Kathmandu University

SCHOOL OF ENGINEERING

Department of Civil and Geomatics Engineering

Assessment and Numerical Modelling of Selected Hydropower Tunnels in Lesser Himalayas with Hydraulic Design

A Case Study of Middle Bhotekoshi HEP and Sanjen HEP

A Final Year Project

Submitted in Partial Fulfillment of the Requirements for the Degree of

Bachelor of Engineering in

Civil Engineering (with Specialization in Hydropower Engineering)

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October 2018

ACKNOWLEDGEMENT

It is our great pleasure to submit this report on "**Assessment and Numerical Modelling of Selected Hydropower Tunnels in Lesser Himalayas with Hydraulic Design**" to the Department of Civil and Geomatics Engineering, Kathmandu University. We would like to thank department for permitting us to carry out this project for the partial fulfillment of the requirement for the Bachelor's degree in Civil Engineering.

We would like to thank our Head of Department, **Assoc. Prof. Dr. Prachand Man Pradhan**, for providing us the opportunity to have professional experience through this final year project.

We express our earnest gratitude and deep respect to our supervisor, **Prof. Dr. Ramesh Kumar Maskey** and **Asst. Prof. Shyam Sundar Khadka**, for his constant availability, encouragements, his motivations towards us regarding the project, his valuable suggestions and sharing his ideas, and pushing us to our potential and making this project what it is now, without which the project would not have been completed.

We would also like thank to our external supervisor **Dr. Pawan Shrestha** for providing us feedback on our works and guiding us during the completion of our project.

We are very much thankful to **Sanjen Hydroelectric Project** and **Middle Bhotekoshi Hydroelectric Project** who assisted us by providing the required data for the completion of our project.

We are also thankful to all our faculties of Department of Civil & Geomatics Engineering, for their suggestions and resources required for the project.

Finally we would like to thank **University Grant Commission (UGC)** and **Faculty Research Grant (FRG) (2073/74)** award no. ***UGC, FRG-73/74-ENGG-03***.

EXECUTIVE SUMMARY

Tunnels in Nepal are not free from various stability problems like squeezing, water leakage, collapse etc because of various causes like weak rock mass quality, fracturing and weathering rock stress anisotropy and ground water effect. Squeezing problems in tunnel occurs due to high overburden and the stress applied. Stress plays a crucial role in developing brittle fractures, rock strength reduction and rock mass instabilities The critical stress is an indicator for the support design of tunnel. To quantify stress state and deformation of tunnel, an empirical, analytical and a two dimensional boundary element numerical investigation have been performed in this project. Finite element Modeling results highlight that the shape and size of the tunnel, existing geological structures and rock-mass parameters have significant influence to the induced stress field and rock deformation, which directly controls the stability of the underground excavation design. The following are the specific objectives of the project work.

- Assessment of squeezing phenomenon of underground structures in different rock mass type in Lesser Himalaya of Nepal using the Empirical, Semi-empirical, Analytical and Finite Element Numerical Modeling.
- Proper Design of support elements to address the squeezing problems in weak rock mass using Numerical Modelling.
- Comparison of Numerical Model with Empirical and Analytical approach This project is done based on extensive literature search supplemented by information from different sources and data from the concerned institutions.

Following summarizes methodology to execute this project:

1. Literature Review:
2. Data Collection
3. Stability Analysis
4. Numerical Investigation
5. Interpretation

Numerical Modeling of the underground structures of mentioned cases was done by six node triangular element using RocScience V 9.0 a finite element analysis software. The obtained results were analyzed and various conclusions were drawn regarding stability of each of the case studies with respect to the degree of rock deformation. This report compiles the case studies of underground structures of two different existing hydropower projects named Sanjen Hydroelectric Project (SHEP) and Middle Bhotekoshi Hydroelectric Project (MBKHEP).

CONTENTS

EXECUTIVE SUMMARY

LIST OF FIGURES i

LIST OF TABLES iv

CHAPTER 1 INTRODUCTION 1

1.1 General 1

1.2 Background 2

1.3 Objectives 3

1.4 Methodology 4

1.5 Scope and Limitations 6

1.5.1 Scope 6

1.5.2 Limitations 6

1.6 Organization of Chapters 7

CHAPTER 2 CONSTRUCTION PRACTISE OF TUNNELS IN NEPAL 8

2.1 General 8

2.2 Tunnels in Nepal 8

2.3 Methods of Tunneling 12

2.3.1 New Austrian Tunneling Method (NATM) 12

2.3.2 Norwegian Method of Tunneling (NMT) 15

2.4 Problem in Underground Structure in Nepal 16

2.5 Rock Support 18

2.5.1 Rock Bolts 19

2.5.2 Shotcrete 20

2.5.3 Grouting 21

2.5.4 Concrete Lining 22

2.5.5 Steel Sets 23

2.5.6 Fore Poling 23

2.6 Literature Review 24
2.6.1 Empirical Approach 24
2.6.2 Analytical Approach 26
2.6.3 Numerical Approach 29
2.7 Squeezing Assessment 30
2.7.1 Empirical Approach 31
2.7.2 Analytical Approach 48
CHAPTER 3 DESCRIPTION OF CASE STUDIES 53
3.1 General 53
3.2 Case Studies 53
3.2.1 Middle Bhotekoshi Hydroelectric Project (MBKHEP) 54
3.2.2 Sanjen Hydroelectric Project (SHEP) 87
3.2.3 Comparison of the results from Analytical and Empirical Approaches 106
CHAPTER 4 EXPERIMENTAL INVESTIGATION 109
4.1 General 109
4.2 Laboratory Testing 111
CHAPTER 5 NUMERICAL INVESTIGATION 116
5.1 General 116
5.2 Phase^2 Software 116
5.3 Numerical assessment using Phase^2 Software 118
5.4 Procedure of modeling a tunnel section 119
5.5 Case Studies 121
5.5.1 Middle Bhotekoshi Hydroelectric Project (MBKHEP) 121
5.5.2 Sanjen Hydroelectric Project 123
5.6 Result and Discussion 131
5.7 Modified Support 131
CHAPTER 6 DESIGN OF HYDROPOWER COMPONENT 142

6.1 Hydraulic Design 142
6.1.1 Intake 142
6.1.2 Approach Canal 145
6.1.3 Settling basin 146
6.1.4 Surge Tank 149
6.2 Optimization of Tunnel 150
6.2.1 Geometry of Tunnel 150
6.2.2 Support Condition 151
6.2.3 Unsupported Span 151
6.2.4 Calculation of head loss 152
6.2.5 Reinforcement of Invert concrete in HRT 154
6.2.6 Case Studies 154
CHAPTER 7 CONCLUSION 160
REFERENCES 161
APPENDIX A 164
A.1 Rates for Optimization 165
A.2 Relation of Area and Perimeter with the Diameter of Tunnel 167
APPENDIX B 170
B.1 Design of Side Intake 171
B.2 Design of Trash rack 172
B.3 Design of approach Canal 173
B.4 Design of Settling Basin 174
B.5 Design of Surge Tank 177
APPENDIX C 180

LIST OF FIGURES

Figure 1-1 Work Methodology 5
Figure 2-1 Location of Tunnels in Nepal (Constructed and Under-construction) along with its density in different province 10
Figure 2-2 Map showing Density of (a) Constructed and (b) Under construction Tunnels according to Provience of Nepal 11
Figure 2-3 The effect of support installation w.r.t time and stiffness. (Singh et al., 2007) 13
Figure 2-4 .Showing the use of invert lining and comparative thickness of modern (Left) and Traditional (Right) (after Singh et al., 2006) 14
Figure 2-5 Heading and Benching (Left) and Multiple Drift excavation (Right) (After Singh et al., 2006) 15
Figure 2-6 Types of Rock Support 18
Figure 2-7 Rock Bolts used in SHEP 20
Figure 2-8 Shotcrete and Grouting in UT-3A 22
Figure 2-9 Forepoling and installation of Steel sets in SHEP 24
Figure 2-10 a) Cylindrical tunnel of radius R driven in the rock mass. b) Cross-section of the rock mass at the section A-A'. c) Cross-Section of the circular support installed at section A-A' (after Carranza-Torres and Fairhurst, 2000) 27
Figure 2-11 Log-Log plot showing the tunnel depth (in meters) versus the rock mass quality Q for prediction of squeezing ground. (Singh et al., 1992) 31
Figure 2-12 Log-Log Plot for prediction of squeezing criteria by Goel et al. (1995) 33
Figure 2-13 Q-Support Chart (Norwegian Geotechnical Institute, 2013) 35
Figure 2-14 Plot of support pressure versus rock mass rating (Q) (after Singh and Goel, 2011) 37
Figure 2-15 Plot of Q/span or Q/Height versus Deformation (mm) (Taken from (Singh B., Goel R.K, 2011)) (after Barton, 2008) 38
Figure 2-16 Plot for correction factor for tunnel closure (Goel et al. (1995)) 39
Figure 2-17 Plot between Span (m) and RMR for estimating the Rock Bolt length by Lowson and Bieniawski (2013) (after Bieiawski, 2013) 42
Figure 2-18 Graph for obtaining thickness of shotcrete by Lowson and Bieiawski (2013) (after Bieiawski, 2013) 42
Figure 2-19 Curve showing the plot between strain and σcm/P where the tunnel is not supported 45

Figure 2-20 Plot of Tunnel strain versus in-situ stress and shear modulus of rock mass for different support pressure. (Shrestha and Panthi, 2018) 46
Figure 2-21 Rock Bolting (Left) and Shotcrete (Right) (Source: Rocscience) 48
Figure 2-22 Sample of Excel file prepared for CCM 49
Figure 2-23 Sample of Deformation Profile of MBKHEP at Section 2886.6m 51
Figure 2-24 Sample of plot of CCM from section at 2886.6m of MBKHEP 52
Figure 3-1 Location of Project Sites 53
Figure 3-2 Joint Rosette Diagram of Upstream Part of Headrace Tunnel (Left) and of Adit Area (right) 56
Figure 3-3 Plan of Middle Bhotekoshi Hydroelectric Project site (after MBKHEP feasibility Report) 57
Figure 3-4 Squeezing Assessment of Headrace Tunnel of MBKHEP 60
Figure 3-5 Squeezing Assessment of Headrace Tunnel of MBKHEP 61
Figure 3-6 Plot between Convergence (mm), Overburden (m) and Chainage (m) 67
Figure 3-7 Plot between Support pressure and Chainage for different Support system 68
Figure 3-8 Deformation for Support Pressure with F.O.S=1 70
Figure 3-9 Deformation for Support Pressure with F.O.S=1.5 70
Figure 3-10 Deformation for Support Pressure with F.O.S=2 71
Figure 3-11 Deformation for Support Pressure with F.O.S =1 71
Figure 3-12 Deformation for Support Pressure with F.O.S=1.5 72
Figure 3-13 Deformation for Support Pressure with F.O.S=2 72
Figure 3-14 Plot of Deformation for Design support using Shrestha and Panthi (2015) and Hoek and Marinos (2000) 73
Figure 3-15 Plot of Deformation for Q- Support classification using Shrestha and Panthi. (2015) and Hoek and Marinos (2000) 74
Figure 3-16 Plot of Deformation for RMR- Support classification using Shrestha and Panthi. (2015) and Hoek and Marinos (2000) 75
Figure 3-17 Plot of CCM for MBKHEP at Section (a) 517m, (b)2886.6m, (c)3190m and (d) 3283.2m based on Observed Condition 81
Figure 3-18 Plot of CCM for MBKHEP at Section (a) 517m, (b)2886.6m, (c)3190m and (d) 3283.2m based on RMR System 82
Figure 3-19 Plot of CCM for MBKHEP at Section (a) 517m, (b)2886.6m, (c)3190m and (d) 3283.2m based on Q System 83
Figure 3-20 Comparison of Sections with known deformations for MBKHEP 85

Figure 3-21 Comparison of Support Pressure at different sections of MBKHEP 85
Figure 3-22 Comparison of closure for MBKHEP 86
Figure 3-23 Squeezing Analysis using Singh et al. (1992).....91
Figure 3-24 Squeezing Analysis Graph using Geol et al. (1994) 92
Figure 3-25 Squeezing Analysis using Jimenez et al. (2011).....92
Figure 3-26 Plot Showing relation between different parameters for underground opening of SHEP.....98
Figure 3-27 Plot of CCM for SHEP at Sections (a) 1563.23m, (b) 1643.084, (c) 1759.88, (d)1938.46 for Observed Condition.....102
Figure 3-28 Plot of CCM for SHEP at Sections (a) 1563.23m, (b) 1643.084, (c) 1759.88, (d)1938.46 for Q-System 103
Figure 3-29 Comparison of Support Pressure from different approaches 104
Figure 3-30 Comparison of Closure for SHEP 106
Figure 3-31 Comparative graph for Observed Condition 107
Figure 3-32 Comparative graph for Q-System 108
Figure 4-1 Schematic Test Setup- Direct Shear Box with Encapsulated Specimen.....109
Figure 4-2 Generalized Shear Stress and Shear Displacement Curve 110
Figure 4-3 Plot between Shear Strength and Displacement for different Normal Loads 113
Figure 4-4 Shear Strength vs. Normal Load 114
Figure 4-5 Direct Shear Test Machine.....115
Figure 4-6 Rock Sample after placing in the Mould.....115
Figure 5-1 Schematic Diagram Showing the process of Analysis.....118
Figure 5-2 Support Capacity Plot for Concrete Lining (designed) for SHEP at chainage 1864.122m 125
Figure 5-3 Support Capacity Plot for Shotcrete (designed) for SHEP at chainage 2094.6m 126
Figure 5-4 Support Capacity Plot for Shotcrete (designed) for MBKHEP at chainage 500m 127
Figure 5-5 Support Capacity Plot for Shotcrete (designed) for MBKHEP at chainage 6795.5m 128
Figure 5-6 Insufficient support for chainage 1864.122m of SHEP 129
Figure 5-7 Plot of Bending moment with points of failure for section 1864.122m of SHEP130
Figure 5-8 Schematic Diagram of Modified Section for MBKHEP.....132
Figure 5-9 Schematic Diagram of Modified Section for SHEP 133
Figure 5-10 Support capacity plot for Modified Support for SHEP at chainage 1864.122m 134

Figure 5-11 Support capacity plot for Modified Support for SHEP at chainage 2094.6m....135
Figure 5-12 Support capacity plot for Modified Support for MBKHEP at chainage 500m..136
Figure 5-13 Support capacity plot for Modified Support for MBKHEP at chainage 6795.5m 137
Figure 5-14 Plot of GRC obtained from Numerical Modelling for MBHEP at sections (a) 300m, (b) 400m, (c) 500m and (d) 6795.5m.....139
Figure 5-15 Plot of GRC obtained from Numerical Modelling for SHEP at sections (a) 1864.22m, (b) 2094m, (c) 2137.7m 141
Figure 6-1 Coefficient of Bend for 90 degree bend (Panthi K., Basnet C., 2018).....153
Figure 6-2 Reduction Factor (Panthi K., Basnet C., 2018).....153
Figure 6-3 Optimization Chart.....156
Figure 6-4 Optimization Chart for MBKHEP 159

LIST OF TABLES

Table 2-1 Length of tunnels according to province of Nepal 12
Table 2-2 Showing Features of NMT (After Singh et al., 2006).....16
Table 2-3 Criteria of ground condition using Rock Mass Number (N) (Singh and Goel, 2011) 32
Table 2-4 Table of ESR (after Singh and Goel, 2011) 35
Table 2-5 Wall factor for respective Q-value (after Singh and Goel, 2011) 36
Table 2-6 Table for correction factor (f2) for tunnel closure (Singh et al., 1992).....38
Table 2-7 Correction factor for tunnel closure (f(N)) (Geol et al. (1995)).....40
Table 2-8 Table showing the classification of support for 10m Horseshoe shape tunnel with drill and blast method. (Singh and Goel, 2011).....41
Table 2-9 Correction factor for tunnel closure f(RMR) (Goel et al., 1995) 44
Table 2-10 Parameters used for Calculation of CCM.....50
Table 3-1 Distribution of Rock Types along Headrace Tunnel and Audit (after MBKHEP Feasibility Report, n.d.) 55
Table 3-2 Rock Mass Classification as observed in MBKHEP HRT.....55
Table 3-3 Showing Orientation of the Joint sets found in MBKHEP HRT.....56
Table 3-4 Different Parameters used for Support Design.....58
Table 3-5 Range of the support Category.....58

Table 3-6 Description of Support system with respective Support Class (after Barton et al., 1993) 58
Table 3-7 Rock Support System Used in MBKHEP Head Race Tunnel 59
Table 3-8 Description of Support system after RMR-Support Guidelines 60
Table 3-9 Squeezing Assessment of MBKHEP Sections 61
Table 3-10 Calculation of Support Pressure for tunnel Section of MBKHEP 62
Table 3-11 Different Parameters for six study section of MBKHEP 64
Table 3-12 Assessment of squeezing ground condition for six study sections 64
Table 3-13 Calculation of Tunnel closure of Designed support 65
Table 3-14 Calculation of Tunnel closure of Defined support using RMR Support Classification Guidelines 65
Table 3-15 Calculation of Tunnel closure of Defined support using Q-Support Chart Guidelines 66
Table 3-16 Calculation of Stiffness and Maximum support pressure 66
Table 3-17 Calculation of Convergence value using Hoeks and Marino (2000) 69
Table 3-18 Calculation of Convergence value using Shrestha et al. (2015) 69
Table 3-19 Value of Parameters used while construction of CCM for MBKHEP (for section at 2886.6) 78
Table 3-20 Calculation of LDP for MBKHEP (for section at 2886.6) 79
Table 3-21 Calculation of GRC for MBKHEP (for section at 2886.6) 79
Table 3-22 Calculation of SCC for MBKHEP (for section at 2886.6) 80
Table 3-23 Comparative Table for Support Pressure obtained from Observed, RMR and Q-System for MBKHEP 84
Table 3-24 Tunnel Closure with and without support for MBKHEP 86
Table 3-25 Distribution of Rock Types along Headrace Tunnel (edited after SHEP Feasibility Report, 2011) 88
Table 3-26 Different Parameters used for Support Design 89
Table 3-27 Range of the support Category 89
Table 3-28 Support Description using Q-Support Chart (NGI (2013)) (after Bashin and Grimstad, 1993) 89
Table 3-29 Support Description of rock support provided in HRT of SHEP 90
Table 3-30 Squeezing Assessment and Rock Burst Condition 93
Table 3-31 Calculation of Support Pressure using Empirical Relations 95
Table 3-32 Calculation of Deformation using Barton (2008) 96

Table 3-33 Value of Parameters used while construction of CCM for SHEP (for section at 1643.084) 99
Table 3-34 Calculation of LDP for SHEP (for section at 1643.084) 100
Table 3-35 Calculation of GRC for SHEP (for section at 1643.084) 101
Table 3-36 Calculation of SCC for SHEP (for section at 1643.084) 101
Table 3-37 Comparative Table for Support Pressure obtained from Observed and Q-System for SHEP 104
Table 3-38 Tunnel Closure with and without support for SHEP 105
Table 3-39 Comparative table for Support Pressure for Observed Condition 106
Table 3-40 Comparative table for Support Pressure for Q-System 107
Table 4-1 Initial Reading for Peak and Residual Shear Stress 111
Table 4-2 Observation table for Normal Load 112
Table 4-3 Observation Table for Shear Load 112
Table 4-4 Peak and Residual Shear stress for different Normal Loads 113
Table 4-5 Values obtained from Direct Shear Test 114
Table 5-1 Parameters used for Numerical Modelling of Sections for MBKHEP 121
Table 5-2 Measurement of Maximum Deformations of Sections for MBKHEP 122
Table 5-3 Parameter Used in Numerical Modeling of sections for SHEP 123
Table 5-4 Deformation measured in the tunnel of SHEP 123
Table 5-5 Maximum Deformation after Support Modification from Modelling for MBKHEP 138
Table 5-6 Deformation after support modification from Modelling for Tunnel of SHEP 138
Table 5-7 Support Pressure of Modified section from Numerical Modelling of MBKHEP 141
Table 5-8 Support Pressure of Modified section from Numerical Modelling of SHEP 141
Table 6-1 Rock Type along the chainage with its Q-Value (after SHEP Feasibilty Report) 154
Table 6-2 Total Construction Cost for SHEP 155
Table 6-3 Head Loss Cost for SHEP 155
Table 6-4 Parameters for Optimization for SHEP 156
Table 6-5 Geological Instigation Data for MBKHEP (after MBKHEP Feasibility Report) 157
Table 6-6 Total Construction Cost for MBKHEP 157
Table 6-7 Head Loss Cost for MBKHEP 158
Table 6-8 Parameters for Optimization for MBKHEP 158

CHAPTER 1 INTRODUCTION

1.1 General

Nepal is a landlocked country with wide variation in its topography. It is located in the central part of the 2500 km long Himalayan arc and covers one third of its length. Geographically, major part of Nepal (83%) falls within the mountainous region and remaining 17% is covered by the alluvial plains of the Gangetic basin. Due to this large variation in topography of Nepal, tunnels provide an economic passage for (a) water conveying, (b) transport, (c) mining and (d) food storage facilities (Panthi, 2004). But the construction and development of tunnel has been a major challenge in Nepal. The presence of major faults like Main Central Thrust (MCT), Main Boundary Thrust (MBT), and Himalayan Frontal Thrust (HFT), or Main Frontal Thrust (MFT) has added to the problem in tunneling and underground construction. Also, the Himalayan region is located at the two-plate boundary: Indian and Eurasian Tectonic plates, therefore Himalayan region is considered seismically active zone.

Proper support system must thus be installed for safe construction of underground structures against all these geological challenges. So accuracy should be maintained from the start of geological investigation, as the results of the investigation plays a crucial role in selection of the tunnel alignment. However, the past tunneling experience in Nepal shows that the accuracy of planning phase geological investigations for underground works has often been rather poor (Panthi, 2008). Due to this negligence in pre-investigation several cases of tunnel failures have been reported over time. For the determination of proper support requirements it is important to identify the major causes for geological uncertainties and failure. According to Panthi (2008) the geological characteristics that have caused major stability problems during tunneling in Nepal are:

a. Weak rock mass quality,
b. High degree of weathering and fracturing,
c. Rock stresses, and
d. Groundwater effect.

These characteristics are identified by strength anisotropy (caused by preferred orientations of mineral grains), immediate tunnel collapse during excavation and leakage of water, stress anisotropy, water leakage (due to high permeability of Himalayan Rocks) respectively.

The support installed must be enough to withstand the pressures of the earth and prevent excessive deformations within the structures. Since underground construction is cost consuming,

optimization must be done to determine the adequate support and prevent over estimation of supports. Keeping this under consideration, this project focuses on assessment of the installed support on two tunnels in Nepal: Sanjen Hydroelectric Project (SHEP) and Middle Bhotekoshi Hydroelectric Project (MBKHEP). It studies the adequacy of the installed support at these sites and compares this result with support determined from other empirical and numerical modelling approaches.

1.2 Background

The compressional tectonic stress regime in the Himalaya has resulted in intense deformation of the rock mass, making it highly folded, faulted, sheared, fractured and deeply weathered. This complex geological setting has caused considerable stability problems (uncertainties) and is a great challenge for successful tunneling. Several cases of squeezing of the tunnel has occurred despite the result of pre-investigation suggesting a good rock mass. Since pre-investigation is done from the surface and it cannot give accurate result at every section along the alignment, investigation should be carried out during excavation and the support condition modified according to the requirement.

Proper support system must be installed during underground construction. Under estimation of support system leads to failure of the structure while over estimation leads to unnecessary increasing cost of the project. Several approaches are available for estimation of tunnel deformability and the support system required to control it. Mathematical, empirical, analytical and numerical modelling approaches can be done for proper estimation of the support system. Mathematical solutions are precise methods; however the need to conduct mathematical calculations usually decreases the user's desire to use this method (Rahmani et al., 2011). Empirical methods are based on the experience gathered by researchers in various parts of the world whereas numerical modelling utilizes computing power and, using various modelling techniques, can be a precise way of solving very complex problems. The analytical approach address the nature of interplay between the rock mass that may vary and the installed support, and the effect of variation in assumed rock properties on the support load. These solutions can be very useful in order to gain insights into tunnel behavior when the excavation takes place in rock masses, which exhibits squeezing condition (Barla, 2000). In country like Nepal, where underground construction practices are still developing, assessment of support can provide valuable insights to the installment of the support system.

During assessment of some tunnels in lesser Himalayan zone of Nepal, it has been seen that there exist major squeezing problem in HRT of Chameliya Hydroelectric project and the support system provided is not able to control the squeezing (Shrestha and Panthi, 2015). While in the HRT of Kulekhani-III Hydroelectric Project, the support system provided is sufficient to withstand the pressure and control squeezing. Numerical, Analytical and empirical approaches were used for the study. Based on the assessments on few sections conclusions can be drawn regarding the sufficiency of the support and interpolation can be done to make the necessary corrections in its design. Although numerical modelling can give relatively precise results than empirical and analytical (if all the data are available) but there are limitations to the available data so analyzing the results of numerical approach with other approaches can help to determine the accuracy of other approaches. Rahmani et al. (2011) on his study on Access tunnel to Shahriar dam crest in East Azerbaijan Province has concluded that the empirical approach gave similar results as that of numerical approach and has recommended that in the absence of the laboratory data for numerical analysis, the results of the empirical methods can be used for the initial stages of project design. However, if the data about rock properties is complete numerical analysis should be used for calibration of the results obtained by using empirical methods.

1.3 Objectives

The main objective of the study is to:

- Assess and analyze the tunnel support system of Hydropower Tunnels located in Lesser Himalayan region of Nepal.

Specific objectives of the study include:

- To determine and analyze the tunnel support system from Rock Mass Classification approach.
- To study the tunnel section deformation pattern using analytical approach.
- To Conduct Direct Shear test to find rock mass parameters.
- To perform Numerical Modelling of selected tunnel system, observing the behavior of ground towards the opening
- To perform comparative study of different methods and study the adequacy of the determined support system against the deformation in the tunnels.

