NUMERICAL OPTIMIZATION OF TUNNELING

A CASE STUDY OF DEFORMATION ANALYSIS OF TUNNELS

By

NIMRAH NADEEM (GROUP LEADER) (NUST2015-124921)

M OSAMA SAEED KHAN (NUST2015-123049)

RIZWAN SAEED GONDAL (NUST2015-128537)

MUHAMMAD HARIS ASHRAF (NUST2015-127923)

Project Advisor: Dr. S. Muhammad Jamil

NUST Institute of Civil Engineering (NICE)

School of Civil and Environmental Engineering (SCEE)

National University of Sciences and Technology (NUST)

Islamabad, Pakistan

(2019)

ABSTRACT

This research comprises of comprehending the current design of Nahakki Tunnel, Swat Twin Tube Tunnel and Shimla Tunnel on the basis of Q, Q and BQ classification systems respectively and analyzing deformations using analytical approaches and Finite Element Modeling (FEM) incorporating PHASE2, to provide a comparison between the analytical and numerical methods, and developing a correlation using neural network.

Artificial neural networks (ANN) or connectionist systems are computing systems vaguely inspired by the biological neural networks. During the research work we have faced numerous challenges like deformations at site in Pakistan are either not monitored and if they are, the data is not extensive.

Analytical methods show considerable variations among each other, modeling require knowledge and skill in software, an empirical correlation is required that can give accurate deformations effectively.

It is found during research that analytical results show similar results like FEM with values lower than FEM. The reasons may be that analytical results don’t take into account the stages of excavation and during analytical calculations the geometry of tunnel is supposed as circular whereas in original the geometry of tunnel is horse shoe.

This is to certify that the

Report entitled

NUMERICAL OPTIMIZATION OF TUNELLING

A CASE STUDY OF DEFORMATION ANALYSIS OF TUNNELS

Submitted by

NIMRAH NADEEM (GROUP LEADER)	(NUST2015-124921)
M OSAMA SAEED KHAN	(NUST2015-123049)
RIZWAN SAEED GONDAL	(NUST2015-128537)
MUHAMMAD HARIS ASHRAF	(NUST2015-127923)

Has been accepted towards the partial fulfillment

of

the requirements

for

Bachelors of Engineering in Civil Engineering

Dr. S. Muhammad Jamil

Department of Construction Engineering and Management,

National Institute of Transportation (NIT),

School of Civil & Environmental Engineering (SCEE),

National University of Sciences & Technology (NUST), H-12 Islamabad.

DEDICATED

TO

OUR

PARENTS

&

TEACHERS

ACKNOWLEDGEMENT

We take this opportunity to evince our profound appreciation to our supervisor, Dr. S. Muhammad Jamil, for his guidance and encouragement. His comments reflected his experience as a counselor and supervisor, he would ask at the end of every meeting if we learned anything. His constant encouragement, support and sheer interest in our project is what kept us going.

Sincere gratitude is extended to Engr. Asim Ayaz, who took keen interest in developing and drafting initial course for our project, helped us through and through by bombarding us with loads of information and options to take.

Thanks is to also be provided to Mr. Umar Khan Jadoon and Mr. Daniyal Omar who took great pain in helping us understand and implement rock mass classification systems for various tunnel alignments we used in our project, also they proved to be ands on learning hubs for finite element modeling and analysis, their combined efforts proved fruitful.

Lastly our family members whose constant support throughout the journey lifted us and spirits to highest morale.

Table of Contents

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 OVERVIEW

Tunnel is one of the important sections of underground structures. It can be well-defined as an underground passageway built for the purpose of transportation, or direct traffic between two points.Tunnels are the "subterranean passageways made without removing the overlaying soil or rock''.

Due to the increasing world's population the space left for future construction is squeezing. A lot of new techniques are being in practice now-a-days to use the underground space to provide working area that can be used to build roads, shopping malls and other sort of infrastructure etc.

In the present time, the use of tunnels is the dire need of time**.** A huge demand on civil and mining engineers is being put to design effective, durable and long lasting support systems for better tunnel designs.

Rock mass classification systems and numerical analysis methods are used together for the purpose to design the tunnel and to ensure safety, economy, effective and durable design for tunnel support systems.

Empirical method alone can't be uses in conjunction with rock mass classification for tunnel design. It is necessary to counter check the results calculated by empirical method and comparing it with using Numerical Analysis method. The popular one is Finite Element Method. This method has been used in many fields of engineering practices for more than thirty years. This method is also being used in our project for calculating deformations for the selected tunnels using phase 2 software.

The finite element method can:

- Model realistic rock behaviour.
- Depict construction methodology.
- Handle difficult hydrological conditions.
- Deal with complicated underground conditions.
- Justification for neighbouring structures and services.
- Deal with underground treatment (e.g. compensation grouting).

A correlation is the dire need of time which can be used at site as an equation to calculate the deformations for better structural health monitoring .For deriving the correlation the help of artificial network is taken .Python programming language is used .Different data points taken from the site but limited in number are fed into algorithm and an attempt is done to derive a correlation which will give approximate correct results.

1.2 PROBLEM STATEMENT.

"Comprehending the current design of Nahakki Tunnel, Swat Twin Tube Tunnel and Shimla Tunnel on the basis of Q, Q and BQ classification systems respectively and analyzing deformations usinganalytical approaches and Finite Element Modeling (FEM) incorporating PHASE2, to provide a comparison between the analytical and numerical methods, and developing a correlation using neural network."

1.3 MAIN OBJECTIVES

1. The first objective was to perform deformation analysis using finite element analysis (Phase2) and other empirical approaches based on the supports provided at the site according to the respective classification systems of the tunnels selected. The variation of the results was then compared.

2. The second objective was to use neural network to derive correlation for approximate accurate tunnel deformation.

1.4 THESIS OUTLINE

Chapter 1 includes introduction.

Rock mass classification systems, equations incorporated (Lade-Duncan, Mohr-Coulomb, and Hoek-Brown) for calculating tunnel deformations and explanation of neural network are incorporated in **Chapter 2**.

Information about Nahakki Tunnel, Swat twin tube tunnel, Shimla tunnel and their respective geotechnical parameters are explained in **Chapter 3**.

Methodology and Analysis of the whole project is explained in **Chapter 4**.

Chapter 5 includes conclusions and recommendations of this study.

CHAPTER 2

LITERATURE REVIEW – EMPIRICAL APPROACHES

2.1 ROCK MECHANICS OVERVIEW

Rock mechanics is the theoretical and applied science of mechanical behavior of a rock; it is that branch of mechanics concerned with the rock's reaction to the physical environment's force fields.

The design procedure involves selecting a tentative design and predicting the anticipated behavior in applied rock mechanics, especially in the field of civil engineering and mining engineering. The theoretical and applied mechanic equations are used. However, some mechanical property of the rock must also be inserted into the equation in about every case. The validity of the solution obtained does not exceed the validity of the used mechanical property. The mechanical characteristics of an intact rock laboratory specimen; may vary greatly from the rock mass mechanical characteristics from which the sample was taken. Acceptance of this fact has given much emphasis to in-situ testing in latest years.

2.2 EMPIRICAL APPROACH

The following rock mass classification systems were studied as an empirical approach to determine the deformations in the rock sections:

- The Rock Quality Designation Index (Deere *et al*, 1967)
- The Rock Structure Rating (Wickham *et al*, 1972)
- Geomechanics or Rock Mass Rating System (Bieniawski, 1973, 1976, 1989)
- Norwegian Geotechnical Institute's Q-System (Barton *et al*, 1974)
- The Geological Strength Index (Hoek*et al*, 1995)

2.3 THE ROCK QUALITY DESIGNATION INDEX (Deere *et al*, 1967)

The rock quality designation index was developed by Deere et al in 1967 to estimate the rock mass quality by taking in account an intact rock core sample from drill core logs. Instead of counting the number and type of fractures or alterations in the rock mass, this method is an indirect approach of a quantitative measurement. It is obtained by taking the ratio of the length of intact rock core pieces that are longer than 100 mm or 4 in to the total core run length.

$$\text{RQD} = \frac{\sum \text{length of core pieces} > 100\ mm}{\text{Total core run length}} \times 100$$

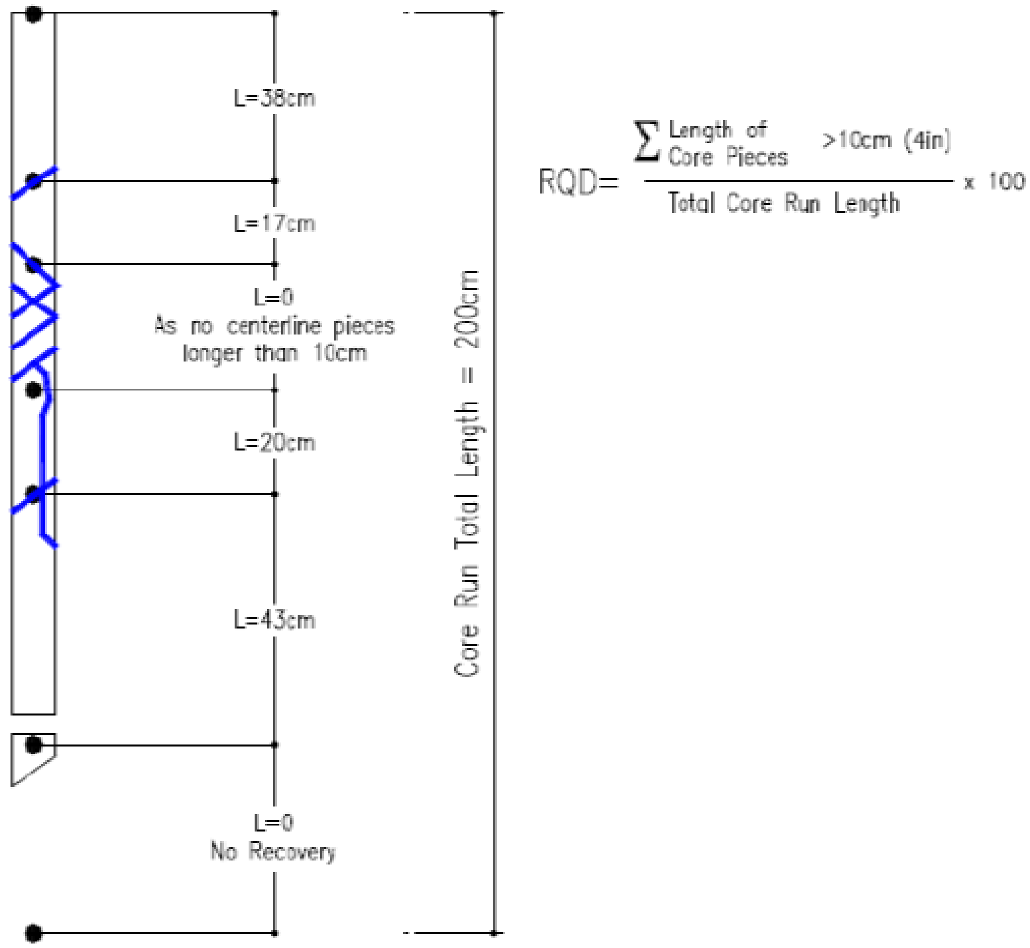


Figure 2.1: Measurement and Calculation of RQD. (after Deere, 1989)

Taking the percentage of the ratio calculated gives the RQD index.

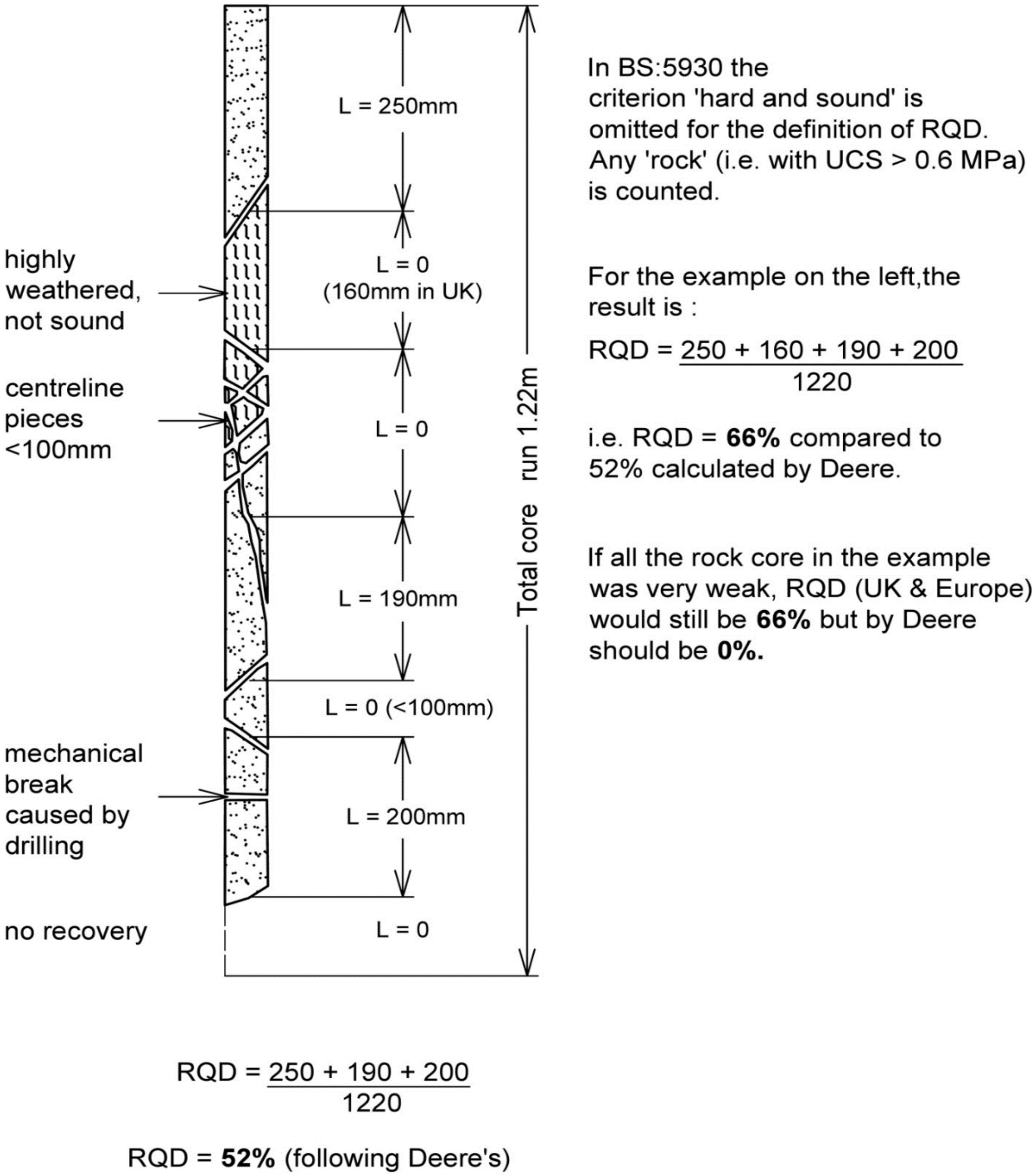


Figure 2.2: Example of calculation of RQD.

The rock quality can hence be determined by comparing the obtained RQD value with the standard values of rock quality as given in the table 2.1

Table 2.1: Relationship between Rock Mass Quality and RQD.

Rock Quality Designation (%)	Rock Mass Quality
<25	very poor
25 < 50	poor
50 < 75	Fair
75 < 90	good
90 < 100	excellent

Thus, this method as developed by Deere et al is a rapid and quite straightforward indication of zones of poor, fair and good rock.

There are several methods for measuring the length of the key elements. It is possible to measure the same core in different ways such as along the center line or from tip to tip. The standard approach suggested by the International Society of Rock Mechanics (ISRM) is to measure the length of the core parts along the center line in order to prevent errors triggered by fractures adjacent to the borehole or in the event of a borehole split by a second joint group.

The drilling process may cause core breaks in some cases. These core breaks however must be fitted together and considered as one piece. Sometimes it is difficult to differentiate between the breaks caused by the drilling process and the natural breaks. In such cases the breaks must be considered in order to get a conservative value of the RQD.

