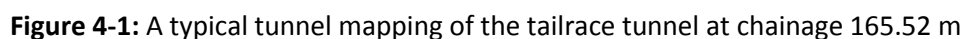
Instrumentation

The total length of the tailrace tunnel is 2 346m. Numerous instrumentation arrays were installed in the tailrace tunnel. The instrumentation consists primarily of surveyed convergence points using multiple point borehole extensometers (MPBX) and instrumented rock bolts. Convergence was measured in the TT from 2010 to 2013.

4.2.2 Rock mass parameters

Comprehensive studies were conducted at the IPSS during the feasibility and design stage to determine the rock mass parameters on the site. Both field tests and laboratory tests were carried out to determine the rock mass parameters. Additional literature was also used to estimate the rock mass parameters. This literature includes scientific papers, books, an internet search, websites etc. Estimations are based on similar rock types, case studies and rock mass conditions. Mudstone and dolerite are the rock types that will be encountered in the selected sections. The rock mass parameters used in the research are shown in Table 4-1.

Intact rock properties were obtained from laboratory testing on borehole cores from site. The uniaxial compressive strength in the dolerite ranges from 242 to 401 MPa with an average value of 284 MPa (Keyter et al., 2008). A general decreasing trend in the intact rock strength and stiffness with depth is evident. Uniaxial compressive strength of mudstone ranges from 41 to 123 MPa with an average of 90 MPa. This range may be due to changes in the mudstone composition with depth.



Density and Poisson's ratio

The rock mass density and Poisson's ratio were obtained from laboratory testing on borehole core from site. Rock mass density was measured to be 2.85g/cm^3 and 2.63g/cm^3 , for dolerite and mudstone respectively. Poisson's ratio for dolerite and mudstone are 0.29 and 0.28 respectively (Keyter et al., 2008).

Young's modulus of Elasticity of intact rocks, E_i

Young's modulus of Elasticity of intact rocks (E_i) was obtained from laboratory testing on borehole core from site. This was measured to be 89.00 GPa and 19.20 GPa for dolerite and mudstone respectively (Keyter et al., 2008).

Hoek and Brown constant, m_i

The Hoek-Brown constant (m_i) values are 16 for dolerite and 7 for mudstone (Keyter et al., 2008).

Disturbance factor, D

The value of disturbance factor is 0.8, which is considered reasonable for a semi-confined rock mass (Keyter et al., 2008). Guideline for the selection of the disturbance factor by Hoek et al. (2002) can be seen in Appendix A1-1.

Table 4-1: Input data for deformation analysis for select tunnel sections (Keyter et al., 2008)

Face Mapping Chainage	Rock type	Weathering	Overburden depth, (m)	Density, g/cm^3	Poisson's ratio	Uniaxial compressive strength (σ_{ci}), MPa	Young's modulus, E_i , GPa	Hoek and Brown constant, m_i	Q-Value	RMR	GSI
520.40	Mudstone	Fresh	95	2.63	0.28	90	19.2	7	4.72	70	75
872.70	Mudstone	Fresh	110	2.63	0.28	90	19.2	7	4.72	65	70
1718.70	Dolerite	Fresh	115	2.85	0.29	284	89.0	16	3.54	55	60
1937.80	Dolerite	Fresh	165	2.85	0.29	284	89.0	16	2.25	57	63
2259.50	Mudstone	Fresh	255	2.63	0.28	90	19.2	7	2.67	68	75

4.2.3 Instrumentation

Ground displacements as well as ground anchor loads during construction of the IPSS tunnels were monitored using installed instrumentation arrays. The instrumentation was critical for the validation of design assumptions and the analysis and timely detection of unanticipated tunnel convergence

and loads in the rock bolts. Some of the instrumentation has been retained in tunnels and in the power caverns after construction to monitor the long term effects during the operation of the scheme. The ground displacements in the TT were measured from 2010 to 2013.

Instrumentation arrays included load cells, multiple point borehole extensometers (MPBX) and optical convergence targets. The MPBX was used to determine the actual ground displacements that were experienced in the TT. Instrumentation was located where areas of concern were perceived to exist, based on tunnel models. Figure 4-2 shows the positions of the MPBX in the TT and defines the location of sidewall A and C.

After the installation of monitoring instrumentation, readings were taken after each blast in the tunnel until the rapid phase of convergence had passed. The frequency and timing of the readings thereafter was decided by the geotechnical engineer on site to provide readings at various construction phases, for design validation.

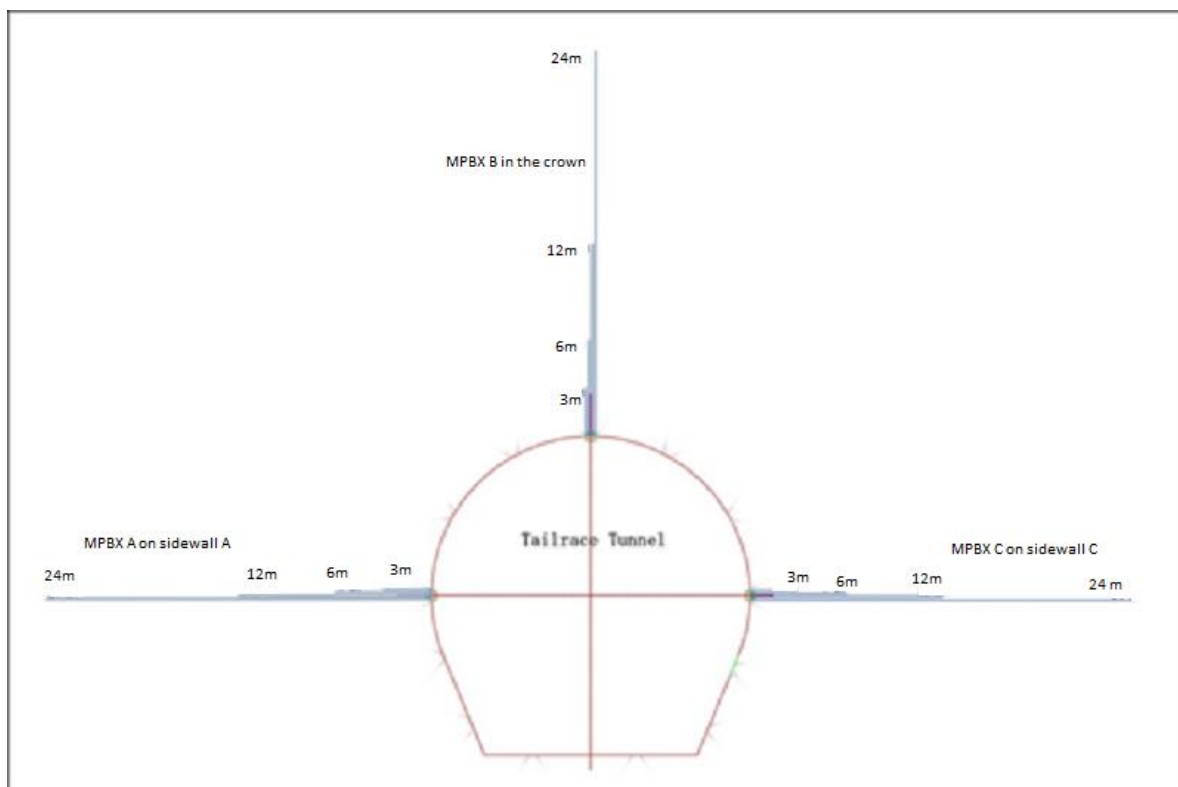


Figure 4-2: Location of Multiple Point Borehole Extensometers installed in the tailrace tunnel

4.2.4 Rock mass support

Rock mass classification systems have been used on the site to determine the rock mass quality and to determine the support to be used in the tunnel. The Q-system proposed by Barton et al. (1974) was used to classify the rock mass quality into the seven classes depending on the value of Q. Q-values considered covered the expected ground conditions, but also extended to cover the potential of a tunnel encountering very poor ground conditions, for example, in a wide shear zone. Six classes of tunnel support were derived, from class A to class E. These support classes have been described in Table 3-5. A range of Q-values from 0.04 to between 10 and 40 was covered, for extremely poor tunnelling condition and good conditions respectively.

To determine the rock support type to be installed in the tunnel, the Q-value and the ratio of span of tunnel and excavation support ratio (ESR) were used. The ESR value for different excavations is given in Appendix A1-2. Figure 4-3 shows the relationship between excavation diameter and rock bolt length used in the IPSS tunnels.

The TT of the IPSS is 11.8m in diameter and the ESR is taken as 1.3. A value of 9.07 was calculated as the equivalent dimension (span/ESR). Using the support chart and the Q-value, the support to be installed in the TT was determined. The required support type was predicted to be fibre reinforced shotcrete and rock bolts. This support type also applies to all the points selected for analysis in the TT.

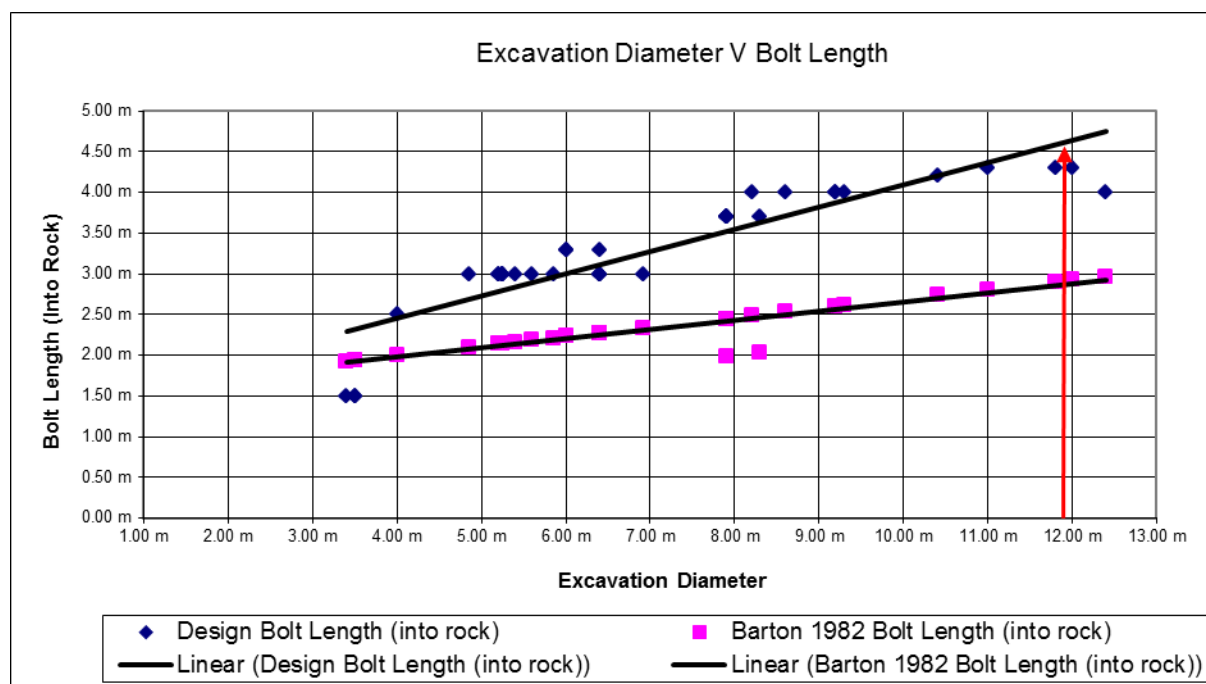


Figure 4-3: The relationship between excavation diameter and rock bolt length

4.3 Modulus of deformation calculations using empirical relationships

The outcome of a comparative study of empirical methods to determine deformation modulus by Shen et al. (2012) is used to choose the empirical methods that are used to calculate the deformation modulus of the rock mass in which the TT is excavated. Relationships were selected based on the quality of data used to derive the relationships and which relationships gave the best results. Modulus of deformation has been calculated using empirical relationships by Serafim and Pereira (1983), Hoek and Diederichs (2006), Hoek and Diederichs (2006) where E_i is considered, Sonmez et al. (2006) and Hoek and Brown (1997). These empirical relationships were discussed in Chapter 2. The input data from Table 4-1 was used to calculate the modulus of deformation for the five tunnel sections which were selected. A comparison of the results from the calculations is shown in Table 4-2.

Table 4-2: A Comparison of modulus of deformation using empirical methods

Chainage (m)	Em (GPa), Serafim and Pereira (1983)	Em (GPa), Hoek and Diederichs (2006)	Em (GPa), Hoek and Diederichs (2006) with E_i	Em (GPa), Sonmez et al. (2006)	Em (GPa), Hoek and Brown (1997)
520.4	31.6	8.4	7.0	1.2	0.7
872.7	23.7	5.6	5.6	0.8	0.7
1718.7	13.3	2.4	15.2	1.5	1.0
1937.8	15.0	3.1	18.1	1.8	1.1
2259.5	28.2	8.4	6.9	1.0	0.7

From Table 4-2 it can be seen that, the Serafim and Pereira (1983) approach results in highest values of the modulus of deformation for mudstone. Although, Hoek and Diederichs (2006) empirical relationship where E_i is considered achieved the highest results for the modulus of deformation in the dolerite, these results are within the same range as the results from the Serafim and Pereira (1983) equation. The empirical relationships by Hoek and Brown (1997), which consider GSI and Sonmez et al. (2006) which consider RMR, yield similar results for the modulus of deformation for both rock types.

Results from Goodman Jack test gave an average deformation modulus of 15.8 GPa and the Plate bearing test yielded a range from 5 to 21 GPa during reloading and from 6 to 19 GPa during unloading. The undisturbed in situ deformation modulus is best represented by the plate bearing

test results. Plate bearing results also correspond well with the Goodman Jack test results. Bearing this mind, the results of the empirical methods fit well within the range of the in situ measured results with the expectation of the Hoek and Brown (1997) method. The Hoek and Diederichs (2006) empirical relationship where E_i is considered, yields results that best fit in the range of in situ measured results. This corresponds with the Shen et al. (2012) results from their comparative study. The above results for modulus of deformation from the empirical methods will be later compared with the back calculated deformation modulus to see which of the five empirical methods yielded the best results.

4.4 Numerical Analysis

In this research, the deformation in the tunnel wall has been determined using the RocScience Phase 2 program (Rocscience, 2014). The resulting deformation is then compared with the deformation measured on site. Numerical analysis was conducted for the five selected tunnel sections. Deformation in the tunnel sections was modelled for elastic material analysis.

In the program, field stress is taken as constant from; “Hydraulic fracturing stress measurements at Braamhoek Pumped Storage Scheme, South Africa” study that was carried out on the site by Golder Associates (2006). This study also gave the field stresses and their direction on sites, which are shown in Table 4-3. The generalized Hoek-Brown failure criterion was used to calculate the rock mass properties. Data from Table 4.1 was used for most of the input parameters in the program.

Fibre reinforced shotcrete and rock bolts are the support type installed in the tunnel. In the numerical model rock bolts are installed first followed by the tunnel lining.

Table 4-3: Stresses and stress orientation at IPSS

Chainage	Maximum horizontal stress (MPa)	Minimum Horizontal stress (MPa)	Vertical Stress (MPa)	Direction of stress (°)
520.4	9.2	1.6	2.4	-58
872.7	7.8	1.4	3.4	-36
1718.7	18.2	9.6	2.9	-38
1937.8	24.3	12.0	2.7	-14
2259.5	9.8	6.4	6.8	-52

4.4.1 Phase 2 model generation

A model was generated for each tunnel section for Phase 2 analysis. For each section, elastic analysis models were generated for analysis with and without support. The field stresses were set as constant and the stress measurements from hydraulic fracture tests was loaded. Figure 4-4 shows the model generated for chainage 520.40m of the tunnel. Support installed in the tunnel at chainage 520.40 is shown in Figure 4-5.