1.4 Methodology

For this project two hydropower sites has been selected: Middle Bhotekoshi Hydroelectric Project (MBKHEP) and Sanjen Hydroelectric Project (SHEP). Both the site lies in lesser Himalayas of Nepal. The main objective of the project is to make a comparative analysis of the support system provided in the site with those obtained from various approaches (Empirical, Analytical and Numerical). The methodology implemented for completion of the project are as follows:

1. **Literature Review**
   - Review of several cases of tunnel failure in Nepal
   - Review of factors affecting the stability of underground structures
   - Review of existing empirical, and analytical methods to evaluate the squeezing potential and design of underground structure
   - Review on numerical investigation methodology
   - Background theories on stability analysis and deformation calculation
2. **Data Collection**
   - Data regarding tunnels of Nepal of both constructed and under construction
   - Geological information on the rock mass condition

Rock mass classification data were obtained from the respective sites. Although the Q-System classification value for MBKHEP were derived from Rock Mass Rating (RMR) value using various relation.

3. **Stability Analysis**

Based on these data, the analysis of squeezing phenomenon has been done using different approaches. The empirical approach; Singh et al. (1992), Jimenez et al. (1995), semi-analytical approach; Hoek and Marinos (2000), analytical approach; Convergence Confinement Method (Carranza-Torres and Fairhurst, 2000) have been used for the squeezing analysis.

4. **Numerical Investigation:**

Based on the values obtained from empirical relations, support system is also analyzed using commercially available Finite element software Rocscience Phase^2

5. **Interpretation**

The results from the analysis is compared with the actual (observed in the site) values and conclusions will be derived regarding the difference in the results.

Implementing all the steps mentioned above, the methodology of the project has been summarized in the flowchart given below.

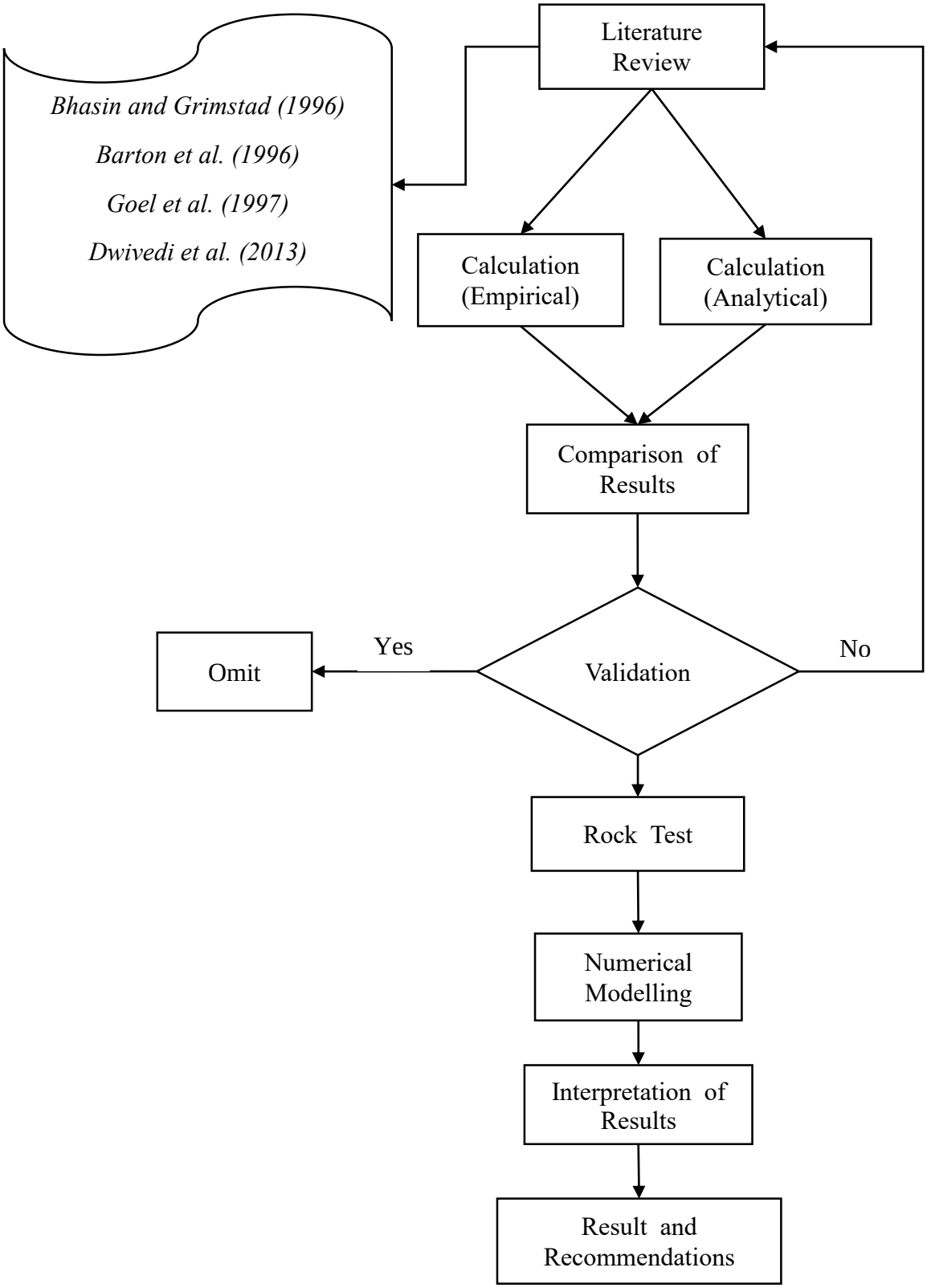


Figure 1-1 Work Methodology

1.5 Scope and Limitations

1.5.1 Scope

The scope of this project is to access the support of underground structures and check its stability. Various theoretical approaches have been used for achieving the objectives of the project. The support obtained from the theoretical approaches will be compared with the actual support provided in the selected sites and the differences in the support system, if present, will be analyzed and the cause will also be studied. The scope can thus be summarized as:

i. Study of squeezing potential at different tunnel section of SHEP and MBKHEP
ii. Analysis of support systems used in the project sites using different approaches.
iii. Recommendation for suitable support system in tunnel
iv. Comparison between the actual support installed at site and support obtained from different approaches
v. 2D Modelling of various sections of the tunnels considered.
vi. Stress analysis by Numerical Modelling in the tunnel sections

1.5.2 Limitations

There are several limitations in the project. Firstly the access to actual data from the sites were limited so assumption and interpolation is done in order to compute the actual support system. Several journals and research papers were referred for the computation of theoretical support so some variations might exist in the computed values. Similarly, Numerical modelling requires inputting many rock parameters which have to be obtained in the laboratory and which are required for precise numerical modelling analysis. However, obtaining all the rock properties through laboratory tests can be very expensive therefore many times the value of these parameters is estimated. This estimation usually increases the error of calculations. Thus the limitations can be summarized as:

i. The overburden calculated was from the provided profile map which may not exactly match the existing overburden.
ii. All the primary data were not enough for the analysis and some secondary data are referred from Generalized Hoek and Brown Failure Criterion and other reports.
iii. Numerical modelling was done according to the secondary data available from initial preliminary geological investigation and existing geological investigation report
iv. Assumptions were made for unavailable data during numerical modelling.

1.6 Organization of Chapters

The report has been divided into seven chapters with each chapter covering different aspects. The first chapter introduces the topic of the project. it also provides a brief background about the cases of failure of some tunnel cases. It talks about the objective of the report along with its scope and limitations. The methodology implemented for the completion of the project has also been included within the chapter.

Chapter 2 describes about the tunneling practice implemented in Nepal. The chapter includes a map for the locations of constructed and under construction tunnels. The chapter also provides a brief description on the types of rock support. The major focus of the chapter lies in literature review for various assessment methods and finally it concludes by giving a brief description of the methods used for this project.

In Chapter 3, the hydropower sites considered for the case study are described. It provides a brief description of the project sites and also includes the results of theoretical approaches obtained from the assessment of the case studies. Following it, chapter 4 consists of the procedure and results obtained from the experimental investigation of rock samples obtained from the selected hydropower. Direct shear test has been conducted in the laboratory of Kathmandu University and the results has been provided.

Chapter 5 covers Numerical Investigation. The parameters used for modelling of case studies have been provided in the chapter. The result obtained from experiment investigation has also been used during modelling. The results of Modelling has been summarized in the chapter while the actual results has been included in the Appendix of the report.

Chapter 6 includes the Hydraulic Design of components of a hydropower. The hydraulic design has been done for one of the two case study. This chapter also includes optimization of tunnels of both the case study. Finally the report has been concluded in chapter 7 which gives the conclusion of assessment of the selected sites.

CHAPTER 2 CONSTRUCTION PRACTISE OF TUNNELS IN NEPAL

2.1 General

Underground tunnels and caverns are used for various purposes like hydropower, storage, transportation, etc. With the increasing use of underground structures, their construction practice is also improving with the development in knowledge and technology. Several methods have been developed for the safe and economic excavation of underground structures. With the introduction of several rock mass classification techniques like Rock Quality Designation (RQD), Rock Mass Rating (RMR), Q-System and several approaches for estimating support pressure, underground excavation has been made easier. Also with the technological development in exaction technique, tunneling can be done in a faster rate which helps save time and money.

In country like Nepal, there is a huge potential of underground constructions for hydropower, transport systems and conservation of environment. However, it is associated with stability problems caused by the fragile regional geology and mountainous topography. As such, various classification techniques should be used for safe design of such structures. Researches must be done to identify the techniques suitable for Himalayan geology. However, for development of underground structures, few research works have been carried out on general basis. All other studies related to geology of underground structures are project specific only.

2.2 Tunnels in Nepal

Being the country with rapid topographical change defines the need of tunnel for movement of man or material. Nepal government is constructing the Kathmandu Nijgadh Fast track of 76.2 Km long connecting Khokana (Lalitpur) to Nijgadh (Bara). The projects consist of 1.35 Km long twin tube tunnel with the purposed tunnel width of 11m for 2 lanes of roadway and 22m for 4 lanes of roadway. Similarly, Japan Government has signed with Nepal government for supporting Nepal government to construct the road tunnel connecting Naubise and Thankot. The proposed project consists of 2.45 Km of tunnel and after completion of this project it is aimed to reduce the traffic congestion and journey time going in and out of the Kathmandu valley through Prithivi highway.

In hydropower sector, the first hydropower tunnel constructed was Tinau Hydroelectric Project situated at Dovan VDC-6 of Palpa District by Their Majesty's Government and the United Mission to Nepal (UMN) in a joint initiative. 2400 m tunnel was constructed with the diameter of 1.8 m and cross sectional area 2.1 m^2.(NEA). The project commenced on 2022 B.S. (1966 A.D.),

taking approximately 11 years for its completion. Similarly other hydropower project that incorporated tunnel structure as the waterway is Kulekhani I hydropower project which was commissioned in 1982 A.D. with the length of headrace tunnel of 6233m followed by Kulekhani II HPP with the headrace tunnel length of 5847.8m. (NEA, n.d.). Adhikhola hydropower plant was commissioned on 1991 A.D. with the installed capacity of 5.1 MW. The headrace tunnel of the project is fully concrete lined having the length of 12,847m with the cross section area of 7.5 m^2, drop shaft of 245m having the diameter of 4.5 m and at last long a tailrace tunnel with the 1080m having cross sectional area of 8.5 m^2. It also consist of cavern of size 37m x 7m x 10m with the shotcrete support together with rock bolts. (Hydro-Solution Nepal, n.d.).

Following them Jhimruk hydropower Plant was commissioned in 1994 A.D is located at Pyuthan district, Nepal. It incorporated the headrace tunnel structure of 1100m length with the cross sectional area of 8.5 m^2 and two inclined penstock tunnel of 280m long. (International Hydropower Association, n.d.); (Himal Hydro & General Construction Ltd., n.d.)

The tunnels with longer length are also in the phase of under construction in Nepal like tunnel of Melamchi Water Supply Project of length 27.5 Km diverting 170 MLD water from Melamchi River to Kathmandu valley. While the Bheri Babai Diversion Multipurpose Project (BBDMP) consist 12 Km headrace tunnel structure which is being constructed using the Tunnel Boring Machine (TBM). According to the authority, the advance rate is 20m/day of excavation in Bheri Babai Multipurpose Project.

From the data available from Department of Electricity Development (DOED) a total of 109 hydropower tunnels has been registered. Among them a total length of 417.83 kms of tunnel is under construction while 67.95 kms has already been constructed. From them the length of tunnels under construction and constructed according to the province has been listed in Table 2-1. A schematic diagram showing the locations of tunnels (both under construction and constructed) has been provided in Figure 2-1 (a). Also Figure 2-1 (b) provides the density of tunnels according to province. It can be seen that most of the tunnels are located in Province 3 and 4. The density of under construction and constructed tunnels according to different province is given in Figure 2-2 (a) and (b).

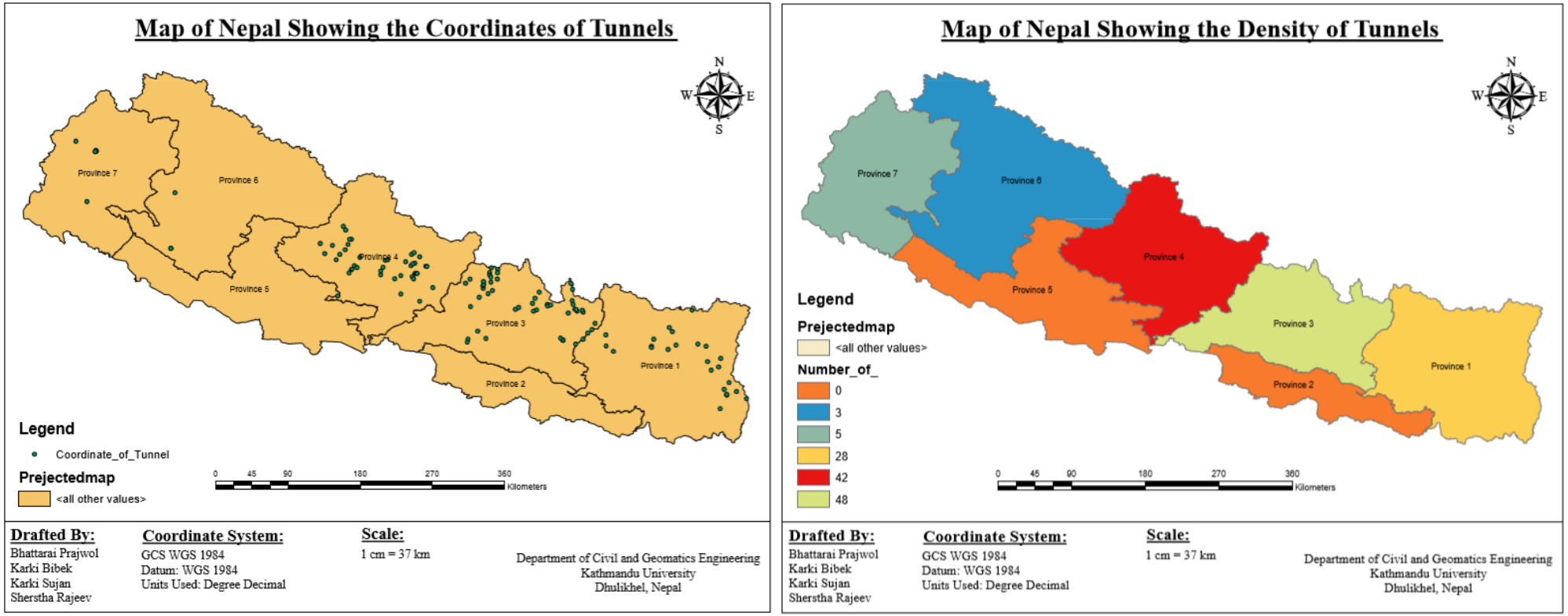


Figure 2-1 Location of Tunnels in Nepal (Constructed and Under-construction) along with its density in different province

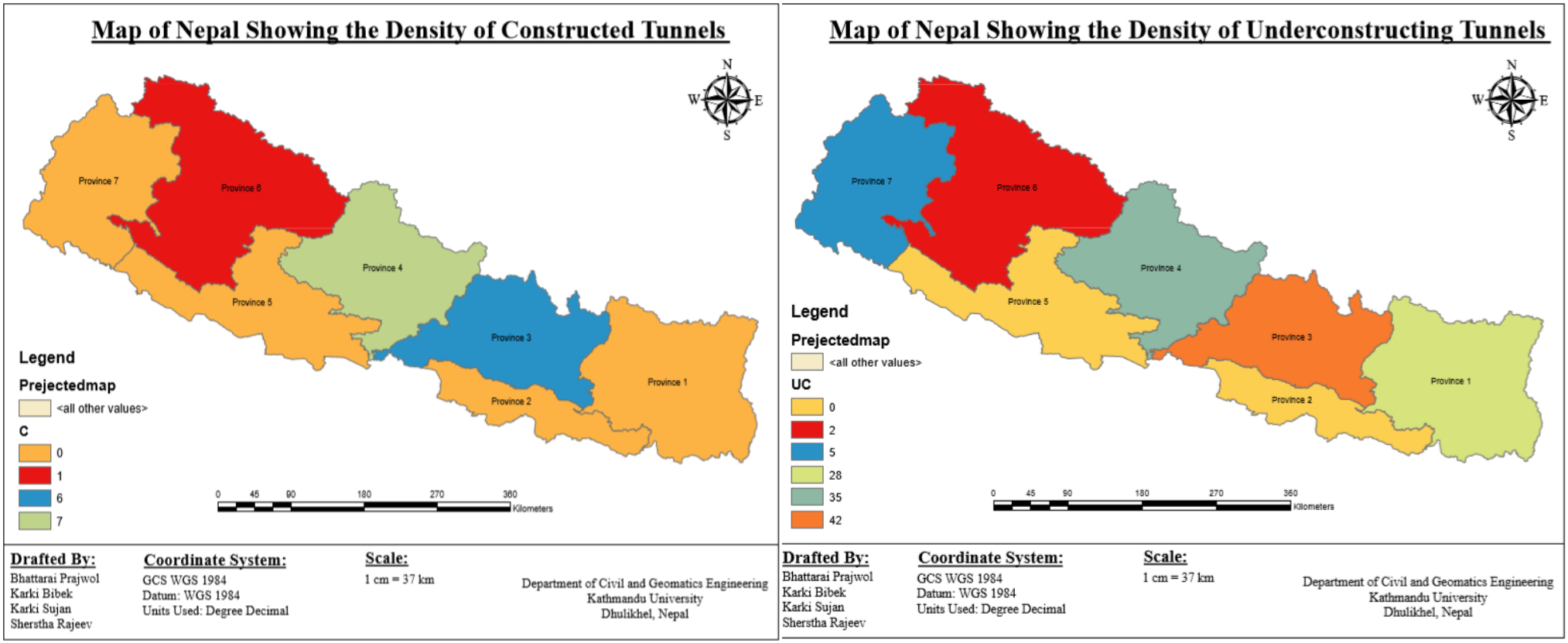


Figure 2-2 Map showing Density of (a) Constructed and (b) Under construction Tunnels according to Provience of Nepal

Table 2-1 Length of tunnels according to province of Nepal

Province	Under Construction (kms)	Constructed (kms)
1	97.7	-
3	180.5	21.9
4	126.6	32.35
6	5.6	13.7
7	7.43	-
Total	417.83	67.95

2.3Methods of Tunneling

2.3.1 New Austrian Tunneling Method (NATM)

Design of support and its installation process for underground structure like tunnels have been very crucial part. Talking about the use of the support system in past centuries, support system like masonry wall of stone and brick were used and in many cases the timber were used as the structural member. But use of these materials became obsolete after use of structural member like steel ribs, shotcrete, and concrete lining etcetera. With increase in the knowledge and the dynamics of the rock mass, new more economical yet safer support system were searched for and introduced. New Austrian Tunneling Method (NATM) is the method that is continuously updating itself through the experience. It is notified by Singh et al.(2006) "*NATM is a misnomer as it is not a method of tunneling but a strategy for tunneling which does have a considerable uniformity and sequence*". NATM defines its approach to be flexible with the philosophy of "Build as you go" where the optimum support condition is assessed making the support not too stiff or too flexible with carefully understanding the time factor being the important aspect of stability by installing the support neither too early nor too late.

The stabilization of underground opening depends upon the redistribution of the stress and controlling the stress release. These techniques will help to reduce the loosening effect of the rock mass around the boundary and which in-turn helps to minimize the hazards and risk. This is the specific strategy of NATM where this method helps surrounding to act as the self-supporting structure with the support installation like shotcrete and systematic rock bolts.

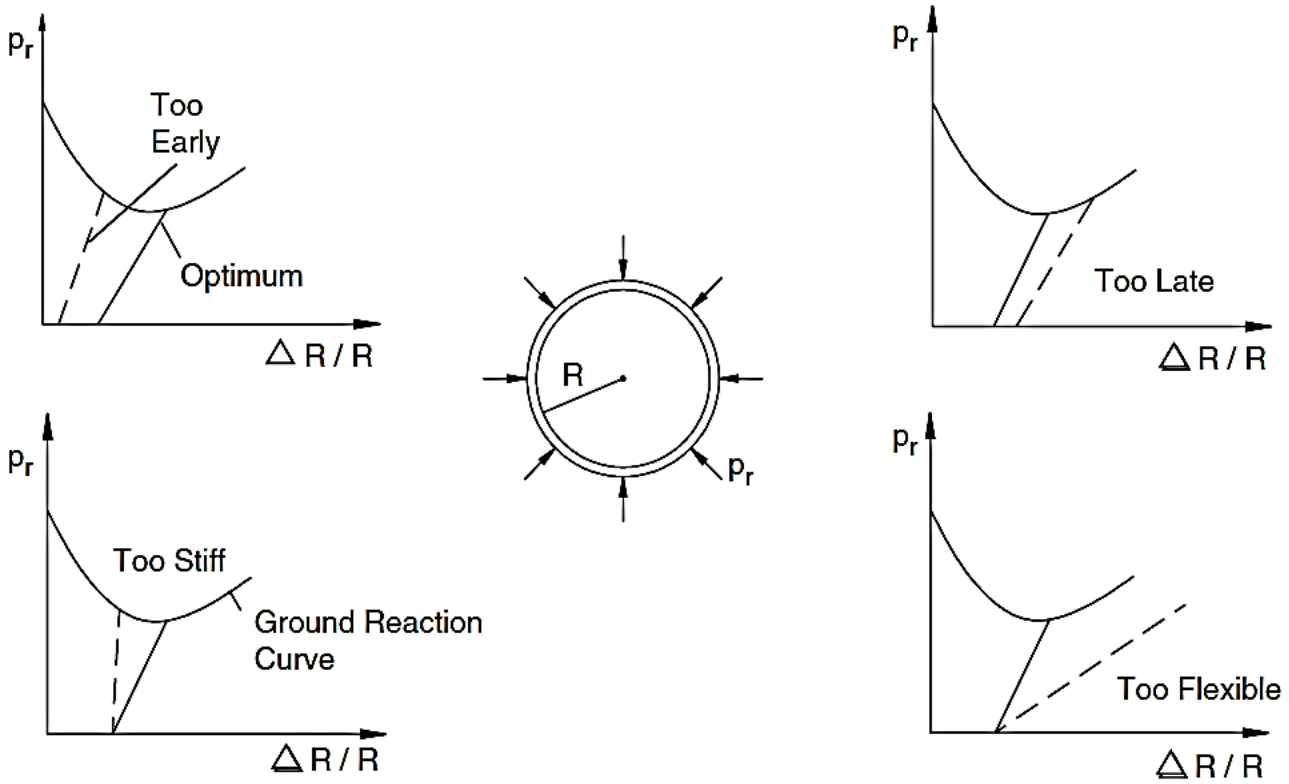


Figure 2-3 The effect of support installation w.r.t time and stiffness. (Singh et al., 2007)

NATM defines the advantages of use of the rock bolt to be one of the major advantages of the support system as it helps to commensurate the complex force introduced by the underground opening to a rock as the supporting structure itself, taking rock as the engineering material. As discussed earlier also, NATM provides the emphasis to time as the crucial factor that interplay in support installation. Because the support installed within the optimum time ensures sustainability of support system. It is also recommended that the spraying to the initial shotcrete of 2.5cm thick will act as the temporary support limiting the initial failure of the rock mass: as we know that if the failure at the initial stage if controlled, it cannot promulgate to larger failure in-turn saving us the money and time. Then after the initial spraying, for permanent support the ground reaction towards the support is taken in account and adequate support is provided as per requirement. The stress-redistribution is taken as utmost importance in NATM to ensure the safety of underground opening and this is done by providing the adequate confining pressure to assess optimum closure with the economical use of support system. The method also recommends to provide the small patches that should be left unshotcreted for effective drainage of groundwater through those patches. It is advised to use full face excavation by NATM method but if not possible the excavation of the opening should be done in multiple stage of excavation of heading and benching

rather than stage drift (*which initiates the complex rearrangement of stress after the introduction of the underground opening*). It's also suggested that for squeezing ground the lining should be provided at the invert such that lining should be flexible resisting the bending failure.

In a whole NATM is the method derived from the experience and is favorable method for weak ground where smooth profile of the opening can be obtained using either perimeter blasting or smooth blasting. The timing and the extent of secondary support is decided significantly by monitoring the performance of the underground construction. The main idea is to use the geological stress of surrounding rock mass to stabilize the tunnel itself. The first use of NATM in soft ground tunnel was done in 1969 AD (after five years of getting worldwide recognition).

In rudimentary concept NATM defines its principle as follows:

- Taking Rock as engineering material by mobilization of its strength.
- Provision for initial shotcrete to localize the failure.
- Monitoring the deformation of the opening.
- Providing adequate support according to the requirement.
- Providing the invert lining to form a complete continuous load bearing support system.