There are also some indirect ways of measuring RQD; such as in 2005Palmstorm, gave a correlation between RQD and the volumetric joint spacing J_v, given as under.

$$RQD = 110 - 2.5J_v$$

2.4 THE ROCK MASS QUALITY, Q-SYSTEM (Barton et al, 1974)

The Rock Mass Quality, Q-system was developed by Barton et al, R. Lien, and J. Lunde in 1974 in Norway at the Norwegian Geotechnical Institute (NGI). The initial source which lead to the development of this system were about 210 case studies from Scandinavia mainly describing the need for shotcrete and fully grouted rock bolts for permanently stabilizing tunnels and caverns. In 1977, Barton gave support estimations for permanent roof support and described 38 support categories for different rock masses. In 1991, Barton gave a correlation between Q and seismic wave velocity V_p given in the figure as:

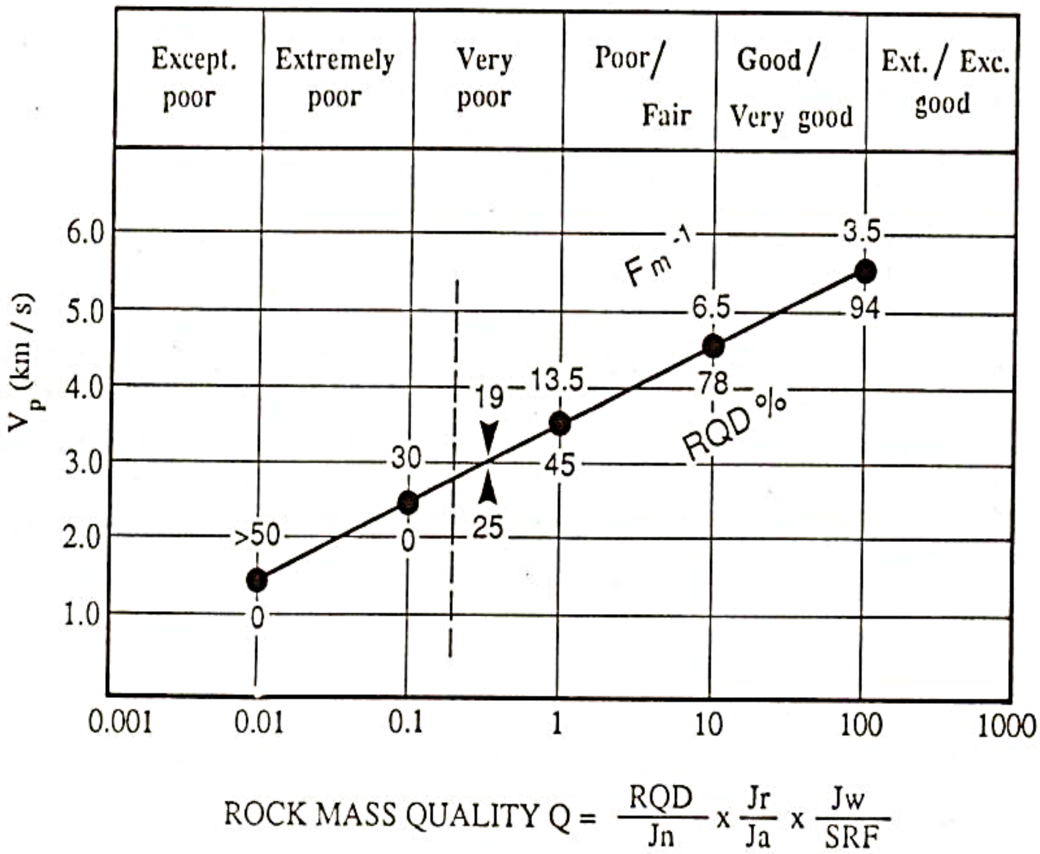


Figure 2.3: Correlation between Q and V_p.

However, in 1993 this system was updated by Grimstad and Barton to include more about 1050 case studies that were from the main road tunnels in Norway, particularly the cases where the Q-system had not been incorporated for estimation of the permanent support. Thus, this update suggested steel-fiber reinforced shortcrete known as sprayed concrete in place of the meshing method. Thus, the earlier three-step procedure was replaced by a more efficient one-step procedure.

Recently it was realized that the need for support systems actually represent the need for enhancing the cohesive and frictional strength of the rock masses. This led to the development of a direct normalization of Q with the uniaxial compressive strength, i.e.

$$Q_c = Q \times \frac{\sigma_c}{100}$$

This was needed so as to obtain correlation of Q with rock mass parameters like deformation modulus and seismic velocity.

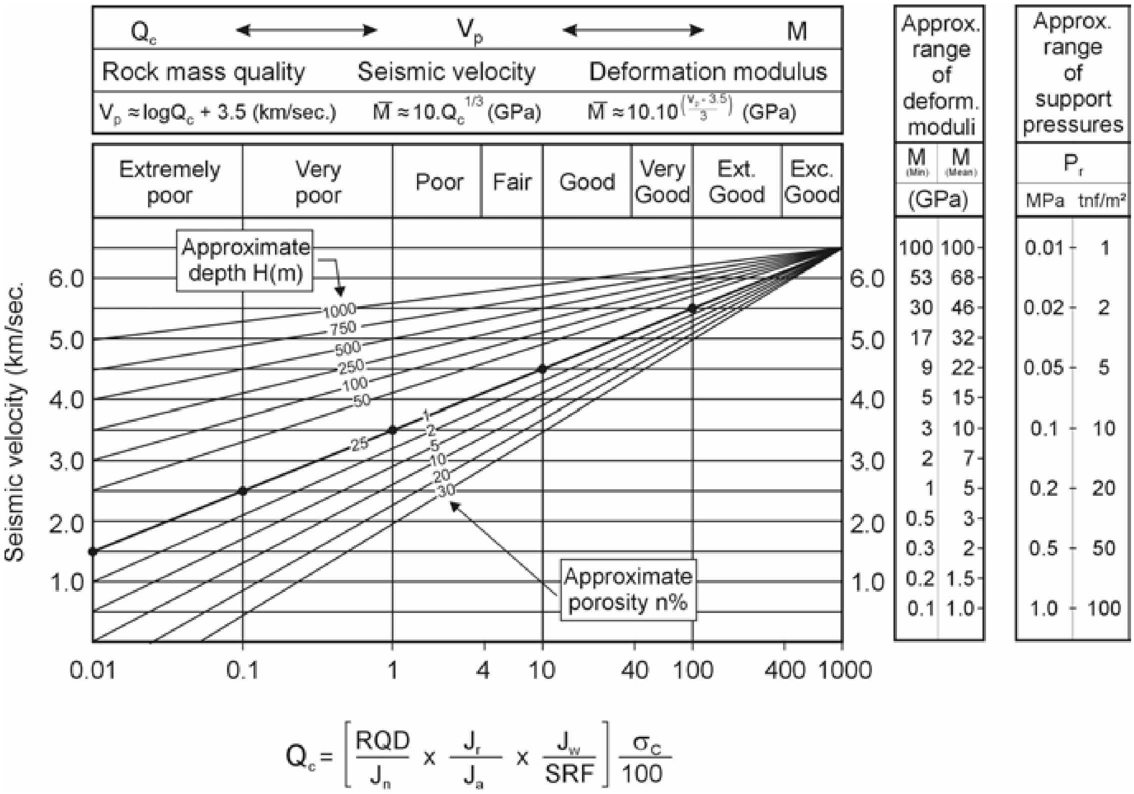


Figure 2.4: Correlation between Q_c, V_p, Emass (or M) and the approximate support pressure.

The Q system is based on the concept of three essential requirements:

- Rock mass categorization based on rock quality.
- Provide optimum dimensions for excavation with a desired factor of safety.
- Estimation of suitable support.

2.4.1 Q-SYSTEM PARAMETERS

The rock mass quality Q is evaluated by the consideration of six parameters that are combined in the following way:

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

where,

1. RQD = Rock quality designation

In case of a very poor rock mass, a minimum value of 10 for RQD should be used.

$$RQD = \frac{\sum \text{length of core pieces} > 100\ mm}{\text{Total core run length}} \times 100$$

2. J_n = Joint set number

This factor incorporates the number of joint sets, if any in the rock mass.

Table 2.2: Joint Set Number, J_n

Massive, No Joints	0.5- 1.00
One Set	2.00
Two Sets	4.00
Three Sets	9.00
Four or more Sets	15.00
Crushed Rocks	20.00

3. J_r = Joint roughness number

The roughness number tells us the degree to which a rock mass has been altered as a result of geological and environmental condition.

Table 2.3: Joint Roughness Number, J_r

Non Continuous Joints	4.00
Rough and wavy	3.00
Smooth `	2.00
Rough and Planar	1.5
Slick Planar	0.5
Filled	1.00
Adjustment factor (when mean point spacing exceeds 3m)	1.00

4. J_a = Joint alteration and wall rock alteration number

The parameter joint alteration number incorporates the infilling material present in between the joints or bedding planes, if any.

Table 2.4: Joint alteration and wall rock alteration number, $J_a$for unfilled rock.

QUANTITY	VALUE
Healed	0.75
Silty and Sandy Coating	3.00
Clay Coating	4.00

Table 2.5: Joint alteration and wall rock alteration number, $J_a$for filled rock.

QUANTITY	VALUE
Sand or Crushed Rock Filling	4.00
Stiff Clay Filling (5mm)	6.00
Soft Clay Filling (5mm)	8.00
Soft Clay Filling (5mm)	15
Swelling Clays (5mm)	20

5. J_w = Joint water reduction factor

The water reduction factor J_w takes the reduction of normal force along the joint into account. This in turn reduces joint shear strength. This factor is important because water presence has an adverse effect on the overall rock mass strength.

Table 2.6: Joint water reduction factor, J_w

Dry Rock	1.00
Medium Water Flow	0.66
Large Inflow	0.5
High Continuous Inflow	0.1

6. SRF = Strength reduction factor

Table 2.7: Strength reduction factor, SRF

Loose Rock with Clay Filled Discontinuities	10.00
Loose Rock with Open Discontinuities	5.00
Rock at Shallow Depth (less than 50m) with Clay Filled Discontinuities	2.05
Rock with Tight Unfilled Discontinuities	1.05

The strength reduction factor (SRF) is composed of:

- Whenever there is an excavation through a shear zone or such rocks that contain clay in between its bedding planes or in between the joints. Loosening pressure is observed which is included in SRF.
- It also incorporates rock stresses which σ_c/σ_1, where σ_c is the uniaxial compressive strength of the intact rock and σ_1 is the major principal stress before excavation takes place.
- Squeezing and swelling pressures in plastic, incompetent rock masses.

The above six parameters are grouped into three quotients to give the overall rock mass quality.

- The first quotient is a relative measure of the block size as the first two parameters are related to the overall structure of the rock mass. The number of joint sets is usually affected by cleavages, foliations, bedding planes etc. The continuities should be taken as a complete joint set if they are strongly developed. If the joints sets are found occasionally in the rock core obtained from the field and are only slightly visible then in such cases they must be taken as a random joint set. The value of J_n is approximately equal to the square of the number of the joints found in a region.
- The second quotient is an indicator of the inter-block shear strength. It is been observed that $\tan^{-1}(J_r/J_a)$ is almost equal to the peak angle of internal friction in clay coated joints. Thus, this gives the frictional characteristics of the rock mass. It must be kept in mind that J_r/J_a of the joint which is most critical for stability must be used in the calculations. It is possible that the joint set that has a minimum value of J_r/J_a is oriented in such a way that it is more stable than other joint sets having higher value of J_r/J_a. In such cases the second less favourable joint set that is oriented in less stable state should be of more value for the design purposes and its value should be used in evaluating Q for conservativeness.

- The third quotient is described as the “active stresses”. Barton et.al defined J_n, J_r and J_a as the controlling parameters in the evaluation of Q value. If joint orientation was to be included in this system then it would have become less general. The joint orientation however is indirectly incorporated in these parameters.

2.4.2 Q SUPPORT SYSTEM

The Q value obtained is related to the support system requirements by defining an equivalent dimension of the underground opening. This equivalent dimension is related to the type and size of excavation.

It is given as:

$$D_e = \frac{D_t}{ESR}$$

where,

D_t = span (distance from the face of excavation to the last support), diameter or roof wall height of the excavation

ESR = excavation span ratio; which depends on the purpose of tunnel

It is given in the table as under:

Table 2.8: Ratings of the excavation support ratio (ESR) (from Barton et. al., 1974).

TYPE OR USE OF UNDERGROUND OPENING	ESR
Temporary mine openings	3.5
Vertical shafts, rectangular and circular respectively	2.0 - 2.5
Water tunnels, permanent mine openings, adits, drifts	1.6
Storage caverns, road tunnels with little traffic, access tunnels, etc.	1.3
Power stations, road and railway tunnels with heavy traffic, civil defense shelters, etc.	1.0
Nuclear power plants, railroad stations, sport arenas, etc.	0.8

The Q-value in Figure 2.5 is related to the total amount of support (temporary and permanent) in the roof. The diagram is based on numerous tunnel support cases. Wall support can also be found using the same figure by applying the wall height and the following adjustments to Q:

For $Q > 10$; use $Q_{wall} = 5Q$

For $0.1 < Q < 10$ use $Q_{wall} = 2.5Q$

For $Q < 0.1$ use $Q_{wall} = Q$

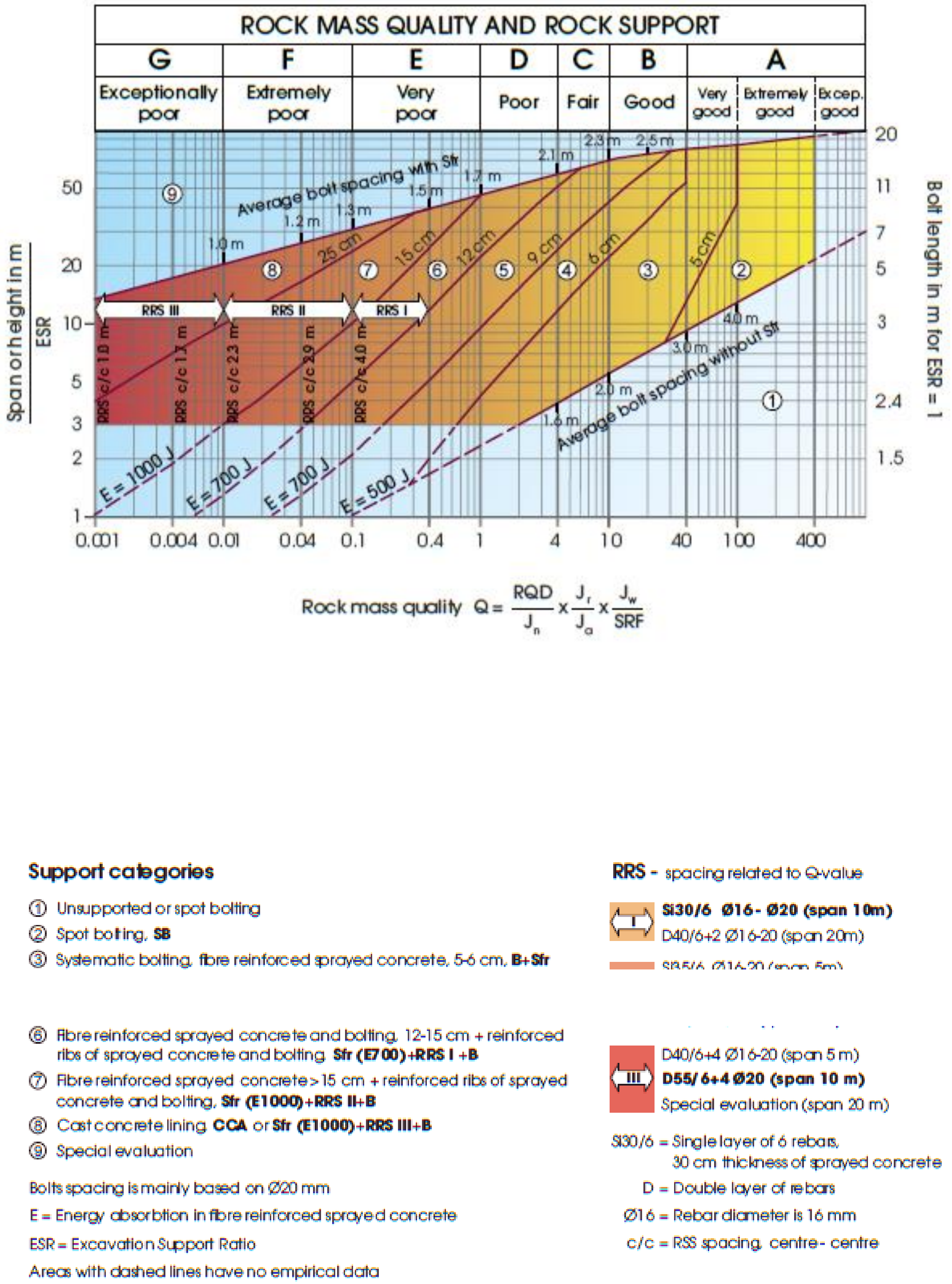


Figure 2.5: The Q system chart for rock support estimate, developed by the Norwegian Geotechnical Institute (NGI),

2.5 GEOMECHANICS OR ROCK MASS RATING SYSTEM (BIENIAWSKI, 1973, 1976, 1989)

Rock mass rating index was introduced by Bieniawski in 1972-1973 in the South African Council of Scientific and Industrial Research (CSIR). He developed this system using his experience which was on shallow tunnels in sedimentary rocks. This system had been modified over the years. This system had been applied to 351 case histories over the span of 15 years which has validated the authenticity, versatility and the ease with which it can be used. This system was modified in the years 1974, 1976, 1979 and 1989.