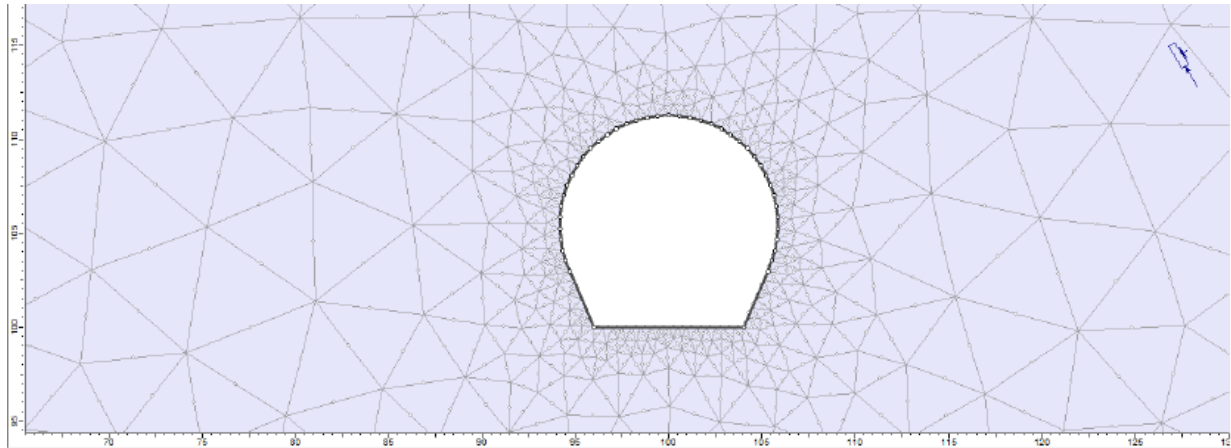


Figure 4-4: Phase2 model at change 520.40m

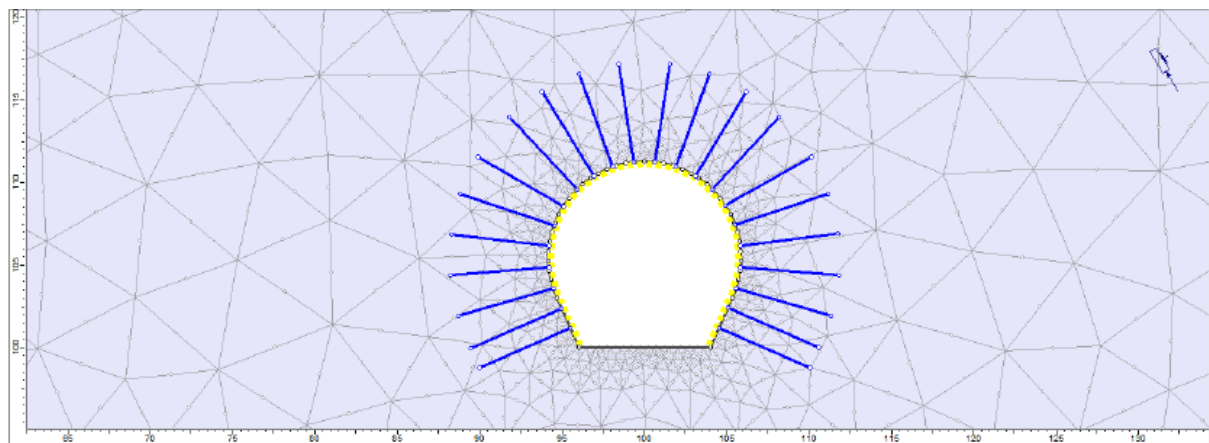


Figure 4-5: Closer view at tunnel excavation and tunnel support at chainage 520.40m

4.4.2 Elastic Analysis

Elastic analysis was used to analyse the stress, strain and deformation around the tunnel. Ratio of available rock mass strength to induced stress at any given point is represented by the strength factor. If the strength factor is less than 1 there will be failure if the rock is left unsupported. Major

principal stresses and total displacement of the tunnel section are analysed and compared with and without support. The results of the model can be seen in the figures below.

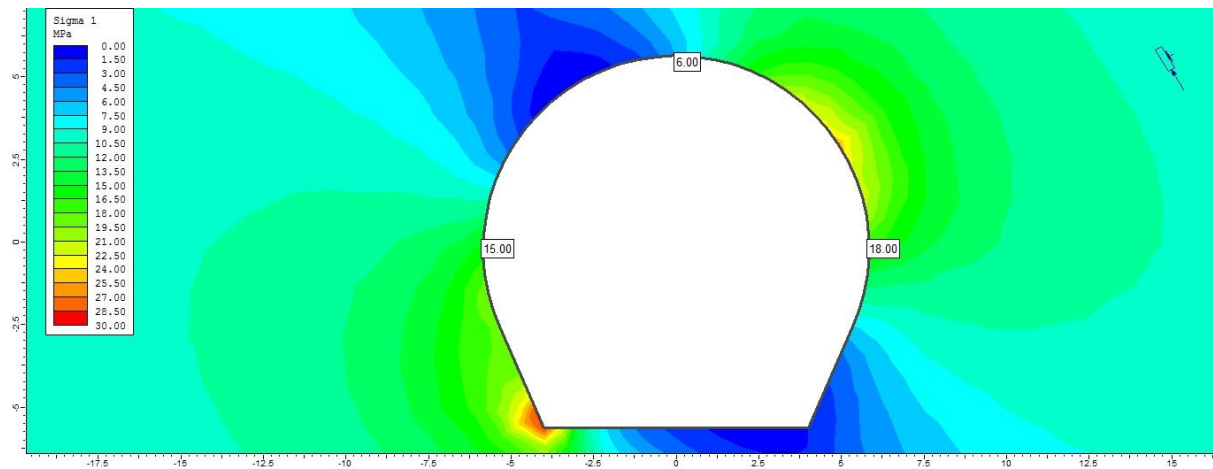


Figure 4-6: Major principal stress after excavation at chainage 520.40m

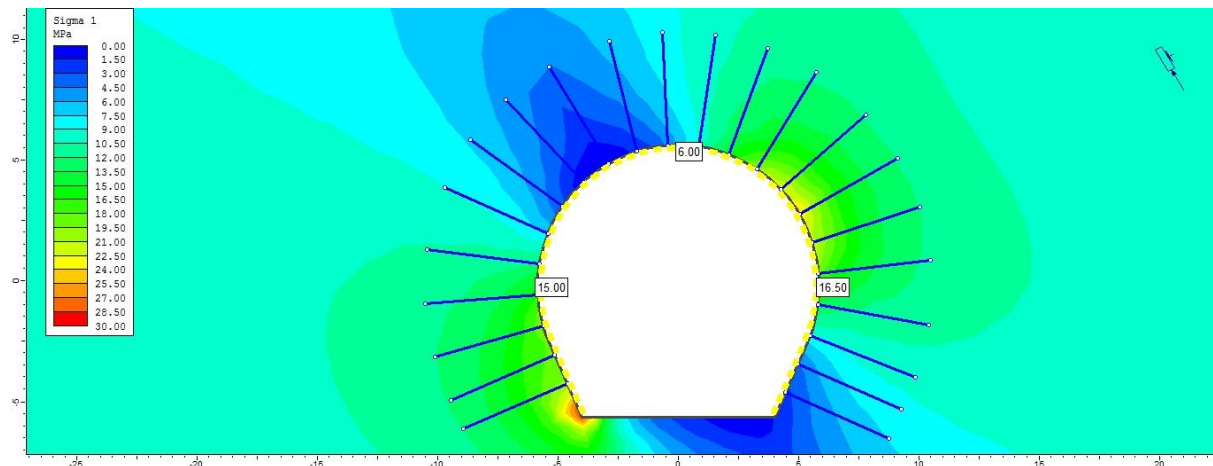


Figure 4-7: Major principal stress after support at chainage 520.40m

Normally, the orientation of stresses are well aligned but this not the case at the IPSS. This may be due to the magnetic influence of the dolerite sill found on site. An average horizontal: vertical in situ stress ratio of 1.0 was determined using data from the whole site, but the range was 0.4 to 4.4 with higher ratios in dolerite. Figure 4-6, shows that more stress develops on the lower left sidewall with the highest stress on the left corner towards the invert as well as the upper right sidewall towards the crown of the excavation. A minor change in major principal stress in the TT after the installation of support is shown in Figure 4-7. Major principal stresses of the other tunnel sections before and after support can be seen in Appendix A2.

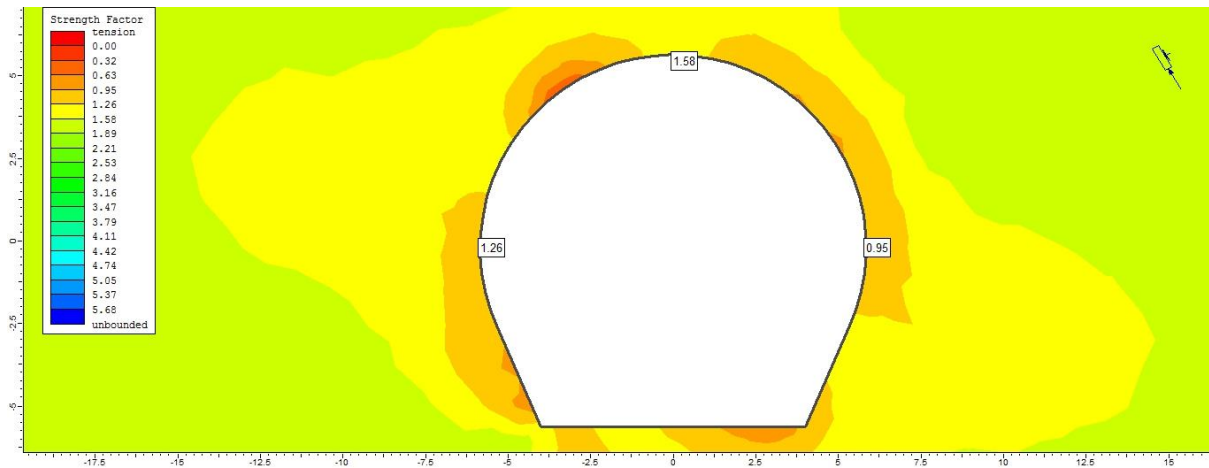


Figure 4-8: The strength factor before application of support at chainage 520.40m

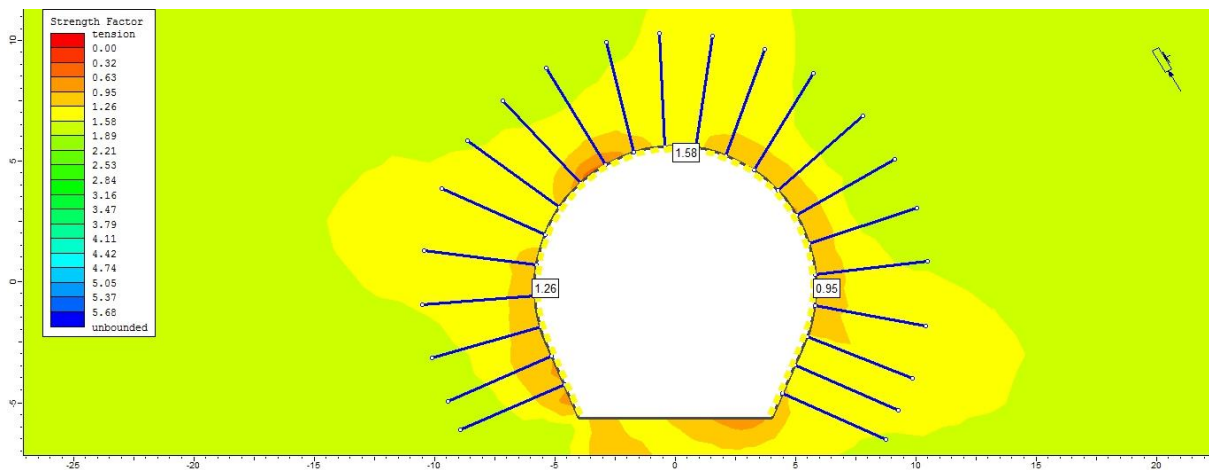


Figure 4-9: The strength factor after application of support at chainage 520.40m

The strength factor is the ratio of available rock mass strength to induced stress, at any given place. All three principal stresses (σ_1 , σ_2 , σ_3) have an effect on the strength factor. If the strength factor is less than one the material will fail if left unsupported. A strength factor of less than one shows overstepping and this is evident in the tunnel sections especially in dolerite. Although some modelled tunnel sections show a strength factor less than one, the TT is stable and adequately supported. No failure has occurred in the tunnel. A strength factor of zero is found on both sidewalls at Ch.1937.80 in dolerite, this may be explained by the high horizontal stresses found in the dolerite. Due to the highly jointed nature of dolerite great care was taken during the excavation of dolerite and adequate support was installed. Plastic analysis is recommended for additional information but this will not be covered in this research. Figure 4-8 and 4-9 show minor changes in the strength factor before and after support is installed at chainage 520.40. Minor changes can also be seen in

the strength factor in the other tunnel sections. The results for the other tunnel sections are given in Appendix A3.

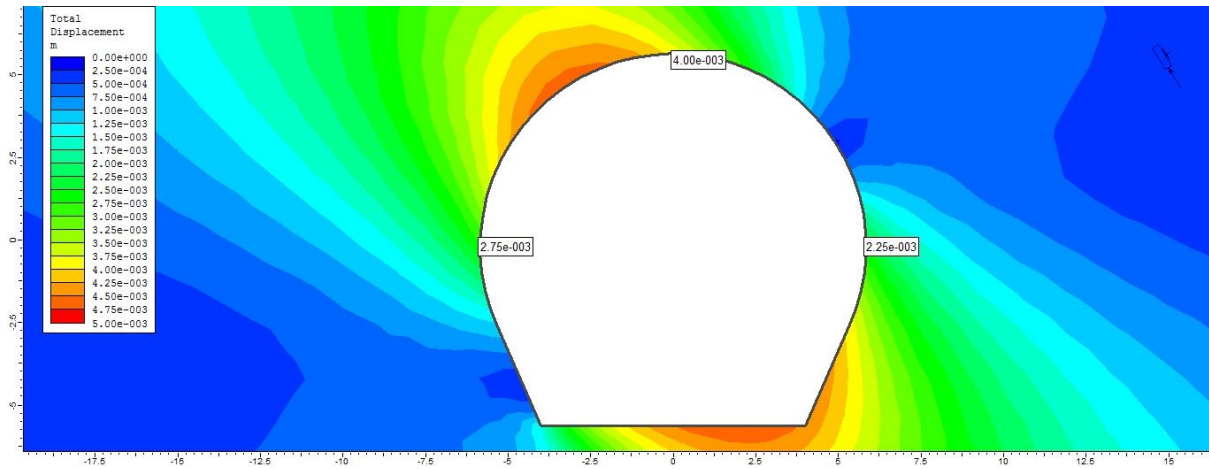


Figure 4-10: Total displacement before the application of support at chainage 520.40m

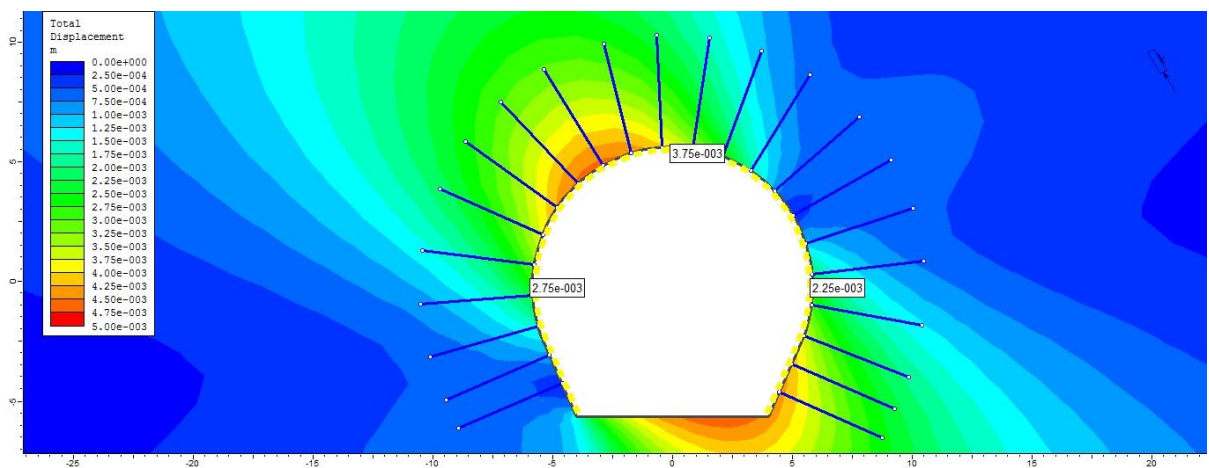


Figure 4-11: Total displacement after the application of support at chainage 520.40m

Figure 4-10 and 4-11 show the displacement in the crown and sidewalls at Ch.520.40m of the tailrace tunnel before and after support installation. At chainage 520.40m, the most displacement occurs in the crown of the tunnel; however this is not the case in the other tunnel sections in mudstone. This relates to the direction of the major principal stress. In dolerite the most displacement occurs in the sidewall. A small reduction in the crown displacement is noted in Figure 4-11 after the application of support. Very little or no change in the displacements occurs in the other tunnel sections after the application of support. Table 4-4 shows the results of the modelled displacement and the in situ measured displacements. The modelled displacement in the other tunnel sections is shown in Appendix A4.