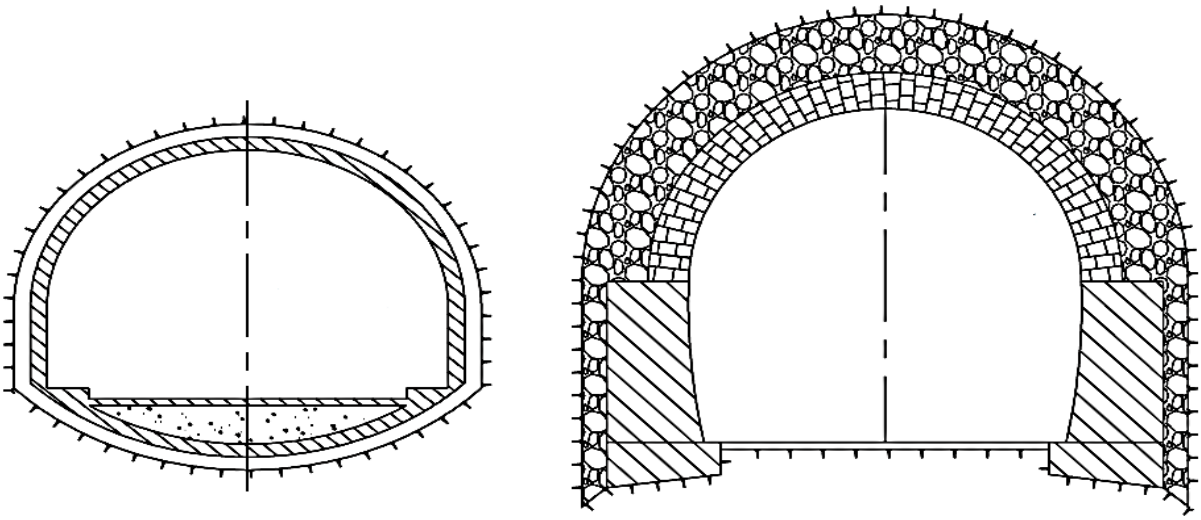


Figure 2-4 .Showing the use of invert lining and comparative thickness of modern (Left) and Traditional (Right) (after Singh et al., 2006)

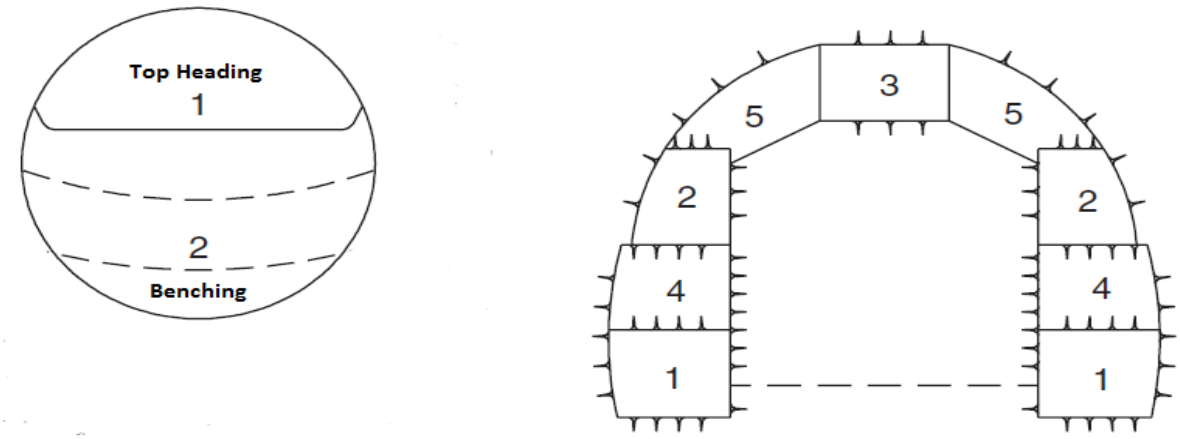


Figure 2-5 Heading and Benching (Left) and Multiple Drift excavation (Right) (After Singh et al., 2006)

2.3.2Norwegian Method of Tunneling (NMT)

NMT is the suitable method of tunneling in the hard ground where we find dominant overbreak and jointing. The most commonly used method of excavation in NMT is Drill and Blast or Hard rock TBM. NMT has been updated with the use of new technology and finding and is being used in last 30 years and uses 1260 case history to back it for its efficiency. NMT describes the actual condition of rock masses in quantitative terms rather than qualitative. NMT uses Q- System for regulating the description of rock mass conditions and support recommendations. As it uses Q-system for support estimation, it is the forward predictive method. NMT emphasizes the use of temporary supports such as rock bolts and shotcrete which can later be incorporated with the permanent support. Final support is selected during tunnel construction based on tunnel logging and use of the Q-system support recommendations. The permanent rock support usually consists of high quality wet process, fibre reinforced shotcrete, and fully grouted, and corrosion protected rock bolts. These may be supplemented by RRS in very poor conditions. The use of nominal thickness, final cast concrete linings for appearance or due to tradition is discouraged due to cost, scheduling and lack of loading when Q-system designed Bolt + fibre reinforced shotcrete for assumed loading levels is already in place. It also values the time aspect of the support installation for which and the qualitative measurement to monetize the aspect of tunneling using Excavation Support Ratio (ESR). NMT view the effectiveness of other geological problem by monitoring the support system, its effectiveness. NMT does not recommends the tunnel support in hard and competent rock unlike the NATM. For the rock mass where the stress is very high and the rock is strong in a sense brittle, application of the rock bolts/anchors is recommended by NMT where the main reason being the transformation of brittle failure towards the ductile failure. This helps the authorities to assess the process and provide appropriate retrofitting as per requirement. The steel

sets and lattice girders are suitable of the overbreak and hard rock conditions. The features of NMT are provided in Table 2-2.

Table 2-2 Showing Features of NMT (After Singh et al., 2006)

S. No.	Features
1.	**Areas of usual application:** Jointed rock giving overbreak, harder end of scale (q_c = 3 to 300 MPa)
2.	**Usual methods of excavation:** Drill and blast, hard rock TBM, hand excavation on clay zones
3.	**Temporary rock reinforcement and permanent tunnel support may be any of the following:** • CCA, S(fr) +RRS + B, B + S(fr), B + S, B, S(fr), S, sb, (NONE) • Temporary reinforcement forms part of permanent support • Mesh Reinforced shotcrete not used • Dry process shotcrete not used • Steel sets or lattice girders not used, RRS and S(fr) are used clay zones and in weak, squeezing rock masses • Contractor chooses temporary support • Owner/consultant choose permanent support • Final concrete lining are less frequently used; i.e., B + S(fr) is usually the final support
4.	**Rock mass characterization for:** • Predicting rock mass quality • Predicting support needs • Updating both during tunneling (monitoring in critical cases only)
5.	**The NMT gives low cost and** • Rapid advance rates in drill and blast tunnels • Improved safety • Improved environment

2.4 Problem in Underground Structure in Nepal

Rock squeezing is a common problem in the Nepal Himalayas while tunneling through low strength rock, fault and shear zones. It reduces the cross-section of a tunnel caused by the in situ stresses, which exceed the rock mass strength. Time of deformation and degree of squeezing

generally depends on overburden pressure and non-swelling clay content. The risk of squeezing increases with increase in clay content and overburden pressure.

There are several cases where the problem of squeezing persisted and Chameliya Hydroelectric Project (CHEP) is one among them. The Chameliya Hydroelectric Project is located on the Chameliya River in Darchula district of the far western region of Nepal and has installed capacity of 30 MW. It is located at about 270 km North West of Dhangadi.

The rock mass quality at several sections of the headrace tunnel was found extremely poor and rock type: talcosic phyllite. Due to severe squeezing and associated deformation, tunnel cross section had reduced considerably in several stretches of tunnel. At several locations in the section between chainage 3+100 to 3+900m, the tunnel wall closure (deformation) was well over 1.0 m and the maximum was recorded above 2.0 m. Due to the excessive deformation, temporary supports were provided at several locations, steel ribs and lattice girders were buckled at several locations and shotcrete lining was also cracked.

The tunnel was excavated by adopting the conventional drill and blast methodology. But for the sections with severe squeezing problems, different methods were attempted to be applied such as over excavation but unsuccessful, fore-poling, sequential excavation (top heading and benching), excavation through light controlled blasting and manual excavations etc. (after Basnet, 2013).

Other tunnel with squeezing problem was Kaligandaki ‘A’ Hydroelectric Project. The Kaligandaki "A" hydroelectric project is located in the western part of Nepal about 200 km west of Kathmandu, Nepal with an installed capacity of 144MW. During tunnel excavation, most of the rock mass along the tunnel alignment was found to be of poor to extremely poor quality and demanding heavy rock support.

There were two major factors that played significant roles for stability at the Kaligandaki headrace tunnel. The first was related to very weak and thinly foliated phyllite with high degree of strength anisotropy that led to considerable reduction on the self-supporting capability of the rock mass. As a result of this, frequent small to medium scale tunnel collapses occurred. The second one was related to tunnel squeezing. Due to high overburden stress and the presence of weak phyllite rock mass, especially graphitic phyllite with low compressive strength, the tunnel squeezed severely at many locations. Severe squeezing was seen between chainage 1+964 and 4+032m where the overburden ranged from 425 to 620m and mainly consists of schistose graphitic phyllite. (Basnet, 2013)

Some tunneling problems were also seen in Khimti Hydroelectric Project located in the lesser Himalayan region about 100 kilometers east of Kathmandu. The headrace tunnel is approximately 7.9 kilometers long with inverted D-shape and 14 square meters in cross-section. Two major stability problems were seen in the tunnel. The first one was related to tunnel collapse due to percolation of ground water into bands of highly weathered and sheared chlorite and talcose mica schist intercalated between fractured gneiss. The second one was related to large leakage through open and permeable joints present in the gneisses and loss of valuable water from the tunnel during operation. Some minor tunnel squeezing was also observed in some locations of the headrace tunnel where chlorite and talcose mica schist was dominant, mainly observed at relatively high overburden (above 200 meters) (Panthi K. K., 2006).

2.5Rock Support

Rock support is used to describe the procedures and materials used to improve the stability and maintain the load bearing capacity of rock near to the boundaries of an underground excavation. The primary objective of a support system is to mobilize and conserve the inherent strength of the rock mass so that it becomes self-supporting. If possible, the installation of rock support should be carried out as an integral part of the excavation cycle to enhance the self -supporting aspects of rock mass improvement.

As mining and civil tunneling progresses to depth, excavation-induced seismicity and rock burst problems increase and cannot be prevented. As an important line of defense, ground control measures and burst-resistant rock support are used to prevent or minimize damage to excavations and thus to enhance workplace safety (Peter and Ming, 2012). There are several types of rock support available and the choice depends upon the quality of the rock mass and ground pressure to be sustained. Some of the common support are:

- Steel sets
- Shotcrete with steel fibers or wire mesh
- Concrete lining
- Grouting
- Spilling or Fore poling
- Rock Bolting

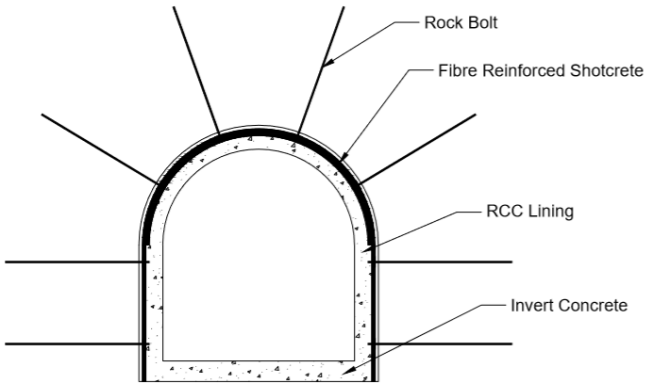


Figure 2-6 Types of Rock Support

Brief description of the supports provided in the sites are provided below:

2.5.1 Rock Bolts

Rock bolts are one of the most common methods of rock reinforcement. The main purpose of bolting is to reinforce loose rock or fractured in-situ rock to prevent caving or spalling (break off in fragments), and to assist the rock mass to form its own supporting structure. Rock bolts are classified into three types:

a. Mechanical Bolts

These bolts have expanding anchor at its end, which after insertion into a hole is either rotated or pressed/hammered against the bottom of the hole. These bolts are typically meant for temporary rock support but together with cement grouting, they provide both immediate and long-term support.

b. Grouted Bolts

Cement grouted bolts are commonly used as a rock support during tunneling. They are preferred to other types of bolts as they are simple and quick to install and can be used with or without any mechanized equipment. The cement grout provide protection against corrosion but the disadvantage is the longer setting time of the cement grout. So in places which demands immediate support, cement grouted bolts prove to be disadvantageous. Although the setting time of the grout could be improved by mixing additives but it would also increase the cost of bolting.

c. Friction Bolts

Split-set and Swellex bolts are two common types of friction bolts. They are quick and easy to install and give instantaneous support but do not provide long term support. The Split-set bolt is hammered into a hole of slightly less diameter than the bolt. They are very suitable for layered formations but the disadvantage is that they cannot be effectively protected against corrosion. The Swellex bolt, however, has a longer life span and is installed by applying high pressure water to the bolt after inserting it to the hole. The high pressure expands the bolt to the final dimension in the hole, therefore utilizing the roughness on the hole surface.

Figure 2-7 Rock Bolts used in SHEP

2.5.2 Shotcrete

Sprayed Concrete or shotcrete is a widely used support method in construction. It is used for temporary or long term support, lining or backfilling. Usually shotcrete is used together with bolting to obtain the best support or reinforcement. Shotcrete can be reinforced by adding steel fibre to the concrete. The most common forms of shotcreting are the dry-mix and wet-mix methods.

a. Dry Mix Method:

In the dry mix method, the aggregate, cement and accelerators are mixed together and propelled by compressed air. Water is added lastly through a control valve on spray nozzle. Advantages of the dry mix process are that the water content can be adjusted instantaneously by the nozzle man, allowing more effective placement in overhead and vertical applications without using accelerators. The dry mix process is useful in repair applications when it is necessary to stop frequently, as the dry material is easily discharged from the hose.

b. Wet Mix Method:

In the wet mix method, the aggregate, cement, additives and water are measured and mixed before transport. Today, wet mix is more widely used because it is easy to mechanize and the transport. The wet-process procedure generally produces less rebound, waste (when material falls to the floor), and dust compared to the dry-mix process. The greatest advantage of the wet-mix process is all the ingredients are mixed with the water and additives required, and also larger volumes can be placed in less time than the dry process.

As shotcrete develops strength with time after application, it may be used effectively soon after excavation. As loading is transferred on to the support system, it is gaining strength and producing a stiffening support member. Most products are shot with up to 5% accelerator if a high early strength is required. This obviously leads to the development of a faster supporting member, but care must be exercised in design to ensure that the support will not become overstressed by load transferred from relaxing ground in high stress environments.

Advantages of Dry mix

- Machines are smaller and cheaper
- Lower costs for cleaning and maintenance
- Stop and restart of shotcreting is simpler
- Longer conveying distances (up to 150m)
- More precise dosage of additives
- Better concrete (pumping of wet mix requires a higher water content)
- Water content can be manually reduced, e.g. when spraying against wet background

Advantages of Wet mix

- Reduced dust production
- Reduced rebound
- Reduced scatter of concrete properties
- Higher capacity

2.5.3 Grouting

Grouting is the method in which a solidifying liquid is pressure injected into the rock mass. The main purpose of grouting is to prevent ground water leakage into the tunnel, and to increase overall of rock mass strength. In drill and blast tunneling, grouting is typically performed before (pre-grouting) or after (post-grouting) excavation.

a. Pre-Grouting

Pre-grouting means the rock mass is grouted excavation begins. Usually pre-grouting is done from the tunnel, but in situations with low overburden it is also possible to do it from the surface.

Probe holes are drilled to map possible fractures and register water flow. This helps to anticipate the need for grouting. Later, grout holes are drilled in a conical fan shape in front of the tunnel face.

b. Post-Grouting

When grouting is done after excavation, grouting holes are drilled from the tunnel in a radial form. In good rock conditions with small water leakage, post-grouting is often adequate. Post-grouting enables better rock mass structure evaluation. On the other hand, water leakage blockage is often more difficult because the water flow tends to flush away the grouting agent before it hardens.

2.5.4 Concrete Lining

Concrete lining in tunnel involves special equipment, including agitator cars for transport, pumps or compressed-air devices for placing the concrete, and telescoping arch forms that can be collapsed to move forward inside forms remaining in place. The invert is generally concreted first, followed by the arch where forms must be left in place from 14 to 18 hours for the concrete to gain necessary strength. The final operation consists of contact grouting, in which a sand-cement grout is injected to fill any voids and to establish full contact between lining and ground. The method usually produces progress in the range of 40 to 120 feet per day.

Figure 2-8 Shotcrete and Grouting in UT-3A

2.5.5 Steel Sets

Steel sets can only respond to loads imposed on them by the inward movement of the rock, hence they are referred as passive support. Since they are generally placed some distance behind the advancing face, most of the short-term movement in the rock has already taken place before the sets are in place and the only load that they are called upon to carry is the dead weight of rock failing around the opening. In hard rock mining, steel sets have very limited application since most support duties can be performed more effectively by rock bolts, or shotcrete or by some combination of these systems. The exception is in mining through faults or in very badly broken ground associated with faults of shear zones. In such cases, it may be impossible to anchor the rock bolts or dowels in the rock mass and steel sets may be required in order to carry the dead weight of the failed material surrounding the opening.

2.5.6 Fore Poling

Fore poling has been frequently used to stabilize the ground around the cutting face, and to control the settlement of ground surface. This method involves the driving of pipes and the injecting of grouting materials into the ground ahead of face prior to excavation. Following instruments are used for fore poling:

c. Pneumatic Rotary Drilling machine
d. Perforated Pipe
e. Jack Hammer
f. Grouting machine

If the strength of the ground is so low that the excavated space is unstable even for a short time, a pre-driven support is applied in such a way that an excavation increment occurs under the protection of a previously driven canopy. Forepoling is achieved by spiling, pipe roof, grouting and freezing.

a. Spiling

This method consists of drilling a canopy of spiles, i.e. steel rods or pipes into the face. A typical length is 4m. In order for the spiles to act not only as beams (i.e. in longitudinal direction) but also to form a protective arch over the excavated space, the surrounding soil is grouted through the steel pipes or sealed with shotcrete. Thus, a connected canopy is formed that consists of grouted soil reinforced with spiles. Spile rods can also be placed into drill hole. The remaining annular gap is filled with mortar, whose setting however may prove to be too slow.

b. Pipe roof

This method is similar to spiling with the only difference that large diameter (>200mm) steel or concrete tubes are jacked into the soil above the space to be excavated. The larger diameter provides a larger bearing capacity. Sometimes, the tubes are filled with concrete. The steel tubes only act as beams and do not form an arch. Pipe roofs do not protect the overburden soils from considerable settlements

Figure 2-9 Forepoling and installation of Steel sets in SHEP

2.6 Literature Review

2.6.1 Empirical Approach

Shrestha and Panthi (2015) describes the influence of stress anisotropy in tunnel deformation by taking the case study of Kaligandaki Hydropower headrace tunnel, which would be very high if the isotropic condition was employed (i.e. *Done by Conventional CCM*). This led to the empirical relationship between the tunnel strain at the spring level (where it is seen maximum) (E), vertical stress (σ_v) and shear modulus of rock (G) along with the stress anisotropy coefficient (k).

$$E = a\left(\frac{2G}{\sigma v(1+k)/2}\right)^b \qquad 2.1$$

Where, a and b are the constant depending on the support pressure (P_i) given by the equations:

$$a = 742.23Pi^2 - 1870.6Pi + 1654.7 \qquad 2.2$$

$$b = -0.1541Pi^2 - 0.2022Pi + 2.003 \qquad 2.3$$

The support pressure that would be taken by the support for providing the stable underground section depends upon the type of the rock environment where the opening is introduced. As these rock environment demonstrate the unique failure behavior that is essential for designer to choose

the competent rock support system (Martina, Kaiserb and Christiansson, 2003). Different rock mass properties like Uniaxial Compressive Strength (UCS) of intact rock mass (σ_{ci}), deformation modulus of the rock, stress ratio and geological mapping are used for obtaining the explicit view of the rock environment that we are dealing with. To find the UCS we generally can employ the simple mean test using like hand pressure test and geological hammer test which gives approximately the UCS comparing with the complex test (Hack and Huisman, 2002). The deformations also play a vital role on estimating the support pressure for tunnel openings. It gives the idea about the stress that are acting on the opening. Nonetheless the Tectonic moment and abundance of young Himalayas in the Himalayan region let the underground structure to be subjected to high in-situ stress in weak rock mass giving rise to the stability problem. Small volumetric expansion caused by the stress induced shear failure resulting the closure of the underground opening in the radial direction can be termed as squeezing (Singh and Goel, 2011). Dwivedi et al. (2013) have given the correlation to find out the support pressure in the tunnel of squeezing ground relating the support pressure with the parameters like joint factor (J_f), vertical insitu stress (σ_v), uniaxial compressive stress of intact rock mass (σ_{ci}) , allowed percentage closure(d) and horizontal insitu stress(σ_h). The graph given for comparison between different empirical approaches defines this correlation as the refined equation.

$$Ps = 9.23 \times 10^{-3}\sigma v\left(\frac{Jf^3 \times \sigma h^{0.1}}{10^7 \times \sigma ci^{0.1} \times \left(d^{0.2} + \frac{Jf}{1434}\right)}\right)^{1.7} \qquad 2.4$$

Dwivedi et al. (2013) also defines the equation was valid for the tunnel excavated by conventional drill and blast method and taking the opening section as the equivalent circular section.

They developed dimensionally correct empirical correlations with correlation factor of 0.94 to predict tunnel deformation for squeezing grounds. He used the data of 53 sections from various case histories of 14 adits/tunnels/mine roadway for the study. Joint factor (as a measure of rock mass quality)/ Barton's rock mass quality, vertical in situ stress, support stiffness and radius of the tunnel are the governing parameters which have been considered for developing the correlations. They studied the geology and tunneling problems of various tunnels including Kalingandaki “A’, Khimti, Chibbro-Khodri Hydroelectric Project, etc. (all the studied tunnels underwent some form of deformation during construction) to formulate an empirical correlation to predict tunnel deformation. They considered two parameters, Joint factor, J_f and rock mass quality index, Q-value to develop two empirical equations

$$\frac{u_p}{a} = \frac{5 \times 10^{-10} \sigma_v {J_f}^3}{K + 0.5} + 0.052 \text{ (Using Jf)} \qquad 2.5$$

$$\frac{u_p}{a} = \frac{0.0191 \sigma_v Q^{-0.2}}{K + 1} + 0.0025 \text{ (Using Q)} \qquad 2.6$$

Where, u_p = predicted radial deformation of tunnel (m),

a = radius of tunnel (m),

J_f = joint factor,

σ_v = vertical in situ stress (0.027H), MPa,

K = support stiffness, MPa, and

Q = rock mass quality

They found that the value of tunnel deformation highly depends upon the quality of rock mass J_f and Q in respective correlations. According to the study, the developed correlations are valid for tunnels excavated by drill and blast method in squeezing ground conditions (where convergence is larger than 1% of opening size).

2.6.2 Analytical Approach

For analytical approach, the Convergence-Confinement method (CCM) given by Carranza–Torres and Fairhust has been preferred. Torres and Fairhust (2000) describes the Convergence-Confinement method as a procedure that allows the load imposed on a support installed behind the face of a tunnel to be estimated If the support is installed immediately in the vicinity of face, it does not carry out full load to which it is supposed to. The part of load is carried by face itself. As tunnel and face advance away from the support, face effect decreases and support must carry more loads. When the tunnel moves well away from face, the support will be subjected to full design load. Torres and Fairhust (2000) describe the application of CCM by taking an example of a tunnel of radius R through a rock mass that is assumed to be subject initially to a uniform field far field stress σ_o A support is installed at a distance L from face at section A-A'. The support is assumed to be of unit length in the direction of tunnel axis. The radius R_{pl} indicates extent of failure zone that developed around the tunnel. They assumed that all the deformation occurs in a plane perpendicular to the axis of tunnel; radial displacement u_r and pressure p_i i.e. the reaction of support on the walls of the tunnel are uniform at the section. This has been shown in Figure 2-10

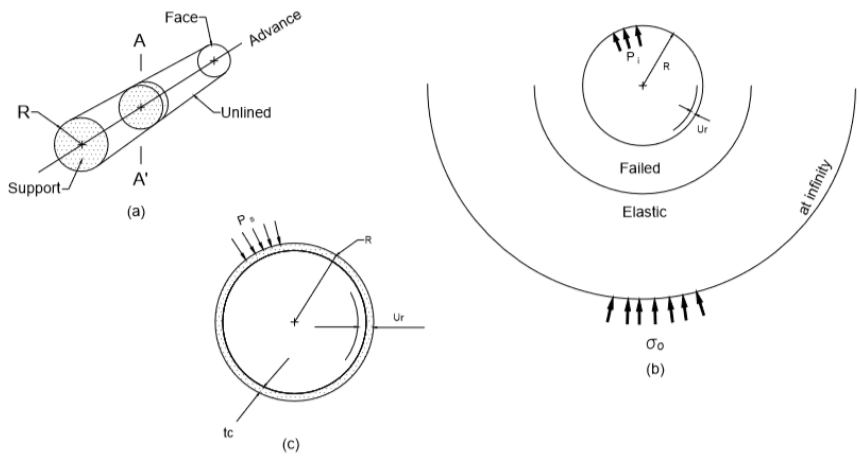


Figure 2-10 a) Cylindrical tunnel of radius R driven in the rock mass. b) Cross-section of the rock mass at the section A-A'. c) Cross-Section of the circular support installed at section A-A' (after Carranza-Torres and Fairhurst, 2000)

It was found that an analysis of the interaction of load-deformation characteristics of the system will be necessary to determine load transferred to the support. The system should thus consist of; the tunnel moving forward, the excavation section normal to tunnel axis and the installation of support at that section.