2.5.1 APPLICATION OF THE PARAMETERS TO DETERMINE THE RMR INDEX

In order to apply the RMR system over a given topology the site must be divided into different sections on the basis of consistency of the rock properties. For each of the section, a rating is selected against the parameters that determine the RMR index rating.

The following six parameters are used to describe the Rock Mass Rating given as under:

1. Uniaxial compressive strength of intact
2. Rock quality designation (RQD)
3. Spacing of discontinuities
4. Condition of discontinuities, given as
   - 4a. length, persistence
   - 4b. separation
   - 4c. smoothness
   - 4d. infilling
   - 4e. alteration / weathering
5. Groundwater conditions
6. Orientation of discontinuities

Table 2.9: RMR classification system and its ratings (after Bieniawski, 1989).

<table>
<tr><th colspan="3">Parameter</th><th colspan="7">Ranges of values</th></tr>
<tr><td rowspan="3">1</td><td rowspan="2">IRS</td><td>PLSI (MPa)</td><td>>10</td><td>4 - 10</td><td>2 - 4</td><td>1 - 2</td><td colspan="3">For this low range UCS is preferred</td></tr>
<tr><td>UCS (MPa)</td><td>>250</td><td>100 - 250</td><td>50 - 100</td><td>25 - 50</td><td>5-25</td><td>1-5</td><td><1</td></tr>
<tr><td colspan="2">Rating</td><td>15</td><td>12</td><td>7</td><td>4</td><td>2</td><td>1</td><td>0</td></tr>
<tr><td rowspan="2">2</td><td colspan="2">Drill core quality RQD (%)</td><td>90 - 100</td><td>75 - 90</td><td>50 - 75</td><td>25 - 50</td><td colspan="3"><25</td></tr>
<tr><td colspan="2">Rating</td><td>20</td><td>17</td><td>13</td><td>8</td><td colspan="3">3</td></tr>
<tr><td rowspan="2">3</td><td colspan="2">JS: joint spacing (mm)</td><td>>2000</td><td>600 - 2000</td><td>200 - 600</td><td>60 - 200</td><td colspan="3"><50</td></tr>
<tr><td colspan="2">Rating</td><td>20</td><td>15</td><td>10</td><td>8</td><td colspan="3">5</td></tr>
<tr><td rowspan="2">4</td><td colspan="2">JC: joint condition</td><td>Very rough surfaces. Not continuous. No separation. Unweathered wall rock</td><td>Slightly rough surfaces. Separation < 1 mm. Slightly weathered walls.</td><td>Slightly rough surfaces. Separation < 1 mm. Highly weathered walls.</td><td>Slickensided surfaces OR Gouge < 5 mm thick OR Separation 1-5 mm. Continuous</td><td colspan="3">Soft gouge > 5 mm thick OR separation > 5 mm. Continuous</td></tr>
<tr><td colspan="2">Rating</td><td>30</td><td>25</td><td>20</td><td>10</td><td colspan="3">0</td></tr>
<tr><td rowspan="4">5</td><td rowspan="3">GW: groundwater condition</td><td>Inflow per 10 m tunnel length</td><td>None</td><td><10 l/min</td><td>10 – 25 l/min</td><td>25 – 125 l/min</td><td colspan="3">>125 l/min</td></tr>
<tr><td>Ratio (Joint water pressure: Major principle stress)</td><td>0</td><td>0.0 - 0.1</td><td>0.1 – 0.2</td><td>0.2 – 0.5</td><td colspan="3">>0.5</td></tr>
<tr><td>General conditions</td><td>Completely dry</td><td>Damp</td><td>Wet</td><td>Dripping</td><td colspan="3">Flowing</td></tr>
<tr><td colspan="2">Rating</td><td>15</td><td>10</td><td>7</td><td>4</td><td colspan="3">0</td></tr>
<tr><td rowspan="3">6</td><td colspan="2">Strike and dip of joints</td><td>Very favourable</td><td>Favourable</td><td>Fair</td><td>Unfavourable</td><td colspan="3">Very unfavourable</td></tr>
<tr><td rowspan="2">RA: Rating adjustment</td><td>Tunnels</td><td>0</td><td>-2</td><td>-5</td><td>-10</td><td colspan="3">-12</td></tr>
<tr><td>Foundations</td><td>0</td><td>-2</td><td>-7</td><td>-15</td><td colspan="3">-25</td></tr>
</table>

By having determined the rating for each of the parameters from the table 2.9, the ratings are added to get a basic value of RMR; which is then adjusted accordingly.

In the case where tunnel boring machine (TBM) is used for carrying out the excavation of the tunnel, it suggested to add 10 points to the value of RMR as heavy blasting may create new fractures and weakness planes in rock resulting in the reduction of RMR value. In the case where controlled blasting is carried out 3 to 5 points should be added to the calculated value of RMR.

Figure 2.6 presents an algorithm which can be considered as an example to determine the basic RMR rating and then applying various adjustments to get the final rating.

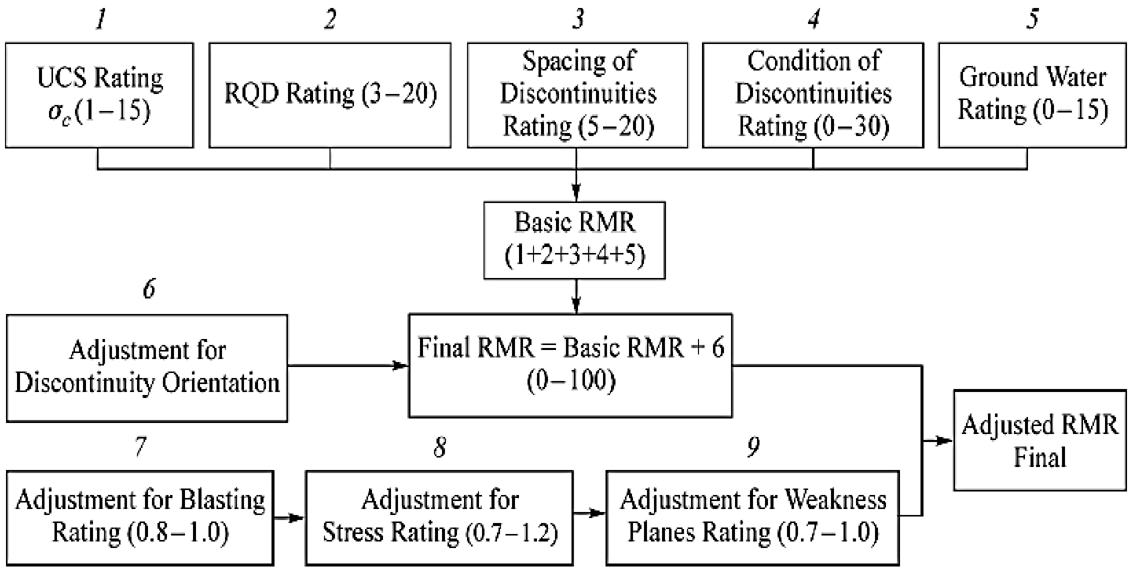


Figure 2.6: RMR calculation algorithm

2.5.2 Application of the Rock Mass Rating

The RMR rating is related to the class of rock mass as well as the cohesion and angle of internal friction of rock mass given as under:

Table 2.10: Rock mass classes determined from total ratings.

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

Thus the rock mass class as determined from the table 2.10 can be used for the estimation of the excavation method and the type of support required for each of the sections into which the site was divided on the basis of consistent rock mass properties for the calculation of RMR rating for each of the section.

Table 2.11: Excavation and support in horseshoe shaped 10 m wide drill and blast excavated rock tunnels with vertical stress < 25 MPa (after Bieniawski, 1989)

Rock Mass Class	Excavation	Support		
		Bolts: 20mm diameter fully grouted	Shotcrete	Steel sets
I - Very good RMR: 100-81	Full face 3 m advance.	Generally no support required except for occasional spot bolting		
II – Good RMR: 80-61	Full face 1.0-1.5 m advance. Complete support 20 m from face.	Locally bolts in crown, 3 m long, spaced 2.5 m, with occasional mesh	50 mm in crown where required	None
III – Fair RMR: 60-41	Top heading & bench, 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5-2 m in crown & walls with wire mesh in crown	50-100 mm in crown & 30 mm in sides	None
IV - Poor RMR: 40-21	Top heading & bench, 1.0-1.5 m advance in top heading. Install support concurrently with excavation 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5m in crown & walls with mesh	100-150 mm in crown & 100 mm in sides	Light to medium ribs spaced 1.5 m where required.
V - Very poor RMR: 20-0	Multiple drifts. 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 11.5 m in crown & walls with wire mesh. Bolt invert	150-200 mm in crown & 150 mm in sides & 50 mm in face	Medium to heavy ribs spaced 0.75 m; lagging & fore poling if required. Close invert

The average stand-up time depends upon the span of the opening and the RMR value. The span is the width of the opening or the distance between the tunnel face and the last support. The smaller of the two is the span of the opening. For an arched roof, the stand-up time is significantly higher than that for a flat roof. This is because the arching effect increases the stability of the roof and hence it can stand longer time while it’s unsupported. Controlled blasting can further increase the stand-up time.

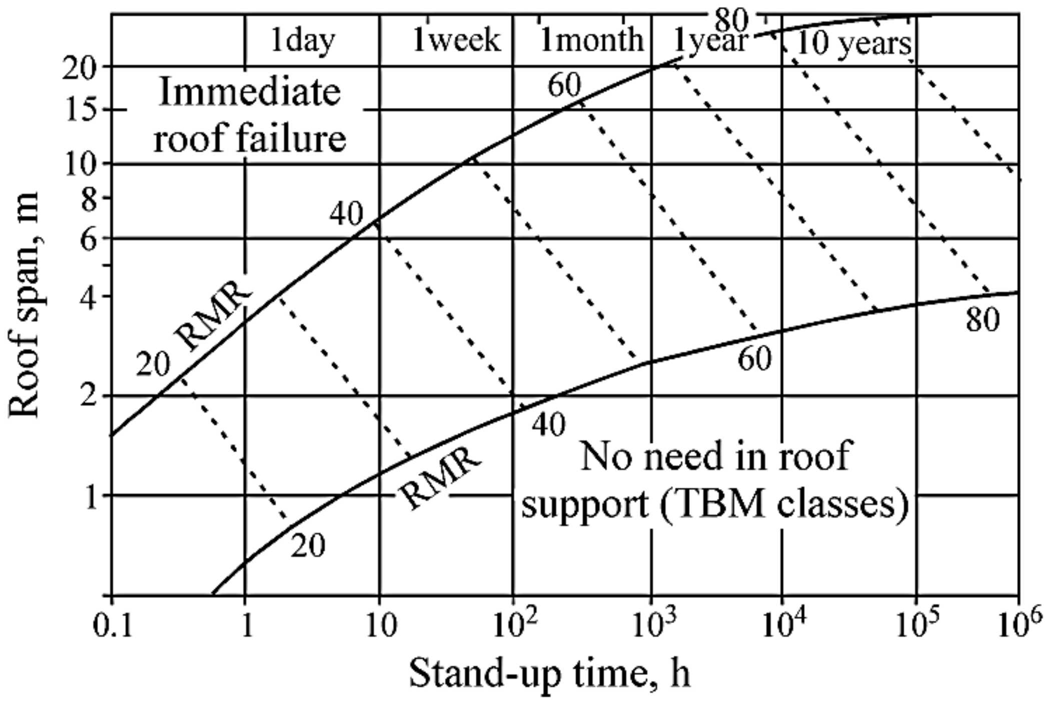


Figure 2.7: Relationship between stand-up time, span and RMR classification

Laufer (1988) concluded that the stand-up time jumps to one class higher of RMR value if TBM is used instead of conventional tunneling boring methods.

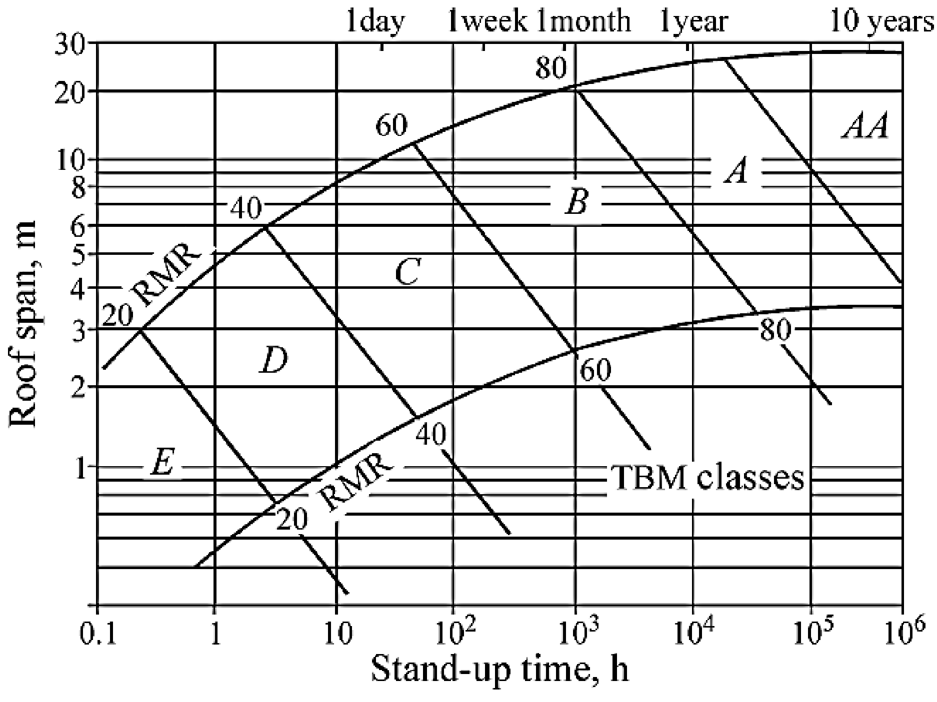


Figure 2.8: Modified relationship between stand-up time, roof span for TBM

CHAPTER 3

ROCK MASS CLASSIFICATION OFABBOTTABADAD, SWAT TWIN TUBE AND NAHAKKI PASS TUNNEL

3.1 INTRODUCTION

Rock mass classification for Abbottabad, Swat twin tube and Nahakki pass tunnel is carried out in this chapter, Rock mass rating system, Norwegian Q system and Geological strength index is used mainly for this purpose, For rock mass classification the properties of rock mass such as intactness percentage of rock mass, discontinuities set and orientation etc. can be calculated using the methods described in the previous chapter, however the geological information required can be taken from site surveys and geological face mappings etc.

In this chapter the rock mass classification for five sections of Abbottabad tunnel and the Nahakki Pass Tunnel is carried out.

3.2 ROCK MASS CLASSIFICATION OF ABBOTTABAD TUNNEL

Abbottabad Tunnel is divided into five sections of different rock masses after geological surveys, the sections are shallow clay, shallow phyllite, deep buried slate, shallow slate and deep buried phyllite.

3.2.1 SECTION 1&2 (SHALLOW CLAY & PHYLLITE)

The very first section of the tunnel comprises of clay mixed with weak phyllite. The uniaxial compressive strength of the core sample from this section shows 12 MPa and percentage of rock intactness is 10 percent, these parameters are used to reach a quantitative value of RMR and Q system.

Q CLASSIFICATIONRATING

Rock Quality Designation (RQD)	10
Joint Set Number (Heavily Jointed)	17
Joint Roughness Number (Smooth, Planar)	01
Joint Alteration Number (Structural Planes filled with mud)	03
Joint Water Reduction Number	0.6
Stress reduction factor	10

Using these values, the Q values are calculated as shown below

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

$$Q = \frac{10}{17} \times \frac{1}{3} \times \frac{0.66}{10}$$

$$Q = 0.04$$

Rock Mass found in these two sections have almost similar rock mass properties, hence they are assigned same values for Q classification, and since the same rock mass properties are used to calculate RMR values both these sections will be assigned a single RMR value as well.

RMR CLASSIFICATION	RATING
A. Rock Quality Designation (RQD =10)	05
B. Uniaxial Compressive Strength (Rc = 12MPa)	02
C. Spacing of the discontinuities (60mm-200mm)	08
D. Conditions of the discontinuities	
Length of the discontinuities (1-3m)	04
Separation of the discontinuities (1-5mm)	01
Roughness of the discontinuities (smooth)	01
Infilling of Joints (Soft mud <5mm)	02
Weathering of Joints (Highly Weathered)	01
	09
E. Ground Water Conditions (Damp Conditions)	10
BASIC RMR VALUE	**36**
F. Discontinuity Orientation (Fair)	-5
TOTAL RMR VALUE	**31**

Rock mass of these sections fall under the section 4 of RMR classification which identifies them as poor-quality rock.