Table 4-4: Deformation in the tailrace tunnel from Phase2 analysis and in situ measurements (the locations of sidewall A and C have been defined in Figure 4-2)

Chainage	Phase2 modelled deformation without support			Phase2 modelled deformation with support			MPBX measured deformation		
	Sidewall A, mm	Crown, mm	Sidewall C, mm	Sidewall A, mm	Crown, mm	Sidewall C, mm	Sidewall A, mm	Crown, mm	Sidewall C, mm
520.40	2.75	4.0	2.25	2.75	3.75	2.25	0.75	3.26	2.11
872.70	3.50	2.25	3.25	3.50	2.25	3.25	1.2	0.5	4.37
1718.70	1.8	1.35	1.65	1.8	1.35	1.65	1.86	1.04	1.68
1937.80	2.85	0.9	2.7	2.85	0.9	2.7	3	1.12	2.4
2259.50	3.25	3.25	3	3.20	3.20	3	2.5	1.58	2.08

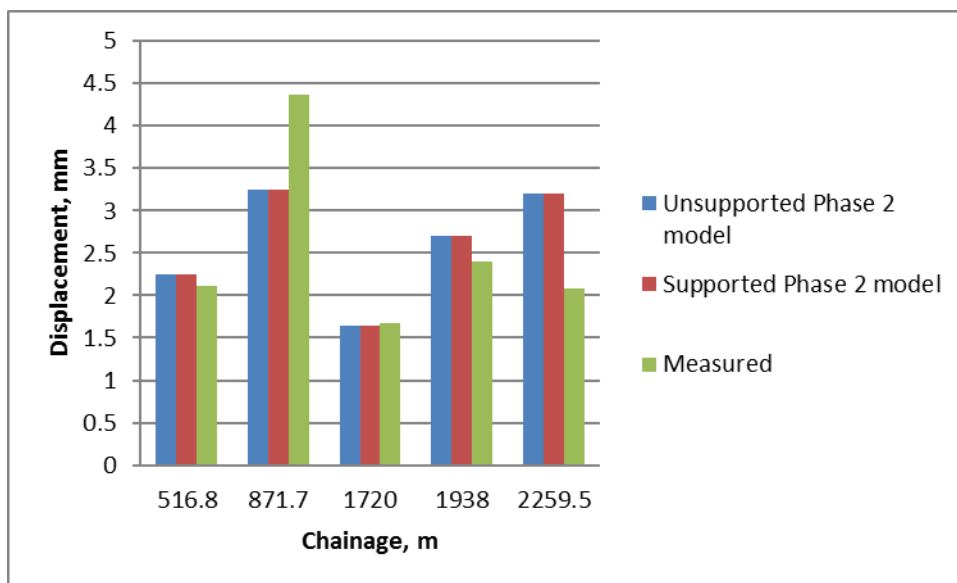


Figure 4-12: Comparison of the Phase2 (elastic analysis) modelled tunnel displacement and measured displacement on sidewall A

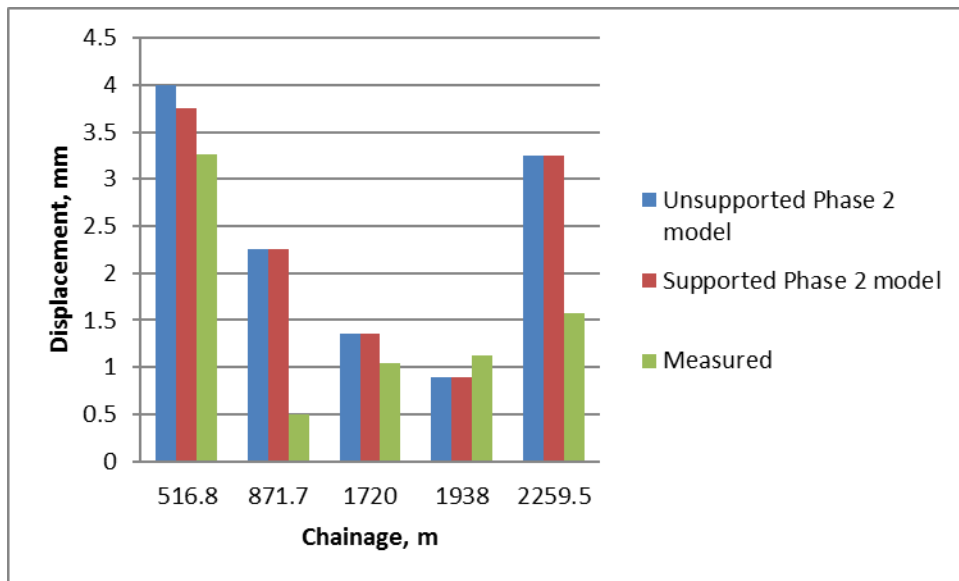


Figure 4-13: Comparison of the Phase2 (elastic analysis) modelled tunnel displacement and measured displacement on crown

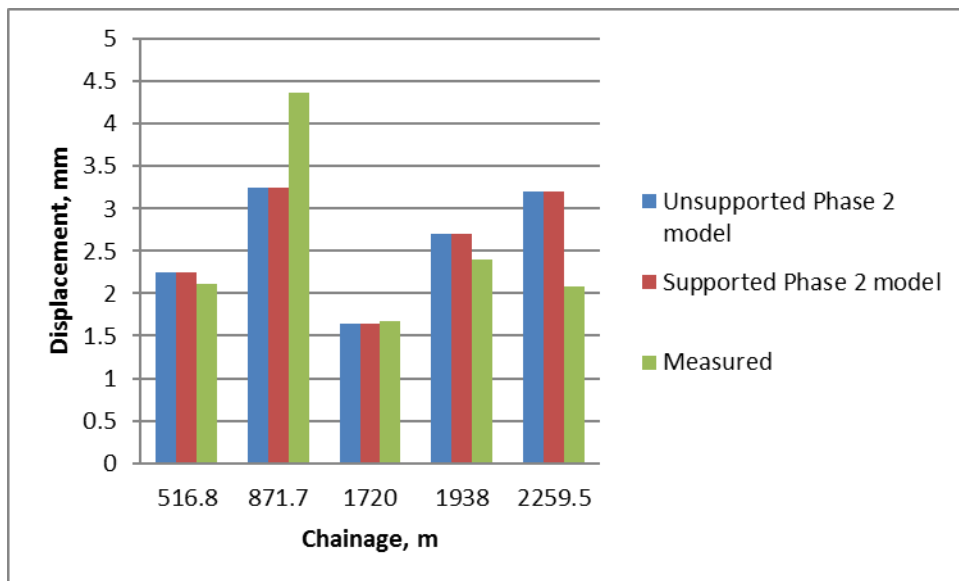


Figure 4-14: Comparison of the Phase2 (elastic analysis) modelled tunnel displacement and measured displacement on sidewall C

The graphs in Figure 4-12 to 4-14, show a comparison of the in situ measured and Phase 2 modelled displacement before and after application of the support in the crown, left sidewall (sidewall A) and the right sidewall (sidewall C). This data is also shown in Table 4-4. The main concern of this study is

to compare the model predicted displacement to that which was measured on site and determine the modulus of deformation. The comparison of the numerically modelled and in situ measured displacements shows that the numerical model does not model the displacements in the mudstones well. However, the Phase2 modelled deformation in the dolerite rock mass at Ch.1937.80 and Ch.1718.70, corresponds well with the in situ measured deformation. This may be explained by the “locked in” and well aligned stresses in the dolerite sill.

4.4.3 Axisymmetric Correction Model

Although the comparison between the instrumentation measured and Phase2 modelled displacements yielded satisfactory results in the dolerite rock mass, it is important to keep in mind that monitoring instrumentation record a fraction of the displacements as it is usually installed after the excavation of the tunnel. For this reason, the results of the MPBX measured deformation needs to be corrected to account for the partial instrumentation measurements. An axisymmetric model was run in the five selected sections of the TT using the Phase 2 programme. The model allows one to analyse a three dimensional excavation which is rotationally symmetrical. Although the input is two dimensional, the results apply to the three dimension problem. Rock mass parameters in Table 4-1 and the stresses and stress orientation in Table 4-3 were used as input values for the model. Figure 4-15 shows the model set up at Ch. 520.40 m. Axisymmetric models for the other tunnel sections are in Appendix A5.

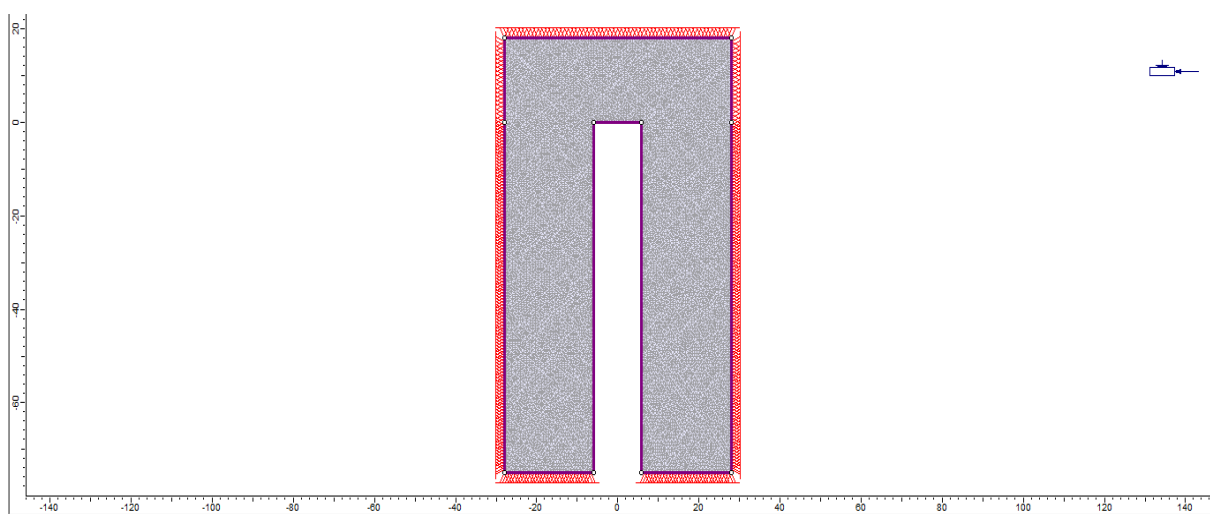


Figure 4-15: Axisymmetric model set up at Ch. 520.40m

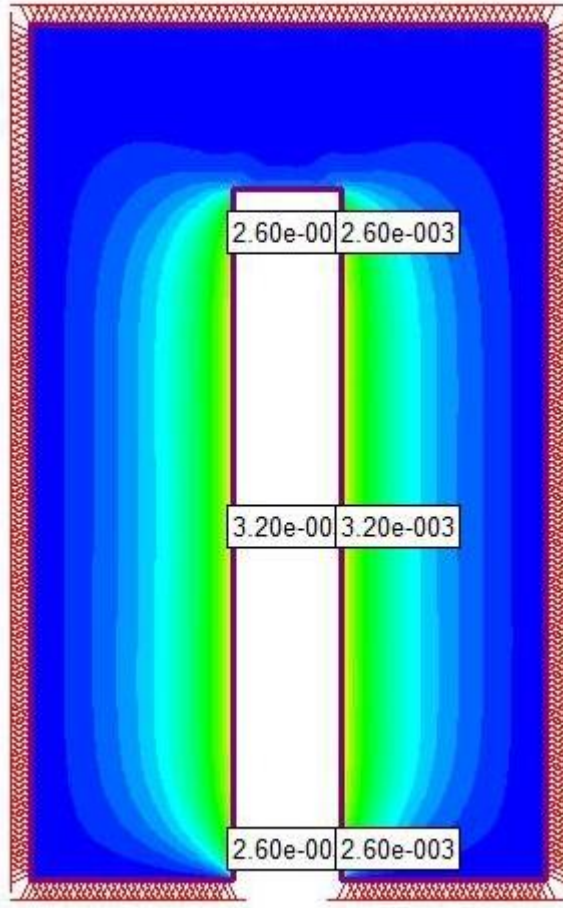


Figure 4-16: Displacement 3m, 35m and 70m behind the tunnel face at Ch.520.4m

Excavation induced stresses reach a maximum at three tunnel diameters behind the tunnel face. Also keeping in mind that monitoring instrumentation is installed between three metres behind the tunnel face deformation measurements at thirty five metres and three metres behind the tunnel face were used to correct the in situ measured displacements. Figure 4-16 shows the axisymmetric model results at 3m, 35m and 70m behind the tunnel face. Results for the other tunnel sections can be found in Appendix A5. From the axisymmetric models it is seen that the maximum deformation occurs at 35 m behind the tunnel face. Results from the MPBX measurements are then corrected. The correction factor was calculated by multiplying the difference between the displacement 35m and 3m behind the tunnel face with the MPBX measured displacement. The corrected deformations are compared to the numerically modelled deformations in the TT, results are shown in Table 4-5. An increase can be seen in the MPBX measured deformation after the correction. The comparison still shows that the Phase2 deformation results in the dolerite rock mass correspond with the corrected measured deformation. An improved correspondence between the modelled and corrected in situ measured results can be seen at Ch.2259.50 in the mudstone rock mass.