Based on the above explanations, Torres and Fairhurst (2000) concluded that CCM has three basic components viz. the Longitudinal Displacement Profile (LDP), the Ground Reaction Curve (GRC) and the Support Characteristics Curve (SCC).

Torres and Fairhurst (2000) have also provided relations for the construction of the three curves. For LDP, the empirical relation suggested by Hoek (1999) can be used:

$$\frac{u_r}{u_r^M} = \left[1 + \exp\left(\frac{-\frac{x}{R}}{1.10}\right)\right]^{-1.7} \qquad 2.7$$

Where u_r = radial displacement; u_r^M = maximum radial displacement; x = distance from the tunnel face; R = tunnel radius.

For GRC, Torres and Fairhurst (2000) have given separate relations for the elastic and plastic part of the curve separated by a pressure point called critical internal pressure. They have given the relationship as follows:

If $p_i \geq p_i^{cr}$,

$$u_r^{el} = \frac{\sigma_o p_i}{2G_{rm}} R \qquad 2.8$$

If $p_i < p_i^{cr}$,

$$R_{pl} = R * e^{\left[2\left(\sqrt{p_i^{cr}} - \sqrt{p_i}\right)\right]} \quad 2.9$$

Where p_i = internal pressure; p_i^{cr} = critical internal pressure; R_{pl} = extent of plastic region; u_r^{el} = radial displacement; G_{rm} = shear modulus of rock mass.

Torres and Fairhurst (2000) explained that a flow rule for the material is necessary to define the plastic part of GRC which can be defined by the relation:

$$\frac{u_r^{pl}}{R}\frac{2G_{rm}}{\sigma_o - p_i^{cr}} = \frac{K_\Psi - 1}{K_\Psi + 1} + \frac{2}{K_\Psi + 1}\left(\frac{R_{pl}}{R}\right)^{K_\Psi + 1} + \frac{1 - 2v}{4(S_o - P_i^{cr})}\left[\ln\left(\frac{R_{pl}}{R}\right)\right]^2 - \left[\frac{1 - 2v}{K_{\Psi+1}}\frac{\sqrt{P_i^{cr}}}{S_o - P_i^{cr}} + \frac{1 - v}{2}\frac{K_\Psi - 1}{(K_\Psi + 1)^2}\frac{1}{S_o - P_i^{cr}}\right] * \left[(K_\Psi + 1)\ln\left(\frac{R_{pl}}{R}\right) - \left(\frac{R_{pl}}{R}\right)^{K_\Psi + 1} + 1\right] \quad 2.10$$

For development of SCC, Torres and Fairhurst (2000) have given the following relation for the development of the elastic part of the curve:

$$p_s = K_s u_r \quad 2.11$$

Where p_s = pressure; K_s = elastic stiffness; u_r = radial displacement.

While for plastic part, they have defined the need to determine the maximum pressure (p_s^{max}) and elastic stiffness (K_s) which are different for different types of support.

For Concrete or Shotcrete Lining

$$p_s^{max} = \frac{\sigma_{cc}}{2}\left[1 - \frac{(R - t_c)^2}{R^2}\right] \quad 2.12$$

$$K_s = \frac{E_c}{(1 - v_c)R}\frac{R^2 - (R - t_c)^2}{(1 - 2v_c)R^2 + (R - t_c)^2} \quad 2.13$$

For Ungrouted Bolts and Cables

$$p_s^{max} = \frac{T_{bf}}{s_c s_l} \quad 2.14$$

$$\frac{1}{K_s} = s_c s_l\left[\frac{4l}{\pi d_b^2 E_s} + Q\right] \quad 2.15$$

For Steel Set Support

$$p_s^{max} = \frac{3}{2}\frac{\sigma_{ys}}{SR\theta}\frac{A_s I_s}{3I_s + DA_s[R - (t_b + 0.5D)](1 - \cos\theta)} \quad 2.16$$

$$\frac{1}{k_s} = \frac{SR^2}{E_s A_s} + \frac{SR^4}{E_s I_s}\left[\frac{\theta(\theta + sin\theta\ cos\theta)}{2sin^2\theta} - 1\right] + \frac{2S\theta t_B R}{E_B B^2} \quad 2.17$$

With the combination of these three curves we can determine the strength of support to be used, the time in which the support should be installed, deformations in the ground leading to the modifications in the support system etc. Torres and Fairhurst (2000) also highlighted the limitations of CCM regarding the shape of the tunnels (for it might not be circular) and unequal principal stresses. They also highlighted that if the rock mass is well described as isotrophic, the CCM method is used to derive a useful estimate of the load imposed on the support but also if the tunnel section is not circular and the principal stresses are unequal, CCM can still be used to determine the fair estimate of the rock support requirement.

2.6.3 Numerical Approach

The empirical (Q-System rock mass classification) approach is not able to predict the deformation of tunnels and the designed support system is not able to control deformation in tunnels in weak rock mass condition. (Khadka et al., n.d.). Although the numerical modeling cannot be used directly to analyze the squeezing phenomenon in the tunnels, its application can be utilized to find the deformation of the tunnel in squeezing environment and the results can be compared with the results that have been found from analytical, semi-analytical and empirical approaches. The numerical analysis will help to define the complex nature of the rock mass and geometry of opening and results from which can be found as close to the reality.

The advantages of numerical analysis over the other analysis according to (Basnet, 2013) are:

g. it is quantitative analysis,
h. it provides better understanding of mechanism,
i. it can be used to verify the results obtained from other methods,
j. it provides the extension of measurement results from field and laboratory, etc.

Numerical modeling means discretization of rock mass into a large number of individual elements and powerful computers are used to handle such a huge amount of data. In rock engineering, the numerical analyses are used mainly to analyze the rock stresses and deformations (Basnet, 2013). There are two categories of numerical models:

a. Continuous models

Rock mass is modeled as a basically continuous medium, only a limited number of discontinuities (joints, faults etc.) may be included here. This is the most commonly used category of numerical models. The methods belonging to this category are; Finite Element Method (FEM), Boundary

Element Method (BEM) and Finite Difference Method (FDM). The most common programs of this category are; ABAQUS, ANSYS, BESOL, PHASE2, FLAC3D etc.

b. Discontinuous models

Rock mass is modeled as system of individual blocks interacting along their boundaries. These models represent the nature of the rock mass more close to the reality. The methods belonging to this category are referred to as the Distinct Element Method (DEM), Discontinuous Displacement Analysis (DDA). The most common program of this category is UDEC.

During Numerical modelling, simulation of rock support is possible. Therefore, evaluation of the effect of bolting and grouting during tunneling is possible. The obtained results depend on the size of block and pattern of the joints. The smaller the size of the mesh elements is, the more accurate will be the model. However, by decreasing the size of mesh elements, the time and volume of calculations will increase. In addition, the definition method of some parameters such as hardness and strength among blocks is usually not confirmed at this stage.

Khadka et al. (n.d.) on comparing the use of 2D and 3D numerical modelling of tunnels in higher Himalayas region of Nepal have found that the 3D effect of an advancing tunnel can be simulated by the 2D analysis by splitting the initial field stress in two different stages for weak rock mass while it was not significant for very weak rock mass. They also found that the face effect can be well described by the 3D analysis compared to the 2D analysis.

2.7 Squeezing Assessment

When there is high insitu stress and weak rock encountered the volumetric expansion of the ground can be observed due to the stress induced shear failure this phenomenon can be term as ***squeezing***. It is the time-dependent phenomenon and if not understood and managed carefully can cause loss of money and time. The interplay of induced stress by introduction of ground opening and material properties of the rock mass on which the opening is introduced, if exceeds the limiting shear stress around the opening causes the creep to start. This phenomenon can limited to a short time period or prevail for long time period. (Barla, 1995)

The competency factor (ratio of uniaxial compressive strength to overburden stress) of the rock mass, geology of the rock mass like thickness of the bedding plane their orientation, rock mass quality, tunnel geometry, porosity of rock mass, advance rate, method employed in tunnelling are some factors influencing the squeezing phenomenon (Aydan, 1996) Sometime swelling phenomenon can be misunderstood to be squeezing ground. Swelling generally occurs due to

increase in moisture content in the rock mineral. When a tunnel is excavated the opening might shrink due to loss of water but when the water content increases in the rock mass, swelling can be observed.(Adyan, 1996) The rock mass like altered gneiss, schist, phyllite, shale mudstone, tuff and clay are vulnerable to large deformation due to development of large stress. (Kovari et al. 1996)

2.7.1 Empirical Approach

2.7.1.1 Empirical Approach for Prediction of Squeezing

a. Singh et al. (1992) approach

Squeezing ground criteria given by Singh et al. (1992) based on the rock mass classification. This approach provides log-log plot between the overburden in meters and the rock mass quality in terms of Q-value. 39 tunnel sections were studied and a critical value of overburden was provided given by:

$$H = 350 \times Q^{0.33}$$

For Squeezing Ground $\quad H \gg 350 \times Q^{0.33}$

For Non − Squeezing Ground $\quad H \ll 350 \times Q^{0.33}$

Where, H = Overburden in meters

Q = Q-value

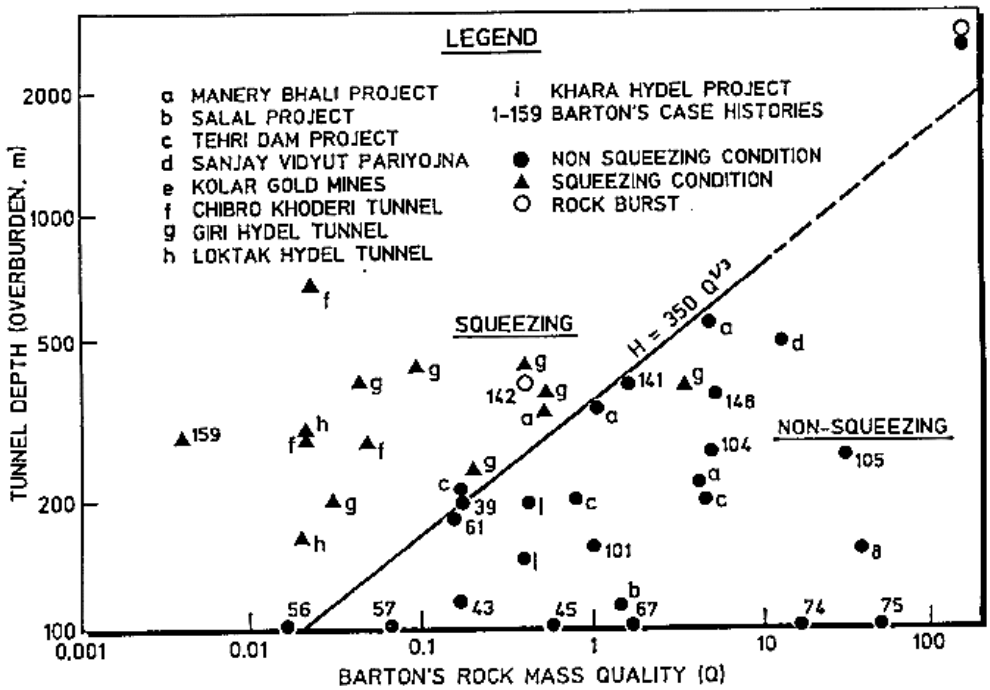


Figure 2-11 Log-Log plot showing the tunnel depth (in meters) versus the rock mass quality Q for prediction of squeezing ground. (Singh et al., 1992)

b. Goel et al. (1995) approach

The squeezing ground criteria given by Goel et al. (1995) is based on rock mass number (N) and the size of the opening. Rock mass number (N) is obtained by making the stress reduction factor (SRF) = 1 in Q classification. It can be also observed that the increase of the opening size also effects the strength of the rock mass making it more vulnerable to instability—as increase in the opening causes lower confinement of the rock mass and high degree of disturbance. This approach provides log-log plot of rock mass number (N) and $HB^{0.1}$ which provides an empirical equation critical equation for overburden given by:

$$H = (275 \times N^{0.33}) \times B^{-0.1}$$

For Squeezing Ground $\quad H \ll (275 \times N^{0.33}) \times B^{-0.1}$

For Non − Squeezing Ground $\quad H \gg (275 \times N^{0.33}) \times B^{-0.1}$

Where, H = Overburden in m

B = Width of tunnel in m

N = Rock Mass Number

Table 2-3 Criteria of ground condition using Rock Mass Number (N) (Singh and Goel, 2011)

S. No.	Ground conditions	Correlations for predicting ground condition
1	Self-supporting	$H < 23.4\ N^{0.88} \cdot B^{-0.1}$ and $1000\ B^{-0.1}$ and $B < 2\ Q^{0.4}$ m
2	Non-squeezing	$23.4\ N^{0.88} \cdot B^{-0.1} < H < 275N^{0.33} \cdot B^{-0.1}$
3	Mild squeezing	$275\ N^{0.33} \cdot B^{-0.1} < H < 450N^{0.33} \cdot B^{-0.1}$ and $J_r/J_a < 0.5$
4	Moderate squeezing	$450\ N^{0.33} \cdot B^{-0.1} < H < 630N^{0.33} \cdot B^{-0.1}$ and $J_r/J_a < 0.5$
5	High squeezing	$H > 630N^{0.33} \cdot B^{-0.1}$ and $J_r/J_a < 0.25$
6	Mild rock burst	$H \cdot B^{0.1} > 1000$ m and $J_r/J_a > 0.5$ and $N > 1.0$

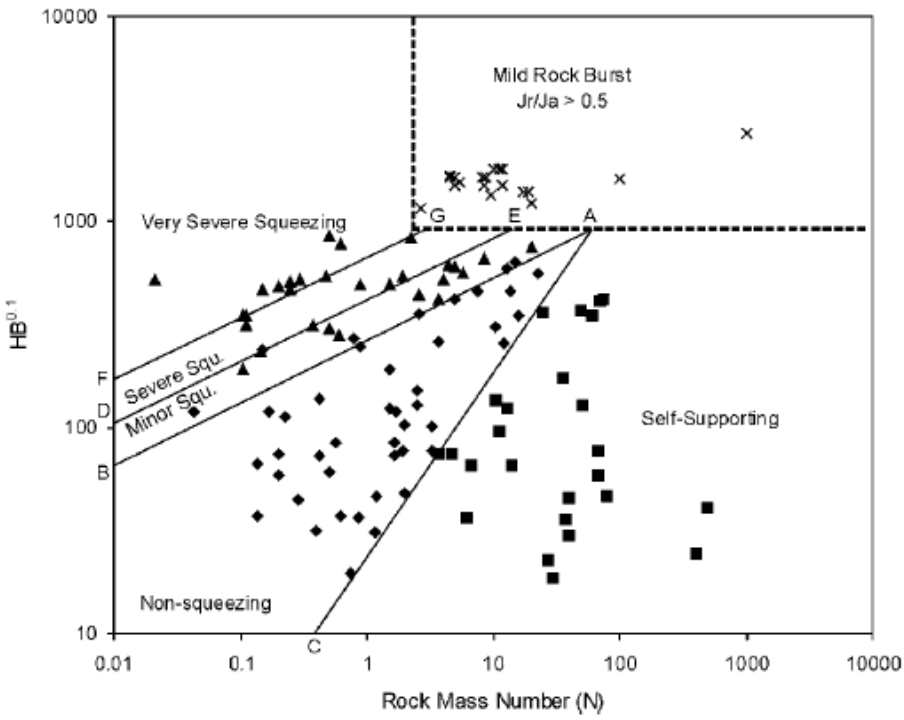


Figure 2-12 Log-Log Plot for prediction of squeezing criteria by Goel et al. (1995)

c. Jimenez et al. (2011) approach

This approach is based statistical analysis of 62 tunnel sections which uses the linear classifier to provide the empirical relation. It provides a log-log space to predict the ground condition either squeezing or Non-squeezing. This also specifies that the empirical relation can be used up to 600-800 meters overburden. The empirical relation for critical overburden is given as:

For Squeezing Ground $H \gg 424.4 \times Q^{0.32}$

For Non − Squeezing Ground $H \ll 424.4 \times Q^{0.32}$

Where H is overburden in meters

2.7.1.2 Empirical Approach for support design

a. Q-System

Barton et al. (1974) proposed a rock mass classification system called "Q-System" at Norwegian Geotechnical Institute. This classification was based on the 200 tunnel and cavern case histories (Singh and Goel, 2011). This system of classification help to quantify the quality of rock mass and with the help of those quantified value, support system for the underground opening can be chosen. Later Grimstad and Barton (1993) updated the Q-System based on 1050 new case histories. This system of rock mass classification defines the rock mass quality based on following six parameters:

- Rock Quality Designation (RQD)
- Joint Set Number (Jn)
- Joint Roughness Number (Jr)
- Joint Alteration Number (Ja)
- Joint Water Reduction Factor (Jw)
- Stress Reduction Factor (SRF)

Using these six parameters, the rock mass quality (Q-value) is given by:

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$$

Furthermore (Singh and Goel, 2011) provides 1260 case records of tunnels to support the strong hold of Q-system as the best method of classification for tunnel support. The value of Q ranges from 0.001 to 1000--lowest representing weak rock mass and highest representing excellent rock mass. Grimstad and Barton (1993) also provided the ground condition and failure that might be seen using the competency factor—ratio of uniaxial compressive strength (σ_c) to in-situ stress (σ_1)—and with the new value of SRF.

For estimation of support system, Barton et al. (1974) introduced a new parameter known as "*Equivalent Dimension*" for unsupported underground opening and given by:

$$De = 2 \times Q^{0.4}$$

This equivalent dimension can also be given as:

$$De = \frac{Limiting\ Span\ or\ Height\ of\ the\ opening\ (in\ meters)}{Excavation\ Support\ Ratio(ESR)}$$

The Excavated Support Ratio (ESR) is a parameter which gives the monetized value of the qualitative aspect of tunnel construction like safety and risk assessment, importance of structure, construction method implemented.

The length of the bolt used is also defined by Barton et al. (1974) as a thumb rule, where the empirical relation for length of the bolt is given by:

For Roof $L = 2 + 0.15 \times W/ESR$

For Tunnel Wall $L = 2 + 0.15 \times H/ESR$

Where, L is the length of the bolt in meters

W is the width of the span of the opening in meters

H is the height of the tunnel wall in meters

Table 2-4 Table of ESR (after Singh and Goel, 2011)

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

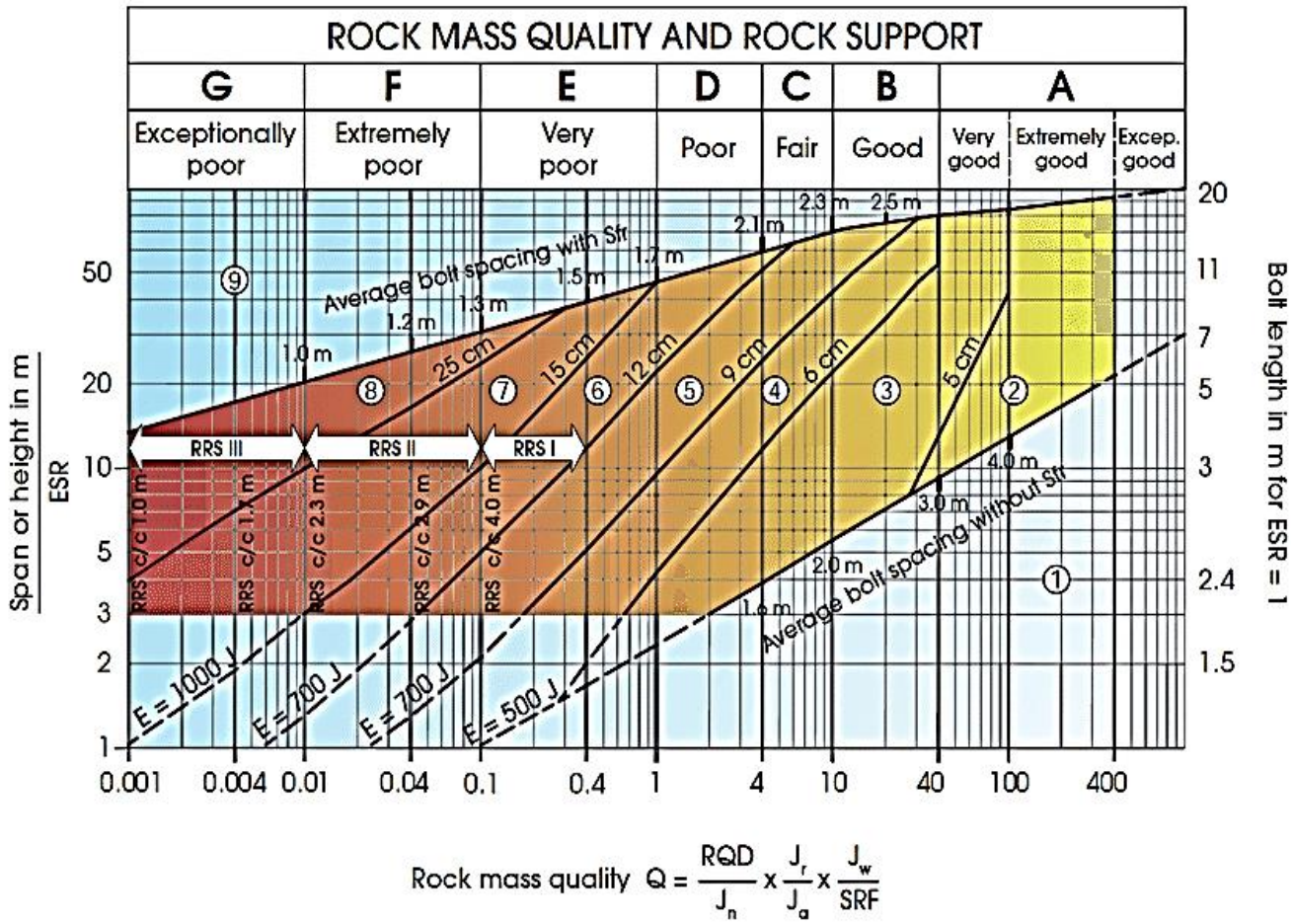


Figure 2-13 Q-Support Chart (Norwegian Geotechnical Institute, 2013)

Calculation of Support Pressure

The ultimate support pressure for tunnel at roof and wall is provided by Barton et al. (1974) by correlating the support capacity of the opening with the rock mass quality given by Q-system of 200 case histories.

For permanent roof support pressure:

$$Pv = \frac{0.2}{Jr} \times Q^{-0.33} \ Mpa$$

For permanent wall support pressure:

$$Pv = \frac{0.2}{Jr} \times Qw^{-0.33} \ Mpa$$

Where, Q_w = wall factor * Q

J_r = Joint Roughness Number

Furthermore, Barton et al. (1974) recommended that if the Joint Set Number (Jn) is less than 3 which means three joint sets or two joint set plus random--is generally defined as the limiting case for three dimension rock block.