3.2.2 RMR FROM THE CORRELATION

RMR can also be calculated using the following correlation

$$\mathrm{RMR} = 9\log\mathrm{Q} + 44$$

$$\mathrm{RMR} = 9\log(0.04) + 44$$

$$\mathrm{RMR} = 31.41$$

Results from this correlation are also very close to the conventionally used methods of finding the RMR values, the results from the correlation also identifies the rock mass as poor. Furthermore, the Q and RMR values for the remaining sections will be shown in the form of tables.

3.3 SUMMARIZED VALUES

Q VALUE

Table 3.1: Summarized Values for Q Classification of Section 1 (Shallow Clay and Phyllite)

SR #	DESCRIPTION	DETAILS	VALUE
1	Rock Quality Designation		10
2	Joint Set Number	Heavily Jointed	17
3	Joint Roughness Number	Smooth, Planar	1
4	Joint Alteration Number	Structural Planes filled with Mud	3
5	Joint Water Conditions	Poor Fissure Water (Wet Clay)	0.6
6	Stress Reduction Factor	Loose Surrounding Rock	10
7	Q value		0.04

Table 3.2: Summarized Values for Q Classification of Section 2 (Shallow Buried Slate)

SR #	DESCRIPTION	DETAILS	VALUE
1	Rock Quality Designation		27
2	Joint Set Number	Heavily Jointed and Crushed Rock Mass	20
3	Joint Roughness Number	Smooth, Planar	1
4	Joint Alteration Number	Structural Planes filled with Mud	3
5	Joint Water Conditions	Poor Fissure Water (Wet Clay)	0.6
6	Stress Reduction Factor	Loose Surrounding Rock	10
7	Q value		0.089

Table 3.3: Summarized Values for Q Classification of Section 3 (Deep Buried Slate)

SR #	DESCRIPTION	DETAILS	VALUE
1	Rock Quality Designation		50
2	Joint Set Number	Heavily Jointed, 4 and more joints	15
3	Joint Roughness Number	Smooth, Planar	1
4	Joint Alteration Number	Structural Planes filled with Mud	3
5	Joint Water Conditions	Poor Fissure Water (Wet Clay)	0.6
6	Stress Reduction Factor	Loose Surrounding Rock	10
7	Q value		0.04

Table 3.4: Summarized Values for Q Classification of Section 4 (Deep Buried Phyllite)

SR #	DESCRIPTION	DETAILS	VALUE
1	Rock Quality Designation		5
2	Joint Set Number	Heavily Jointed, Crushed Rock Mass	15
3	Joint Roughness Number	Smooth, Planar	1
4	Joint Alteration Number	Structural Planes filled with Mud	3
5	Joint Water Conditions	Poor Fissure Water (Wet Clay)	0.6
6	Stress Reduction Factor	Loose Surrounding Rock	10
7	Q value		0.022

RMR VALUES

Section 1 (Shallow Clay and Phyllite)

DISCONTINUITY CONDITION	RATING
1- Length (1-3m)	04
2- Separation (1-5mm)	01
3- Roughness (Smooth)	01
4- Infilling of joints (Soft Mud< 5mm)	02
5- Weathering of joints (Highly Weathered)	01
	09

Table 3.5: Summarized Values for RMR Classification of Section 2 (Shallow Buried Slate)

SR #	DESCRIPTION	VALUE	REMARKS	RATING
1	Rock Quality Designation	10		05
2	Uniaxial Compressive Strength	12		02
3	Spacing of Discontinuities	60mm-200mm		08
4	Condition of Discontinuities		N/A	9
5	Orientation of Discontinuity		Fair	5
6	Ground Water Conditions		Damp	10

BASIC RMR = 36

Total RMR = BASIC RMR – Orientation of Discontinuity = 36 – 5 = 31

Section 2 (Shallow Buried Slate)

DISCONTINUITY CONDITION	RATING
1- Length (1-3m)	04
2- Separation (1-5m)	01
3- Roughness (Smooth)	01
4- Infilling of joints (No filling)	06
5- Weathering of joints (Highly Weathered)	01
	13

Table 3.6: Summarized Values for RMR Classification of Section 2 (Shallow Buried Slate)

SR #	DESCRIPTION	VALUE	REMARKS	RATING
1	Rock Quality Designation	27		08
2	Uniaxial Compressive Strength	30.58		04
3	Spacing of Discontinuities	60mm-200mm		08
4	Condition of Discontinuities		N/A	13
5	Orientation of Discontinuity		Fair	5
6	Ground Water Conditions		Damp	10

BASIC RMR = 43

Total RMR = BASIC RMR – Orientation of Discontinuity = 43 – 5 = 38

Section 3 (Deep Buried Slate)

DISCONTINUITY CONDITIONS	RATING
1- Length (1-3m)	04
2- Separation (1-5m)	01
3- Roughness (Smooth)	01
4- Infilling of joints (Soft mud <5mm)	02
5- Weathering of joints (Moderately Weathered)	03
	11

Table 3.7: Summarized Values for RMR Classification of Section 3 (Deep Buried Slate)

SR #	DESCRIPTION	VALUE	REMARKS	RATING
1	Rock Quality Designation	50		08
2	Uniaxial Compressive Strength	30		04
3	Spacing of Discontinuities	60mm-200mm		08
4	Condition of Discontinuities		N/A	11
5	Orientation of Discontinuity		Fair to unfavorable	7.5
6	Ground Water Conditions		Damp	10

BASIC RMR = 44

Total RMR = BASIC RMR – Orientation of Discontinuity = 44 – 7.5 = 37.5

Section 4 (Deep Buried Phyllite)

DISCONTINUITY CONDITIONS	RATING
1- Length (1-3m)	04
2- Separation (1-5m)	01
3- Roughness (Smooth)	01
4- Infilling of joints (Soft mud <5mm)	02
5- Weathering of joints (Highly Weathered)	01
	09

Table 3.8: Summarized Values for RMR Classification of Section 4 (Deep Buried Phyllite)

SR #	DESCRIPTION	VALUE	REMARKS	RATING
1	Rock Quality Designation	5		02
2	Uniaxial Compressive Strength	12Ma		02
3	Spacing of Discontinuities	60mm-200mm		08
4	Condition of Discontinuities		N/A	11
5	Orientation of Discontinuity		Fair	5
6	Ground Water Conditions		Damp	10

BASIC RMR =31

Total RMR = BASIC RMR – Orientation of Discontinuity = 31 - 5 = 26

3.4 ROCK MASS CLASSIFICATION OF SWAT TWIN TUBE

As the name suggests Swat twin tube consists of two portals, a northbound portal and a southbound portal, we will look into each portals rock mass properties differently, starting from northbound tunnel.

3.4.1 NORTHBOUND PORTAL

The northbound portal consists of only one type of rock mass and that is graphitic schist, given below are the table for Q and RMR classification respectively.

Table 3.9: Summarized Values for Q Classification of Northbound Portal (Graphitic Schist)

SR #	DESCRIPTION	DETAILS	VALUE
1	Rock Quality Designation	Relatively intact rock mass	92
2	Joint Set Number	Heavily jointed, crushed rock mass	6
3	Joint Roughness Number	Slightly rough, slightly undulated	1.75
4	Joint Alteration Number	Healed structural planes	0.75
5	Joint Water Conditions	No inflow of water	1
6	Stress Reduction Factor	Good surrounding rock	1
7	Q value	Roof value of Q	35.7

DISCONTINUITY CONDITIONS	RATING
1- Length (3-10m)	02
2- Separation (0.1-0.5mm)	04
3- Roughness (Smooth)	03
4- Infilling of joints (--)	06
5- Weathering of joints (--)	06
	21

Table 3.10: Summarized Values for RMR Classification of Northbound Portal (Graphitic Schist)

SR #	DESCRIPTION	VALUE	REMARKS	RATING
1	Rock Quality Designation	92		20
2	Uniaxial Compressive Strength	34.7 MPa		04
3	Spacing of Discontinuities	0.06 m -2 m		15
4	Condition of Discontinuities		N/A	21
5	Orientation of Discontinuity		Unfavorable	10
6	Ground Water Conditions		Dry	15

BASIC RMR = 75

Total RMR = BASIC RMR – Orientation of Discontinuity = 75 - 10 = 65

3.4.2 SOUTHBOUND TUNNEL

Southbound portal also consists of graphitic schist entirely.

Table 3.11: Summarized Values for Q Classification of Southbound Portal (Graphitic Schist)

SR #	DESCRIPTION	DETAILS	VALUE
1	Rock Quality Designation	Relatively intact rock mass	57
2	Joint Set Number	3 random sets	12
3	Joint Roughness Number	Smooth, planar	1
4	Joint Alteration Number	Structural planes filled with swelling clay	4
5	Joint Water Conditions	Dripping water inflow	0.5
6	Stress Reduction Factor	Medium to moderate stresses	1
7	Q value	Roof value of Q (poor rock mass)	0.59115

DISCONTINUITY CONDITIONS	RATING
1- Length (0.1-1m)	06
2- Separation (0.1-0.5mm)	04
3- Roughness (Smooth)	01
4- Infilling of joints (Swelling clay)	05
5- Weathering of joints (Highly Weathered)	00
	16

Table 3.12: Summarized Values for RMR Classification of Southbound Portal (Graphitic Schist)

SR #	DESCRIPTION	VALUE	REMARKS	RATING
1	Rock Quality Designation	57		13
2	Uniaxial Compressive Strength	34.7 MPa		04
3	Spacing of Discontinuities	0.2 m -0.6 m		10
4	Condition of Discontinuities		N/A	16
5	Orientation of Discontinuity		Fair	5
6	Ground Water Conditions		Dripping	4

BASIC RMR = 47

Total RMR = BASIC RMR – Orientation of Discontinuity = 47 – 05 = 45

3.5 ROCK MASS CLASSIFICATION OF NAHAKKI TUNNEL

3.5.1 SOUTH PORTAL (MICA SCHIST)

Table 3.13: Summarized Values for Q Classification of South Portal (Mica Schist)

SR	DESCRIPTION	DETAILS	VALUE
1	Rock Quality Designation	Relatively intact rock mass	16
2	Joint Set Number	3 joint sets	18
3	Joint Roughness Number	Rough undulating joints	3
4	Joint Alteration Number	Moderately-slightly weathered joints	2.5
5	Joint Water Conditions	Dry	1
6	Stress Reduction Factor	Medium to moderate stresses	1
7	Q value	Roof value of Q (poor rock mass)	1.06

DISCONTINUITY CONDITIONS	RATING
1- Length (1-3m)	04
2- Separation (0.1-1mm)	04
3- Roughness (slightly rough)	03
4- Infilling of joints (hard filling < 5mm)	04
5- Weathering of joints (Moderately Weathered)	03
	18

Table 3.14: Summarized Values for RMR Classification of South Portal (Mica Schist)

SR #	DESCRIPTION	VALUE	REMARKS	RATING
1	Rock Quality Designation	16		4
2	Uniaxial Compressive Strength	20 MPa		03
3	Spacing of Discontinuities	0.05 m -1.00 m		8
4	Condition of Discontinuities		N/A	18
5	Orientation of Discontinuity		Favorable	2
6	Ground Water Conditions		Dry	15

BASIC RMR = 48

Total RMR = BASIC RMR – Orientation of Discontinuity = 48 – 02 = 46

3.5.2 MIDDLE SECTION 1 (MARBLE)

Table 3.15: Summarized Values for Q Classification of middle section 1 (Marble)

SR #	DESCRIPTION	DETAILS	VALUE
1	Rock Quality Designation	Relatively intact rock mass	21.5
2	Joint Set Number	3 joint sets, random	9
3	Joint Roughness Number	Rough undulating joints	2.5
4	Joint Alteration Number	Slightly altered joint walls	2.0
5	Joint Water Conditions	Wet	0.66
6	Stress Reduction Factor	Medium to moderate stresses	1
7	Q value	Roof value of Q (poor rock mass)	1.185

DISCONTINUITY CONDITIONS	RATING
1- Length (1-3m)	04
2- Separation (0.1-1mm)	04
3- Roughness (Slightly rough)	03
4- Infilling of joints (hard filling < 5mm)	04
5- Weathering of joints (Slightly Weathered)	05
	20

Table 3.16: Summarized Values for RMR Classification of middle section 1 (Marble)

SR #	DESCRIPTION	VALUE	REMARKS	RATING
1	Rock Quality Designation	21.5		3
2	Uniaxial Compressive Strength	50 MPa		4
3	Spacing of Discontinuities	0.05 m -1.00 m		8
4	Condition of Discontinuities		N/A	20
5	Orientation of Discontinuity		Favorable	2
6	Ground Water Conditions		Damp	10

BASIC RMR = 45

Total RMR = BASIC RMR – Orientation of Discontinuity = 45 – 02 =

3.5.3 MIDDLE SECTION 2 (MICA SCHIST)

Table 3.17: Summarized Values for Q Classification of middle section 2 (Mica Schist)

SR #	DESCRIPTION	DETAILS	VALUE
1	Rock Quality Designation	Relatively intact rock mass	43.5
2	Joint Set Number	2 joint sets, random	6
3	Joint Roughness Number	Rough undulating joints	1.5
4	Joint Alteration Number	Slightly altered joint walls	2.0
5	Joint Water Conditions	Wet	0.66
6	Stress Reduction Factor	Medium to moderate stresses	1
7	Q value	Roof value of Q (poor rock mass)	3.58

DISCONTINUITY CONDITIONS	RATING
1- Length (1-3m)	04
2- Separation (0.1-1mm)	04
3- Roughness (Slightly rough)	03
4- Infilling of joints (hard filling < 5mm)	04
5- Weathering of joints (Slightly Weathered)	05
	20

Table 3.18: Summarized Values for RMR Classification of middle section 2 (Mica Schist)

SR #	DESCRIPTION	VALUE	REMARKS	RATING
1	Rock Quality Designation	43.5		8
2	Uniaxial Compressive Strength	20 MPa		3
3	Spacing of Discontinuities	0.05 m -1.00 m		10
4	Condition of Discontinuities		N/A	20
5	Orientation of Discontinuity		Favorable	2
6	Ground Water Conditions		Damp	10

BASIC RMR = 50

Total RMR = BASIC RMR – Orientation of Discontinuity = 50 – 02 = 48

3.5.4 MIDDLE SECTION 3 (QUARTZATIC MARBLE AND PHYLLITE)

Table 3.19: Summarized Values for Q Classification of middle section 3 (Quartzatic Marble and Phyllite)

SR #	DESCRIPTION	DETAILS	VALUE
1	Rock Quality Designation	Relatively intact rock mass	19
2	Joint Set Number	3 joint sets, random	12
3	Joint Roughness Number	Rough undulating joints	1.5
4	Joint Alteration Number	Highly weathered joints	4
5	Joint Water Conditions	Wet	0.66
6	Stress Reduction Factor	Medium to moderate stresses	1
7	Q value	Roof value of Q (poor rock mass)	0.392

DISCONTINUITY CONDITIONS	RATING
1- Length (3-10m)	02
2- Separation (0.1-1mm)	04
3- Roughness (Rough)	05
4- Infilling of joints (hard filling < 5mm)	02
5- Weathering of joints (highly Weathered)	01
	14

Table 3.20: Summarized Values for RMR Classification of middle section 1 (Quartzatic Marble and Phyllite)

SR #	DESCRIPTION	VALUE	REMARKS	RATING
1	Rock Quality Designation	19		2
2	Uniaxial Compressive Strength	40 MPa		4
3	Spacing of Discontinuities	0.06 m -2.00 m		8
4	Condition of Discontinuities		N/A	14
5	Orientation of Discontinuity		Favorable	2
6	Ground Water Conditions		Wet	7

BASIC RMR = 35

Total RMR = BASIC RMR – Orientation of Discontinuity = 35 – 02 = 33

3.5.5 EXIT NORTH PORTAL (SCHIST)

Table 3.21: Summarized Values for Q Classification of exit section (Schist)

SR #	DESCRIPTION	DETAILS	VALUE
1	Rock Quality Designation	Relatively intact rock mass	19
2	Joint Set Number	3 joint sets, random	12
3	Joint Roughness Number	Rough undulating joints	1.5
4	Joint Alteration Number	Highly weathered joints	4
5	Joint Water Conditions	Wet	0.66
6	Stress Reduction Factor	Medium to moderate stresses	1
7	Q value	Roof value of Q (poor rock mass)	0.392

DISCONTINUITY CONDITIONS	RATING
1- Length (3-10m)	02
2- Separation (0.1-1mm)	04
3- Roughness (slightly rough)	03
4- Infilling of joints (soft filling < 5mm)	02
5- Weathering of joints (highly Weathered)	01
	12

Table 3.22: Summarized Values for RMR Classification of exit section (schist)

SR #	DESCRIPTION	VALUE	REMARKS	RATING
1	Rock Quality Designation	10		2
2	Uniaxial Compressive Strength	20 MPa		2
3	Spacing of Discontinuities	>60mm		5
4	Condition of Discontinuities		N/A	12
5	Orientation of Discontinuity		Fair	5
6	Ground Water Conditions		Wet	7

BASIC RMR = 28

Total RMR = BASIC RMR – Orientation of Discontinuity = 28 - 5 = 33

CHAPTER 4

DEFORMATION ANALYSIS

4.1 INTRODUCTION

To calculate the deformation at the crown of the tunnel a number of analytical techniques were used the techniques can be broadly divided into two categories:

- Empirical Approaches
- Finite Element Analysis

For the finite element analysis (FEA) PHASE2 software was used, model of each section of the selected tunnels was made and analyzed by inputting the relevant field details.