Table 4-5: Corrected deformation results (location of sidewall A and C are defined in Figure 4-2)

Chainage	Phase 2 modelled deformation			Corrected MPBX measured deformation		
	Sidewall A, mm	Crown, mm	Sidewall C, mm	Sidewall A, mm	Crown, mm	Sidewall C, mm
520.4	2.75	4.0	2.25	1.35	3.86	2.71
872.7	3.50	2.25	3.25	1.8	1.1	4.97
1718.7	1.80	1.35	1.65	2.07	1.25	1.89
1937.8	2.85	0.9	2.70	3.36	1.48	2.76
2259.5	3.25	3.25	3.00	3.3	2.38	2.88

4.4.4 Hazard Warning

The use of hazard warning level is recommended during the construction of a tunnel to assess the stability of the tunnel. Hazard warning levels used in Figure 4-17 and 4-18 were defined using the critical strains at the five points in the TT. Laboratory results were used to determine the critical strain. Equations 2-56 and 2-57 discussed in Chapter 2 were used to determine the strain in the rock mass around the tunnel.

The resulting strains calculated from the equations 2-56 and 2-57 are plotted in relation to the uniaxial compressive strength. Results for the crown and the sidewalls were plotted separately in Figure 4-17 and Figure 4-18. The plots show the relationship between measured strain in the crown and the sidewall with the hazard warning levels. Table 4-6 shows the calculated strains at the different chainages.

Table 4-6: Calculated strain and uniaxial compressive strength

Chainage	Strain (ϵ_0) at sidewall A	Strain (ϵ_0) at Crown	Strain (ϵ_0) at sidewall C	Uniaxial Compressive Strength at sidewall A, σ_c (MPa)	Uniaxial Compressive Strength at crown, σ_c (MPa)	Uniaxial Compressive Strength at sidewall C, σ_c (MPa)
520.4	0.055	0.071	0.05	2.3	3.0	2.1
872.7	0.063	0.039	0.05	2.7	1.7	2.1
1718.7	0.03	0.024	0.036	5.9	4.8	7.1
1937.8	0.025	0.016	0.025	5.0	3.2	5.0
2259.5	0.048	0.058	0.044	2.0	2.4	1.9

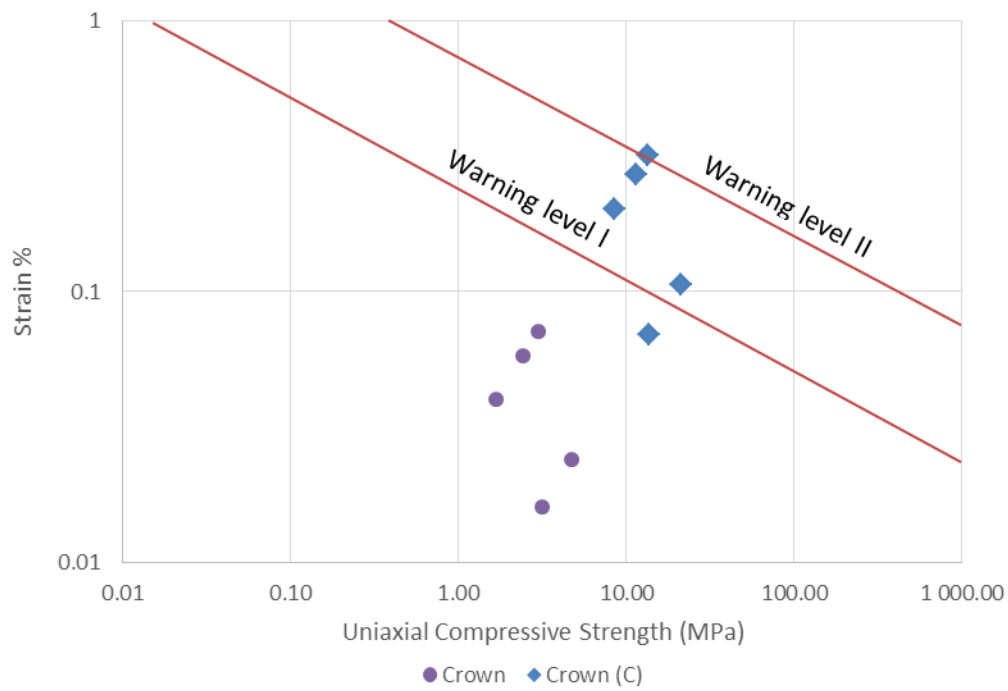


Figure 4-17: Relationship between strain and uniaxial compressive strength in the crown and hazard warning levels

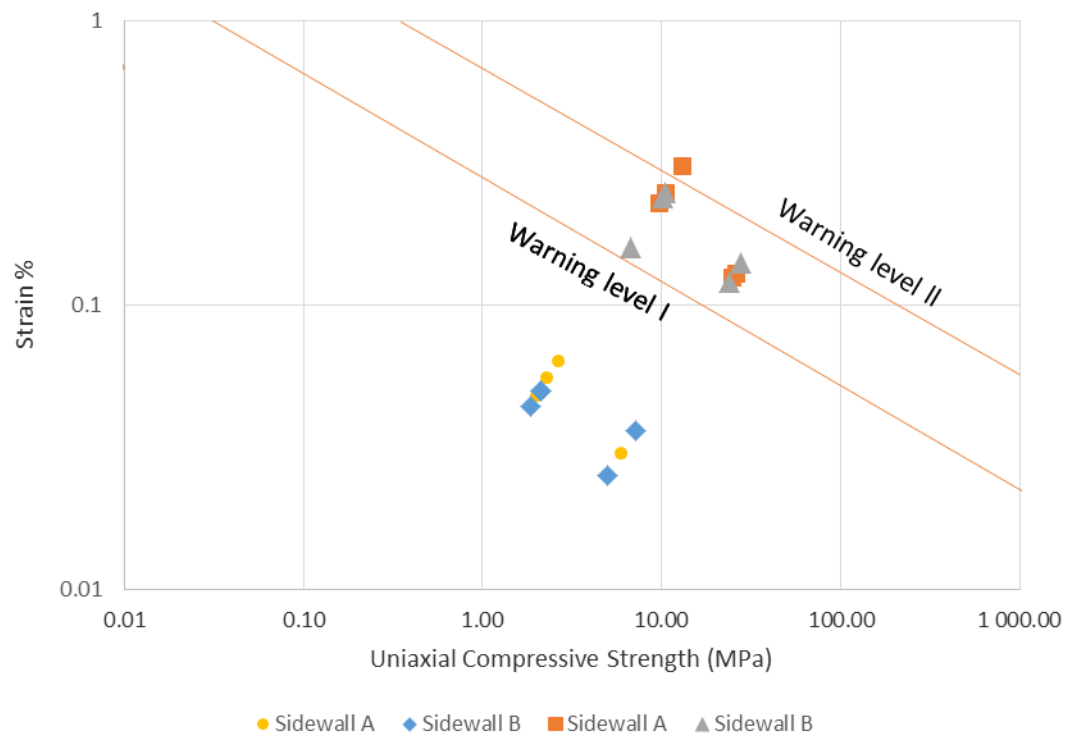


Figure 4-18: Relationship between strain and the uniaxial compressive strength in the sidewalls and hazard warning levels

From the graphs, it can be seen that the strains in the TT are lower than the lower bound critical strain. Showing that the TT was stable and there were no problems experienced during excavation. This evidence agrees with what happened on site.

4.5 Modulus of deformation back analysis

The Kirsten (1976) formula was used to determine the modulus of deformation for the rock mass using the deformation results from the in situ and modelled measurements. The axisymmetric corrected in situ measurements were used for the back analysis as they account for the complete deformation that occurred in the tunnel. Derivation of this formula has been described in Chapter 2. Modulus of deformation (E) as well as Poisson's ratio (ν) for the mudstone and the dolerite was determined.

Phase 2 models were run for three sets of elastic modulus and Poisson's ratio values as given in Table 4-7. These values were then used to solve for the influence coefficients.

Table 4-7: Elastic modulus and Poisson's ratio inputs for finite element model

Number of set	E(GPa), mudstone	ν , mudstone	E(GPa), Dolerite	ν , Dolerite
1	19.2	0.28	89.0	0.29
2	7.5	0.3	24.0	0.3
3	6	0.3	15.0	0.3
4	4.0	0.28	19.0	0.29

Three models were run to determine the influence coefficients P, Q, R, p, q and r, at the chosen chainages for the different rock masses. The coefficients P, Q, R, p, q and r have been defined by the function 2-37 in Chapter 2. The fourth model was used as an example to prove the back analysis of the moduli. Vertical and horizontal displacements from the fourth model were used together with the expressions of A and B to determine the back analysed values for the modulus of deformation. Functions 2-38 to 2-43 in Chapter 2 describe the expressions A and B. Results of the back analysis are shown in Table 4-8 for mudstone and Table 4-9 for dolerite.

Table 4-8: Rock mass modulus and Poisson's ratio results from back analysis in mudstone

Chainage	E (GPa)	ν
520.40	4.3	0.28
872.70	4.3	0.28
2259.50	4.4	0.28

Table 4-9: Rock mass modulus and Poisson's ratio results from back analysis in dolerite

Chainage	E (GPa)	ν
1718.70	19.2	0.29
1937.80	20.1	0.29

The vertical and horizontal displacements of the selected tunnel sections were also analysed separately. An average for the modulus of deformation and Poisson's value in the two rock types were also calculated. Results are shown on Table 4-10 and 4-11.

Table 4-10: Rock mass modulus and Poisson's ratio results from back analysis in mudstone using horizontal and vertical displacement separately

Chainage	Horizontal Analysis		Vertical Analysis	
	E (GPa)	ν	E (GPa)	ν
520.70 - 872.70	4.3	0.28	4.3	0.29
520.70 - 2259.50	4.3	0.28	4.2	0.28
872.70 – 2259.50	4.4	0.28	4.3	0.28
Average	4.3	0.28	4.3	0.28

Table 4-11: Rock mass modulus and Poisson's ratio results from back analysis in dolerite using horizontal and vertical displacement

Chainage	Horizontal Analysis		Vertical Analysis	
	E (GPa)	ν	E (GPa)	ν
1718.70-1937.80	22.1	0.29	19.3	0.29

The back analysed average modulus of deformation and Poisson's ratio in the mudstone are 4.3 GPa and 0.28 respectively and in the dolerite 20.2 GPa and 0.29 respectively. In situ Goodman Jack test conducted in the exploration tunnel yielded an average modulus of deformation of 15.8 GPa. A modulus of deformation ranging from 5 to 21 GPa during the reloading cycles and 6 to 19 GPa during unloading cycles was measured from the plate bearing tests. The plate bearing test results are more representative of the undisturbed in situ deformation modulus and correspond well with the Goodman Jack test Results. Back analysed results are within the range of the results of the in situ tests.

4.6 Summary of results

After the calculation of the modulus of deformation, the results from the back calculated deformation moduli are compared to the modulus from the empirical relationship, to determine which of the five methods yields the 'best fit' results. The results from all six methods can be seen on Table 4-12. From the back analysis, an average modulus of deformation is 4.3 GPa with a Poisson's ratio of 0.28 and 20.2 GPa with a Poisson's ratio of 0.29 for mudstone and dolerite respectively. Looking at Table 4-12, the Hoek and Diederichs (2006) empirical relationship achieves results that are the closest to the back analysed results as compared to the other empirical relationships. This close relation may be due to the fact the initial rock mass modulus is considered in both methods. Although the Hoek and Diederichs (2006) empirical relationship had the best fit results, all the empirical equations used were effective in calculating the rock mass deformation modulus. This shows the effectiveness of rock mass classification systems in the determination of empirical equations for rock mass modulus. The comparison of the empirical methods and back analysed results to the in situ test results illustrates that both these methods determined the modulus of deformation in the TT effectively.

Numerical models are vital in the design stage of the engineering projects but the quality of the results of these models is highly reliant on the quality of data input into the model. From the analysis it can be seen that the Phase2 numerical model, modelled the stress, strain and the deformation in the TT adequately. Results showed that the model predicted deformation in the dolerite more accurately than it did in the mudstone. Figures 4-12 to 4.14 show a comparison of the results. The axisymmetric model showed that the maximum deformation occurs at 35 m behind the tunnel face, where induced stress reaches a maximum distance. Results from the axisymmetric correction had an increasing effect on the MPBX measured deformation. The stability of the TT was also exhibited using the hazard warning levels and that no problems were experienced.

Table 4-12: A comparison modulus of deformation calculations using empirical methods versus back analysed modulus of deformation

Chainage (m)	Em (GPa), Serafim and Pereira (1983)	Em (GPa), Hoek and Diederichs (2006)	Em (GPa), Hoek and Diederichs (2006) wth Ei	Em (GPa), Sonmez et al. (2006)	Em (GPa), Hoek and Brown (1997)	Em (GPa), Kirsten (1976)
520.4	31.6	8.4	7.0	1.2	0.7	4.3
872.7	23.7	5.6	5.6	0.8	0.7	4.3
1718.7	13.3	2.4	15.2	1.5	1.0	19.2
1937.8	15.0	3.1	18.1	1.8	1.1	20.1
2259.5	28.2	8.4	6.9	1.0	0.7	4.4

5. DISCUSSION AND CONCLUSION

The focus of this research has been to evaluate how good numerical methods are in the estimation of the behaviour of the rock mass by comparing their predictions with actual field data. This has been done by analysing the deformation of the tailrace tunnel (TT) at the Ingula Pumped Storage Scheme (IPSS). Stress, strain and deformation in the TT were modelled elastically with the use of the Phase2 Finite Element Method (FEM). Empirical and back analysis methods were applied to determine the modulus of deformation of the rock mass. Five tunnel sections were analysed in the research project, three in mudstone and two in dolerite.

5.1 Discussion of results

In this study, five empirical methods and one back analysis method were used to determine the modulus of deformation. The empirical relationships of Serafim and Pereira (1983), Hoek and Diederichs (2006), Hoek and Brown (1997) as well as Sonmez et al. (2006) were employed. Rock mass classification systems are the basis of these empirical relationships. The results for the empirical relationships are shown in Table 4-2. For the back analysis of the deformation modulus, results from elastic numerical analysis and in situ measurements were used in a closed form developed by Kirsten (1976). In situ and laboratory test results were used as the inputs to determine the deformation modulus and Poisson's ratio. This equation yields an average modulus of deformation of 20.2 GPa with a Poisson's ratio of 0.29 in the dolerite and 4.3 GPa with a Poisson's ratio of 0.28 in the mudstone.

Goodman Jacking test results yielded an average deformation modulus of 15.8 GPa while the plate bearing test results ranged from 5 to 21 GPa during reloading cycles and from 6 to 19 GPa during unloading cycles. Plate Bearing test best represent the deformation modulus of the undisturbed rock mass. Back analysed and empirically determined deformation modulus were compared to the in situ measured deformation modulus. The comparison showed that both the empirical methods and back analysis defined the deformation modulus well. Results from the back analysis are considered to be the most accurate in calculating the deformation modulus, as this method makes use of the actual measured displacements and the in situ and laboratory results as inputs for the Phase2 numerical analysis. When the results of the empirical methods are compared to those of the back analysis; the Hoek and Diederichs (2006) empirical relationship, which takes into account the intact

rock mass modulus yields the results that best correspond with those from the back analysis. This close relationship is owing to the fact that both the Kirsten (1976) back analysis and Hoek and Diederichs (2006) method consider the intact rock mass modulus. The empirical relationship by Hoek and Diederichs (2006) which takes into account the intact rock mass modulus also yields the best results when compared to the plate bearing test results.

The Phase2 elastic model was used to model the stress, strain and deformation in the TT. Modelled deformations were then compared to deformations measured by the multiple point borehole extensometers (MPBX) installed in the TT. Rock mass stresses and rock mass parameters are the main factors that control the deformation in the rock mass. Therefore, the quality of the numerical analysis is dependent on the input rock mass parameters and stress. The stress analysis shows overstressing in the tunnel due to high horizontal stresses experienced at the IPSS, especially in the dolerite rock mass. Although that is the case, the TT was adequately supported and is stable.