For Permanent roof support pressure:

$$Pv = \frac{0.2 \times Jn^{0.5}}{3 \times Jr} \times Q^{-0.33} \ Mpa$$

For Permanent wall support pressure:

$$Pv = \frac{0.2 \times Jn^{0.5}}{3 \times Jr} \times Qw^{-0.33} \ Mpa$$

Table 2-5 Wall factor for respective Q-value (after Singh and Goel, 2011)

Range of Q	**Wall factor Q_w**
>10	5.0 Q
0.1-10	2.5 Q
<0.1	1.0 Q

For the poor rock mass (that is value of Q less than 4) Bhasin and Grimstad recommended support pressure correlation depending upon the span of the underground opening and given as:

$$Pv = \frac{40 \times W}{Jr} \times Q^{-0.33} \ kPa$$

Where, Pv is the Support Pressure in kPa; W is the width of underground opening; and J_r is the Joint Roughness Number

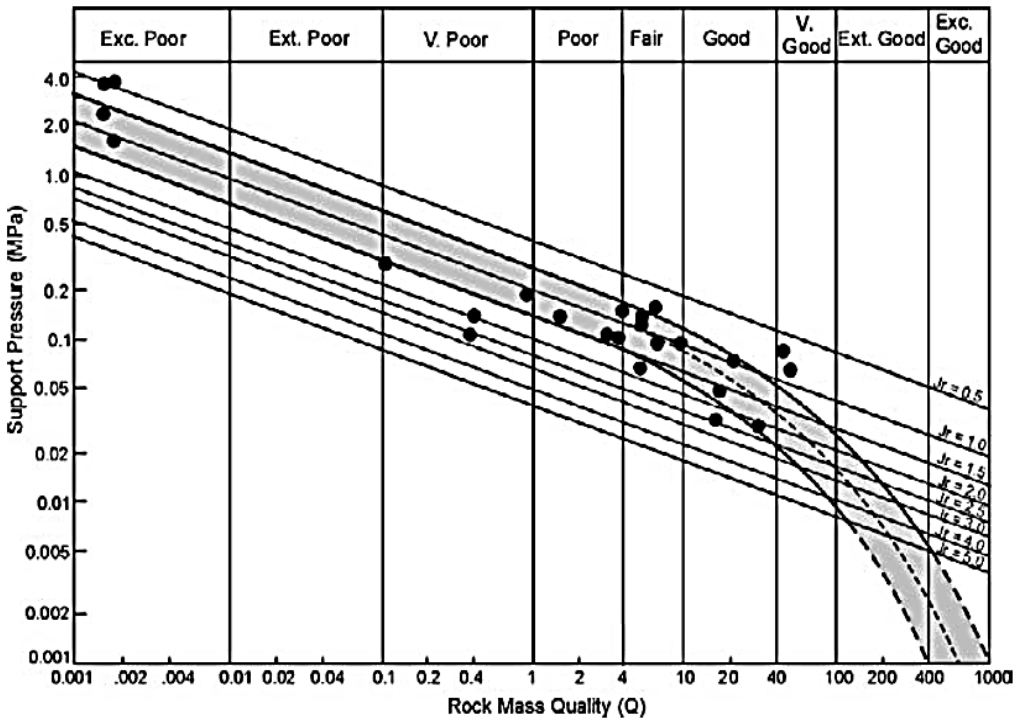


Figure 2-14 Plot of support pressure versus rock mass rating (Q) (after Singh and Goel, 2011)

Singh et al. (1992) provided a correlation of short term rock mass quality (Q') with vertical support pressure by studying 30 instrumental tunnel sections. This empirical relationship also provides the correction factor for overburden, tunnel closure and time factor for time period between excavation and support installation.

$$Pv = \frac{0.2}{Jr} \times Q'^{-0.33} \times f1 \times f2 \times f3 \;\; Mpa$$

$$f1 = 1 + \frac{H - 320}{800} \geq 1$$

$$f3 = log(9.5 \times T^{0.25})$$

Where, Q' is 5 times Q-value (Taken as the short term support pressure just after excavation)

f1 is Correction factor for overburden

f2 is Correction factor for tunnel closure

f3 is Correction factor for time elapsed between excavation and support installation

H is the overburden in meters

T is the time elapsed between excavation and support installation

Table 2-6 Table for correction factor (f2) for tunnel closure (Singh et al., 1992)

S. No.	Ground Condition	Support System	Tunnel Closure (%)	f_w' or f_r'
1.	Non-Squeezing ($H < 350\ Q^{1/3}$)	-	<1	1.0
2.	Squeezing ($H > 350\ Q^{1/3}$)	Very stiff	1-2	>1.80
3.	Moderately Squeezing	Stiff	2-4	0.85
4.	-do-	Flexible	4-6	0.70
5.	Highly Squeezing	Very flexible	6-8	1.15
6.	-do-	Extremely flexible	>8	1.80

For predicting tunnel closure Barton (2008) provided the correlation between Q-value, opening width or height, and competency factor and is given by: (Singh B., Goel R.K, India)

$$\Delta v = \frac{Width}{100 \times Q} \times \sqrt{\frac{\sigma v}{\sigma c}}\ mm$$

$$\Delta v = \frac{Height}{100 \times Q} \times \sqrt{\frac{\sigma h}{\sigma c}}\ mm$$

Where, σv is in-situ vertical stress

σh is in-situ horizontal stress

σc is Uniaxial Compressive Strength of intact rock mass

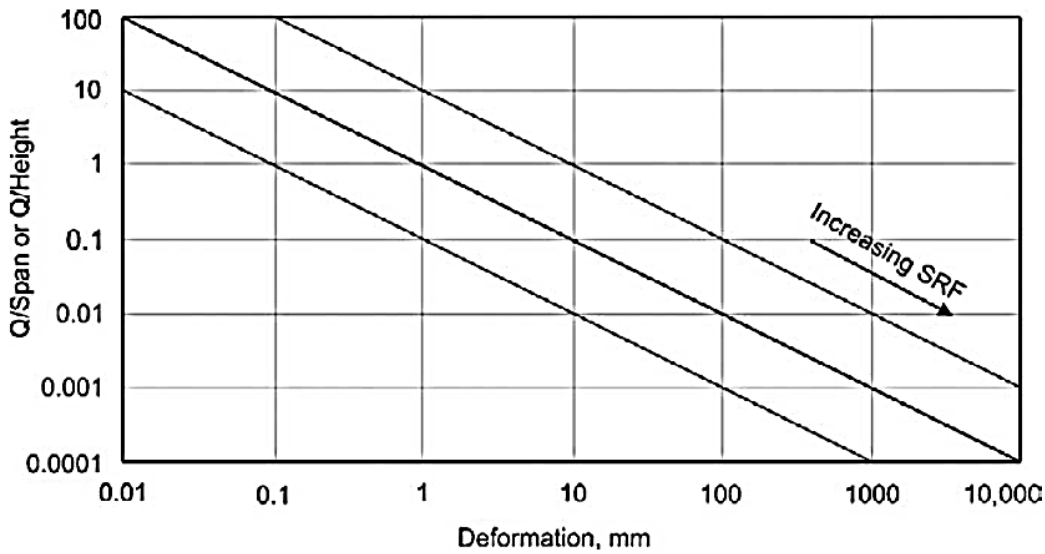


Figure 2-15 Plot of Q/span or Q/Height versus Deformation (mm) (Taken from (Singh B., Goel R.K, 2011)) (after Barton, 2008)

Calculation of support pressure using Rock Mass Number (N)

Goel et al. (1995) provided correlation between rock mass number (N), overburden (H) in meters, tunnel radius (R) in meters and tunnel closure by studying eight tunnels in Himalayas. The support pressure for Non-squeezing and squeezing ground are given below.

For Non − Squeezing Ground $$P = \left(\frac{0.12 \times H^{0.1} \times R^{0.1}}{N^{0.33}}\right) - 0.038$$

For Squeezing Ground $$P = \left(\frac{f(N)}{30}\right) \times 10^{\left(\frac{H^{0.6} \times R^{0.1}}{50 \times N^{0.33}}\right)}$$

Where, P is Support pressure in MPa

f(N) is the correction factor for tunnel closure and given by the figure below.

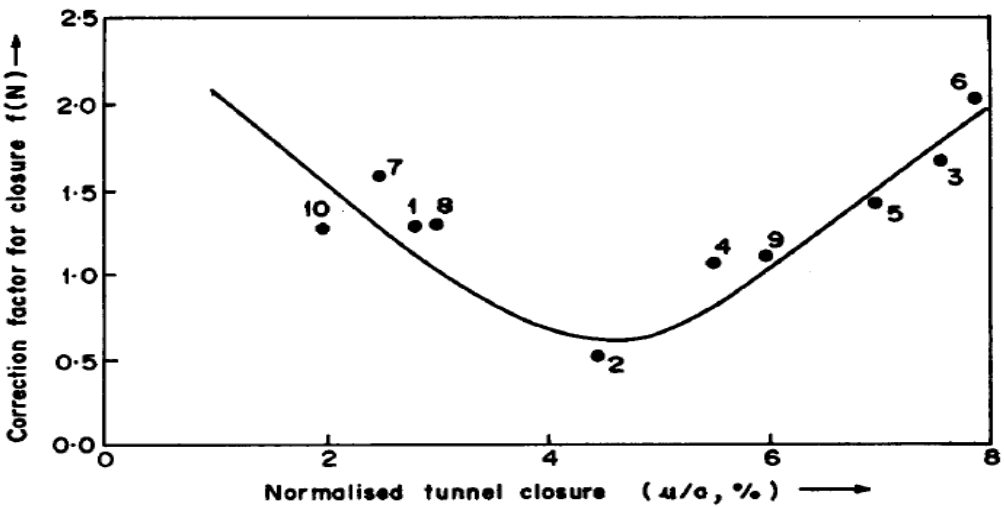


Figure 2-16 Plot for correction factor for tunnel closure (Goel et al. (1995))

b. Rock Mass Rating

Rock Mass Rating is rock mass classification technique developed by Bieniawski in 1973 A.D. This classification was founded upon the experiences he acquired in shallow tunnels in sedimentary rock (Singh B., Goel R.K, India). This classification uses six parameters to quantify the rock mass in their combinations. These six parameters are given below:

1. Uniaxial compressive strength(UCS) of intact rock
2. Rock Quality Designation
3. Joint condition
4. Joint spacing
5. Joint orientation
6. Groundwater Condition

Bieniawski (1989) provided the support condition guidelines under which support condition is defined for underground openings. This guideline is defined for the tunnel with horseshoe cross-section with 10m span where the vertical stress is less than 25 Mpa and the construction method employed is drill and blast tunneling.

Table 2-7 Correction factor for tunnel closure (f(N)) (Geol et al. (1995))

S. No.	Degree of Squeezing	Normalized Tunnel Closure, %	Correction Factor, f(N)
1.	Very Mild Squeezing ($270\ N^{0.33}\ B^{-0.1} < H < 360\ N^{0.33}\ B^{-0.1}$)	1-2	1.5
2.	Mild Squeezing ($360\ N^{0.33}\ B^{-0.1} < H < 450\ N^{0.33}\ B^{-0.1}$)	2-3	1.2
3.	Mild to Moderate Squeezing ($450\ N^{0.33}\ B^{-0.1} < H < 540\ N^{0.33}\ B^{-0.1}$)	3-4	1.0
4.	Moderate Squeezing ($540\ N^{0.33}\ B^{-0.1} < H < 630\ N^{0.33}\ B^{-0.1}$)	4-5	0.8
5.	High Squeezing ($630\ N^{0.33}\ B^{-0.1} < H < 800\ N^{0.33}\ B^{-0.1}$)	5-7	1.1
6.	Very High Squeezing ($800\ N^{0.33}\ B^{-0.1} < H$)	>7	1.7

Rock Bolt

25mm diameter of rock bolt with the length and the spacing of the rock bolt obtained by the empirical relation given by Lowson and Bieniawski (2013).

$$if\ 20 < RMR \leq 85 \qquad S = \left(0.5 + 2.5 \times \frac{RMR - 20}{65}\right)$$

$$if\ \ 10 < RMR \leq 20 \qquad S = 0.25 + \frac{(RMR - 10)^{1.5}}{140}$$

$$if\ RMR\ \leq 10 \qquad S = 0.25$$

Where, S is the spacing of the rock bolt in meters

The length of the rock bolt to be used is given by

$$Span = \frac{(L + 2.5)^{\frac{RMR+25}{52}}}{3.6}$$

Where, the span defines the width of the opening in meters and L is the length of the rock bolt to be used in meters.

For lower value of RMR specifically for the support Category R-III, R-IV and R-V, the obtained value of length of the rock increases beyond acceptable limit. So, the limiting value of the length of the rock bolt is taken as 6m which also specified by the graph given by Lowson and Bieniawski (2013).

Table 2-8 Table showing the classification of support for 10m Horseshoe shape tunnel with drill and blast method. (Singh and Goel, 2011)

		Supports		
Rock mass class	**Excavation**	**Rock Bolts (20mm diameter, fully grouted)**	**Conventional shotcrete**	**Steel sets**
Very good rock RMR = 81-100	Full face, 3m advance	Generally, no support required except for occasional spot bolting		
Good rock RMR = 61-80	Full face, 1.0-1.5 m advance; complete support 20 m from face	Locally, bolts in crown 3m long, spaced 2.5m, with occasional wire mesh	50mm in crown where required	None
Fair rock RMR = 41-60	Heading and benching; 1,5-3 m advance in 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
Poor rock RMR = 21-40	Top heading and bench; 1.0-1.5m advance in top heading; install support concurrently with excavation 10m from face	Systematic bolts 4-5m long, spaced 1-1.5m in crown and wall with wire mesh	100-150mm in crown and 100mm in sides	Light to medium ribs spaced 1.5m where required
Very poor rock RMR < 20	Multiple drifts; 0.5-1.5m advance in top heading; install support concurrently with excavation; shotcrete as soon as possible after blasting	Systematic bolts 5-6m long, spaced 1-1.5m in crown and walls with wire mesh; bolt invert	150-200mm in crown, 150mm in sides, and 50mm on face	Medium to heavy ribs spaced 0.75m with steel lagging and forepoling if required; close invert

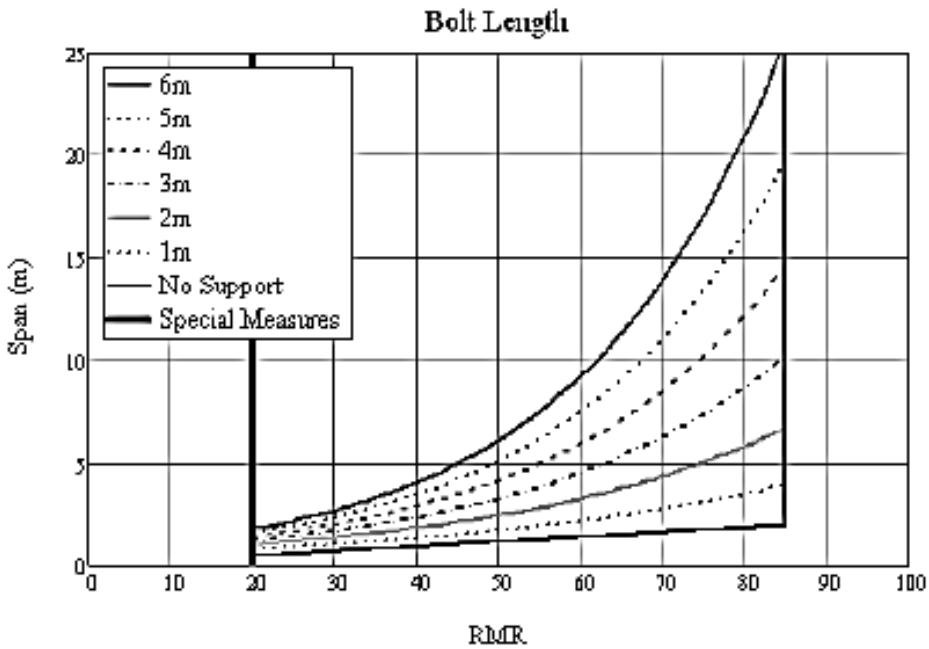


Figure 2-17 Plot between Span (m) and RMR for estimating the Rock Bolt length by Lowson and Bieniawski (2013) (after Bieiawski, 2013)

Shotcrete

Lowson and Biensiawsk(2013) takes the thickness of shotcrete as the function of RMR and span of opening(in meters).

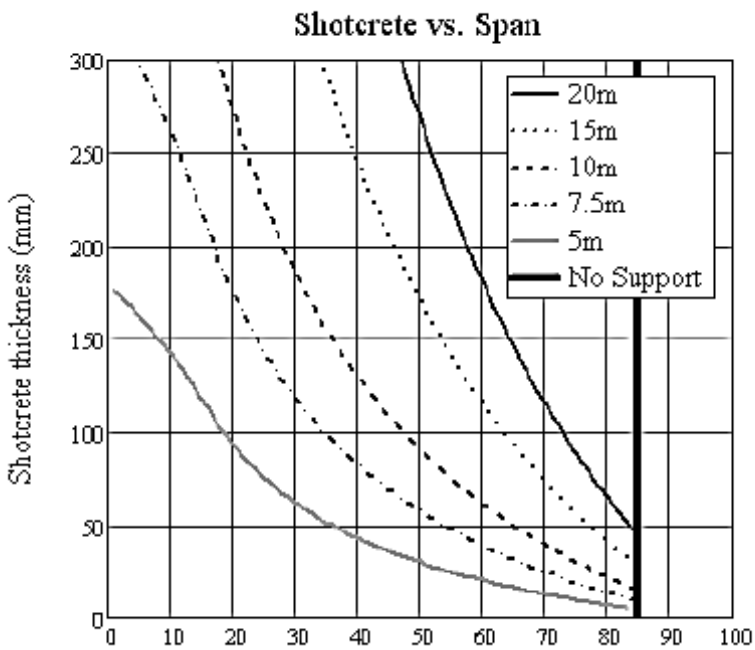


Figure 2-18 Graph for obtaining thickness of shotcrete by Lowson and Bieiawski (2013) (after Bieiawski, 2013)

When using the steel ribs for weak ground shotcrete is placed between the steel rib and excavated rock surface. The thickness of shotcrete (t) to be placed is given by Lowson and Biensiawski (2013):

$$t = \sqrt{\frac{3 \times S^3 \times D \times \gamma r \times \gamma f}{4 \times f}} \; mm$$

Where, S is the spacing between the steel ribs, D is the rock density and γr, γf are the partial factor on loading and shotcrete flexural strength.

The value of partial factor on loading and shotcrete flexural strength is taken as 1.5 and 1.5 respectively and the flexural strength of the rib is taken as 5 MPa as specified by Lowson and Biensiawski (2013)

Calculation of support Pressure

Goel and Jethwa (1991) have provided the empirical equation by correlating RMR value, overburden, and width of the underground opening and support pressure of 30 instrumental tunnels of India. The empirical correlation is given by:

$$Pv = \frac{7.5 \times B^{0.1} \times H^{0.5} - RMR}{20 \times RMR}$$

Where, B is the width of the underground opening in meters

H is the overburden in meters

Pv is the short term support pressure given in Mpa

Furthermore, Goel et al. (1995) proposed the empirical equation of estimating the support pressure of underground opening for both squeezing and non- squeezing ground condition. This empirical relation is given by:

For Non-Squeezing ground:

$$Pv = 2.32 - 0.035 \times RMR + 0.001 \times H + 0.03 \times a \; Mpa$$

For squeezing ground:

$$Pv = ((f(RMR)/12) \times 10^{(1.8 \times H^{0.4} \times a^{0.1}/RMR^{1.2})} \; Mpa$$

Table 2-9 Correction factor for tunnel closure f(RMR) (Goel et al., 1995)

S. No.	Degree of Squeezing	Tunnel Closure (%)	Correction Factor, f
1.	Very Mild Squeezing	1-2	1.35
2.	Mild Squeezing	2-3	1.1
3.	Mild to Moderate Squeezing	3-4	0.73
4.	Moderate Squeezing	4-5	0.57
5.	High Squeezing	5-7	0.78
6.	Very High Squeezing	>7	1.1

2.7.1.3 Semi Empirical Approach

Hoek and Marinos (2000) provided semi empirical relation for estimating tunnel strain using curve fitting technique. This method assumes that the analysis to be based on the simple closed-form solution with the tunnel geometry being circular in a hydrostatic stress field. The support is considered to be act uniformly over the entire boundary of the underground opening.

$$Strain(\varepsilon) = [0.002 - 0.025 \times (Pi/P)] \times (\sigma cm/P)^{[2.4\times(Pi/P)-2]}$$

$$\sigma cm = (0.0034 \times mi^{0.8}) \times \sigma ci\{1.029 + 0.025 \times e^{(-0.1mi)}\} \times GSI$$

Where, Strain is the obtained with respect to diameter of the underground opening

Pi is the internal support pressure

P is the in-situ stress given by depth multiplied by unit weight

σcm is the Uniaxial Compressive Strength of the Rock mass

mi is the rock mass constant defined by friction characteristics of rock mass

σci is the Uniaxial Compressive Strength of Intact Rock mass

GSI is Geological Strength Index

For unsupported condition, the above equation for strain can be obtained by replacing support pressure (P) equal to zero.

The subsequent tunnel closure was obtained for four tunnel sections of MBKHEP using the Hoek and Marinos (2000) for each tunnel support classification for support pressure with F.O.S 1, 1.5 and 2 respectivley shown in Figure 3-11, Figure 3-12 and Figure 3-13 and their corresponding data of measurement is given in Table 3-17. Again a comprehensive plot given in Figure 3-13, Figure 3-14 and Figure 3-15 show the comparison with actual measured deformation and closure obtained using Shrestha and Panthi (2018).

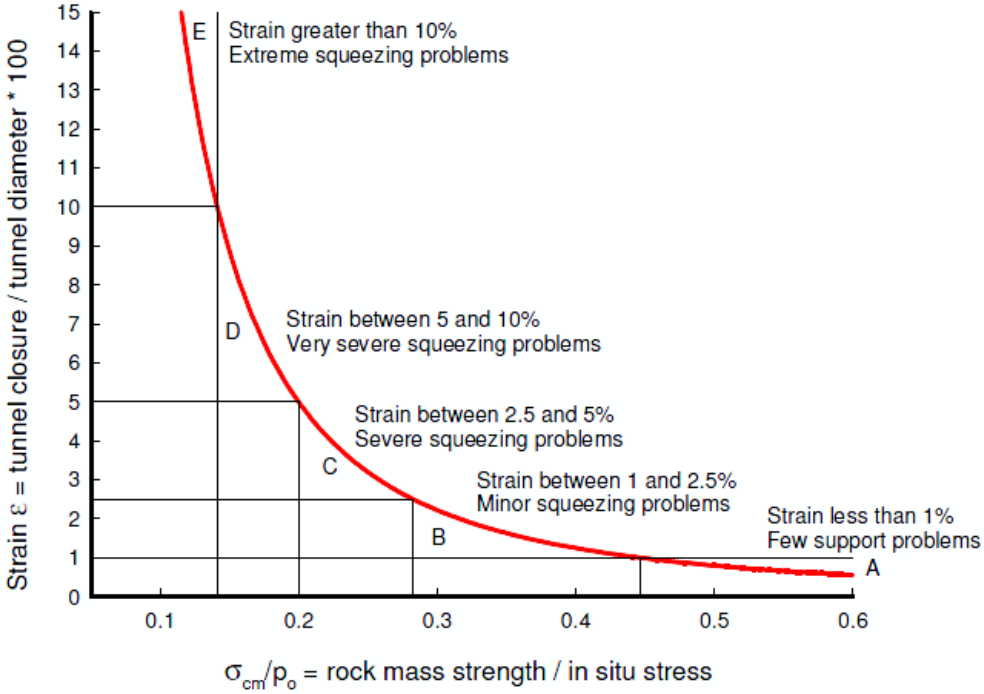


Figure 2-19 Curve showing the plot between strain and σcm/P where the tunnel is not supported

Strain Calculation using Shrestha (2015)

Shrestha and Panthi (2015) provided a relation of tunnel strain with the shear modulus of rock mass (G), support pressure (Pi), in-situ stresses by studying 77 section of Kaligandaki hydroelectric tunnel. Later Shrestha and Panthi (2018) studied 3 instrumental tunnels of Nepal Himalyas and gave then generalized empirical relation for tunnel closure. This correlation so obtained incorporates the underground anisotropy and shows its influence in the convergence of the underground opening. The empirical relation is given as:

For instantaneous tunnel closure:

$$\varepsilon ic = 3065 \times \left(\frac{\sigma v \times (1+k)/2}{2G \times (1+Pi)}\right)^{2.13}$$

For final tunnel closure:

$$\varepsilon fc = 4509 \times \left(\frac{\sigma v \times (1+k)/2}{2G \times (1+Pi)}\right)^{2.09}$$

Where, εic and εfc represents the tunnel strain for instantaneous and final closure respectively.

Pi is the support pressure (MPa)

G is the shear modulus of rock mass (MPa)

σv is the vertical in-situ stress acting in the opening(MPa)

k is in-situ stress ratio

The relation for final tunnel strain was used to calculate the closure and the final closure so obtained were compared with actual tunnel convergence data obtained from the MBKHEP site itself. The comparison is done for different support pressure giving the factor of safety of 1, 1.5 and 2 where their plot is given in Figure 3-14, Figure 3-15 and Figure 3-16for designed support system, defined support system using Q-Chart Guidelines and defined support system using RMR guidelines respectively along with closure obtained using semi empirical relation given by Hoek and Marinos (2000). Furthermore, the final tunnel closure of the different support system were estimate using the relation given above and compared for support pressure with factor of safety 1 and the result plot is shown in Figure 3-6. Moreover, individual plot of convergence for different F.O.S is given in Figure 3-8, Figure 3-9 and Figure 3-10which compares the convergence obtained from the different support definitions.

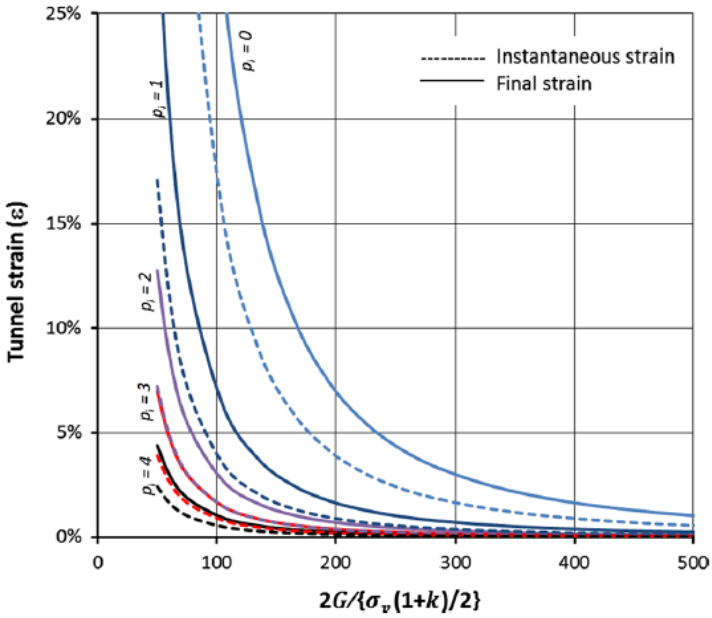


Figure 2-20 Plot of Tunnel strain versus in-situ stress and shear modulus of rock mass for different support pressure. (Shrestha and Panthi, 2018)

<u>Calculation of Support pressure and stiffness of support</u>

The relation used to calculate the stiffness and maximum support pressure of different support system given in the Table 3-12, Table 3-13, Table 3-14 and Table 3-15 according to RocScience is given below as:

- **<u>For Steel Sets</u>**

$$Pssmax = \frac{A \times \sigma ys}{S \times R}\ MPa$$

$$Kss = \frac{E \times A}{S \times R^2}\ Mpa$$

Where, *Pssmax* and *Kss* are maximum support pressure and stiffness of support

A is the cross sectional area of the section in sq. meters

E is the young's modulus of steel (MPa)

σys is yield strength of the steel (MPa)

R is the radius of the tunnel (m)

- **<u>For Shotcrete</u>**

$$Psmax = \frac{\sigma cc}{2} \times \left(1 - \frac{(R-T)^2}{R^2}\right) Mpa$$

$$Ksc = \frac{E}{2} \times \frac{(R^2 - (R-T)^2)}{(1-\upsilon^2) \times (R-T) \times R^2} Mpa$$

Where, σcc is the UCS of concrete or shotcrete (MPa)

E is the young's modulus of elasticity of concrete (MPa)

υ is the Poisson's ratio of the concrete or shotcrete

T is the thickness of shotcrete or concrete in meters

R is the radius of tunnel in meters

- **<u>For Rock Bolts</u>**

$$Psbmax = \frac{T}{Sc \times Sl} Mpa$$

$$Ksb = \frac{E \times \Pi \times D^2}{4 \times L \times Sc \times Sl} Mpa$$

Where, T is the ultimate bolt load obtained from pull out test (MN)

D is the diameter of the bolt in meters

E is the young's modulus of bolt (MPa)

Sc is the circumferential spacing of the rock bolt (m)

Sl is the longitudinal spacing of the rock bolt (m)

L is the length of the rock bolt (m)

Using these relations above the maximum support pressure is obtained for six different sections of MBKHEP and the comparison of support classification where made in-terms of maximum support pressure for these different sections and the result is shown in Figure 3-7.