For the empirical approaches three criterions were used, Mohr-Coulomb, Hoek-Brown and Lade-Duncan. In manner similar to the FEA each section of the selected tunnels was analyzed and results were compiled.

4.2 EMPIRICAL APPROACHES

Deformation analysis for the three tunnels was carried out using three empirical approaches namely: Lade-Duncan, Mohr-Coulomb and Hoek-Brown criteria. All three criteria have similar basic assumptions:

- The cross-section of the tunnel is circular and the length of the tunnel is infinite, these assumptions are made to ensure plain strain conditions.
- The overburden and internal support stresses are assumed to be distributed uniformly.
- The rock mass is assumed as continuous, homogenous and isotropic elasto-plastic material.

4.2.1 MOHR-COULOMB

The deformation analysis by the Mohr- Coulomb approach is based on the Mohr-Coulomb strength criterion that is a critical combination of principle stresses causes failure. The equation derived for deformation in the plastic zone is as follows:

$$u_p = \frac{R_p^2(P\sin\emptyset + \cos\emptyset)}{2r_oG} \qquad (1)$$

This criterion takes into account the principal stresses while the influences of the intermediate stresses are completely ignored.

4.2.2 HOEK- BROWN

The deformation analysis in this case is dependent on the uniform resistance provided the internal support,p_i. No failure occurs when the support resistance is greater than the critical support pressure which is defined by:

$$p_{cr} = \frac{2p_0 - \partial_{cm}}{1+k} \qquad (2)$$

In (2) K is defined by the following equation:

$$k_1 = \frac{1+\sin\emptyset}{1-\sin\emptyset} \qquad (3)$$

For the tunnels selected the internal resistance provided by the supports was less than the critical support pressure thus the displacement was calculated in the plastic zone. The equation derived from the Hoek- Brown criterion for the plastic zone deformation is as follows:

$$u_p = \frac{r_{o(1+\vartheta)}}{E}\left[2(1-\mu)(p_o - p_{cr})\left(\frac{r_p}{r_o}\right)^2 - (1-2\vartheta)(p_o - p_i)\right] \qquad (4)$$

For the Hoek-Brown Criterion plastic radius R_p is defined by:

$$R_p = r_o\left[\frac{2(p_o(k-1)+\sigma_{cm})}{(1+k)(k-1)p_i+\sigma_{cm}}\right]^{\frac{1}{k-1}} \qquad (5)$$

4.2.3 LADE-DUNCAN

The Mohr-Coulomb and the Hoek-Brown criterion do not take into account the effect of intermediate principal stresses on the damage and the yield characteristics of pure hydrostatic stresses. The Lade-Duncan criterion takes into account the intermediate principal stresses in a simple and easy to use expression.

The Lade-Duncan failure criterion can be expressed by the following equation:

$$\frac{I_1^3}{I_3} = k_2 = \frac{(3-\sin\emptyset)^3}{(1+\sin\emptyset)(1-\sin\emptyset)^2} \qquad (6)$$

Where $I_1$and I_3 are the first and third tensor invariants

$$I_1 = \partial_1 + \partial_2 + \partial_3$$

$$I_3 = \partial_1 \, \partial_2 \, \partial_3 \qquad (7)$$

The plastic zone deformation is given by the following:

$$u_p = \frac{R_p^2}{2r_o G}\left[p_o - \frac{p_i}{\left(\sqrt{\frac{k}{k-27}}-1\right)^2}\left(\ln\frac{R_p}{r_o} + \sqrt{\frac{k}{k-27}} - 1\right)^2\right] \qquad (8)$$

4.3 APPLICATION OF THE EMPIRICAL APPROACHES TO THE SELECTED TUNNELS.

The selected tunnels were divided into number of sections based on their geology. The deformations were calculated on the crown based on these divisions, the overburden pressure and the tunnel geometry. The internal friction angle was read against RMR values from the correlation (Bieniawski) given in Figure 4.1.

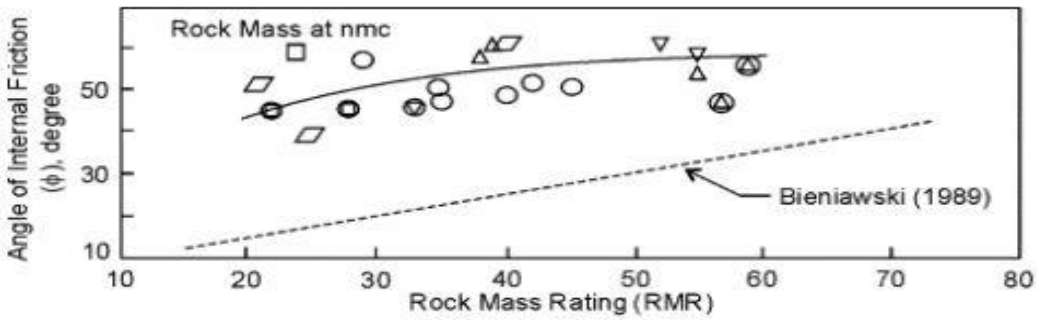


Figure 4.1: Angle of internal friction vs RMR

The Rupture Modulus, G was calculated from the following relation:

$$G = \frac{E_m}{2(1+\vartheta)} \qquad (9)$$

Section details and sample calculations for one section of each of the selected tunnels are shown in the following paragraphs.

4.3.1 NAHAKKI PASS TUNNEL

The Nahakki pass tunnel has a radius of 5.6 m with the depth of overburden ranging from 35 to 50 m for different sections. The unit weight of rocks is taken as 27 $\mathrm{KN/m^3}$.The details of the sections are given in Table 4.1 and Table 4.2

Table 4.1: Nahakki Pass tunnel division based on geology

SECTION	LITHOLOGY	AVG. ELEVATION (m)
Entrance section (RD 15+010 – 15+020)	Mica Schist	35
Middle section-1 (RD 15+020 – 15+050)	MARBLE	40
Middle section-2 (RD 15+050 – 15+130)	Mica Schist	45
Middle section-3 (RD 15+130 – 15+160)	Quartzatic/Phylite	50
Exit section (RD 15+415 – 15+680)	Schist	40

Table 4.2: Nahakki Pass tunnel section properties

SECTION	YOUNG's MODULUS INTACT ROCK, E_i (MPa)	YOUNG's MODULUS ROCK MASS, E_m(MPa)	RMR	UNIAXIAL COMPRESSIVE STRENGTH. UCS (MPa)	Poison's ratio,ϑ
Entrance section	13500	1880	46	20	0.25
Middle section-1	42500	6785	43	50	0.3
Middle section-2	13500	2470	48	20	0.25
Middle section-3	22000	2040	33	40	0.25
Exit section	13500	857	23	20	0.25

The section selected for sample calculation is the entrance section.

4.3.1.1 MOHR- COULOMB CRITERION

- Radius of tunnel, r_o = 5.60 m
- Plastic Radius, R_p = 11 m
- Internal Friction Angle, $\emptyset = 28^0$
- Rupture Modulus, G = 752 MPa
- Overburden pressure, p_o = 0.91 MPa
- Internal Support Pressure, p_i = 0.18 MPa

Substituting these values in (1) gives plastic displacement of 0.017 m.

4.3.1.2 HOEK-BROWN CRITERION

- Radius of tunnel, r_o = 5.60 m
- Internal Friction Angle, $\emptyset = 28^0$
- Active Earth Pressure constant, K_1 = 2.77
- Plastic Radius, R_p = 8.90 , From (5)
- Overburden pressure, p_o = 0.91 MPa
- Internal Support Pressure, p_i = 0.18 MPa

Substituting these values and the required values from Table 4.2 in (4) gives the plastic zone deformation to be 0.007 m.

4.3.1.3 LADE- DUNCAN CRITERION

- Radius of tunnel, r_o = 5.60 m
- Internal Friction Angle, $\emptyset = 28^0$
- Material constant, K_2 = 35.21, From (6)
- Plastic Radius, R_p = 11 m
- Overburden pressure, p_o = 0.91 MPa
- Internal Support Pressure, p_i = 0.18 MPa

Substituting these values and the required values from Table 4.2 in (8) gives the plastic zone deformation to be 0.006m.

4.3.2 SHIMLA TUNNEL, ABBOTTABAD

Shimla tunnel has a radius of 6.64 m with the depth of overburden ranging from 69 to 120 m for different sections. The unit weight of rocks is taken as $27 KN/m^3$. The details of the sections are given in Table 4.3 and Table 4.

Table 4.3: Shimla tunnel division based on geology

SECTION	LITHOLOGY	AVG. ELEVATION (m)
Section 1	Shallow Clay/Phyllite	69
Section 2	Shallow Slate	70
Section 3	Deep Buried Slate	120
Section 4	Deep Buried Phyllite	107.5

Table 4.4: Shimla tunnel section properties

SECTION	YOUNG's MODULUS INTACT ROCK, E_i (MPa)	YOUNG's MODULUS ROCK MASS, E_m(MPa)	RMR	UNIAXIAL COMPRESSIVE STRENGTH. UCS (MPa)	Poison's ratio,ϑ
Section 1	20000	692	34	12	0.3
Section 2	20000	944	38	30	0.3
Section 3	20000	1489	36.5	30	0.3
Section 4	20000	782	26	15	0.3

The section selected for sample calculation is Section 1.

4.3.2.1 MOHR-COULOMB CRITERION

- Radius of tunnel,r_o= 6.64 m
- Plastic Radius, R_p= 14.34 m
- Internal Friction Angle, $\emptyset = 17^0$
- Rupture Modulus, G = 266.31MPa
- Overburden pressure,p_o = 1.86 MPa
- Internal Support Pressure,p_i= 0.58 MPa

Substituting these values in (1) gives plastic displacement of 0.022 m.

4.3.2.2 HOEK-BROWN CRITERION

- Radius of tunnel, r_o = 6.64 m
- Internal Friction Angle, $\emptyset$ = 17^0
- Active Earth Pressure constant, K_1 = 1.83
- Plastic Radius, R_p = 14.34 , From (5)
- Overburden pressure, p_o = 1.86MPa
- Internal Support Pressure, p_i = 0.58 MPa

Substituting these values and the required values from Table 4.4 in (4) gives the plastic zone deformation to be 0.013 m.

4.3.2.3 LADE-DUNCAN CRITERION

- Radius of tunnel, r_o = 6.64 m
- Internal Friction Angle, $\emptyset$ = 17^0
- Material constant, K_2 = 30.13, From (6)
- Plastic Radius, R_p = 14.34 m
- Overburden pressure, p_o = 1.86 MPa
- Internal Support Pressure, p_i = 0.58 MPa

Substituting these values and the required values from Table 4.4 in (8) gives the plastic zone deformation to be 0.045m.

4.3.3 SWAT TWIN TUBE TUNNEL

Swat twin tube tunnel has a radius of 5.53 m with the depth of overburden ranging from 102 to 205 m for different sections. The unit weight of rocks is taken as 26 KN/m^3.The details of the sections are given in Table 4.5 and Table 4.6

Table 4.5: Swat tunnel division based on geology

SECTION	LITHOLOGY	AVG. ELEVATION (m)
Entrance section (RD 00+000 – 00+250)	Graphitic Schist with Quartz veins	102
Middle section-1 (RD 00+250 – 00+500)	Graphitic Schist with Quartz veins	250
Middle section-2 (RD 00+500 – 00+750)	Graphitic Schist with Quartz veins	175
Middle section-3 (RD 00+750 – 01+000)	Graphitic Schist with Quartz veins	121

Table 4.6: Swat tunnel section properties

SECTION	YOUNG's MODULUS INTACT ROCK, E_i (MPa)	YOUNG's MODULUS ROCK MASS, E_m(MPa)	RMR	UNIAXIAL COMPRESSIVE STRENGTH. UCS (MPa)	Poison's ratio,ϑ
Section 1	16000	3210.9	65	34.7	0.215
Section 2	16000	3210.9	65	34.7	0.215
Section 3	16000	3210.9	65	34.7	0.215
Section 4	16000	3210.9	65	34.7	0.215

The section selected for sample calculation is Section 1.

4.3.3.1 MOHR-COULOMB CRITERION

- Radius of tunnel,r_o= 5.53 m
- Plastic Radius, R_p= 6.67 m
- Internal Friction Angle, Ø = 40^0
- Rupture Modulus, G = 1321.35MPa
- Overburden pressure,p_o = 3.39 MPa
- Internal Support Pressure,p_i= 1.02 MPa

Substituting these values in (1) gives plastic displacement of 0.007 m

4.3.3.2 HOEK- BROWN CRITERION

- Radius of tunnel, r_o= 5.53 m
- Internal Friction Angle, $\emptyset$ = 40^0
- Active Earth Pressure constant, K_1= 1.28
- Plastic Radius, R_p= 6.42 , From (5)
- Overburden pressure, p_o = 3.39 MPa
- Internal Support Pressure, p_i= 1.02 MPa

Substituting these values and the required values from Table 4.6 in (4) gives the plastic zone deformation to be 0.008 m.

4.3.3.3 LADE-DUNCAN CRITERION

- Radius of tunnel, r_o= 5.53 m
- Internal Friction Angle, $\emptyset$ = 40^0
- Material constant, K_2= 28.44, From (6)
- Plastic Radius, R_p= 6.67 m
- Overburden pressure, p_o = 3.39 MPa
- Internal Support Pressure, p_i= 1.02 MPa

Substituting these values and the required values from Table 4.6 in (8) gives the plastic zone deformation to be 0.009 m.

4.4 FINITE ELEMENT ANALYSIS

In order to analyze the deformation the generalized Hoek-Brown criterion is used. The reduction factors used in the criterion are obtained using the GSI values. For analysis purpose the tunnels have been divided into number of sections based on the lithology. The GSI values have been assessed using field observations and geological surveys carried out at the project site while the rock properties have been assessed using lab results performed on core logs from the project site.

As a sample, section 2 of Shimla tunnel Abbottabad is considered.

4.4.1 FIELD STRESS APPLICATION

The field stresses are applied based on the overburden. Figure 4.2 shows the application of field while Figure 4.3 shows the stages of stress reduction. Stage 1 represents no excavation whereas stage 11 represents complete excavation. The internal stresses are reduced uniformly between these stages. Figure 4.4 shows the model with no excavation and Figure 4.5 shows the model which is completely excavated.