A comparison of the results from the numerical model and in situ measurements shows that Phase2 models deformation in the dolerite more accurately than it does in the mudstones. This could be related to the locked in situ stresses in the dolerite. Stresses in the mudstones were affected by the dolerite intrusions which can be seen when comparing the modelled and measured results. Due to monitoring instrumentation being installed after tunnel excavation, 3m behind the tunnel face, these deformation results were corrected using a Phase2 axisymmetric model. Deformation results at 3 m and 35m behind the tunnel face were modelled and used to correct the monitoring instrumentation results. This correction increased the measured deformations, improving the correspondence between the Phase2 modelled deformation and the in situ measured deformation.

Stability in the TT was also examined with the use of the Hazard Warning Levels. The strain around the TT was plotted against the uniaxial compressive strength of the rock mass in the TT. Results show that the TT is very stable and no problems were encountered during the construction of the tunnel which, corresponds with what happened on site.

5.2 Conclusion and recommendations

Numerical modelling is a very useful and important tool for the engineering design of underground excavations for the characterization of rock mass behaviour. Models are based on the input

geotechnical parameters determined during the design stage. The elastic numerical analysis presented in this research project shows that numerical modelling predicted the deformation in the dolerite rock mass well. The importance of monitoring which includes, installation of instrumentation, continuous data collection through mappings and field tests and the use of rock mass classification systems during construction is also shown in the research. This monitoring system assisted in the verification of any design assumptions and gave warning of the likelihood of problems during construction.

The modulus of deformation is one of the important parameters required in the analysis of rock mass behaviour. Over the years, rock mass classification systems have formed the basis for the development of empirical relationships for determining the modulus of deformation. Empirical relationships that make use of rock mass classification systems, applied in the research project have successfully managed to determine the deformation modulus. The Kirsten (1976) developed back analysis of the measured and elastically modelled deformation provided the most satisfactory determination of the modulus of deformation in the rock mass surrounding the TT. Even though that is the case, empirical relationships are open to further improvement as more in situ data becomes available. It is therefore recommended that more work is done on the development of back analysis methods with regards to the determination of the modulus of deformation.

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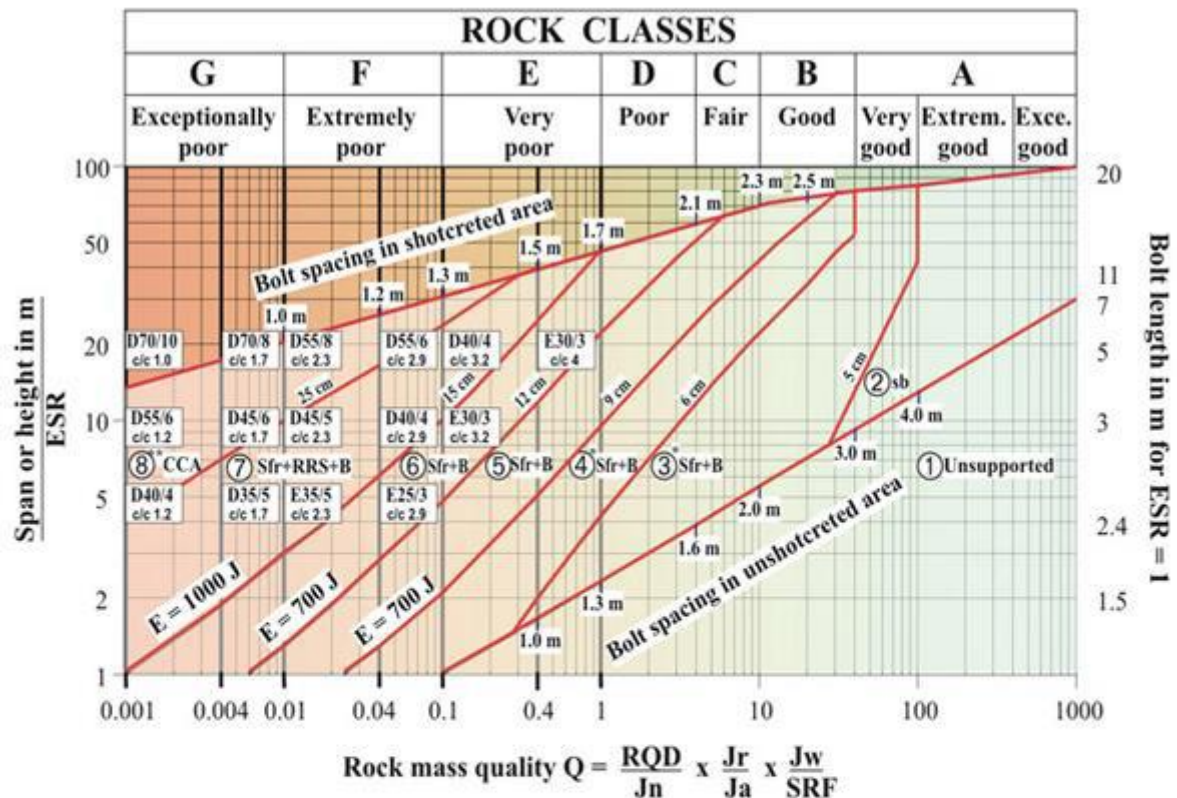
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7. APPENDIX

Appendix A1-1: Selection of disturbance factor, D (Hoek et al., 2002)

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	$D = 0$
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	$D = 0$ $D = 0.5$ No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	$D = 0.8$
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	$D = 0.7$ Good blasting $D = 1.0$ Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	$D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation

Appendix A1-2: Q-system chart and various excavation support ratio categories(Grimstad and Barton, 1993)