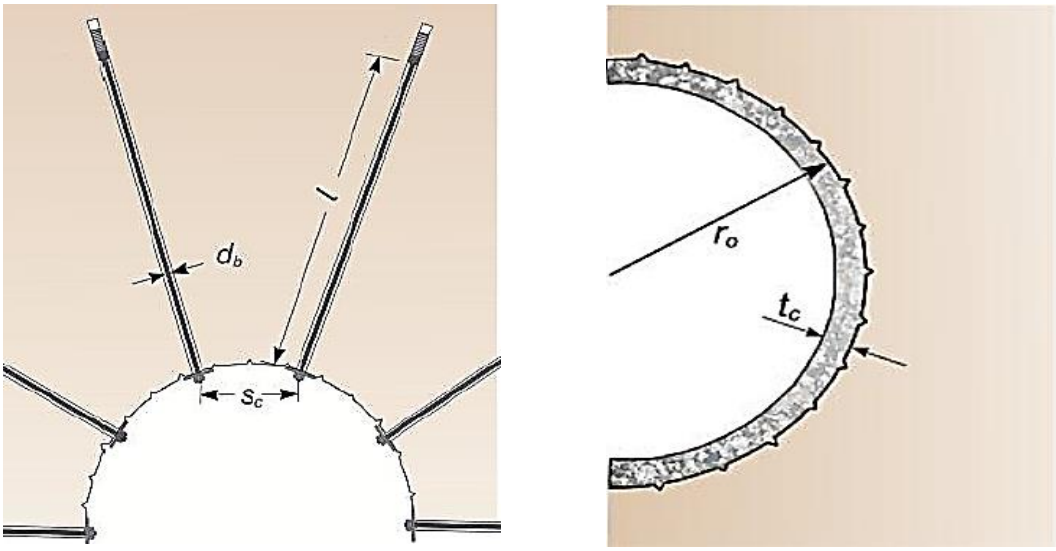


Figure 2-21 Rock Bolting (Left) and Shotcrete (Right) (Source: Rocscience)

2.7.2 Analytical Approach

The Convergence Confinement Method is the process of determining the support pressure exerted to the support which is placed at the near vicinity of the tunnel face i.e behind the tunnel face. If a support is installed very near to the tunnel face, it doesn't carry the full load that it has been designed for. It is because of the reason that some part of the load is being carried by the face of the tunnel which is termed as the face effect. But when the face moves away from the support, the support starts to carry more load and eventually it will carry the designed load at the future.

The Convergence Confinement Method is used in our Project in order to determine the support pressure to be provided at the different sections of the tunnels taken into consideration. The results obtained from the use of this method will be used to compare and validate with the results obtained from Empirical and Numerical Analysis.

2.7.2.1 Preparation of Excel File

In order to conduct the Analytical Method, Excel file containing different parameters required to determine the Support Pressure of various sections was prepared The excel file was prepared in such a way that changing a few values/parameters (which differs from every section to section) all other parameters or values are automatically calculated. The graph of CCM is automatically generated with the help of this file.

The following figure shows a sample for excel file where the shaded box represents the input box. It means that the parameters should be entered by user. The parameters within the white boxes are calculated automatically. So when the values are entered in the input boxes, all the parameters shown above along with the parameters required for Longitudinal Deformation Profile, Ground

Reaction Curve and the Support Characteristics Curve are automatically calculated along with the respective graph.

Chainage	
OverBurden	
Dimension	
R(m)	
H(m)	
σ_0 (MPa)	
ν	
L(m)	
Q	
Ei(Gpa)	
Erm(Gpa)	
Disturbance Factor	
Friction Angle	

k	
σ_v (MPa)	
σ_h (MPa)	

m_i	
GSI	
mb	
s	
a	
σ_{ci} (MPa)	
S_0	
P_i^{cr} (MPa)	
p_i^{cr} (MPa)	
$C(\sigma_{ci})$	
E_{rm} (GPa)	
G_{rm} (GPa)	
Ψ (degree)	
K_Ψ	

For Bolts	
bolt provided	
l (m)	
d_b (mm)	
E_b (Gpa)	
Q_b (m/MN)	
T_{bf} (MN)	
sc (m)	
sl (m)	
Ps_{max} (Mpa)	
k_s (Mpa/m)	
u_{rbmax} (mm)	

For Shotcxrete	
σ_{cc}(Mpa)	
ν_c	
t_c (mm)	
E_c (GPa)	
P_s^{max} (MPa)	
K_s (MPa/m)	
u_r^{max} (mm)	

For Steel Sets	
σys(Mpa)	
Es(Gpa)	
As	
Sl	
ro	
Pss Max	
Kss	
Ur max	

Combination of Supports	
Ks(MPa/m)	
urmax(mm)	
Psmax(Mpa)	

For Lattice Girder Steel Properties	
lattice girder provided	

Figure 2-22 Sample of Excel file prepared for CCM

The definition of the parameters used in the above figure is provided in the Table 2-10.

Table 2-10 Parameters used for Calculation of CCM

Symbol	Designation
R	Radius of Tunnel
A	Radius of Tunnel
L	Length of support from tunnel face
H	Overburden
V	Poissons Ratio
Q	Q Value
E_i	Modulus of Elasticity of Intact Rock
E_{rm}	Modulus of Elasticity of Rock Mass
Φ	Friction Angle
Ψ	Dilation Angle
m_i	Rock mass parameter
GSI	Geological Strength Index
m_b	Rock mass parameter
S	Rock mass parameter
C_i	Uni-axial Compressive Strength
Pi_{cr}	Scaled Internal Pressure
Pi_{cr}	Internal Pressure
G_{rm}	Shear Modulus of Rock

Symbol	Designation
Qb	Deformation Load Constant
Tbf	Ultimate Load Obtained from Pullout Test
sc	Ciucumferential Bolt Spacing
sl	Longitudinal Bolt Spacing
Pbmax	Maximum Support Pressure (Bolt)
Kb	Stiffness of Bolt
Urbmax	Deformation of Bolt
Sig cc	Unconfined Compressive Strength of Concrete/Shotcrete
vc	Poisson's ratio of Shotcrete/Concrete
tc	Thickness of Shotcrete/concrete
Ec	Young's Modulus of Shotcrete / concrete
Ps max	Max Support Pressure due to shotcrete
Ks	Stiffness of Shotcrete
Ur max	Deformation of Shotcrete
Ks_all	Stiffness of Combined Support
ur max_all	Deformation of Combied Support
Ps max_all	Support Pressure due to combined Support
Es	Young's modulus of Steel

L	Length of Rock Bolt
d_b	Diameter of RockBolt
E_b	Young's Modulus of Rock Bolt Material

Sig ys	Yeild strength of steel
As	Cross Sectional Area of Steel Section
S	Spacing of steel sets along the tunnel axis

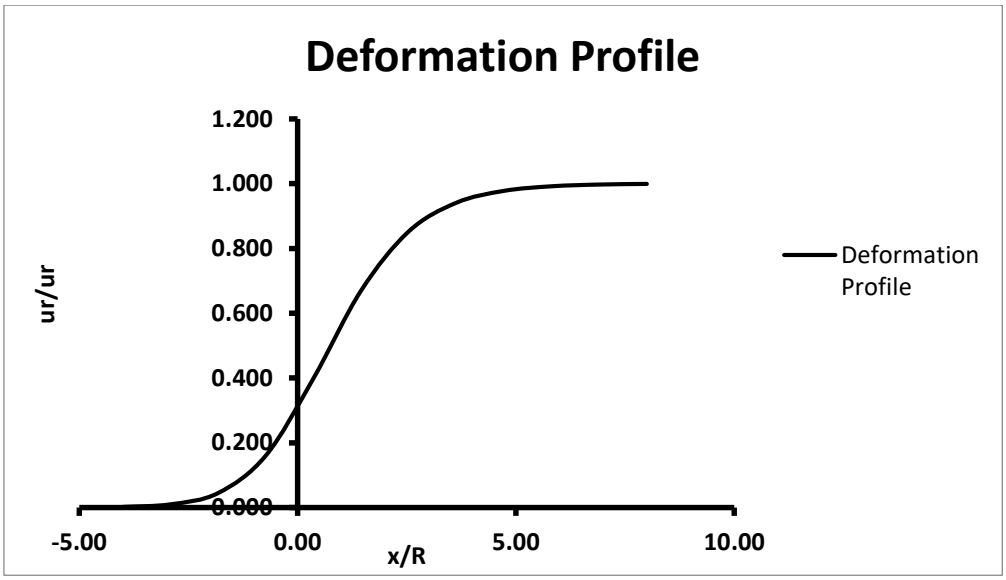


Figure 2-23 Sample of Deformation Profile of MBKHEP at Section 2886.6m

This figure shows the deformation profile of the ground .This figure is obtained by plotting x/R in x axis and ur/ur max in Y- axis (the values are provided later in Table 3-20). Ahead of the face there is low to no deformation. As the theory implies, at the face there is about 30% of deformation and the deformation increases as the excavation proceeds further behind the face. The curve stops at a certain point showing that the deformation shall not increase further though the distance from the face increases.

Figure 2-24 above shows the Longitudinal Deformation Profile (LDP), Ground Reaction Curve (GRC) and Support Characteristics Curve (SCC) of the tunnel section at HRT of MBKHEP at the chainage 2+866.6m (the values used for plotting are provided in Table 3-20, Table 3-21 and Table 3-22). Initially the internal pressure in the tunnel is same as the insitu stress in the Point A (as shown in the figure). But as the tunnel face moves further away, the internal pressure is released. The rock mass exhibits the elastic behavior till it reaches the internal pressure value of 3.12 MPa as indicated by point D in the figure. Beyond point D, the rock mass exhibits plastic behavior. The ordinate of point B (intersection of GRC and SCC) represents the designed support pressure while the abscissa represents the deformation or the convergence of support and the ground

together. Similarly, the ordinate of point C represents the maximum support pressure that the support can bear before failure and the abscissa of point C represents maximum deformation the support can undergo. The point E in the graph above represents the unsupported closure of the tunnel.

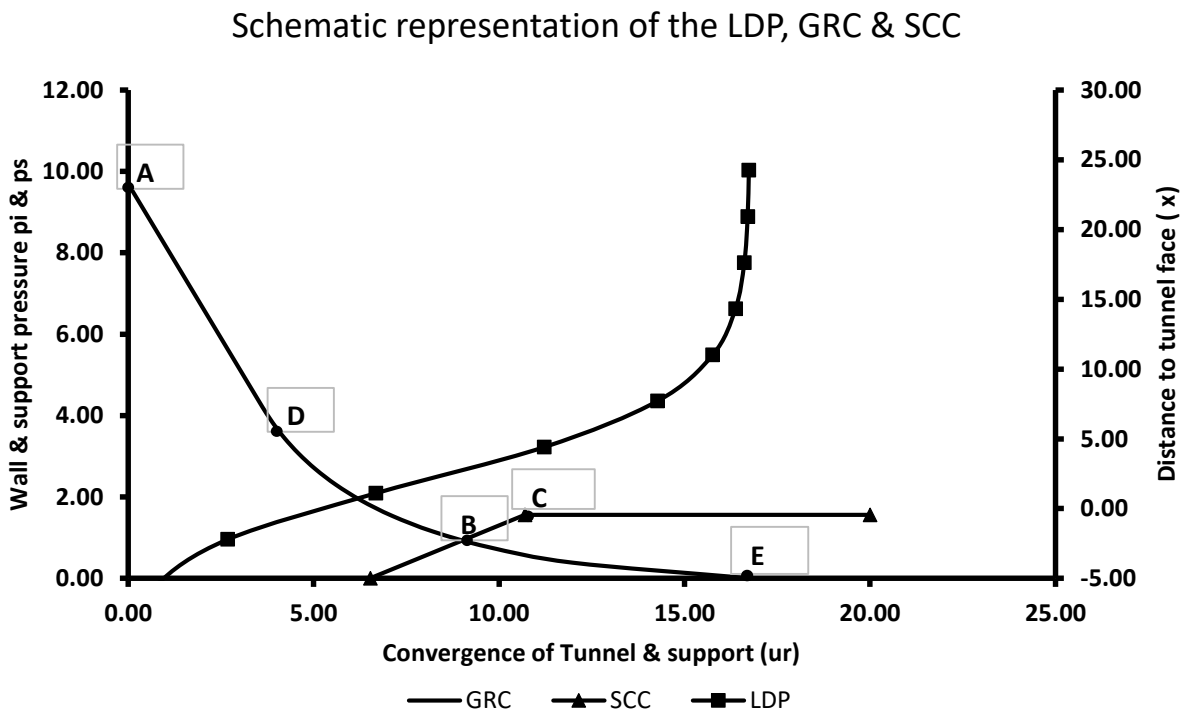


Figure 2-24 Sample of plot of CCM from section at 2886.6m of MBKHEP

A support system consisting of shotcrete of thickness 12 cm and pattern bolting with 3.8m length and 25mm dia. bolts with circumferential and longitudinal spacing 2m is provided for the section. The designed support pressure is found to be 0.8 MPa and the capacity of the supported is calculated to be 1.56 MPa. Hence, the factor of safety is found to be 1.95.

CHAPTER 3 DESCRIPTION OF CASE STUDIES

3.1 General

Topography regime and abundance of water resources favors Nepal as a high potential for development of Hydropower projects. The steep terrain and fast flowing rivers fulfills the basic requirements. But due to the mountainous topography, it is difficult to divert the water to the feasible site. As such underground structures like tunnels and caverns provide economic and safe solution. For the study of tunnels for this project, hydropower lying in middle Himalayan region have been selected. A brief description of the selected sites have been provided in the following section.

3.2 Case Studies

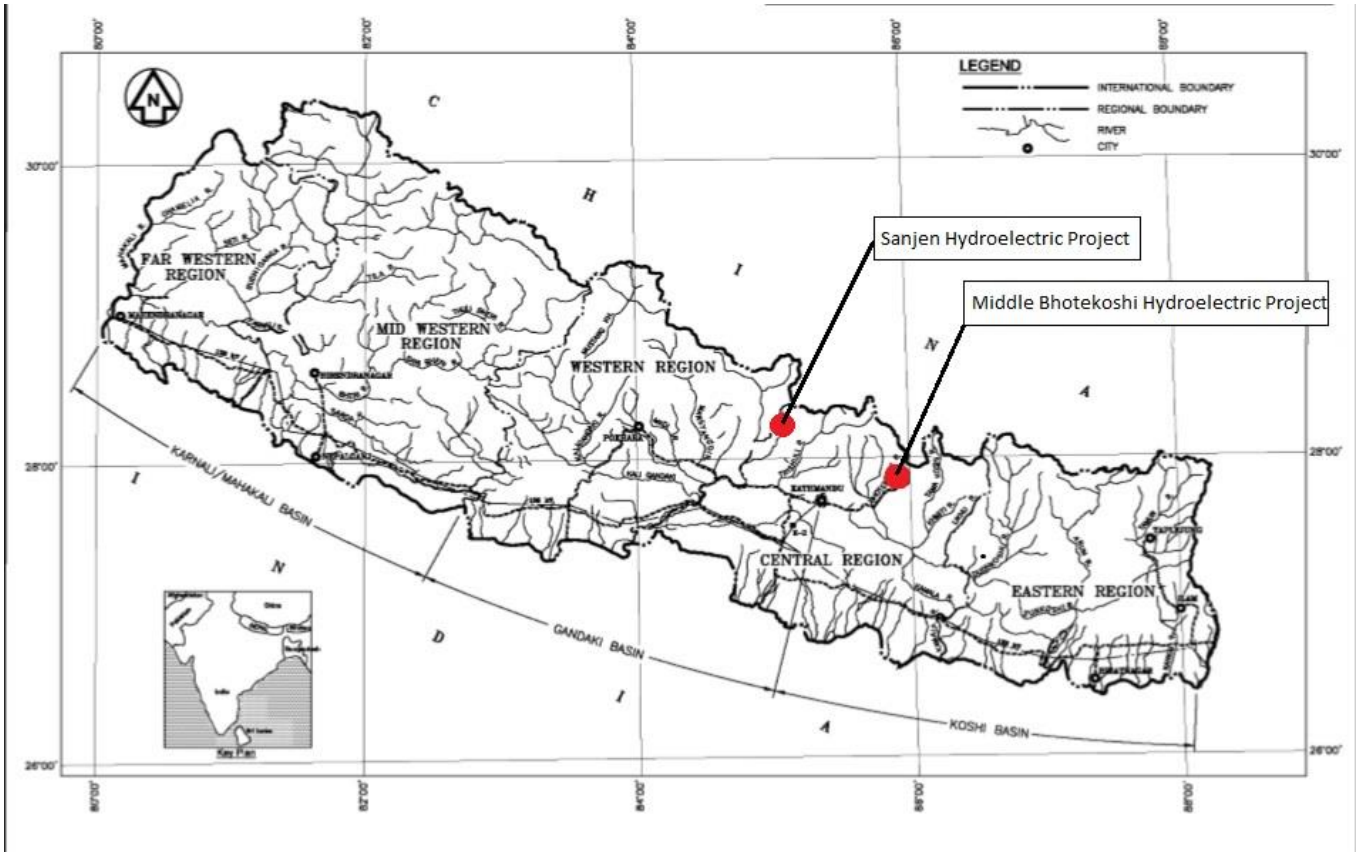


Figure 3-1 Location of Project Sites

3.2.1 Middle Bhotekoshi Hydroelectric Project (MBKHEP)

3.2.1.1 Project Description

The Middle Bhotekoshi Hydroelectric Project is located in Sindhupalchowk District of Bagmati Zone about 101 km road head distance on Araniko Highway towards north-east of Kathmandu. The Project is basically a run-of-river type scheme having the capacity of 102 MW with the design discharge of 50.8m^3/sec at Q40 exceedance and available gross head of 235m and a net head of 222m. The headworks site is located about one hundred meters downstream from the confluence of Chaku khola and Bhote Koshi River. The project has a gated weir type designed for 100 years return period and encompasses downstream stilling pool as energy dissipating structure. The desanding basin provided is of triple type chamber and the water is taken towards the powerhouse situated at Jambo, Sindhupalchok using headrace tunnel. The total headrace tunnel length is 7124 m from headrace inlet to the surge tank.

An inverted "D" shaped, 5.7 m x 5.7 m size and 7124 m long headrace tunnel is the major structure of the project. The tunnel passes through the rough hill on the left bank of the Bhotekoshi River at an average elevation of 1165 m. The intake portal is located near the Chaku Bazaar and the outlet portal above the Khoran village. The present mapping shows that the headrace tunnel will encounter different types of rock masses as shown in Table 3-1

In total, 1350 m of headrace tunnel length will lie on Quartzite rock, 1070 m of it lie on Dolomite rock and the remaining 4780 m length lie on different type of Phyllitic schist rock

3.2.1.2 Geology of the Tunnel

Geologically, the project area lies in the Lesser Himalayan Unit in northern part of the Central Development Region of Nepal. The main rock types in the project area are phyllitic schist, quartzite and dolomite. The well-known thrust zones, namely the Main Central Thrust (MCT) and the Main Boundary Thrust (MBT) are located about 10km to the north and more than 100km to the south of the project area respectively.

Major rock type in the headrace tunnel alignment is phyllitic schist. In different places along the headrace tunnel alignment, almost 20 m thick beds of schistose quartzite are present interbedded in the phyllitic schist rock. Maximum rock cover is present at chainage 2173 m which is about 825m. Similarly, minimum rock cover of about 50 m lies immediately downstream of the audit and headrace tunnel intersection.

Table 3-1 Distribution of Rock Types along Headrace Tunnel and Audit (after MBKHEP Feasibility Report, n.d.)

S.N.	Tunnel	Chainage (m)	Rock Type
1	Headrace	0 - 50	Quartzite
		50 - 1120	Dolomite
		1120 - 2420	Quartzite
		2420 - 2480	Talcose Phyllitic schist
		2480 - 7198	Phyllitic schist
2	Adit	Entire length	Phyllitic schist

Table 3-2 Rock Mass Classification as observed in MBKHEP HRT

Chainage From-to (m)	Length (m)	RMR Value	Barton's Q	Classification	Description
0-50	50	45-57	2.9	III	Fair
50-150	100	55	2.5	III-IV	Fair-Good
150-880	730	64	18.8	II	Good
880-1300	420	29	1.4	IV	Poor
1300-2350	1050	65	9.36	II-III	Good-Fair
2350-2550	150	17	0.8	V	Very Poor
2550-3900	1350	58	7.1	III	Fair
3900-5300	1400	44	3.1	III-IV	Fair-Poor
5300-5560	260	29	1.3	IV	Poor
5560-7000	1440	54	6.6	III	Fair
7000-S/T		44	6	III	Fair

Talking about the quartzite found at the project site, it is of fine grained nature and generally maintains the steep cliffs and are thin to medium bedded. Whilst dolomite found is a bluish grey in color with thin to medium foliation. Furthermore, the soft talcose phyllite rock with green color was found between the phyllitic schist and quartzite rock. For which the effective cohesion was estimated to be in the range of 0 to 10 KPa. It can be clearly seen above form the Table 3-1 phyllitic schist contributes the most of geology along the tunnel with thick foliation bedding. According to the MBKHEP feasibility study, statistical analysis were done and a detailed mapping of the joint was obtained where it showed 3 tunnel joint set along the tunnel. The headrace tunnel

section are divided into 2 sections (that is namely upstream from audit and downstream of the audit) and their statistical analysis is given below in the table:

Table 3-3 Showing Orientation of the Joint sets found in MBKHEP HRT

Joint Sets	**Orientation of Joint sets**	
	Upstream of the audit	**Downstream of the audit**
Foliation Plane	12°/054°	15°/058°
Joint Set 1	75°/175°	80°/191°
Joint Set 2	80°/245°	

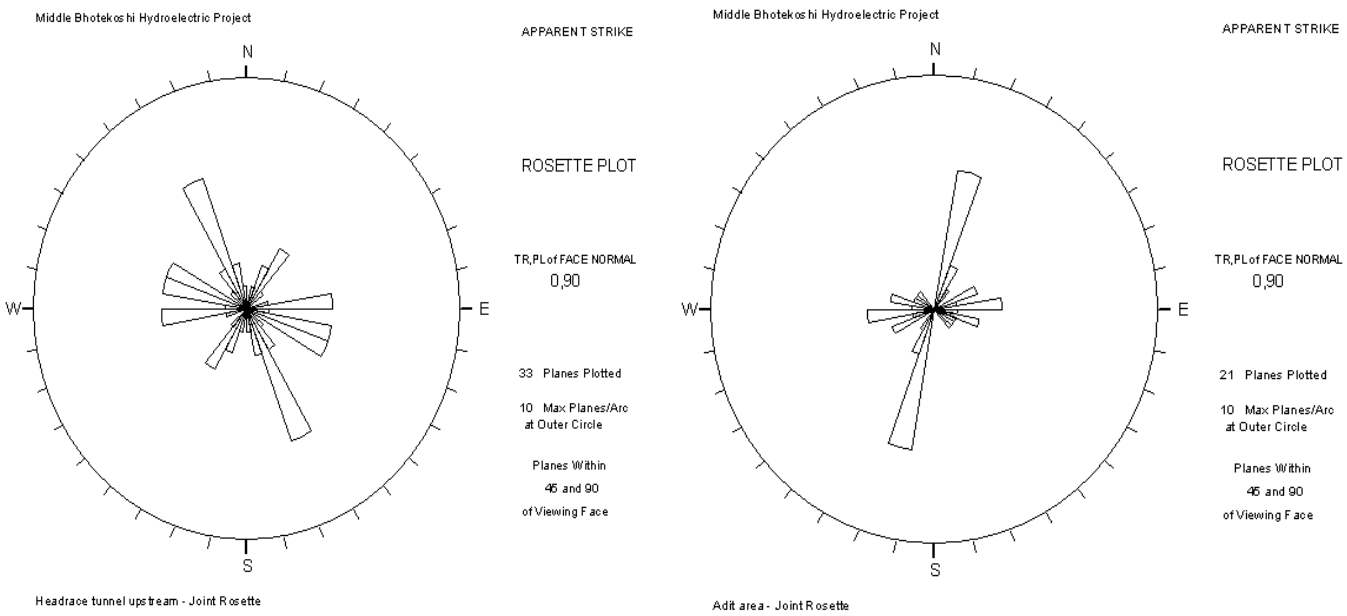


Figure 3-2 Joint Rosette Diagram of Upstream Part of Headrace Tunnel (Left) and of Adit Area (right)