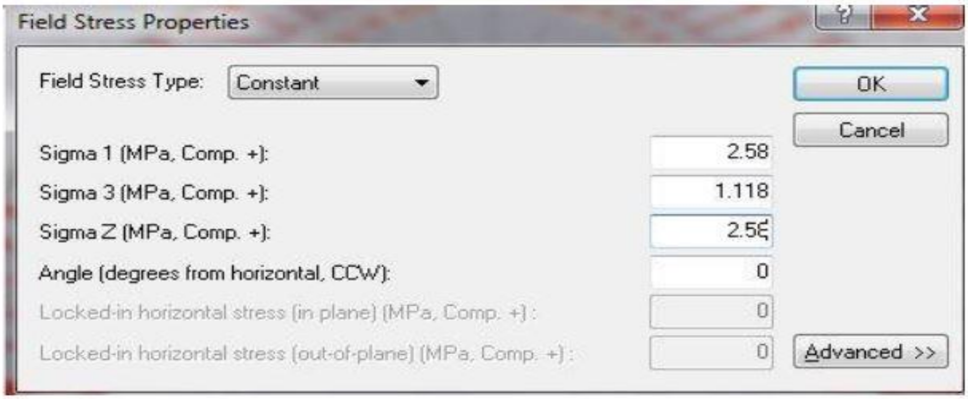
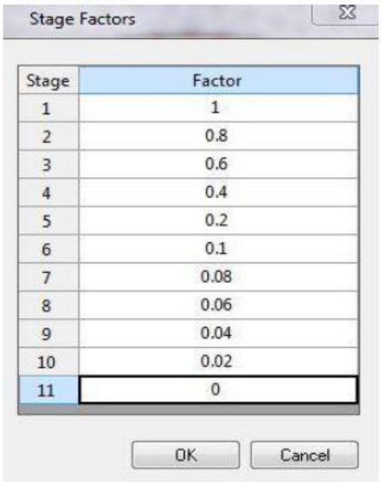


Figure 4.2: Application of field stresses



Stage	Factor
1	1
2	0.8
3	0.6
4	0.4
5	0.2
6	0.1
7	0.08
8	0.06
9	0.04
10	0.02
11	0

Figure 4.3: Load split factor for each stage

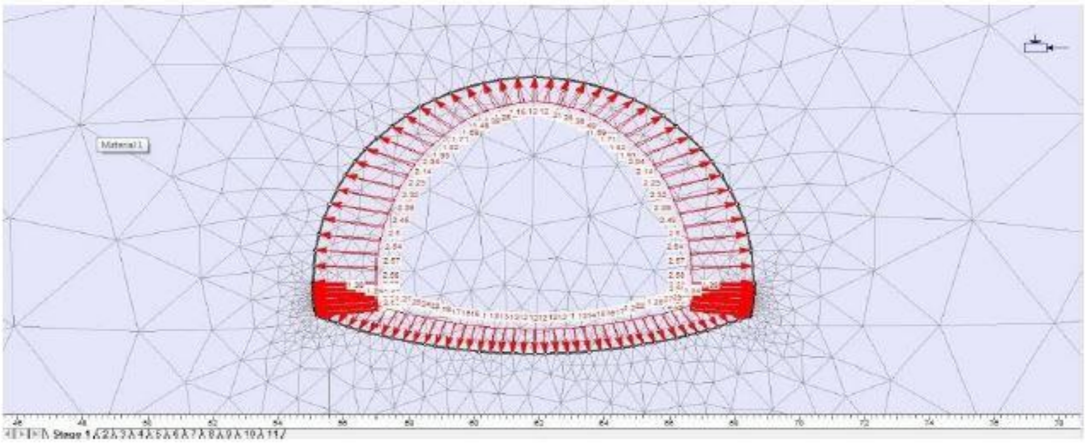


Figure 4.4: Section before excavation

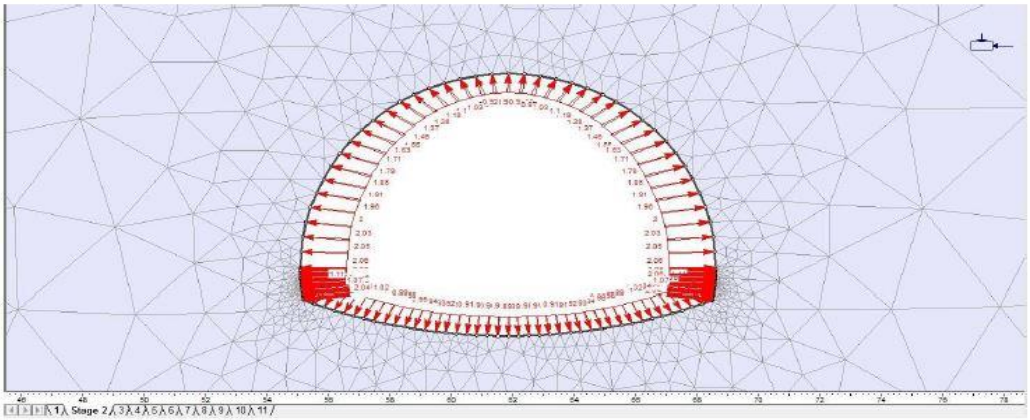


Figure 4.5: Section after excavation

4.4.2 DEFINING MATERIAL PROPERTIES

The material properties for the selected section are assigned to the model. Figure 4.6 shows the material properties assigned and Figure 4.7 values of GSI and disturbance factors assigned to the model.

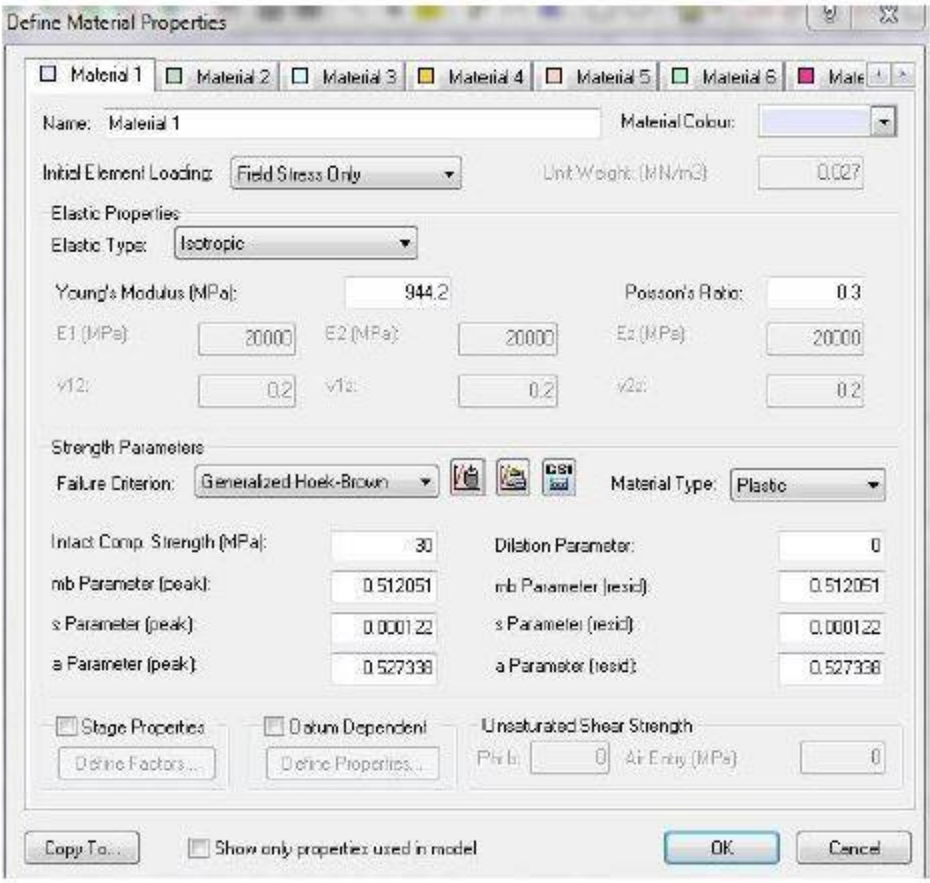


Figure 4.6: Material Properties

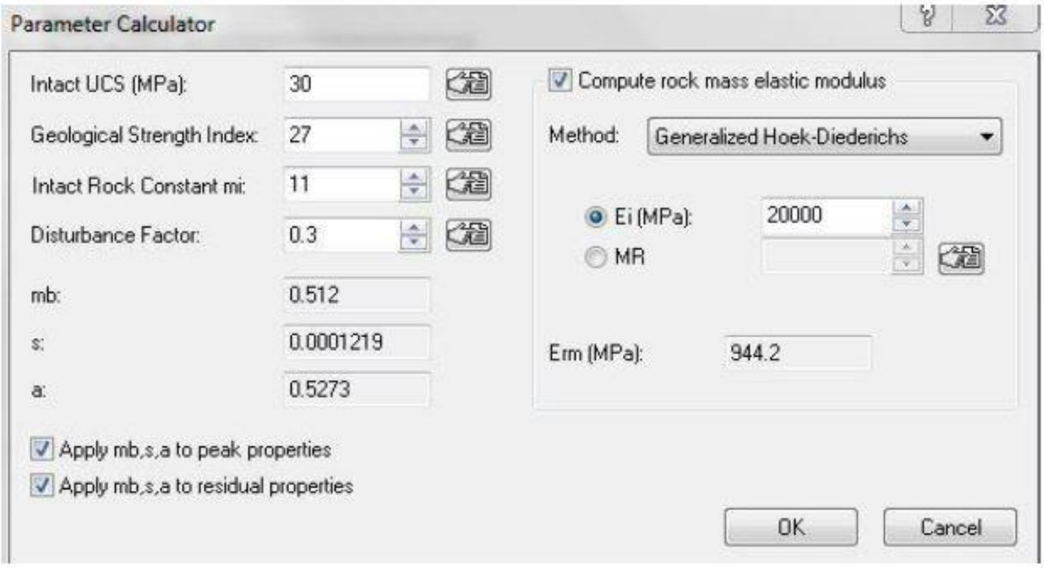


Figure 4.7: GSI and disturbance factor

4.4.3 ANALYSIS

After the properties are defined the model is analysed to get the deformation. Figure 4.8 and 4.9 shows analysis of the model.

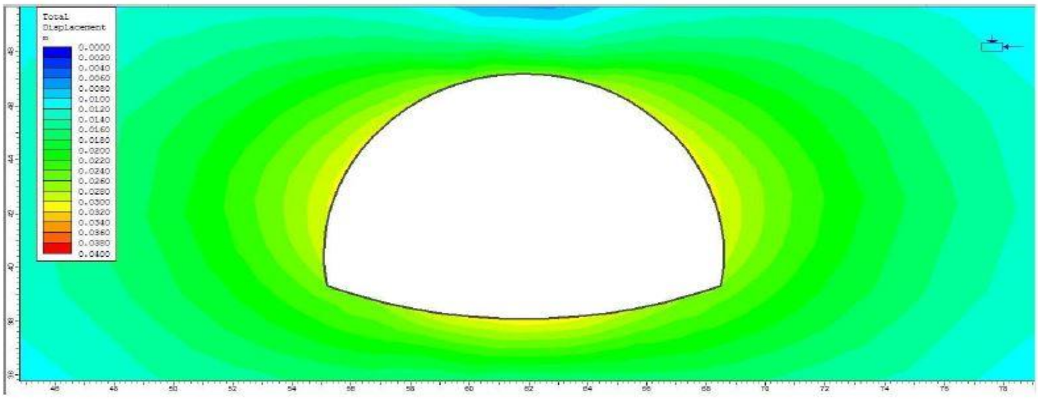


Figure 4.8: Deformation analysis of model

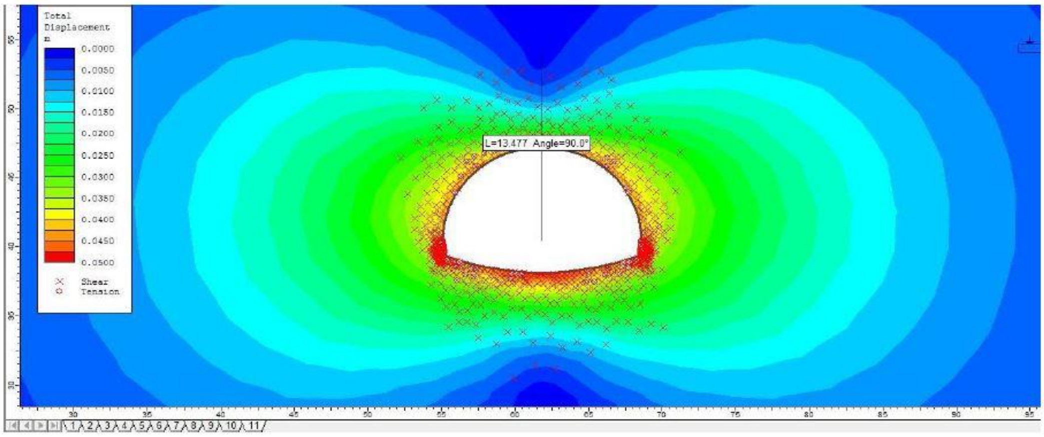


Figure 4.9: Yielded elements around the tunnel

4.4.4 DISPLACEMENT IN THE TUNNEL RELAXATION PHASE

To determine the maximum displacement before the installation of supports the following calculation needs to be done:

- Plastic radius,R_p/Tunnel radius,r_o=2.03
- Maximum deformation without support= 0.048
- Distance from tunnel face/Tunnel radius,r_o=0.3

From these calculations and the graph (Vlachopoulos and Diederichs) shown in Figure 4.10

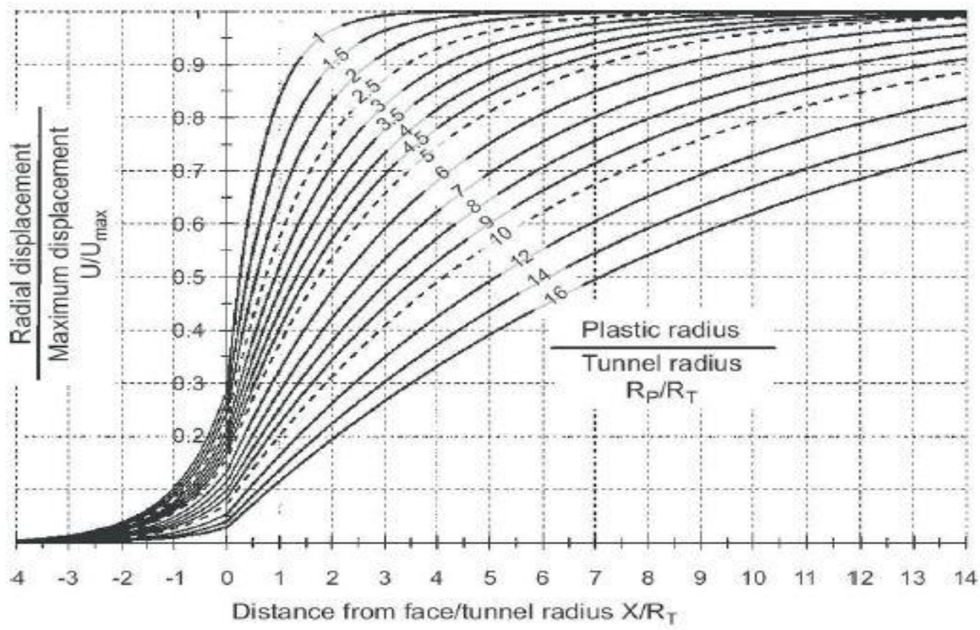


Figure 4.10: Relationship to estimate radial displacement

From the graph, radial displacement/maximum displacement = 0.475.

So, displacement before installation of support = 0.475 x 0.048 = 0.022m.

Using these calculations and the graph between stage number and displacements, Figure 4.11, the load split factor at the time of installation of supports can be calculated.

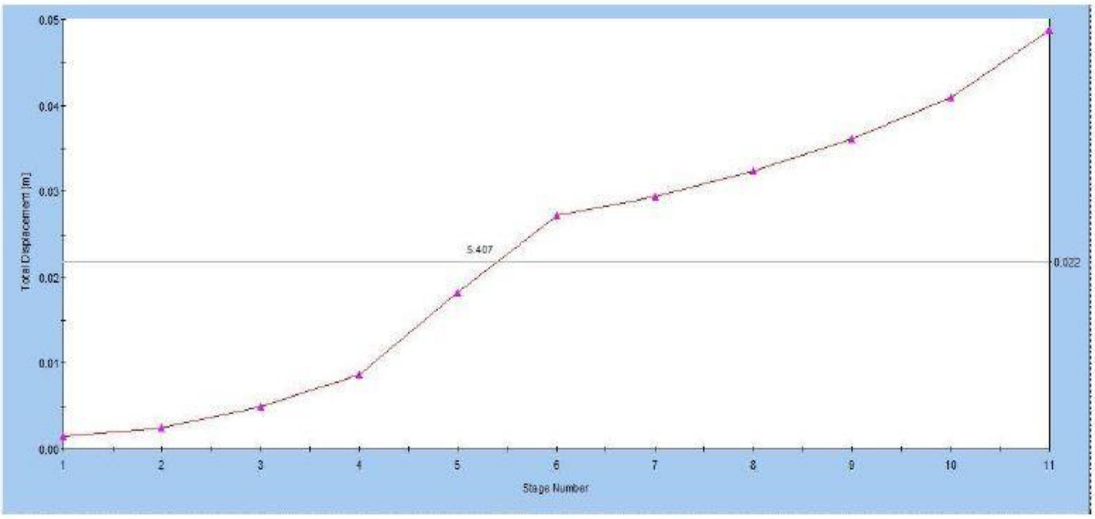


Figure 4.11: Displacement vs load stage

The stage number for this displacement comes out to be 5.407 and from interpolation of load split factor for different stages the load split for this stage comes out to be 0.15. The stages are again defined as shown in Figure 4.12.