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

Appendix A2: Major Principal stresses

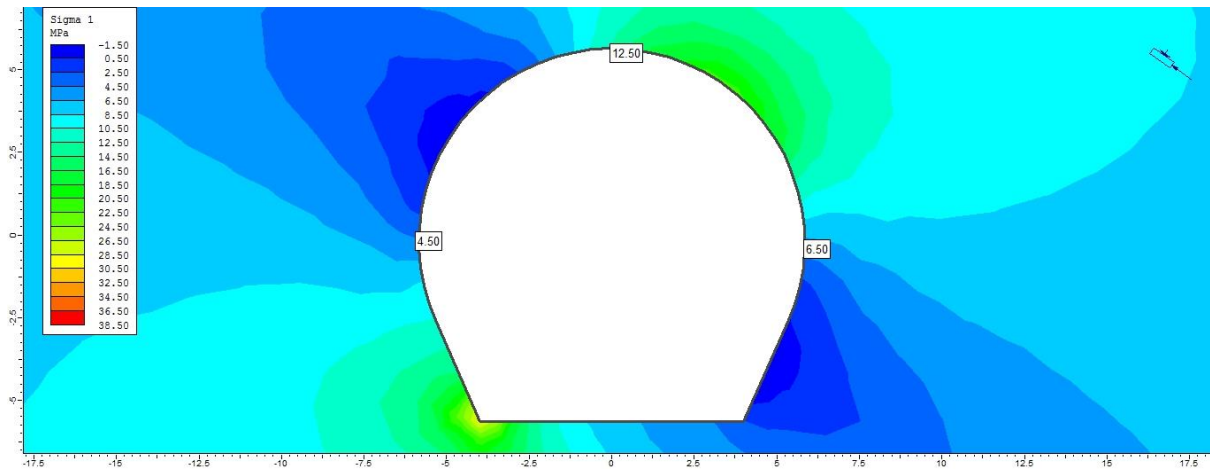


Figure A-1: Major principal stress after excavation at chainage 872.70m

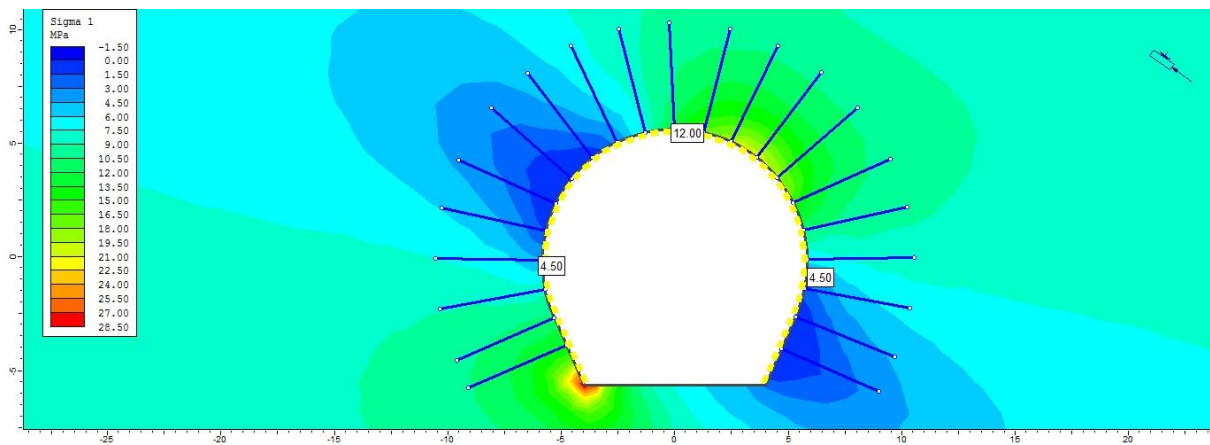


Figure A-2: Major principal stress after support at chainage 872.70m

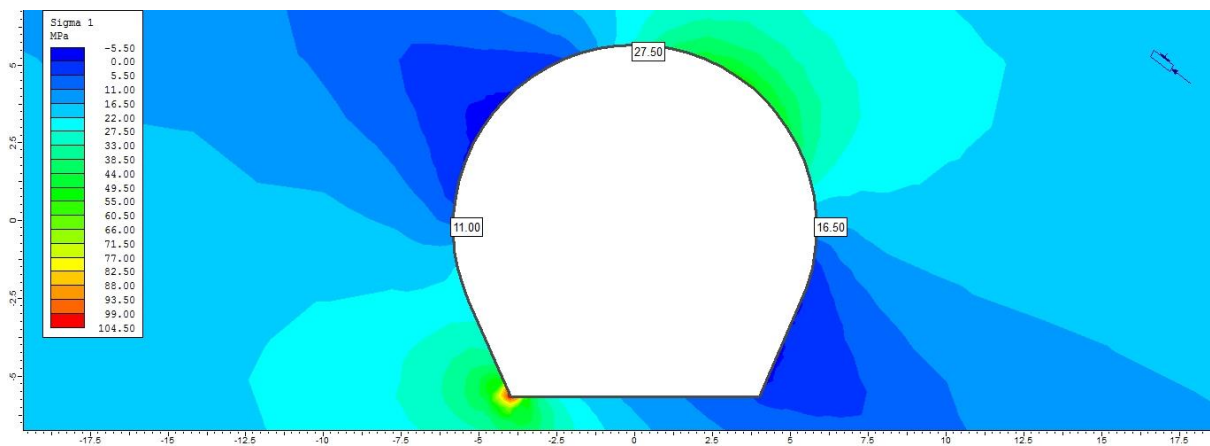


Figure A-3: Major principal stress after excavation at chainage 1718.70m

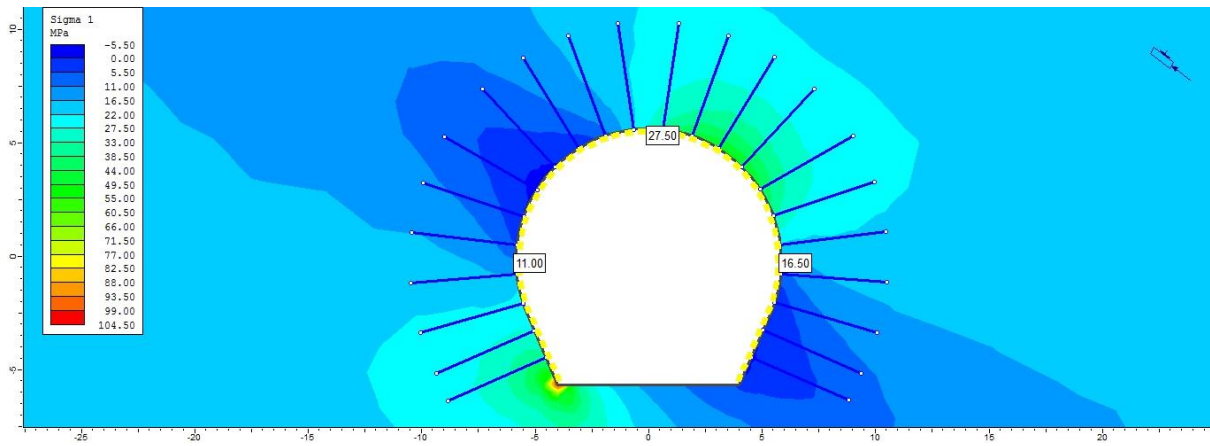


Figure A-4: Major principal stress after support at chainage 1718.70m

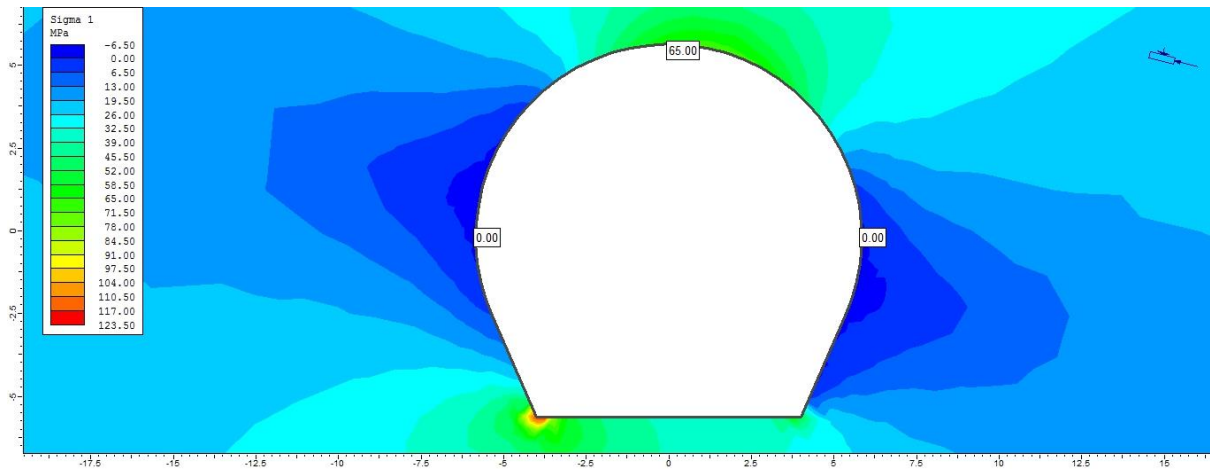


Figure A-5: Major principal stress after excavation at chainage 1937.80m

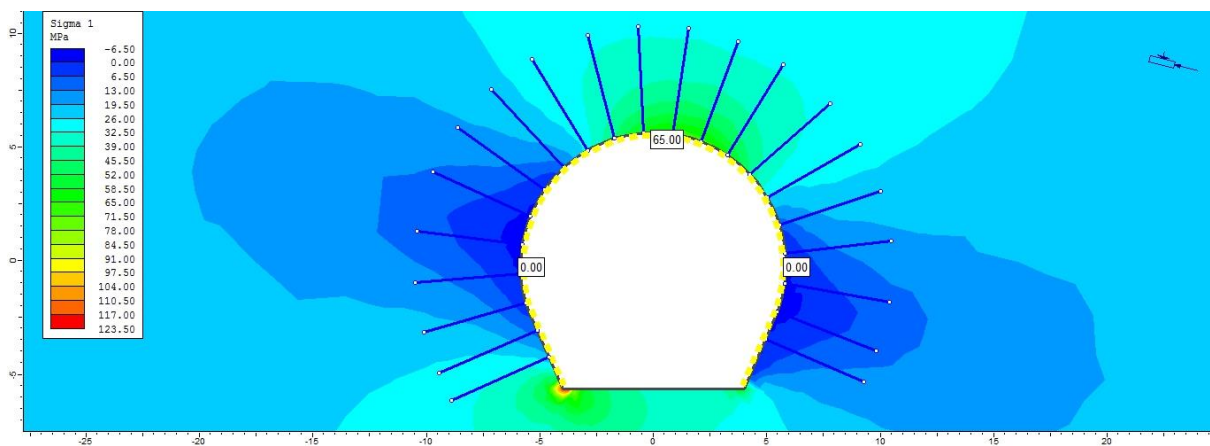


Figure A-6: Major principal stress after support at chainage 1937.80m

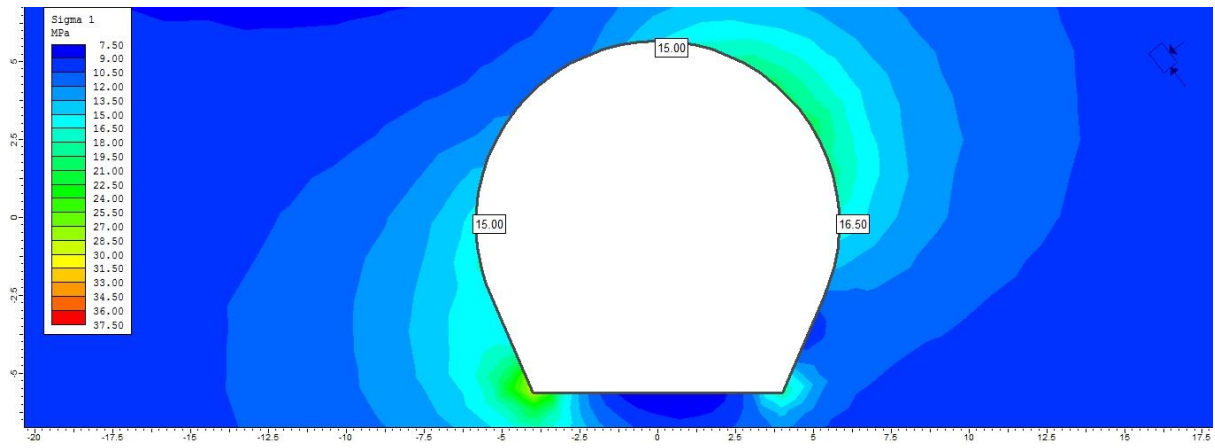


Figure A-7: Major principal stress after excavation at chainage 2259.50m

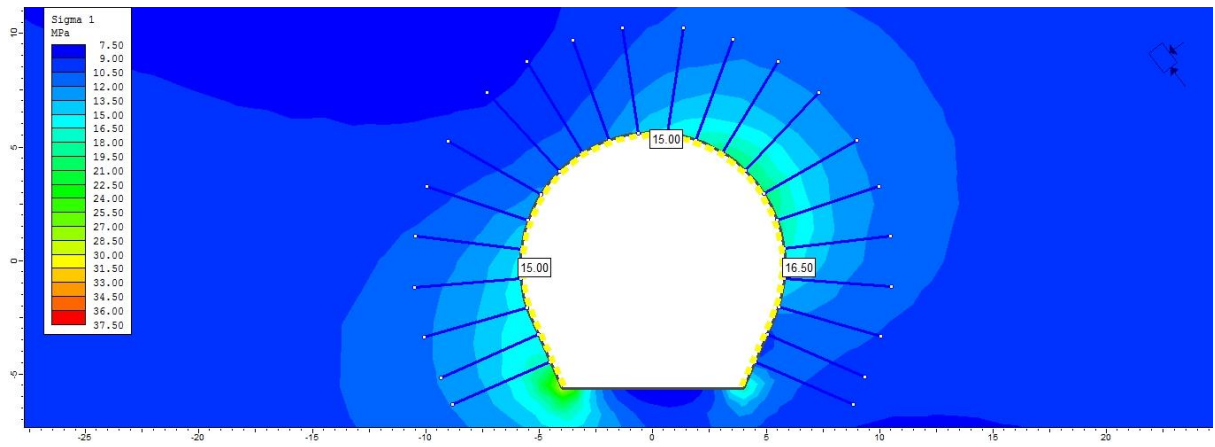


Figure A-8: Major principal stress after support at chainage 2259.50m

Appendix A3: Strength factor

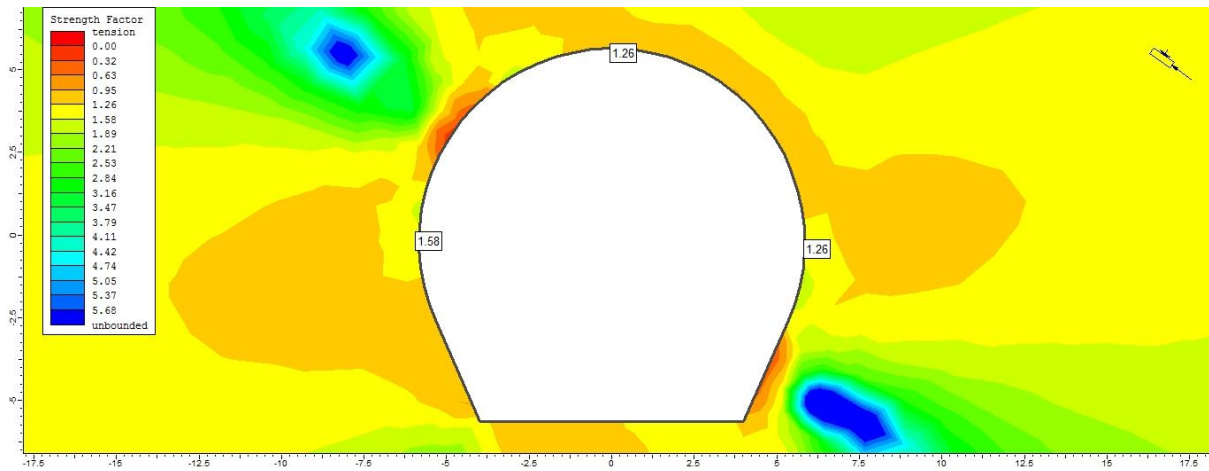


Figure A-9: Strength factor after excavation at chainage 872.70m

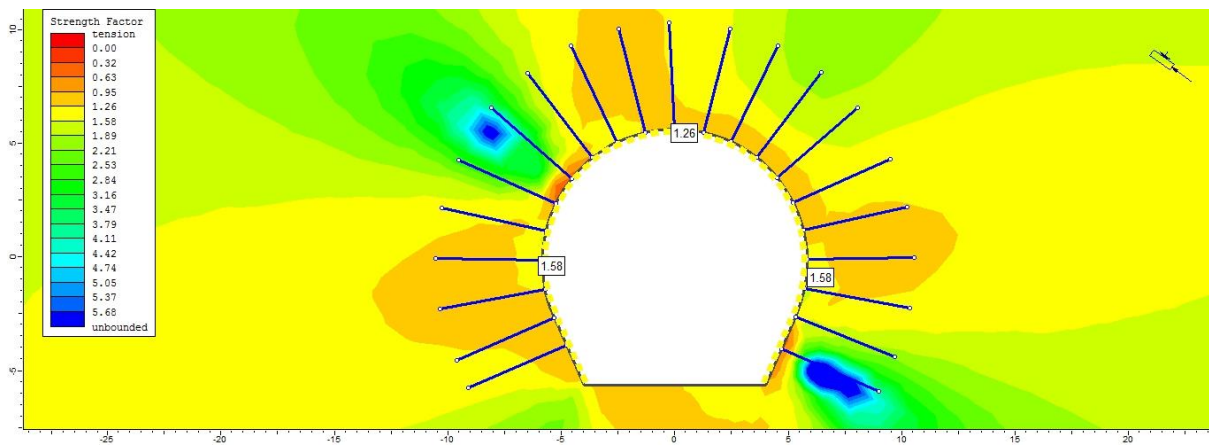


Figure A-10: Strength factor after application of support at chainage 872.70m

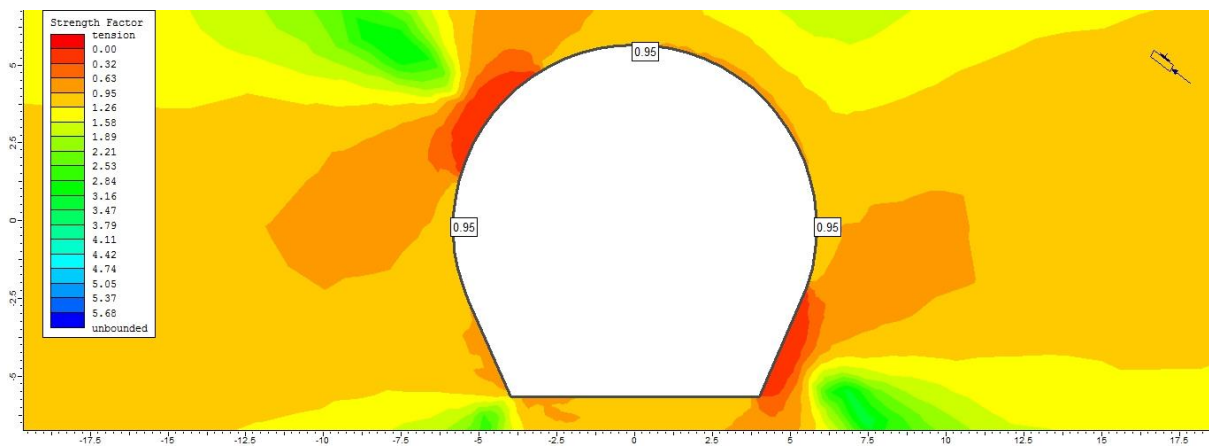


Figure A-11: Strength factor after excavation at chainage 1718.70m

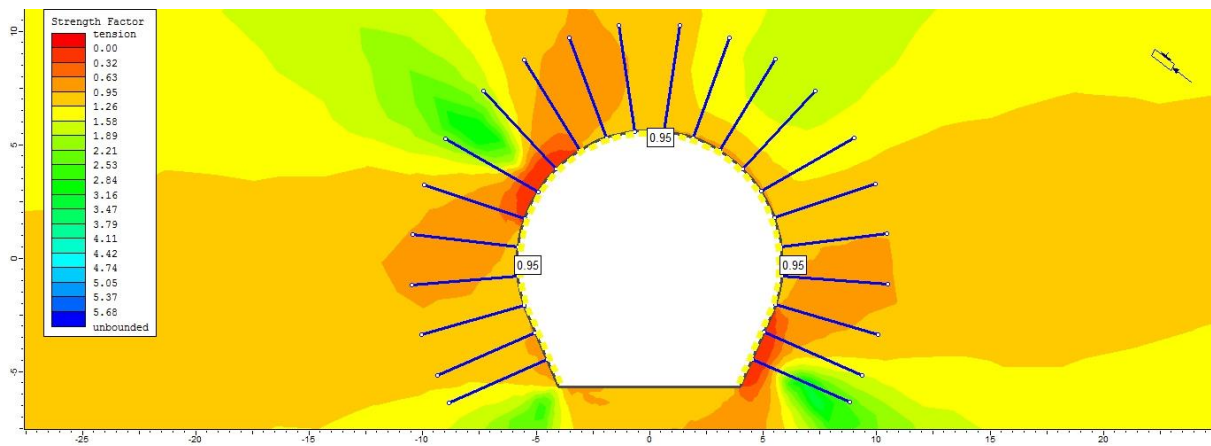