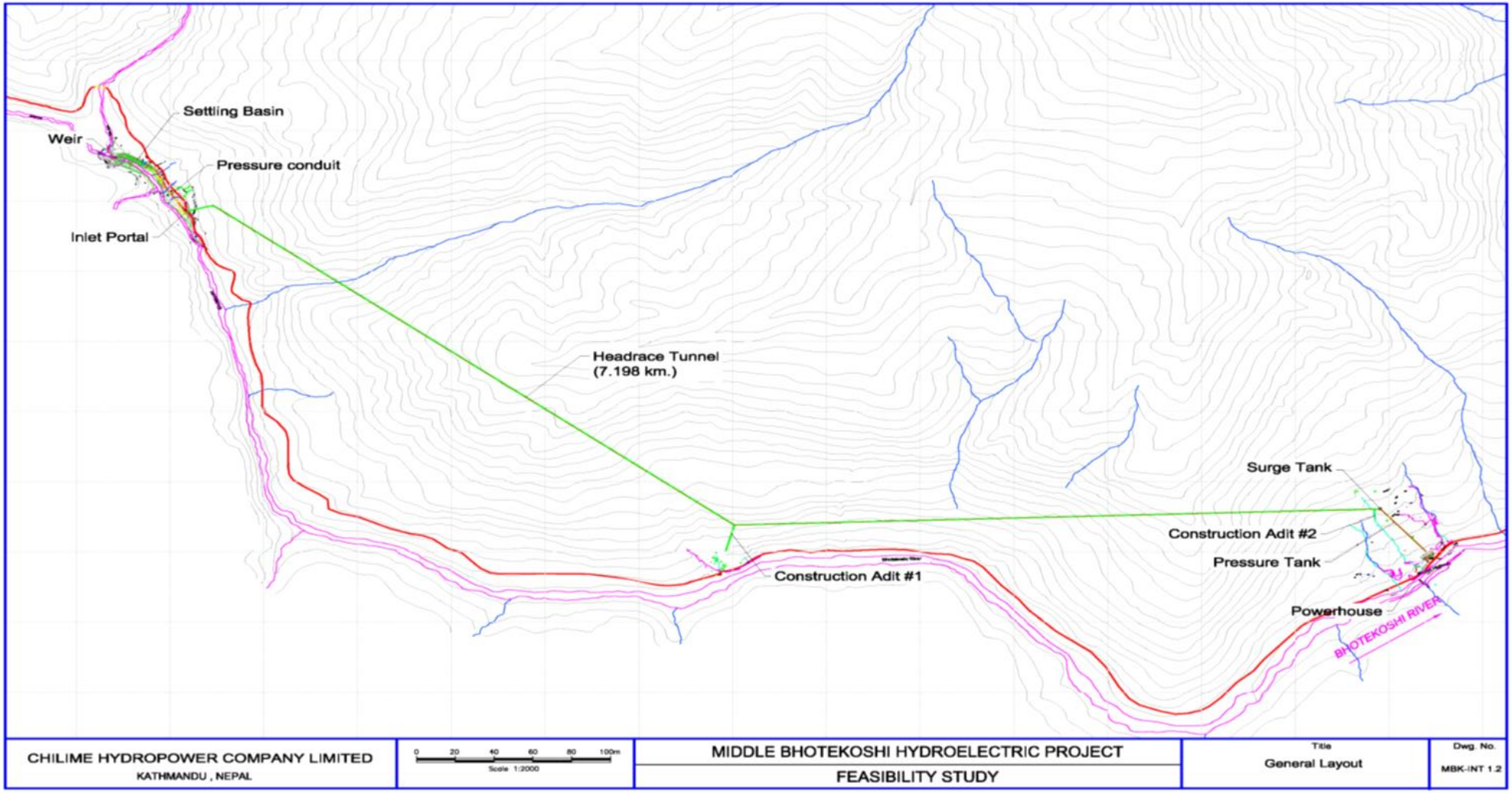


Figure 3-3 Plan of Middle Bhotekoshi Hydroelectric Project site (after MBKHEP feasibility Report)

3.2.1.3 Support Classification

a. Based on Q-Chart Guidelines (NGI (2013)) (after Barton and Grimstad (1993))

Table 3-4 Different Parameters used for Support Design

Parameters	Values
Diameter(m)/Span(m)	5.7
ESR	1.6
Equivalent Dimension (De)	3.563
Factor of Safety	1
Effective (De)	3.563
Length of Bolt(m)	2.4
Diameter of Bolt (mm)	25

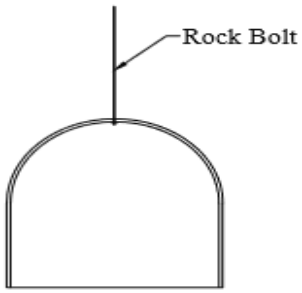
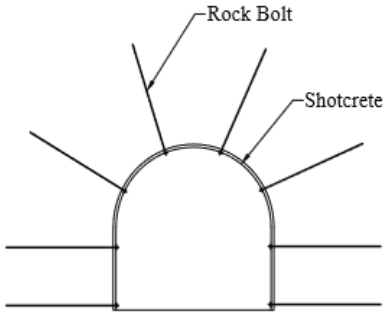
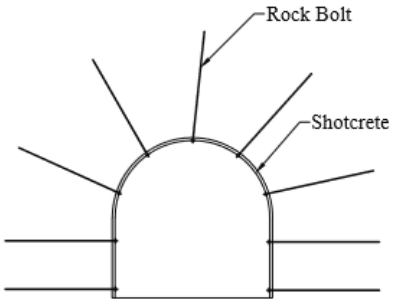
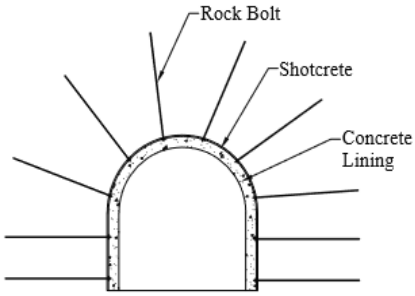
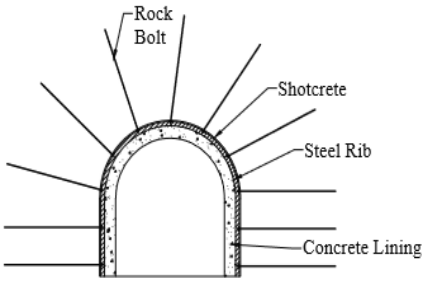
Table 3-5 Range of the support Category

Defining the support class		
Maximum Q-Value	**Minimum Q-value**	**Support Category**
1000	4	S1-I
4	1	S1-II
1	0.1	S1-III
0.1	0.04	S1-IV
0.04	0.01	S1-V
0.01	0.001	S1-VI

Table 3-6 Description of Support system with respective Support Class (after Barton et al., 1993)

Support Class	Description of Support
S1-I	Systematic Bolting @ 2.1m c/c and Plain shotcrete of 4 cm thickness
S1-II	Pattern Bolting @ 1.7m c/c and Fiber reinforced shotcrete of 5 cm thickness
S1-III	Pattern Bolting @ 1.3m c/c and Fiber reinforced shotcrete of 12cm thickness
S1-IV	Pattern Bolting @ 1.2m c/c and Fiber reinforced shotcrete of 15cm thickness
S1-V	RRS with 6 Bar Reinforcement at a single layer of diameter 16 mm with 35cm thickness of shotcrete @ 2.3m c/c with pattern bolting of 1m spacing c/c
S1-VI	RRS with Double layer reinforcement of nos. 6 and 4 with the thickness of shotcrete 40 cm and spacing 1m c/c

Table 3-7 Rock Support System Used in MBKHEP Head Race Tunnel

Range of RMR Value	**Range of Q-Value**	**Category of Rock Mass**	**Support Classification**		**Description of Support**
81-100	>40	RI	SI		SFRS of thickness 100mm with spot bolting with 25mm diameter bolt and length of 3.8m
61-80	10 - 40	RII	SII		SFRS of thickness 100mm with pattern bolting of 2m c/c and diameter 25mm with length 3.8m
41-60	4 – 10	RIII	SIII		SFRS of thickness 100-150mm with pattern bolting 1.75m to 2m with diameter 25mm and length 3.8m
21-40	1 – 4	RIV	SIV		SFRS of thickness 50mm and concrete lining of thickness 30 to 45 cm with pattern bolting at 1.5m c/c with diameter of bolt 25mm and length 3.8m.
0-20	<1	RV	SV		Steel Ribds ISMB 75 X 150 @ 0.25 to 0.33m c/c with SFRS OF 50mm thickness and concrete lining of thickness 45cm

b. Based on RMR –Support Guidelines (Lowson and Bieniawski (2013)) (after Bieniawski (1984))

Table 3-8 Description of Support system after RMR-Support Guidelines

Support Range		Support Division	Bolt Length (m)	Spacing of Bolt (using RMR guidelines) (m)	Limiting Bolt Spacing (m)	Thickness of Shotcrete	Steel Ribs
Minimum	Maximum						
100	81	R-I	1.90	2.85	3.00	No Shotcrete Required	No
80	61	R-II	3.71	2.08	2.50	50 mm	No
60	41	R-III	6.00	1.31	1.50	100 mm	No
40	21	R-IV	6.00	0.54	1.00	175 mm	No
20	11	R-V	6.00	0.26	1.00	50 mm with Concrete Lining of 10cm	ISHB 150mm x 150 mm

3.2.1.4 Empirical Approach for prediction of Squeezing

a. Singh et al. (1992)

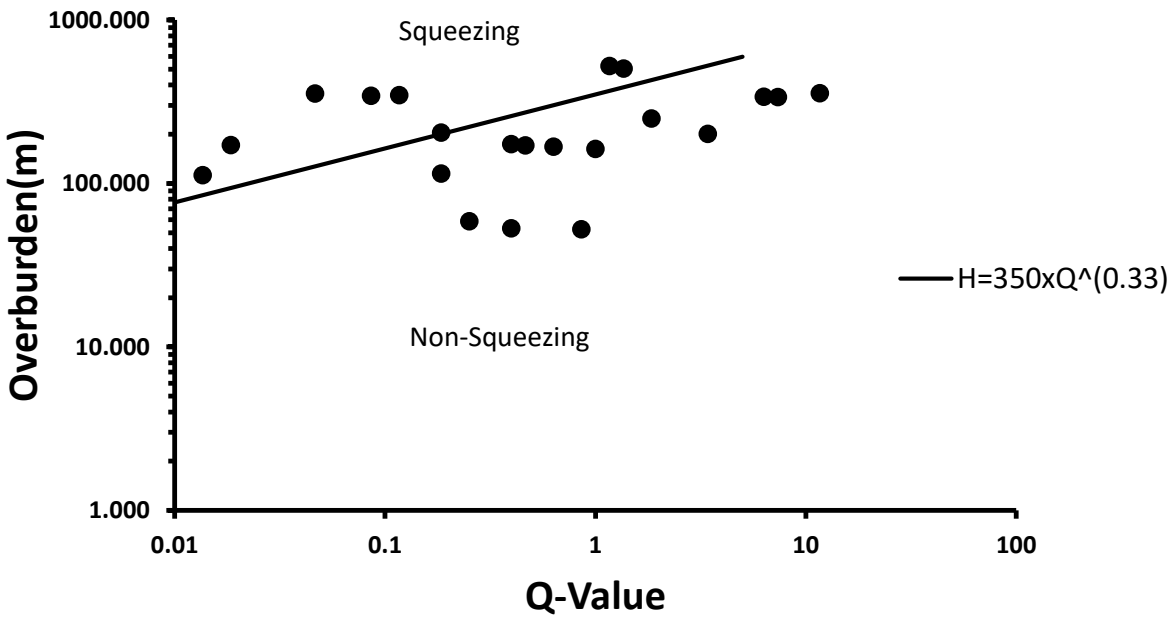


Figure 3-4 Squeezing Assessment of Headrace Tunnel of MBKHEP

b. Jimenez et al. (2011)

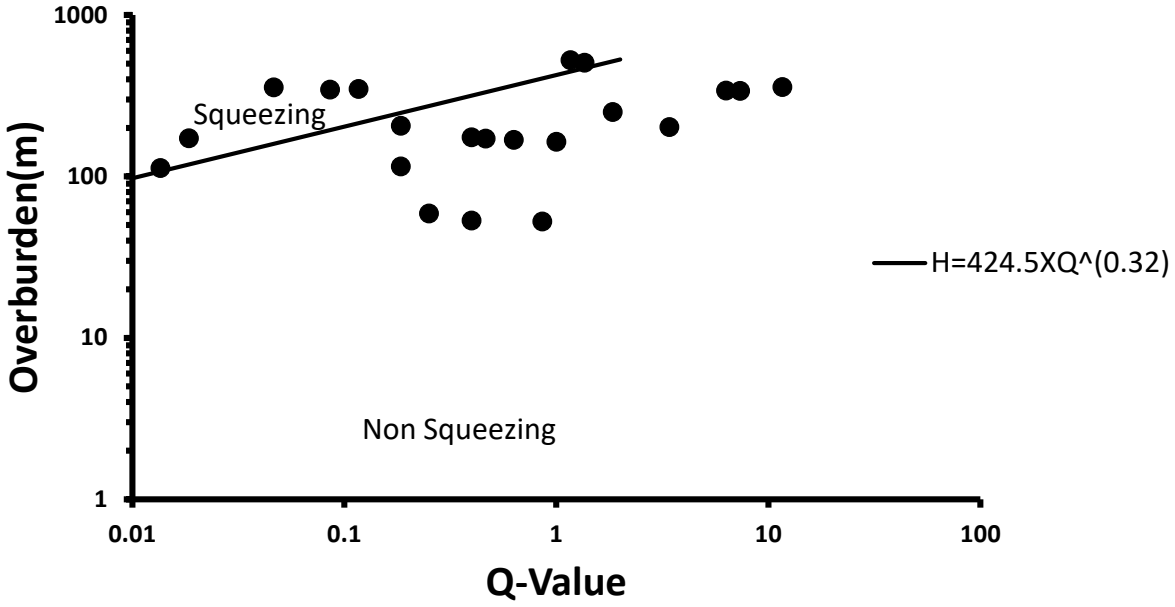


Figure 3-5 Squeezing Assessment of Headrace Tunnel of MBKHEP

Table 3-9 Squeezing Assessment of MBKHEP Sections

Chainage (m)		Overburden (m)	Singh et. al (1992)		R. Jimenez et. al(2011)	
Starting	Ending		Critical Overburden (m)	Status	Critical Overburden (m)	Status
404.3	406.6	250.59	428.616	No Squeezing	516.546	No Squeezing
517	518	294.50	72.789	Squeezing	92.564	Squeezing
512	513.3	292.28	48.536	Squeezing	62.485	Squeezing
509.2	512.5	289.42	59.439	Squeezing	76.052	Squeezing
2912.2	2914.7	505.93	387.318	Squeezing	468.211	Squeezing
2886.6	2889.2	523.56	368.187	Squeezing	445.768	Squeezing
3190	3193.1	355.13	127.077	Squeezing	158.894	Squeezing
3171.7	3173.2	357.23	787.169	No Squeezing	931.341	No Squeezing
3283.2	3285.6	339.85	642.788	No Squeezing	765.201	No Squeezing
3297.5	3300.3	338.14	676.189	No Squeezing	803.727	No Squeezing
3264.4	3266.9	344.98	155.621	Squeezing	193.394	Squeezing
3248.2	3250.5	347.10	172.214	Squeezing	213.358	Squeezing
3756	3758.5	201.61	524.890	No Squeezing	628.698	No Squeezing
3793.4	3795.6	205.56	200.479	Squeezing	247.234	No Squeezing
4002.6	4004.7	174.22	258.266	No Squeezing	316.064	No Squeezing

4028.5	4031	163.35	350.000	No Squeezing	424.400	No Squeezing
4017.5	4020.4	168.38	300.655	No Squeezing	366.248	No Squeezing
4011.8	4014.4	170.92	271.686	No Squeezing	331.978	No Squeezing
6772	6770.4	161.45	72.789	Squeezing	92.564	Squeezing
6777.3	6775.6	158.49	65.776	Squeezing	83.903	Squeezing
6856.6	6854.3	115.19	200.479	No Squeezing	247.234	No Squeezing
6867.5	6865.3	112.55	84.736	Squeezing	107.261	Squeezing
6872.7	6870.6	171.91	93.771	Squeezing	118.334	Squeezing
7082	7083.3	58.86	221.854	No Squeezing	272.757	No Squeezing
7106.5	7109	52.49	332.712	No Squeezing	404.056	No Squeezing
7124.8	7127.5	53.19	258.266	No Squeezing	316.064	No Squeezing

3.2.1.5 Support System Provided

Table 3-10 Calculation of Support Pressure for tunnel Section of MBKHEP

Chainage		**RMR Value**	**Support Class (As per Design)**	**Support Class (As per Q-chart)**	**Support Class(As per RMR support classification)**	**Advance Rate (m)**	**Overburden (m)**	**Geol and Jethwa(1991) (Mpa) (Short term roof pressure)**	**Support Pressure (MPa)**	
Starting	**Ending**								**Bhasin and Grimstad (1996)**	**Geol et al.(1995)**
404.3	406.6	54	3	S1-II	R-III	2.3	250.59	0.081	0.186	0.771
517	518	19	5	S1-VI	R-V	1	294.50	0.353	1.096	0.310
512	513.3	11	5	S1-VI	R-V	0.8	292.28	0.644	1.644	1.041
509.2	512.5	15	5	S1-VI	R-V	3.3	289.42	0.456	1.343	0.472
2912.2	2914.7	52	3	S1-II	R-III	2.5	505.93	0.143	0.206	0.136
2886.6	2889.2	51	3	S1-II	R-III	2.6	523.56	0.150	0.217	0.138
3190	3193.1	30	4	S1-IV	R-IV	3.1	355.13	0.230	0.628	0.189
3171.7	3173.2	66	2	S1-I	R-II	1.5	357.23	0.078	0.101	0.458
3283.2	3285.6	62	2	S1-I	R-II	2.4	339.85	0.083	0.124	0.581
3297.5	3300.3	63	2	S1-I	R-II	2.8	338.14	0.080	0.118	0.544
3264.4	3266.9	34	4	S1-IV	R-IV	2.5	344.98	0.194	0.513	0.167
3248.2	3250.5	36	4	S1-III	R-IV	2.3	347.10	0.181	0.463	0.160
3756	3758.5	58	3	S1-II	R-III	2.2	201.61	0.059	0.152	0.583
3793.4	3795.6	39	4	S1-III	R-IV	2.2	205.56	0.114	0.398	0.135

4002.6	4004.7	44	3	S1-III	R-III	2.1	174.22	0.084	0.309	1.045
4028.5	4031	50	3	S1-II	R-III	2.5	163.35	0.064	0.228	0.824
4017.5	4020.4	47	3	S1-III	R-III	3	168.38	0.073	0.265	0.934
4011.8	4014.4	45	3	S1-III	R-III	2.6	170.92	0.080	0.294	1.007
6772	6770.4	19	5	S1-VI	R-V	1.6	161.45	0.248	1.096	0.234
6777.3	6775.6	17	5	S1-VI	R-V	1.7	158.49	0.280	1.213	0.269
6856.6	6854.3	39	4	S1-III	R-IV	2.3	115.19	0.073	0.398	1.161
6867.5	6865.3	22	4	S1-V	R-IV	2.2	112.55	0.165	0.942	0.176
6872.7	6870.6	24	4	S1-V	R-IV	2.1	171.91	0.194	0.851	0.186
7082	7083.3	41	3	S1-III	R-III	1.3	58.86	0.034	0.360	1.035
7106.5	7109	49	3	S1-III	R-III	2.5	52.49	0.016	0.240	0.748
7124.8	7127.5	44	4	S1-III	R-III		53.19	0.024	0.309	0.924

Table 3-11 Different Parameters for six study section of MBKHEP

Chainage (m)		RMR	Overburden (m)	Vertical Pressure (Mpa)	Q-value	Uniaxial Compressive strength (σi) (Mpa)	Modular Ratio (Hoek et al. (2006))	Modulus of Elasticity of Intact Rock (Ei) (Mpa)	Stress Ratio(k) (Sheorey 1994)	Modulus of rock mass (Erm) (Mpa) (Sonmez et al. (2006))	Shear Modulus (G) (Mpa)	In-plane Horizontal Stress(σh) (Mpa)
Starting	Ending											
3283.2	3285.6	62	339.85	9.18	6.310	34.508	675.000	23292.900	0.893	4967.645	1480.086	8.193
2886.6	2889.2	51	523.56	14.14	1.166	34.508	675.000	23292.900	0.724	2331.484	698.985	10.241
7106.5	7109	49	52.49	1.42	0.858	34.508	675.000	23292.900	3.519	2022.134	607.050	4.988
3190	3193.1	30	355.13	9.59	0.046	34.508	675.000	23292.900	0.872	517.186	158.029	8.363
6867.5	6865.3	22	112.55	3.04	0.014	34.508	675.000	23292.900	1.862	296.443	91.663	5.658
517	518	19	294.50	7.95	0.009	80.000	350.000	28000.000	1.112	290.862	90.445	8.838

Table 3-12 Assessment of squeezing ground condition for six study sections

Chainage (m)		Overburden (m)	Singh et. al (1992)		R. Jimenez et. al(2011)	
Starting	Ending		Critical Overburden (m)	Status	Critical Overburden (m)	Status
3283.2	3285.6	339.85	642.788	No Squeezing	765.201	No Squeezing
2886.6	2889.2	523.56	368.187	Squeezing	445.768	Squeezing
7106.5	7109	52.49	332.712	No Squeezing	404.056	No Squeezing
3190	3193.1	355.13	127.077	Squeezing	158.894	Squeezing
6867.5	6865.3	112.55	84.736	Squeezing	107.261	Squeezing
517	518	294.50	72.789	Squeezing	92.564	Squeezing

3.2.1.6 Calculation of Tunnel deformation for study sections using Shrestha and Panthi (2015)

Table 3-13 Calculation of Tunnel closure of Designed support

Chainage (m)		For Designed Support (MPa)			Tunnel Closure FOS=1		Tunnel Closure FOS=1.5		Tunnel Closure FOS=2	
Starting	Ending	Support pressure with F.O.S =1	Support pressure with F.O.S =1.5	Support pressure with F.O.S =2	Instantaneous Closure (mm)	Final strain(mm)	Instantaneous Closure (mm)	Final Closure (mm)	Instantaneous Closure (mm)	Final Closure (mm)
3283.2	3285.6	1.385	0.923	0.693	0.130	0.250	0.206	0.393	0.270	0.513
2886.6	2889.2	2.044	1.363	1.022	0.788	1.465	1.352	2.488	1.883	3.445
7106.5	7109	2.044	1.363	1.022	0.062	0.120	0.106	0.205	0.148	0.283
3190	3193.1	6.209	4.139	3.105	1.554	2.852	3.194	5.786	5.157	9.256
6867.5	6865.3	6.209	4.139	3.105	1.059	1.958	2.177	3.971	3.514	6.353
517	518	6.689	4.459	3.345	3.855	6.958	7.996	14.234	13.006	22.943

Table 3-14 Calculation of Tunnel closure of Defined support using RMR Support Classification Guidelines

Chainage (m)		For Q-Chart Support (MPa)			Tunnel Closure FOS=1		Tunnel Closure FOS=1.5		Tunnel Closure FOS=2	
Starting	Ending	Support pressure with F.O.S =1	Support pressure with F.O.S =1.5	Support pressure with F.O.S =2	Instantaneous Closure (mm)	Final Closure (mm)	Instantaneous Closure (mm)	Final Closure (mm)	Instantaneous Closure (mm)	Final closure (mm)
3283.2	3285.6	0.603	0.402	0.302	0.304	0.575	0.404	0.760	0.473	0.888
2886.6	2889.2	1.290	0.860	0.645	1.445	2.656	2.250	4.102	2.923	5.303
7106.5	7109	1.757	1.171	0.879	0.076	0.148	0.127	0.244	0.173	0.330
3190	3193.1	2.171	1.447	1.086	8.935	15.872	15.513	27.275	21.812	38.106
6867.5	6865.3	4.109	2.739	2.055	2.204	4.020	4.285	7.718	6.593	11.780
517	518	4.309	2.873	2.155	8.486	15.089	16.615	29.175	25.718	44.790

Table 3-15 Calculation of Tunnel closure of Defined support using Q-Support Chart Guidelines

Chainage (m)		**For RMR Support Classification (MPa)**			**Tunnel Closure FOS=1**		**Tunnel Closure FOS=1.5**		**Tunnel Closure FOS=2**	
Starting	**Ending**	**Support pressure with F.O.S =1**	**Support pressure with F.O.S =1.5**	**Support pressure with F.O.S =2**	**Instantaneous Closure (mm)**	**Final sClosure (mm)**	**Instantaneous Closure (mm)**	**Final Closure (mm)**	**Instantaneous Closure (mm)**	**Final Closure (mm)**
3283.2	3285.6	0.710	0.473	0.355	0.264	0.502	0.363	0.685	0.434	0.816
2886.6	2889.2	1.452	0.968	0.484	1.249	2.302	1.995	3.646	3.640	6.577
7106.5	7109	1.452	0.968	0.484	0.098	0.189	0.156	0.300	0.285	0.541
3190	3193.1	2.589	1.726	0.863	6.863	12.253	12.330	21.772	27.738	48.239
6867.5	6865.3	2.589	1.726	0.863	4.677	8.409	8.401	14.942	18.900	33.107
517	518	2.491	1.661	0.830	20.724	36.239	36.960	63.932	81.992	139.720

Table 3-16 Calculation of Stiffness and Maximum support pressure

Chainage (m)		**RMR**	**Support Class**			**Stiffness of Support System (MPa)**			**Maximum Support Pressure (P_{max}) (MPa)**		
Starting	**Ending**		**As per Design**	**As per Q-chart**	**As per RMR support classification**	**As per Design**	**As per Q-chart)**	**As per RMR classification**	**As Per Design**	**As Per Q-chart**	**As Per RMR classification**
3283.2	3285.6	62	2	S1-I	R-II	249.017	105.16	124.197	1.385	0.603	0.71
2886.6	2889.2	51	3	S1-II	R-III	375.438	231.786	249.859	2.044	1.29	1.452
7106.5	7109	49	3	S1-III	R-III	375.438	315.865	249.859	2.044	1.757	1.452
3190	3193.1	30	4	S1-IV	R-IV	1320.937	394.932	446.681	6.209	2.171	2.589
6867.5	6865.3	22	4	S1-V	R-IV	1320.937	794.718	446.681	6.209	4.109	2.589
517	518	19	5	S1-VI	R-V	152.266	1025.542	50.851	6.689	4.309	2.491