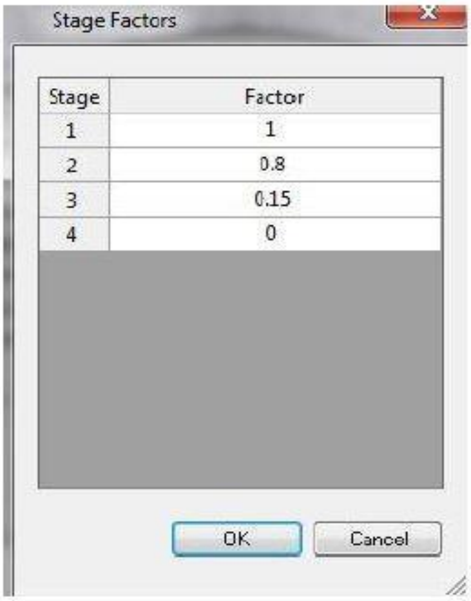


Figure 4.12: Load split factor for maximum displacement after support installation

4.4.5 SUPPORT PROPERTIES AND SUPPORT INSTALLATION

After the maximum displacement has been found the supports are installed. First the rock bolts and then the liner are installed. The properties of the supports are the same as those used in the field for Nahaki pass and Swat twin tube tunnel. For the Shimla tunnel the support system used are those as recommended by the Q-System. Figure 4.12 to Figure 4.14 show the installation of support in the model.

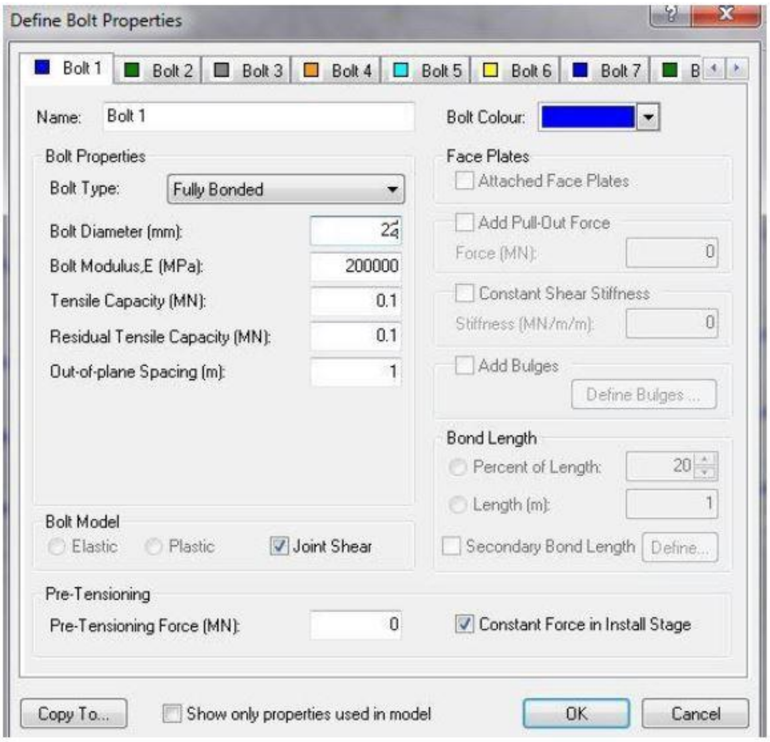


Figure 4.13: Rock bolt properties

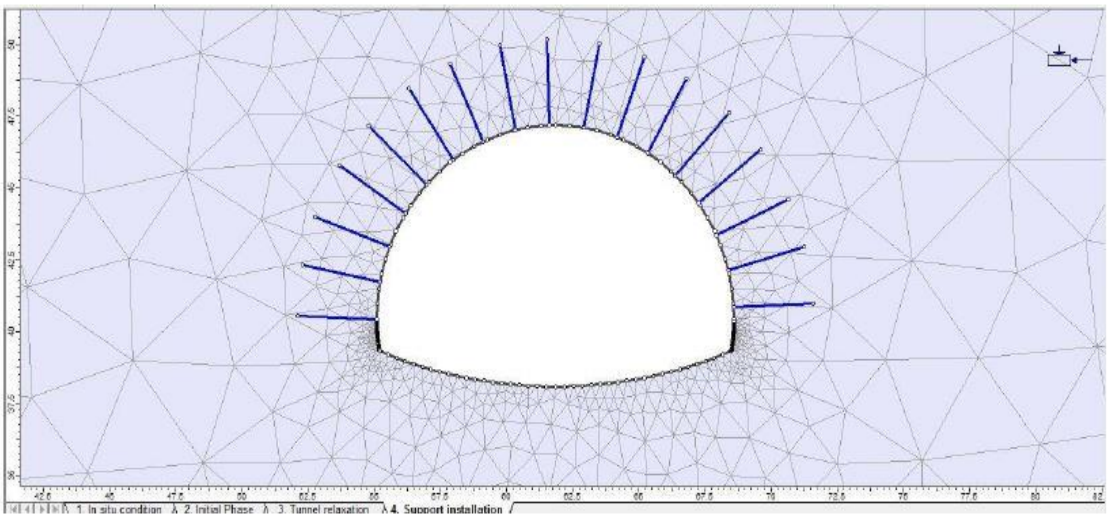


Figure 4.14: Installation of rock bolts

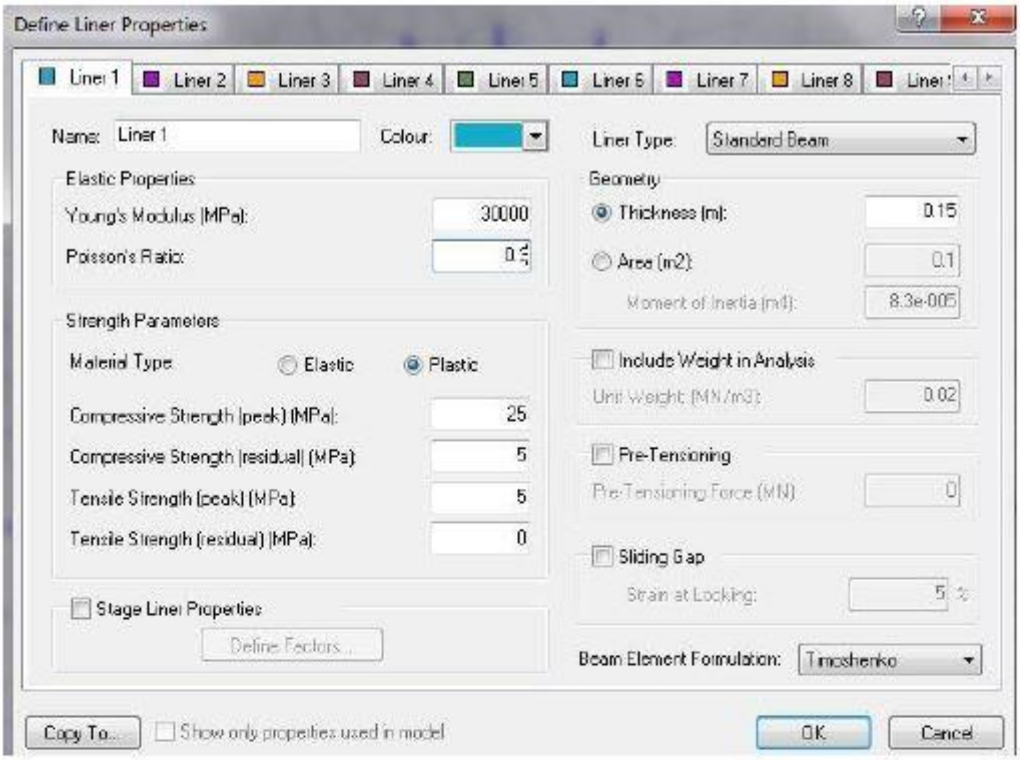


Figure 4.15: Liner properties

4.4.6 ANALYSIS OF MODEL AFTER SUPPORT INSTALLATION

The model is again analysed for deformation after support installation as shown in figure 4.15. The maximum displacement now comes out to be 0.028m against 0.048m before the support installation. The maximum displacement at the crown comes out to be 0.018m.

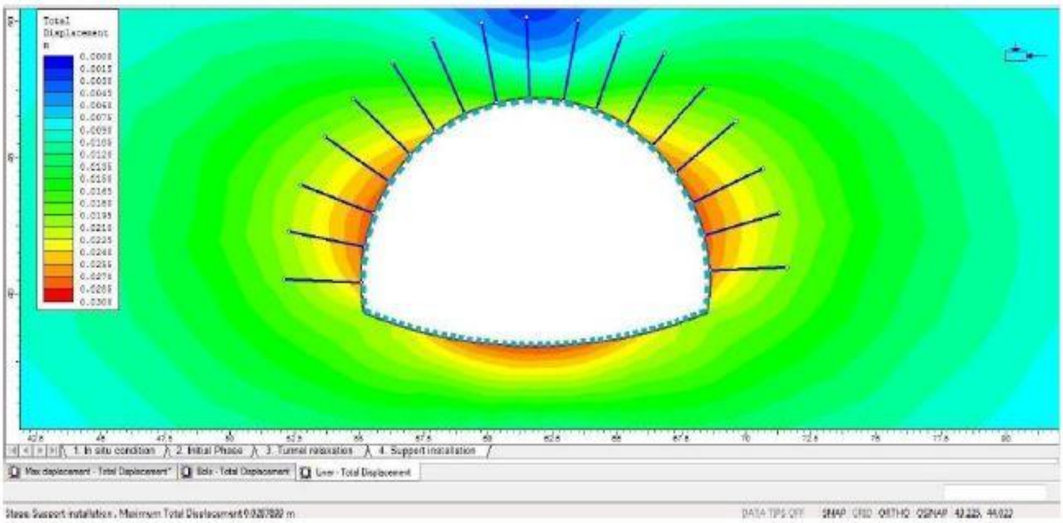


Figure 4.16: Deformation analysis after support installation

4.5 DEFORMATION ANALYSIS OF EACH SECTION

Based on the sample FEM model and the calculations shown in the previously, every section of the selected tunnels were analyzed.

4.5.1 NAHAKI PASS TUNNEL

The deformations of each section of Nahakki pass tunnel are summarized in Table 4.7.

Table 4.7: Deformation of sections (Nahakki tunnel)

SECTION	FEM (m)	MOHR-COULOMB (m)	HOEK-BROWN (m)	LADE-DUNCAN (m)
Entrance section	0.0220	0.0170	0.00750	0.0060
Middle section-1	0.0048	0.0038	0.0113	0.0035
Middle section-2	0.0135	0.0137	0.0070	0.0035
Middle section-3	0.0135	0.0136	0.0325	0.0087
Exit section	0.0500	0.1100	0.0370	0.0077

4.5.2 SHIMLA TUNNEL, ABBOTTABAD

The deformations of each section of Shimla tunnel are summarized in Table 4.8.

Table 4.8: Deformation of sections (Shimla tunnel)

SECTION	FEM (m)	MOHR-COULOMB (m)	HOEK-BROWN (m)	LADE-DUNCAN (m)
Section 1	0.0280	0.0217	0.0133	0.0450
Section 2	0.0180	0.0036	0.0161	0.0345
Section 3	0.6000	0.1370	0.0283	0.0541
Section 4	0.0195	0.0550	0.0419	0.0549

The deformations of each section of Swat tunnel are summarized in Table 4.9.

Table 4. 9: Deformation of sections (Swat tunnel)

SECTION	FEM (m)	MOHR-COULOMB (m)	HOEK-BROWN (m)	LADE-DUNCAN (m)
Section 1	0.0058	0.0070	0.0078	0.0094
Section 2	0.0130	0.0100	0.0080	0.0100
Section 3	0.0160	0.0090	0.0110	0.0090
Section 4	0.0070	0.0070	0.0030	0.0070

4.6 VARIATION OF EMPIRICAL APPROACHES

The variations of the deformations calculated by the empirical approaches (Mohr-Coulomb, Hoek-Brown and Lade-Duncan) from the deformations obtained by PHASE2 were recorded. The deformations obtained by the PHASE2 analysis are used as a benchmark because of the following reasons:

- Deformations are not observed in the field
- FEM being the latest technique has more consistent and reliable results

4.6.1 NAHAKI PASS TUNNEL

The following figures show the variation between the deformations obtained by different analytical techniques for all the sections of the Nahaki pass tunnel. Figure 4.16 shows the variation between different analytical approaches when the sections are arranged in order of increasing overburden.

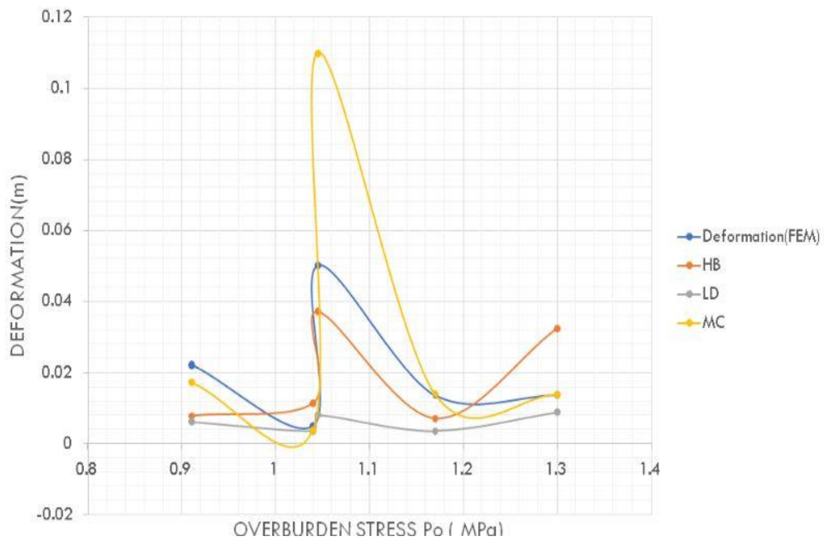


Figure 4.17: Comparison between approaches (Nahaki pass tunnel)

Figure 4.17 shows the variation of the empirical approaches (Mohr-Coulomb, Hoek-Brown and Lade-Duncan) from the deformations obtained by PHASE2 in the form of percentage.

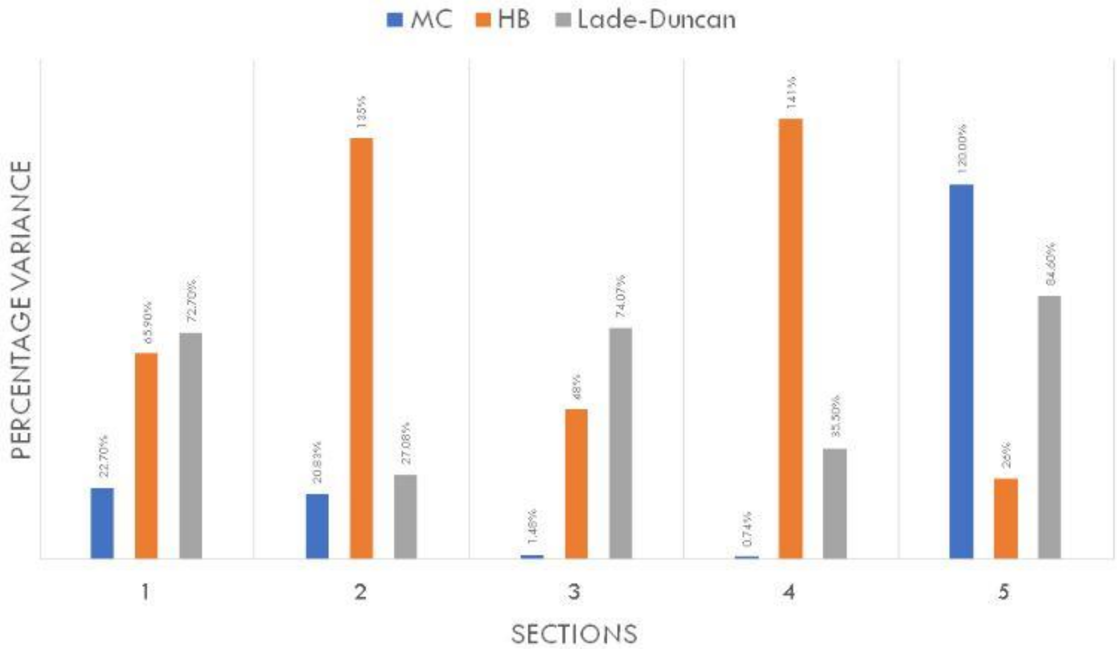


Figure 4.18: Percentage variation from FEM results (Nahaki pass tunnel)

4.6.2 SHIMLA TUNNEL, ABBOTTABAD

The following figures show the variation between the deformations obtained by different analytical techniques for all the sections of the Shimla tunnel. Figure 4.18 shows the variation between different analytical approaches when the sections are arranged in order of increasing overburden.