Figure A-12: Strength factor after application of support at chainage 1718.70m

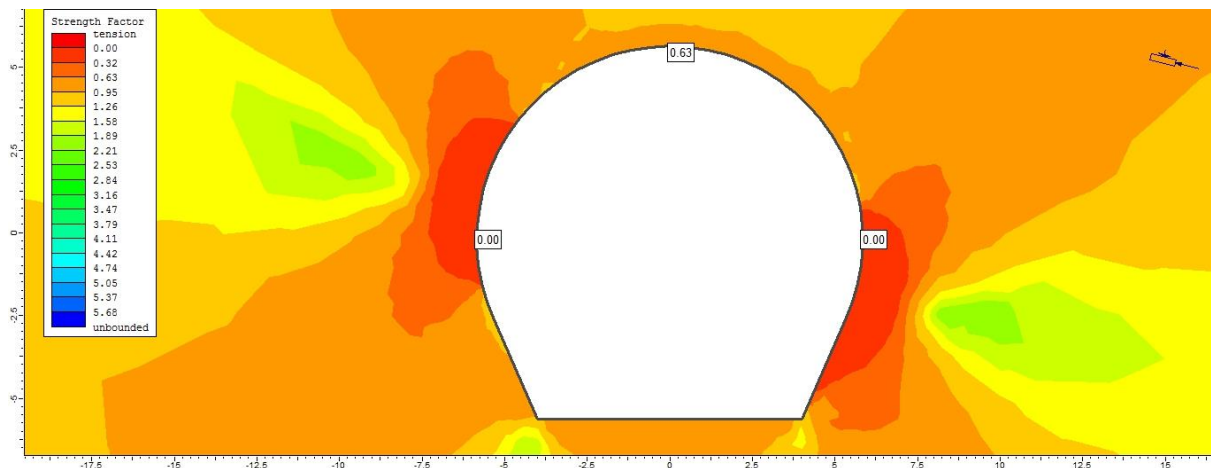


Figure A-13: Strength factor after excavation at chainage 1937.80m

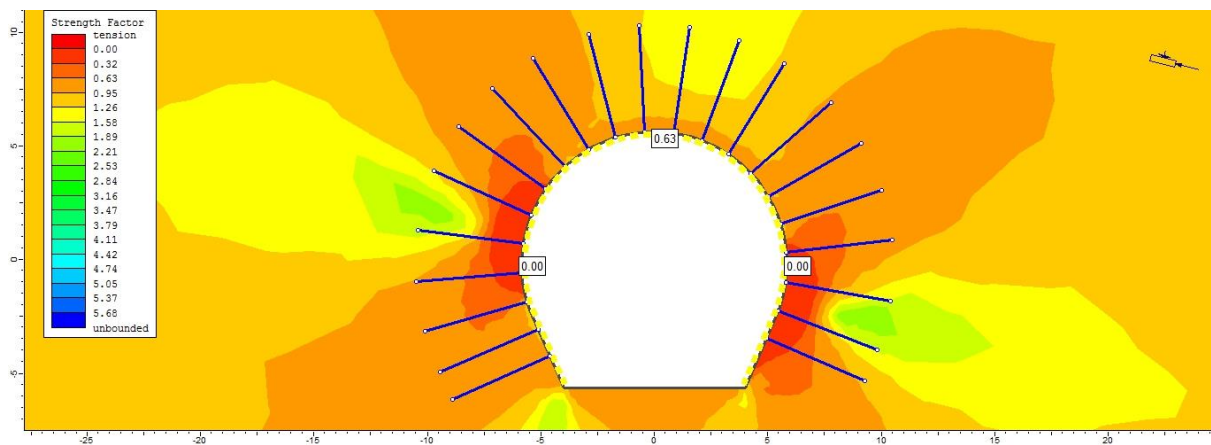


Figure A-14: Strength factor after application of support at chainage 1937.80m

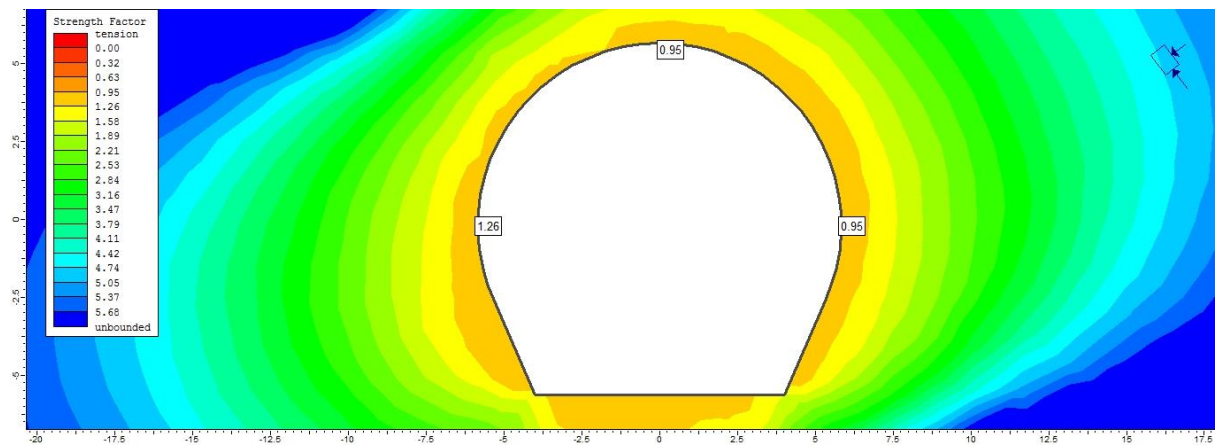


Figure A-15: Strength factor after excavation at chainage 2259.50m

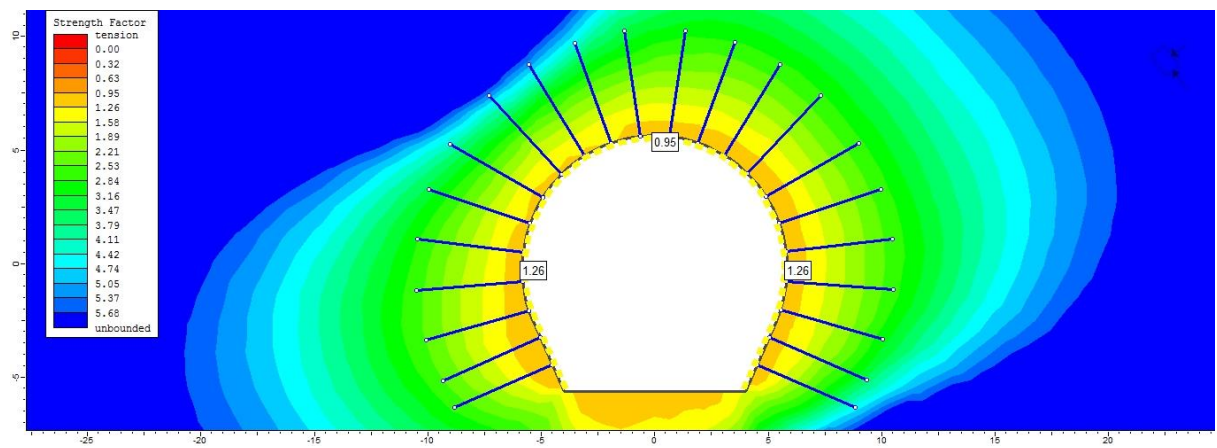


Figure A-16: Strength factor after application of support at chainage 2259.50m

Appendix A4: Displacement results

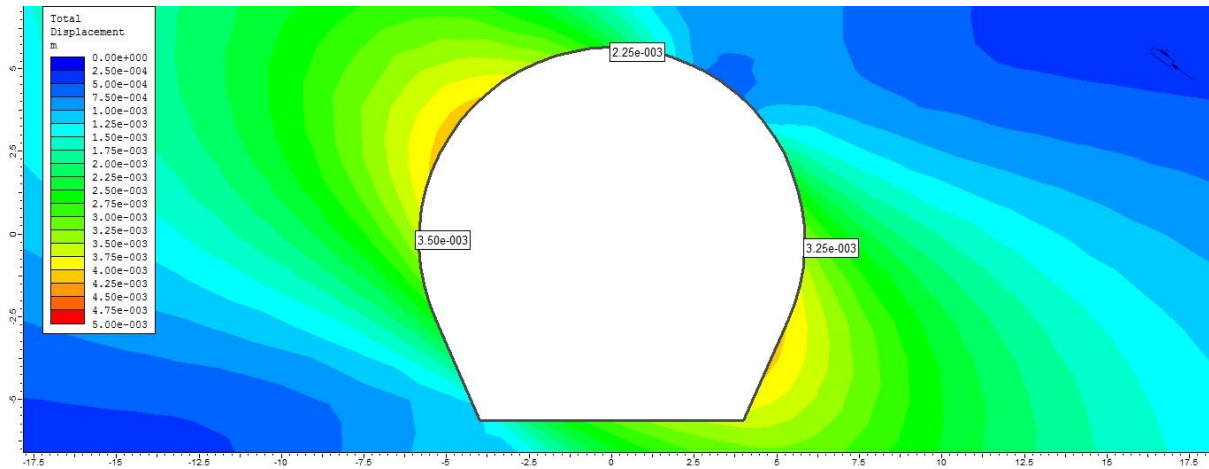


Figure A-17: Total displacement before the application of support at chainage 872.70m

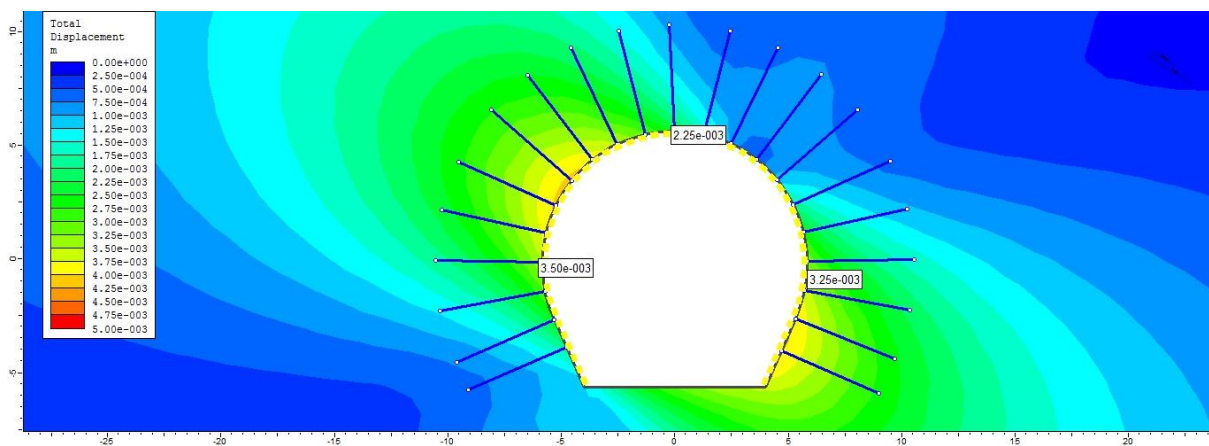


Figure A-18: Total displacement after the application of support at chainage 872.70m

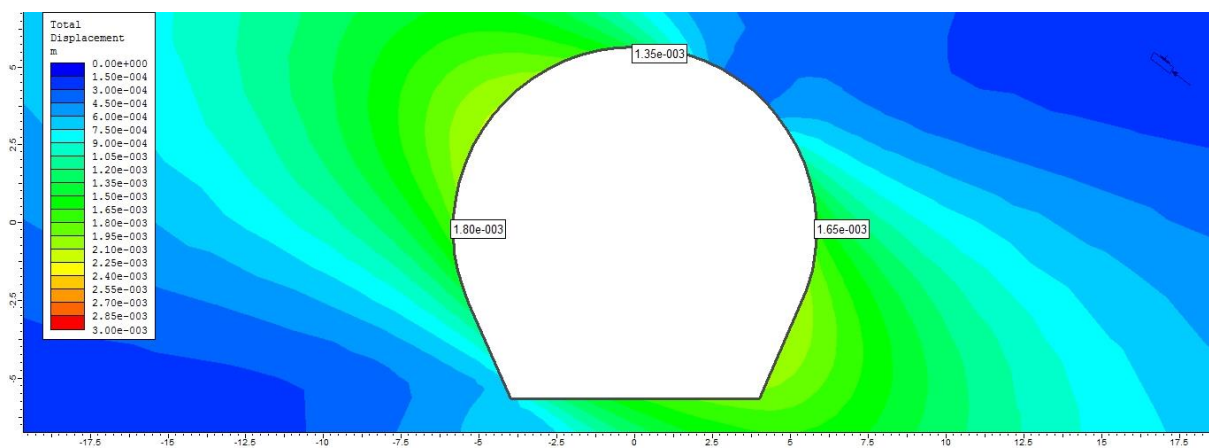


Figure A-19: Total displacement before the application of support at chainage 1718.70m

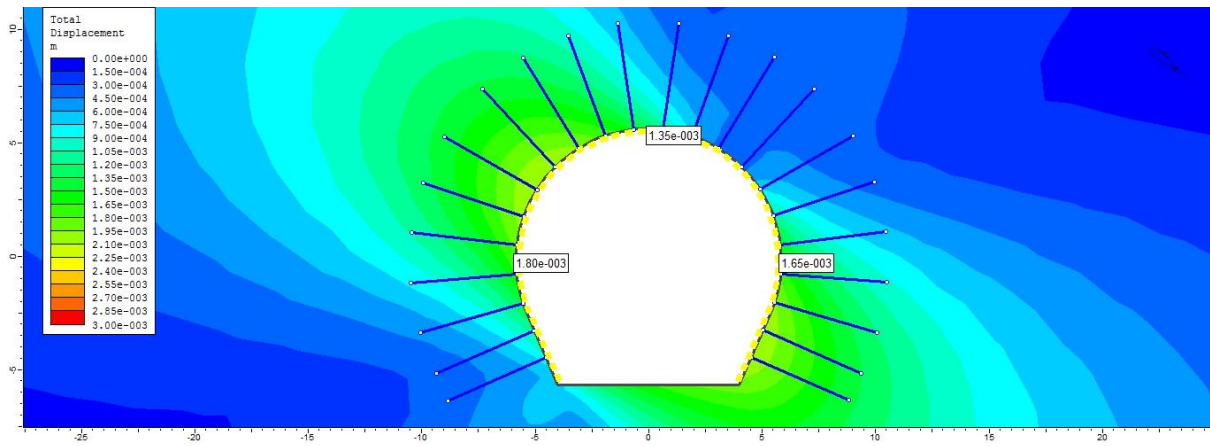


Figure A-20: Total displacement after the application of support at chainage 1718.70m

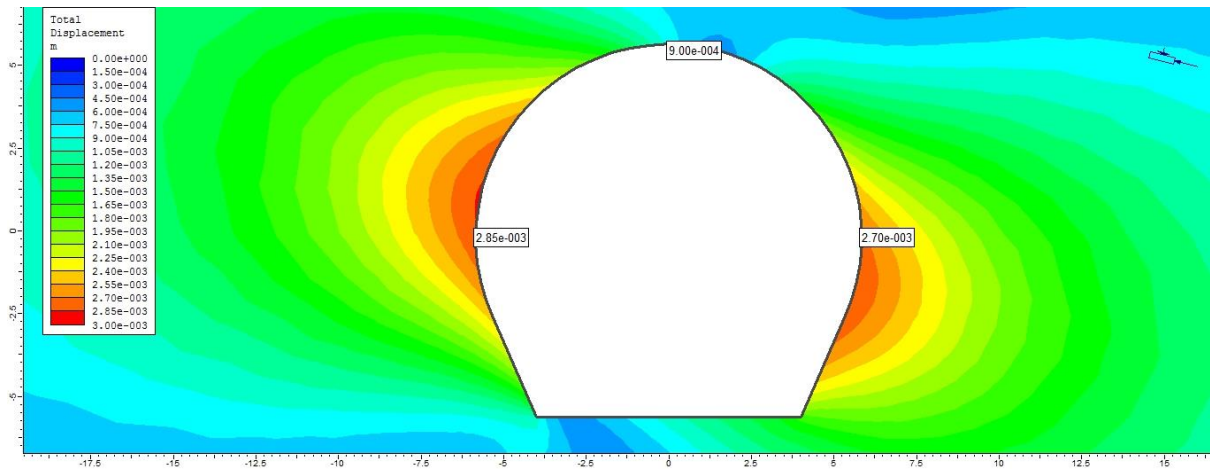


Figure A-21: Total displacement before the application of support at chainage 1937.80m

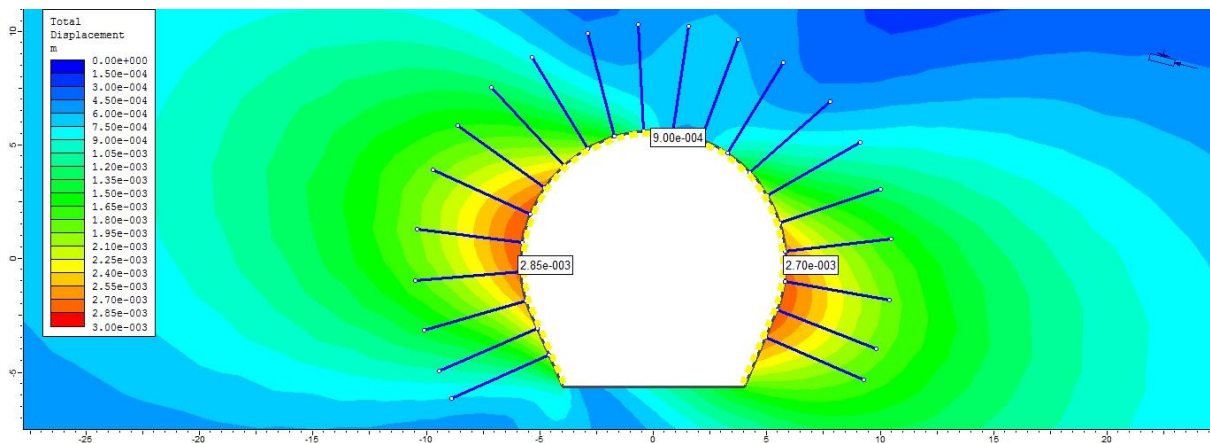


Figure A-22: Total displacement after the application of support at chainage 1937.80m

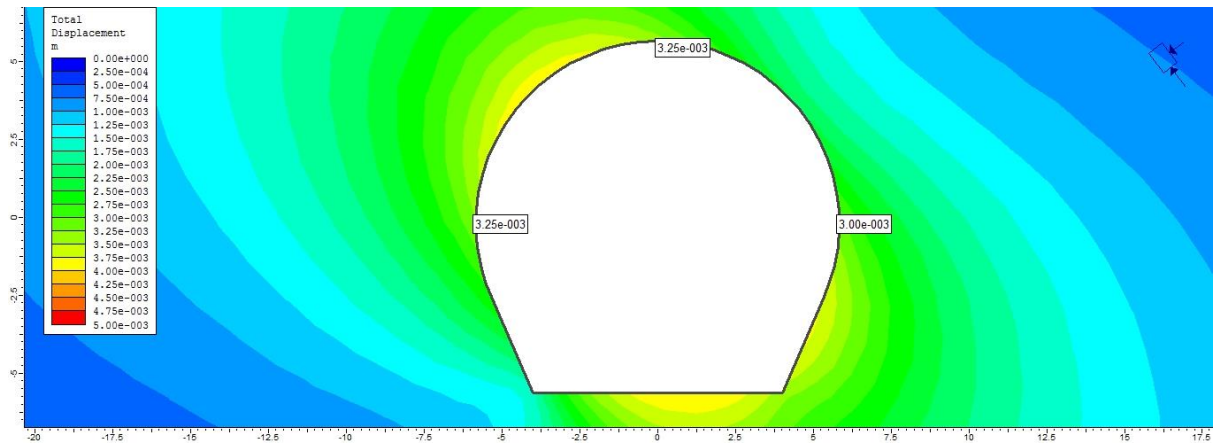


Figure A-23: Total displacement before the application of support at chainage 2259.50m

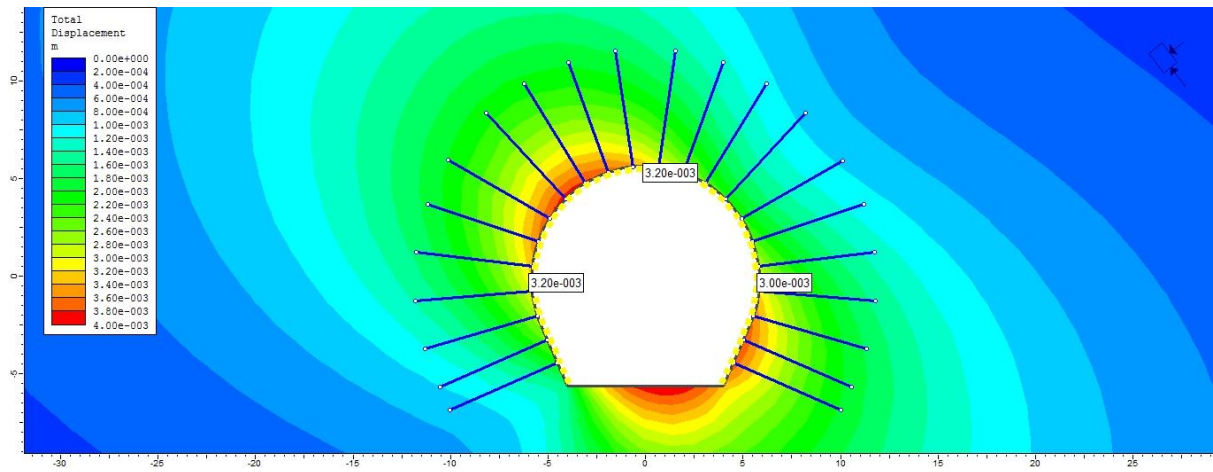


Figure A-24: Total displacement after the application of support at chainage 2259.50m

Appendix A5: Axisymmetric model

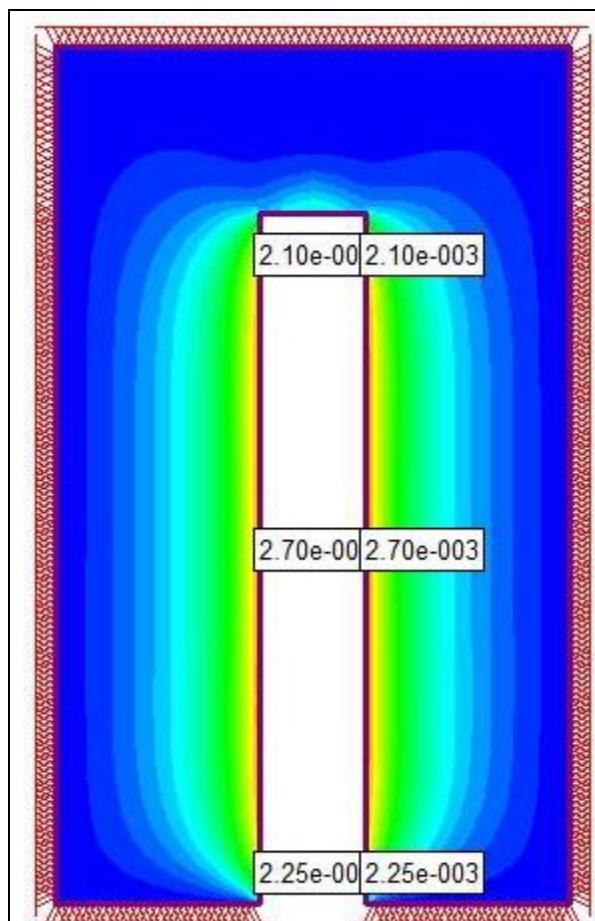


Figure A-25: Total displacement of axisymmetric model 3m, 35m and 70m behind the tunnel face at chainage 872.70m

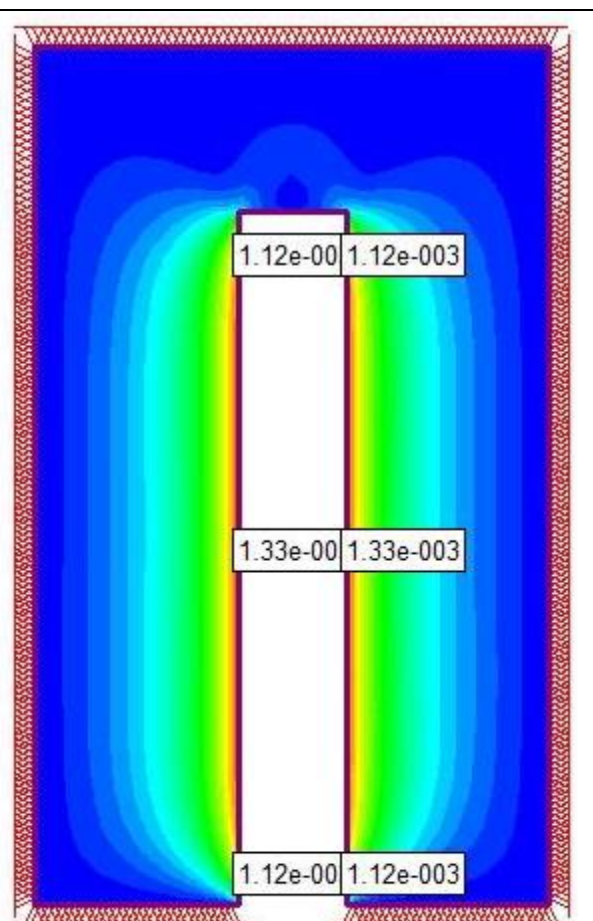


Figure A-26: Total displacement of axisymmetric model 3m, 35m and 70m behind the tunnel face at chainage 1718.70m

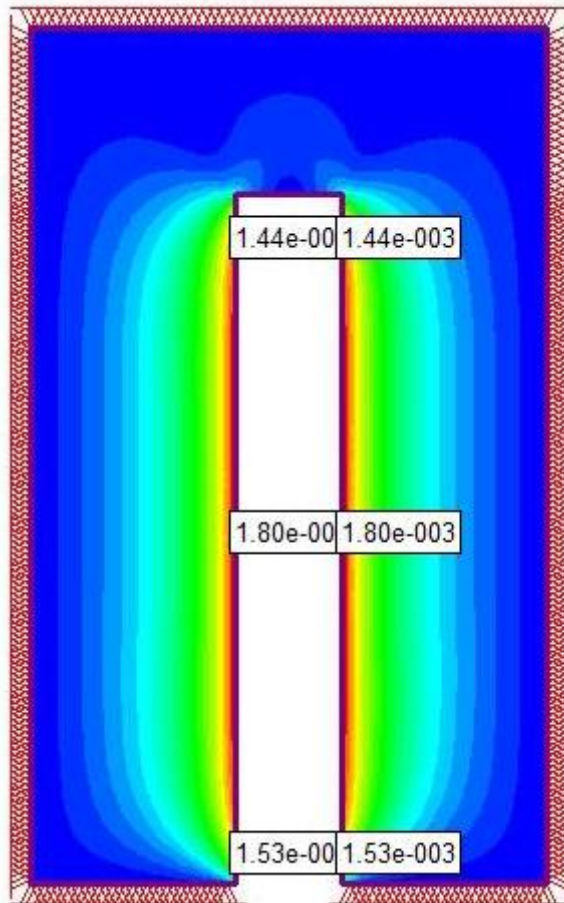


Figure A-27:Total displacement of axisymmetric model 3m, 35m and 70m behind the tunnel face at chainage1937.80m

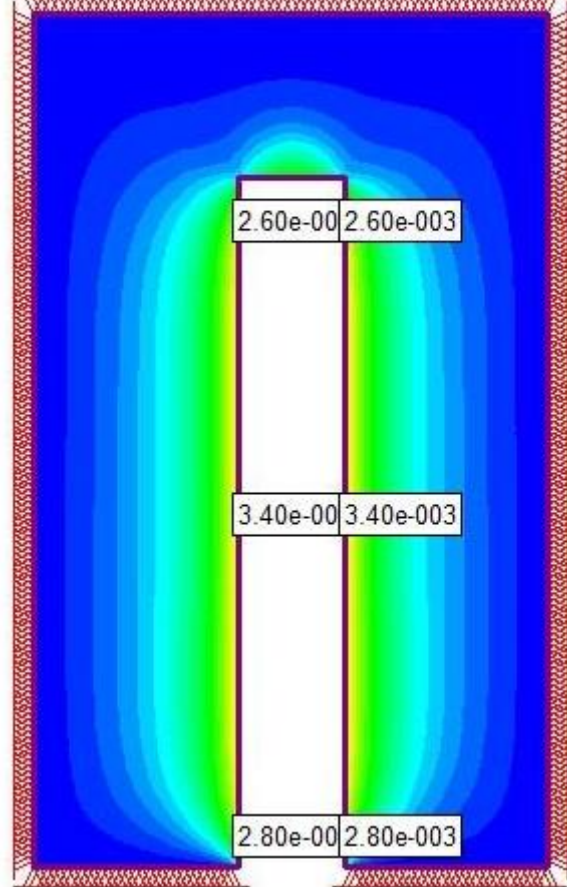


Figure A-28: Total displacement of axisymmetric model 3m, 35m and 70m behind the tunnel face at chainage 2259.50m

Appendix B: Eskom non-disclosure agreement

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		Revision	Rev 3
		Revision Date	May 2016
		Group Technology & Commercial	

A

NON-DISCLOSURE AND CONFIDENTIALITY AGREEMENT

between

ESKOM HOLDINGS SOC LIMITED

("Eskom")

and

(full name and surname)

Amanda Sphathiswe Moyo

(Identity number AND unique number)

880801690082 AND 4271095

("the Requestor/employee")

Employee Business Area

third party contractor

The Project

Ingula Pumped Storage Scheme

Employee Asm

240- 6315269

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Definitions

1.1 In this agreement, unless inconsistent with context:

1.1.1 "project participation" means the evaluation, negotiations, accessing information and any other activity relating to geotechnical information of the Ingula Pumped Storage Scheme, the "The project" as recorded by the employee.

1.1.2 "confidential information" includes any information or knowledge relating to Eskom or any of its subsidiaries ("the companies"), acquired by the employee, whether independently or in association with others, by virtue of the employee's project participation or during the course of or incidental to the employee's employment with Eskom, including, but not limited to:

1.1.2.1. all information in whatever form (tangible or intangible), documentation, specifications, know-how, trade secrets, accounts and computer readable data relating or pertaining to the project/s;

1.1.2.2. methods of operation and mode by which Eskom or the companies conducts their business with regard to the project/s.

2 Acknowledgement

2.1 The employee acknowledges that during the course of his or her project participation and/or during the course and scope of his or her employment or relationship with Eskom, he or she will be exposed and will have access to, and will learn of certain confidential information relating to the project/s.

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2.2 Notwithstanding any provisions in this agreement, confidential information excludes any information which:

2.2.1 was in the public domain prior to it being disclosed to the employee or the employee acquiring knowledge of such information;

2.2.2 came into the public domain through no fault on the part of the employee; or

2.2.3 is or has been disclosed to the employee by a third party and the employee is able to demonstrate that the third party is legally entitled to such information.

2.2.4 is required to be disclosed under applicable law or by a governmental order, decree and regulation (provided that the employee shall give written notice to Eskom prior to such disclosure).

2.3 The employee acknowledges that the restraints imposed in this agreement are reasonable in all respects and necessary in order for Eskom to protect its rights and proprietary interests.

3 Confidentiality and non-disclosure

3.1 The employee agrees and undertakes that:

3.1.1 during the period of his or her involvement in Ingula Pumped Storage Scheme project/s, or

3.1.2 if he or she is no longer involved in Group Commercial project/s but is still in the employ of Eskom, or

Employee ASH

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3.1.3 if he or she has left the employ of Eskom,

he or she will not disclose to any other person, including third parties, or make use for his or her benefit, or that of any other person, whether directly or indirectly, any confidential information of Eskom or the companies regarding the project/s without the prior written consent of Eskom. Eskom shall provide the employee with a list of employees who have signed confidentiality agreements regarding the project/s and the employee shall only grant access to the confidential information to these employees.

3.2 The employee agrees and undertakes to protect Eskom's confidential information and to handle and store the confidential information in such a way as to prevent any unauthorised disclosure.

3.3 The obligation to maintain the confidentiality of the information will not apply to the information excluded by clause 2.2 above.

4 Return of the information

The employee is required to deliver to Eskom whenever required to do so, and in any event, when leaving Eskom's employment, documents, correspondence, notes, computer discs or any documents concerning or pertaining to Eskom, project/s or any of the companies.

5 Duration

5.1 Notwithstanding the date of signature hereof, this agreement shall be binding on the employee with effect from the date on which the confidential information was first disclosed to the employee.

5.2 This Agreement shall commence upon the date referred to in clause 5.1 and shall endure for a period of three (3) years from date hereof.

6 General

6.1 This agreement contains all the express provisions agreed upon by the parties with regard to the subject matter of the agreement and parties waive their right to rely on any alleged provision not contained in this agreement

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- 6.2 This agreement is subject to and shall be governed by the laws of the Republic of South Africa.
- 6.3 No contract varying, adding to, deleting from or cancelling this agreement, and no waiver of any right under this agreement, will be effective unless reduced to writing and signed by or on behalf of the parties.
- 6.4 The granting of any indulgence by a party under this agreement will not constitute a waiver of any rights by the grantor or prevent or adversely affect the exercise by the grantor of any existing or future right of the grantor.
- 6.5 In the event that the employee should breach the provisions of this agreement and fail to remedy such breach within seven (7) calendar days from date of a written notice to do so, then Eskom shall be entitled to invoke all remedies available to it in law.
- 6.6 This agreement replaces any non-disclosure and confidentiality agreement previously signed by the employee in respect of project/s.
- 6.7 In the event of any one or more of the provisions of this agreement being held for any reason to be invalid, illegal or unenforceable in any respect, such invalidity, illegality or unenforceability shall not affect any other provision of this agreement, and this agreement shall be construed as if such invalid, illegal or unenforceable provisions were not a part of this agreement, and this agreement shall be carried out as nearly as possible in accordance with its original terms and intent.
- 6.8 The employee will not make or issue any formal or informal announcement or statement to the press or any third party in connection with this agreement or the project/s.

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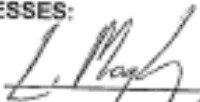
 Eskom	NDA	Unique Identifier	240- 64228874
		Revision	Rev 3
		Revision Date	May 2016
		Group Technology & Commercial	

SIGNED AT Johannesburg ON 22 September 2014 IN THE
PRESENCE OF THE UNDERSIGNED WITNESSES

Amanda Siphathiswe Moyo
NAME


SIGNATURE

WITNESSES:

1. 
2. 

SIGNED AT INGOLA ON 01/10/2014 IN THE PRESENCE
OF THE UNDERSIGNED WITNESSES

DESIGAN PADAYACHEE
NAME


SIGNATURE

For and on behalf of Eskom duly
authorised

WITNESSES:

1. 
2. 

Employee ASM

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