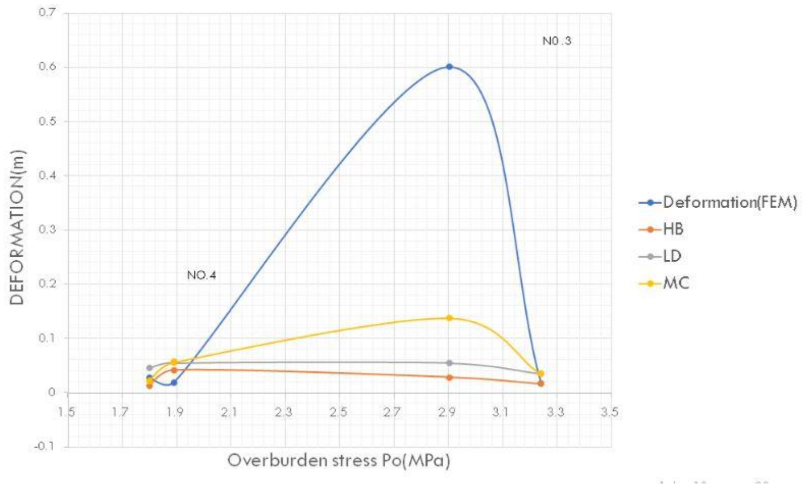


Figure 4.19: Comparison between approaches (Shimla tunnel)

Figure 4.19 shows the variation of the empirical approaches (Mohr-Coulomb, Hoek-Brown and Lade-Duncan) from the deformations obtained by PHASE2 in the form of percentage.

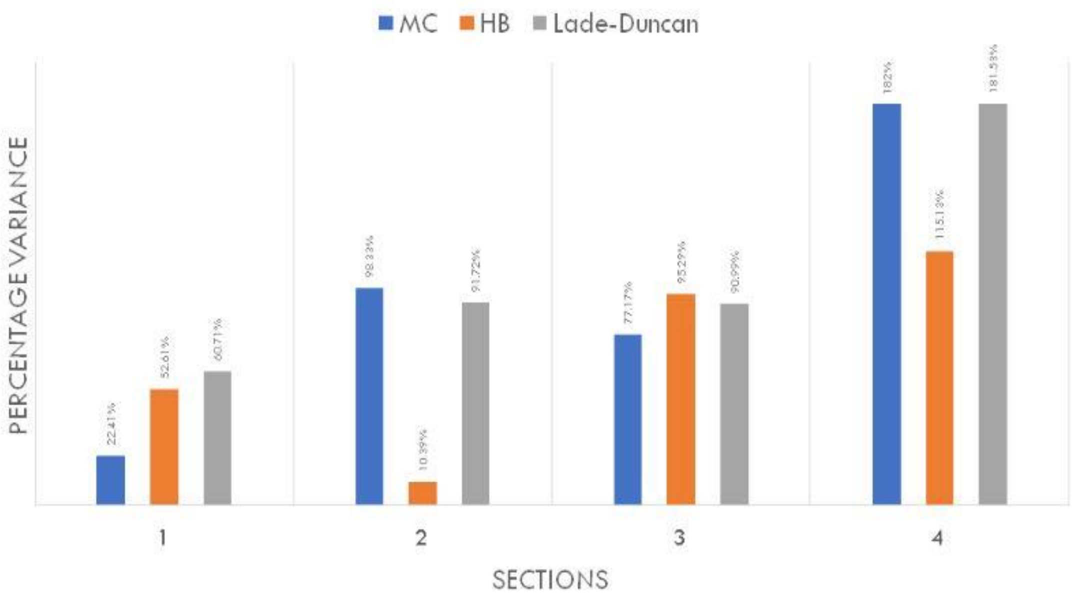


Figure 4.20: Percentage variation from FEM results (Shimla tunnel)

4.6.3 SWAT TWIN TUBE TUNNEL

The following figures show the variation between the deformations obtained by different analytical techniques for all the sections of the Shimla tunnel. Figure 4.20 shows the variation between different analytical approaches when the sections are arranged in order of increasing overburden.

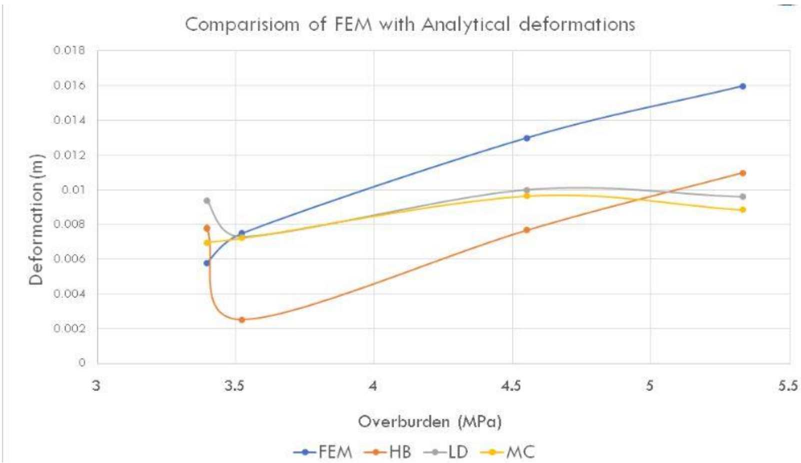


Figure 4.21: Comparison between approaches (Swat tunnel)

Figure 4.21 shows the variation of the empirical approaches (Mohr-Coulomb, Hoek-Brown and Lade-Duncan) from the deformations obtained by PHASE2 in the form of percentage.

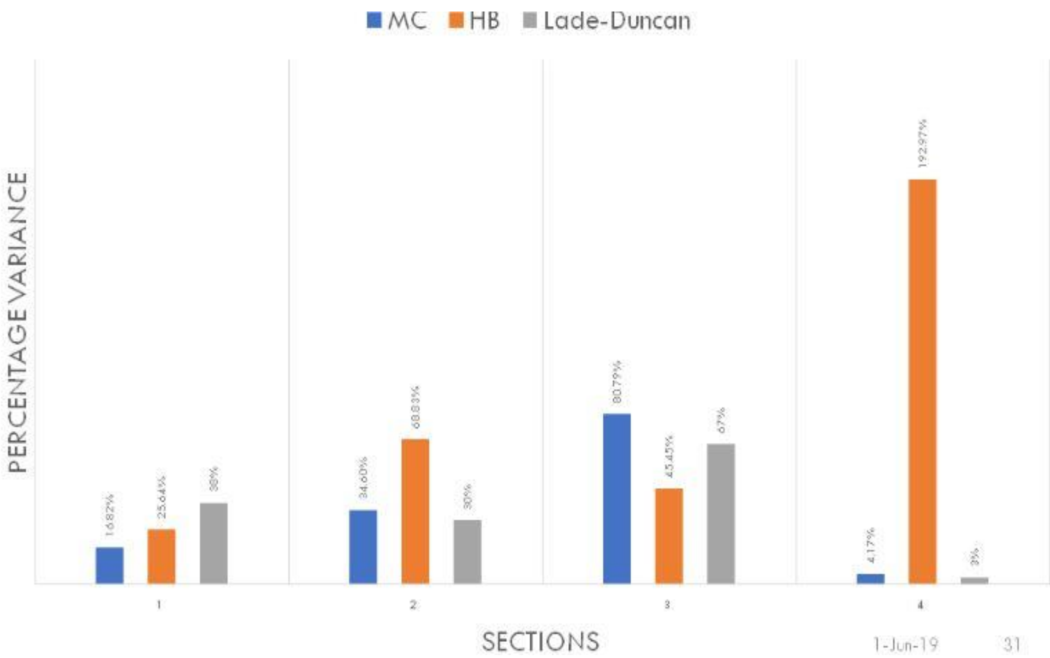


Figure 4.22: Percentage variation from FEM results (Swat tunnel)

4.7 CORRELATION USING NUERAL NETWORK

Given the massive variation of empirical approaches from the FEM benchmark it can be concluded that the approaches are not consistent and there is no one best approach to approximate the deformations.

To deal with this problem it was decided to use neural network to correlate parameters common in all the empirical approaches to come up with a single equation that can yield reasonable deformation results.

A neural network has a number of artificial neurons called units arranged in a series of layers. The three main layers are:

- The input layer which takes in the data
- The hidden layer which processes the data
- The output layer which generates the desired output of the model

The model is trained using a set of data, in training the inputs are assigned a certain weight the hidden layer then processes the weighted input and output is produced. The difference between the original output and the output of the model is the error. The error is back propagated to adjust the weights.

The model is then tested using another set of actual data to gauge its working and further improve the performance. Figure 4.22 shows the layers of the neural network.

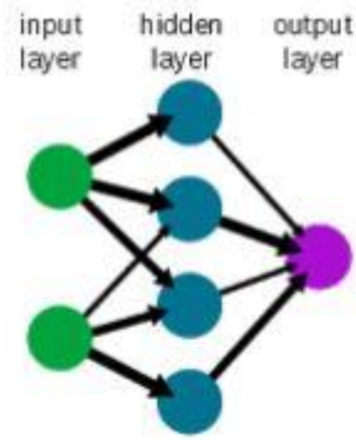


Figure 4.23: Layers of neural network

Python programming language along with scikit-learn libraries for machine learning were used in our project. The neural network was based on linear regression. The parameters selected were:

- Radius of tunnel, r_o
- Young's modulus of rock mass, E_m
- Poisson's ratio, ϑ
- Plastic Radius, R_p
- Overburden pressure, p_o
- Internal Support Pressure, p_i

The result of the model was a linear equation that correlates these parameters. The equation is as follows:

$$u_p = 6.28 \times 10^{-4} R_p + 3.51 \times 10^{-2} r_o + 5.36 \times 10^{-6}, E_m - 7.53 \times 10^{-1} \vartheta - 10.32 \times 10^{-4} p_o - 8.53 \times 10^{-3} p_i \qquad (9)$$

Figure 4.23 shows the results of (9) and the results from PHASE2 side by side.

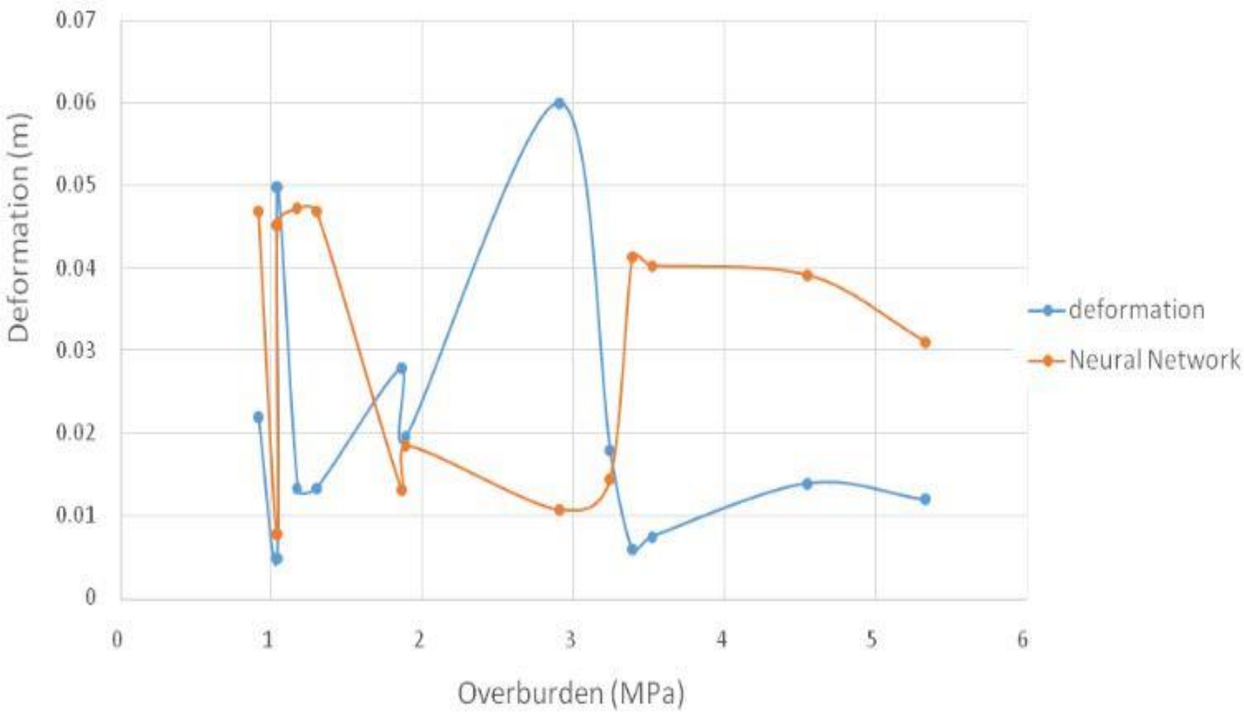


Figure 4.24: Neural network V/S FEM results

The results have been arranged in order of increasing overburden and have the results of all the sections of the selected tunnels. The results are acceptable to an overburden of 1.8 MPa beyond this the results show great variation. The reason for the variation is lack of data points to train the neural network model.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 REVIEW

The objectives of this project were the following:

- Analysis of crown deformation of selected tunnels using:
    1) Empirical approaches
    2) Finite Element Modelling
- Comparison between these approaches.
- Developing an equation using Neural Networks to estimate the crown deformation which is consistent with the FEM benchmark.

5.2 CONCLUSIONS

The following conclusions were made from this project:

- Empirical results show similar trend to FEM results, with values generally lower than FEM results.
- The possible reasons for variation in data are :
    1) The analytical methods don’t take into account stages of excavation.
    2) The geometry is supposed to be circular whereas in model it is horse shoe.
- Though following the same trend, the empirical approaches show inconsistent variation from the FEM benchmark.
- The neural network results show consistency till 1.8 Mpa. This is because the data set is concentrated in this region and thus better results are achieved.
- Machine learning is a continuous process and is improved continuously with the provision of more data.
- The model shows that if data is extensive it can be versatile enough to work in a number of environments.

5.3 RECOMMENDATIONS

Based on the conclusions following recommendations were made:

- Field observation of deformations should be recommended for future projects so that the neural network model is trained on real life data which in turn will result in more accurate deformation predictions.
- Monitoring of deformation should be done extensively in a number of different locations as to provide a comprehensive data set for the training of the neural network model. Larger data set results in better training of the model which in turn results in better prediction of deformation.
- It is further recommended that a co-relation between displacements of circular tunnel and horse shoe tunnel is developed so that displacement on the crown, walls and invert can be easily estimated.

REFERENCES

Barton, N.R., Lien, R. and Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. Rock Mech. 6(4), 189-239.

Bieniawski, Z.T. 1973. Engineering classification of jointed rock masses. Trans S. Afr. Inst. Civ. Engrs 15, 335-344.

Bieniawski, Z.T. 1976. Rock mass classification in rock engineering. In Exploration for rock engineering, proc. of the symp., (ed. Z.T. Bieniawski) 1, 97-106. Cape Town: Balkema.

Bieniawski, Z.T. 1989. Engineering rock mass classifications. New York: Wiley

Deere, D.U., Hendron, A.J., Patton, F.D. and Cording, E.J. 1967. Design of surface and near surface construction in rock. In Failure and breakage of rock, proc. 8th U.S. symp. rock mech., (ed. C. Fairhurst), 237-302. New York: Soc. Min. Engrs, Am. Inst. Min. Metall. Petrolm Engrs.

Hoek, E., 1999. Support for very weak rocks associated for with faults and shear zones. In: Proceedings of International Symposium on Rock Support and Reinforcement Practice in Mining, Kalgoorlie, Australia, March 14–19

Lippmann, R.P., 1987. An introduction to computing with neural nets. IEEE Accost. Speech Signal Process. Mag., April: 4-22.

MingZheng Zhu, Yugui Yang, Feng Gao, and Juan Liu, “Analytical Solution of Tunnel Surrounding Rock for Stress and Displacement Based on Lade–Duncan Criterion,” Advances in Civil Engineering, vol. 2018, Article ID 5363658, 7 pages, 2018.

P. K. Woodward, “Earth pressure coefficients based on the Lade–duncan failure criterion,” Engineering Structures , vol. 19, no. 9, pp. 733–737, 1997

Ravandi, Ebrahim & Rahmannejad, Reza. (2013). Wall displacement prediction of circular, D shaped and modified horseshoe tunnels in non-hydrostatic stress fields.

Tunnelling and Underground Space Technology. 34. 54–60. 10.1016/j.tust.2012.11.001.

Vlachopoulos, N., Diederichs, M.S., 2009. Improved Longitudinal Displacement Profiles for Convergence Confinement Analysis of Deep Tunnels. Rock Mech.& Rock Eng.. 42:2, 131-146.

X. Huang, J. Zhang, L. Yang, S. Yang, and X. Wang, "Elastoplastic analysis of the surrounding rock mass in circular tunnel based on the generalized nonlinear unified strength theory," International Journal of Mining Science and Technology, vol. 26, no. 5, pp. 819–823, 2016.