Comparison of Minimum convergence using different Support Condition

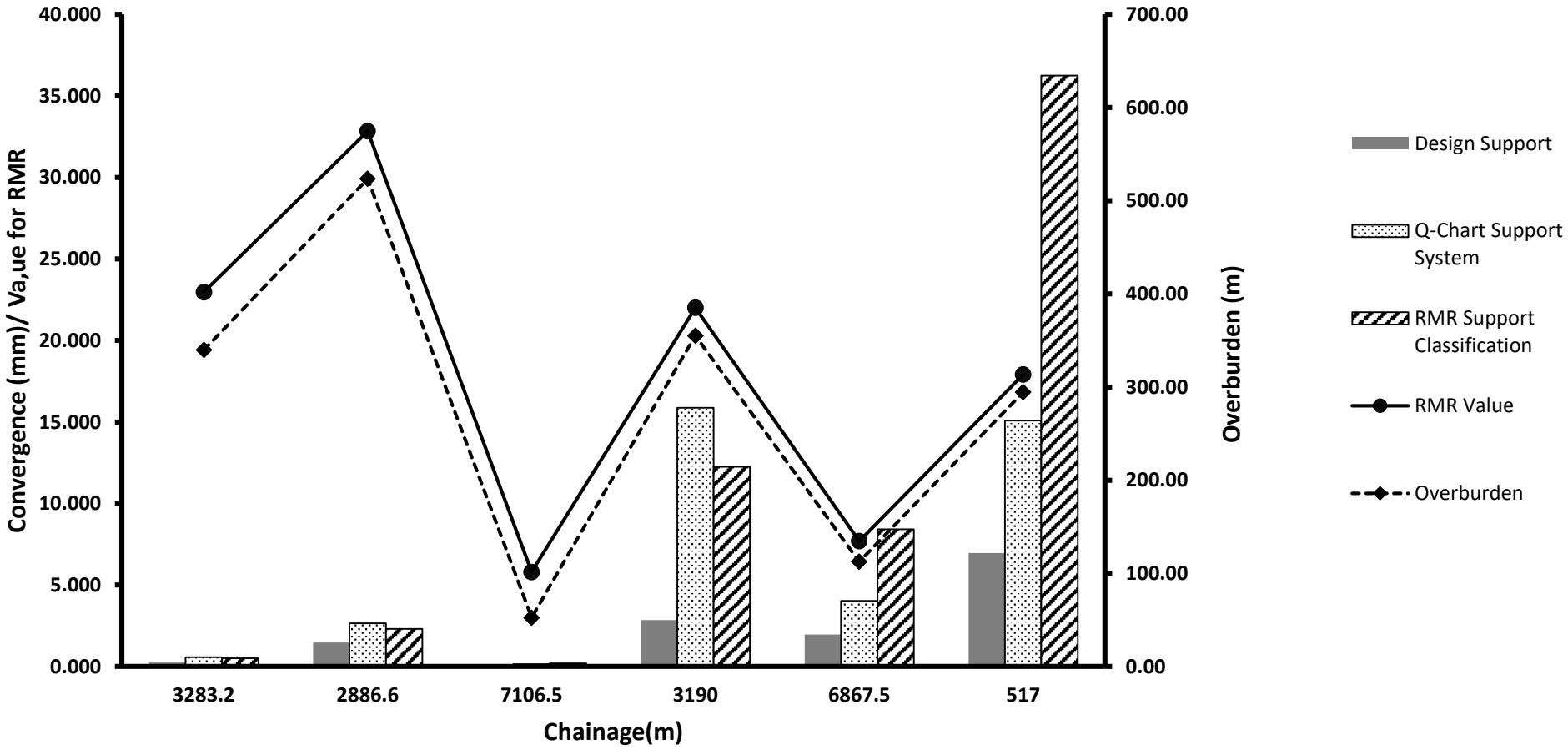


Figure 3-6 Plot between Convergence (mm), Overburden (m) and Chainage (m)

Comparison of Support Pressure

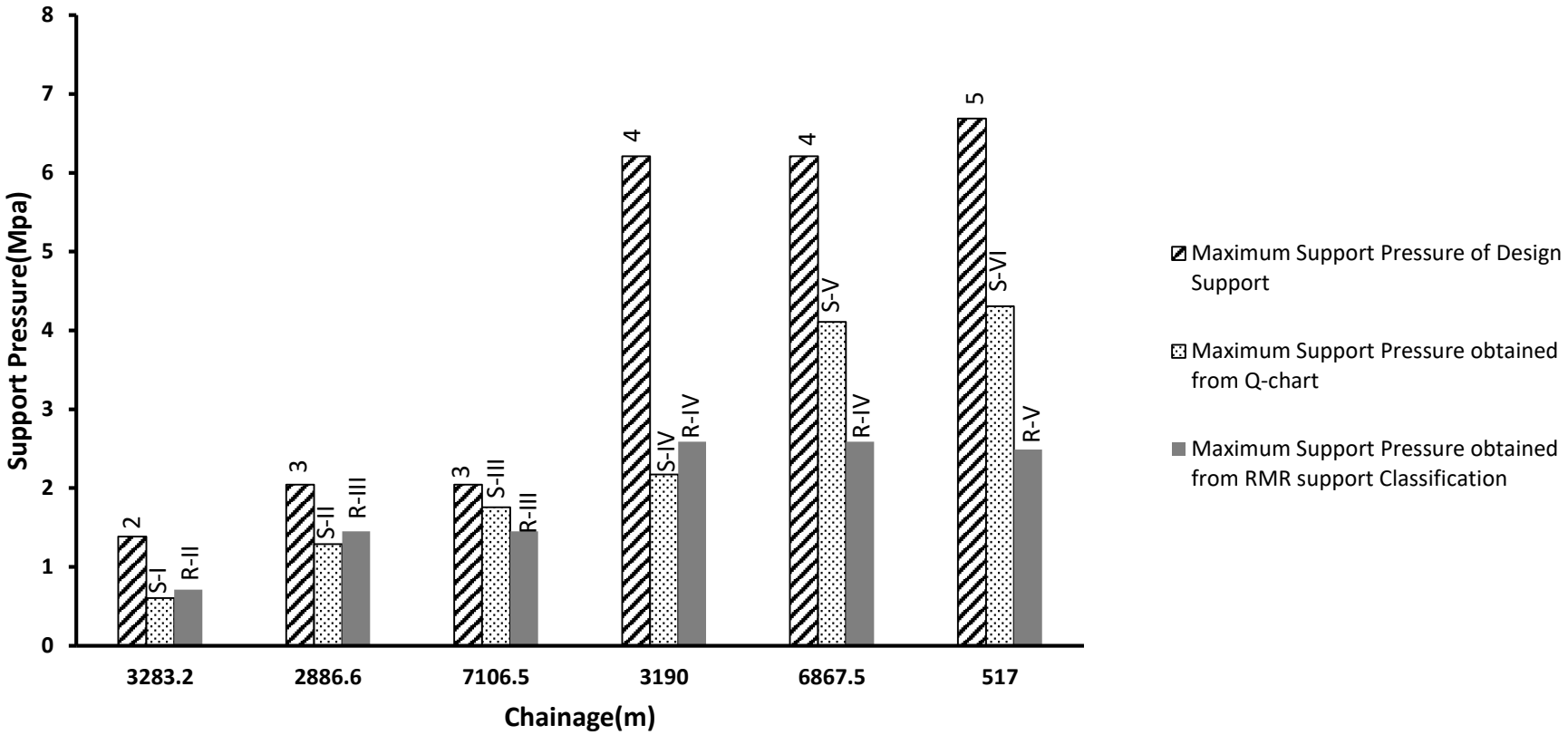


Figure 3-7 Plot between Support pressure and Chainage for different Support system

Table 3-17 Calculation of Convergence value using Hoeks and Marino (2000)

Chainage (m)	Maximum Deformation Observed (mm)	Rock Type	Tunnel Support (As per design)	RMR value	Overburden (m)	Vertical stress (σv) (MPa)	Correlated GSI value (Ceballos et. al 2014)	UCS of intact rock (σci) (MPa)	Value of mi (Heok (2006)	UCS of rock mass (Hoek and Marino (2000)) (MPa)	F.O.S=1(mm)			F.O.S=1.5 (mm)			F.O.S=2 (mm)		
											Designed support	Q-chart support system	RMR support system	Designed support	Q-chart support	RMR support system	Designed support	Q-chart support	RMR support system
0+300	1.5	Dolomite	3	42	181.62	4.904	37	80	9.00	6.535	4.9	4.8	5.3	5.4	5.7	5.9	5.8	6.0	6.2
0+400	2.5	Dolomite	3	49	246.23	6.648	44	80	9.00	8.552	6.1	5.8	6.1	6.2	6.4	6.6	6.5	6.6	6.8
0+500	4	Dolomite	3	41	286.35	7.731	36	80	9.00	6.289	12.0	11.7	12.8	13.1	13.8	14.6	14.4	14.9	15.5
6+795.90	4.3	Phyllitic Schist	4	31	147.43	3.981	26	34.508	9.00	1.847	-3.4	6.6	3.2	-2.5	15.7	11.6	0.3	22.5	18.4

Table 3-18 Calculation of Convergence value using Shrestha et al. (2015)

Chainage (m)	Maximum Deformation	Rock Type	Tunnel Support (As per design)	RMR value	Overburden(m)	Vertical stress (σv) (MPa)	Stress Ratio (k) (Shorey (1994))	UCS of intact rock (σci) (MPa)	Poisson Ratio (υ) (Vasarhelyi et. al (2017))	Shear Modulus (G) (MPa)	F.O.S=1(mm)			F.O.S =1.5 (mm)			F.O.S =2 (mm)		
											Designed support	Q-chart support system	RMR support system	Designed support	Q-chart support	RMR support system	Designed support	Q-chart support	RMR support system
0+300	1.5	Dolomite	3	42	181.62	4.904	1.525	80	0.657	444	0.9	1.1	1.4	1.6	1.9	2.3	2.2	2.5	3.0
0+400	2.5	Dolomite	3	49	246.23	6.648	1.424	80	0.666	730	0.6	0.7	0.9	1.0	1.1	1.4	1.3	1.5	1.8
0+500	4	Dolomite	3	41	286.35	7.731	1.13	80	0.655	413	1.9	2.4	3.0	3.3	3.9	4.8	4.6	5.3	6.3
6+795 .90	4.3	Phyllitic Schist	4	31	147.43	3.981	0.814	34.508	0.638	75	2.0	11.1	8.6	4.1	19.1	15.3	6.5	26.7	21.9

Comparison of Deformation for different support system using Shrestha and Panthi (2015)

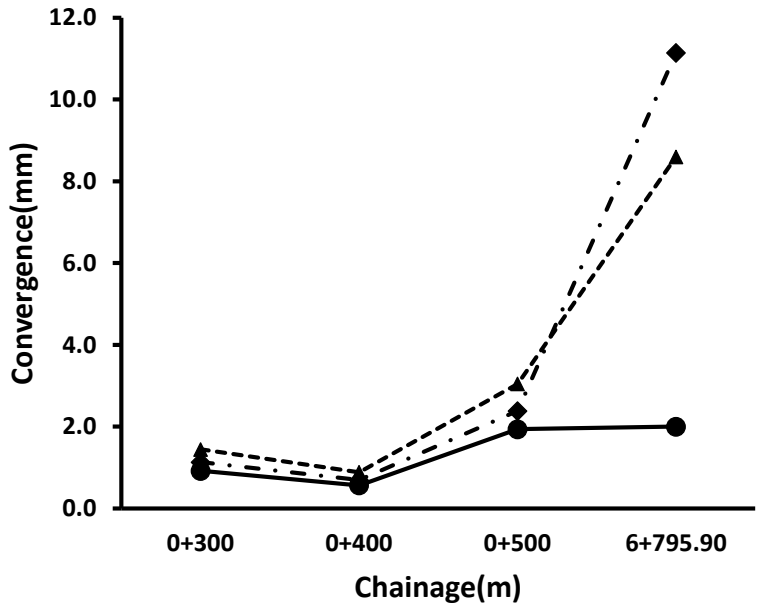


Figure 3-8 Deformation for Support Pressure with F.O.S=1

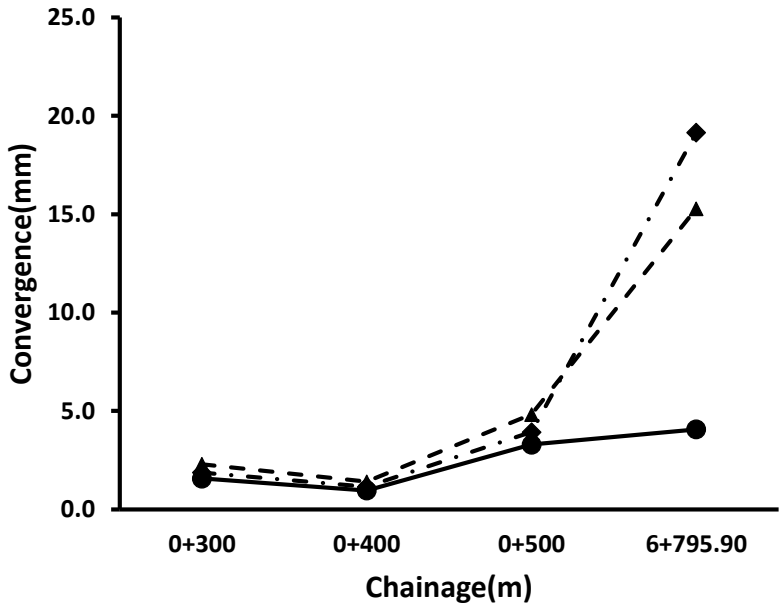


Figure 3-9 Deformation for Support Pressure with F.O.S=1.5

Design Suppport System Q-Chart Support system RMR Support Classification

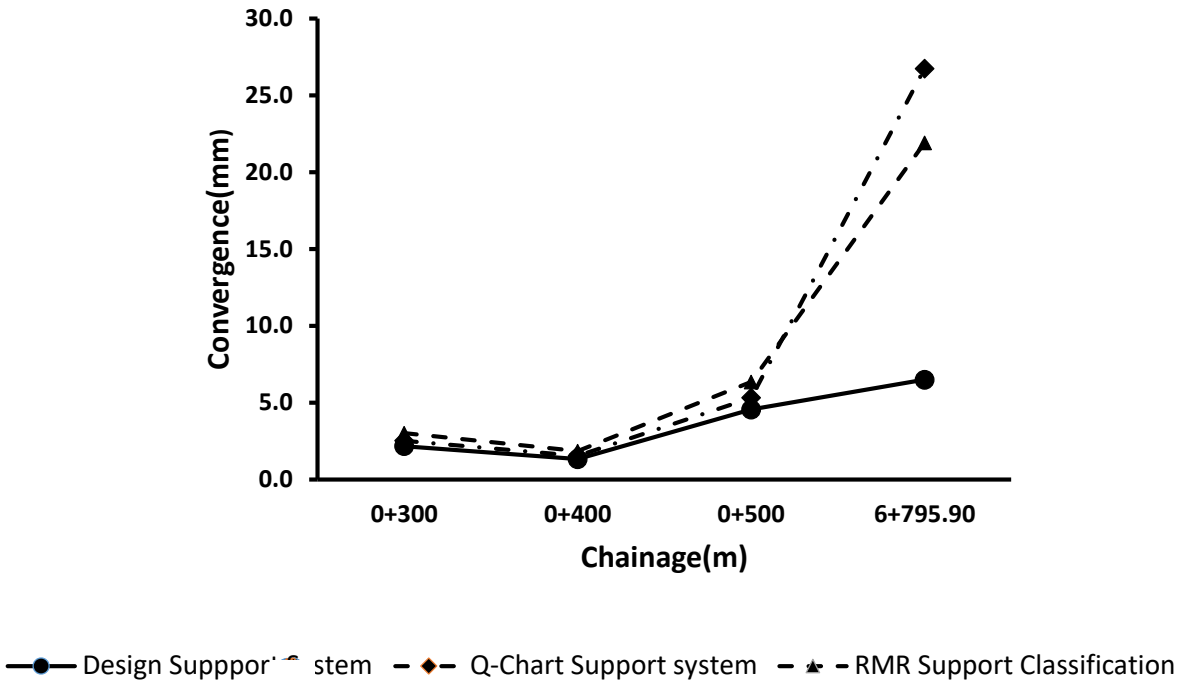


Figure 3-10 Deformation for Support Pressure with F.O.S=2

Comparison of Deformation for different support system using Hoeks and Marinos (2000)

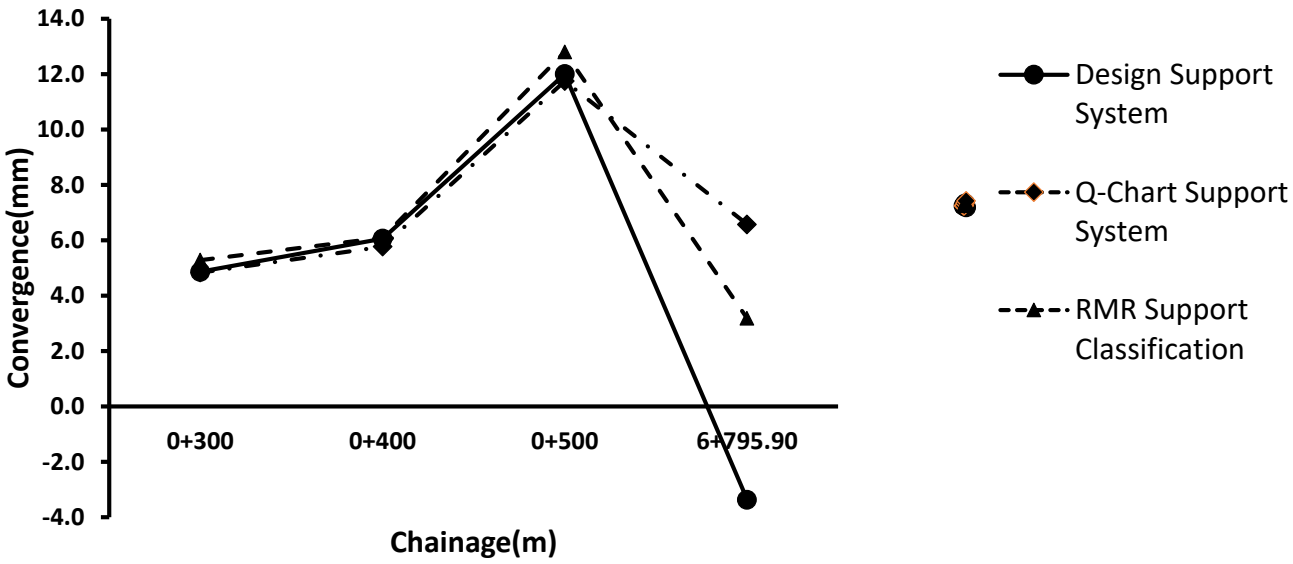


Figure 3-11 Deformation for Support Pressure with F.O.S =1

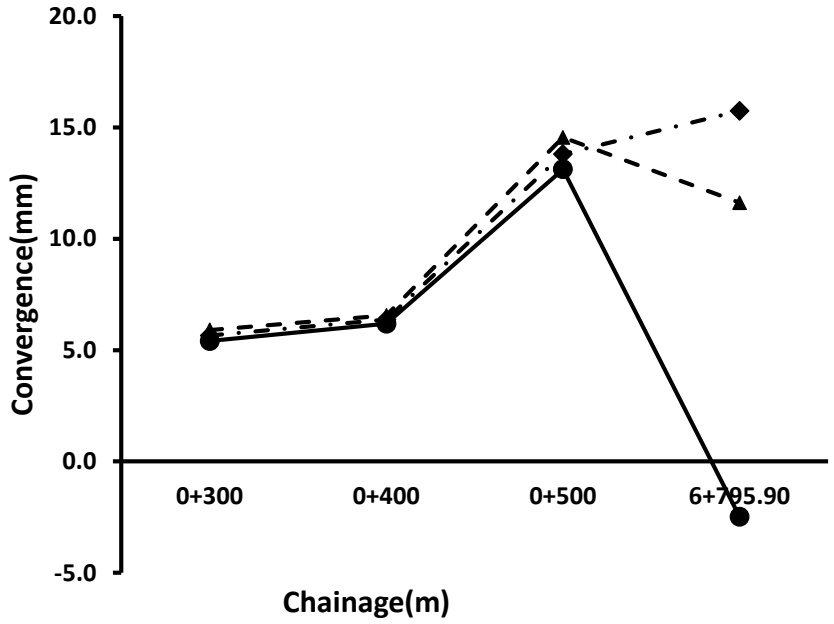


Figure 3-12 Deformation for Support Pressure with F.O.S=1.5

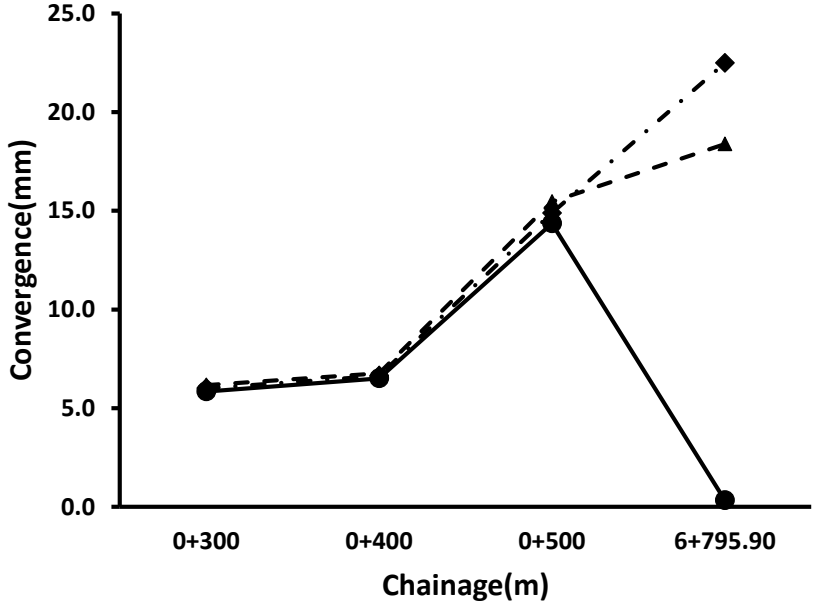


Figure 3-13 Deformation for Support Pressure with F.O.S=2

Design Support System Q-Chart Support System RMR Support Classification

Comparison of Convergence according to Different approach with actual Observed Deformation (Design Support System)

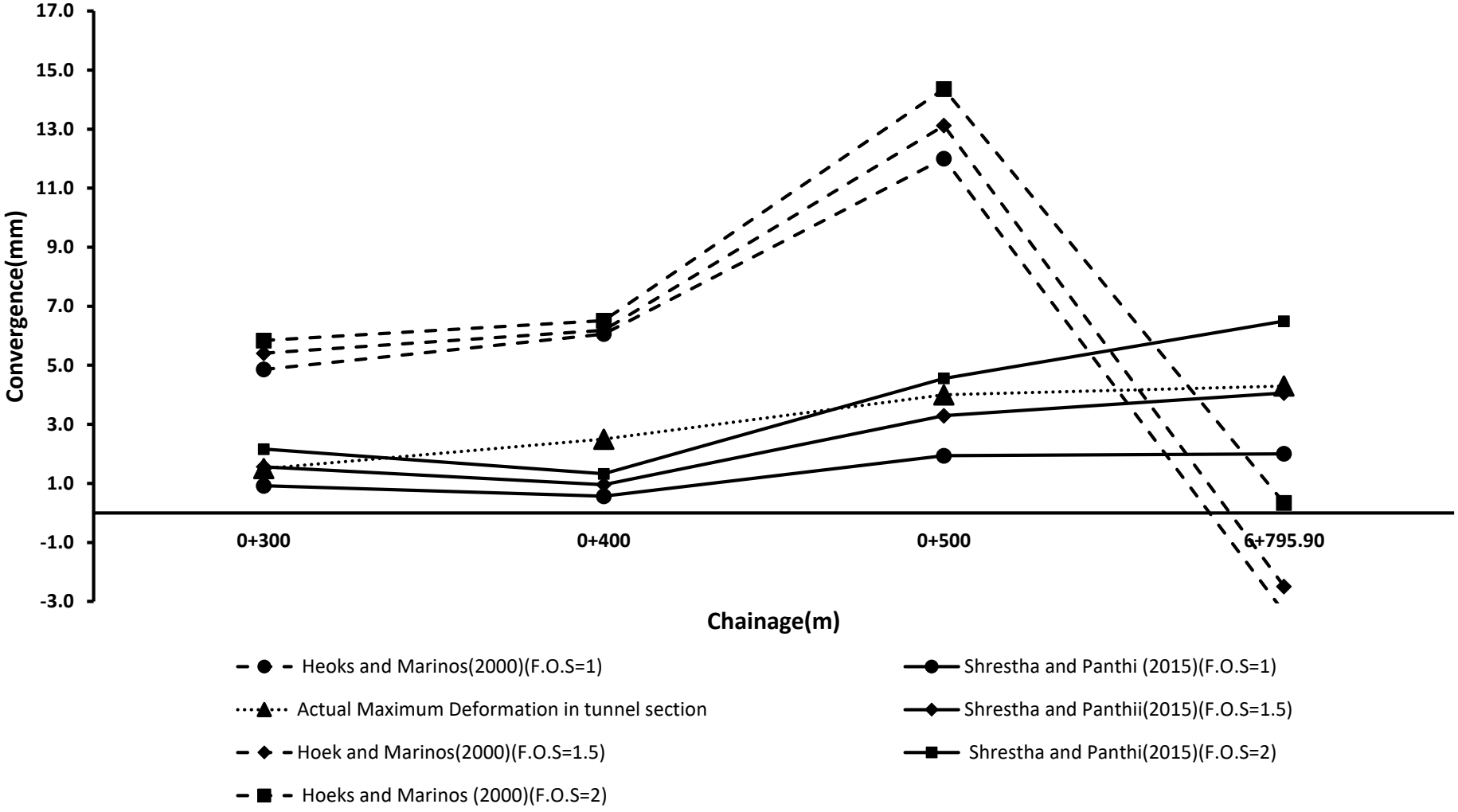


Figure 3-14 Plot of Deformation for Design support using Shrestha and Panthi (2015) and Hoek and Marinos (2000)

Comparison of Convergence according to Different approach with actual Observed Deformation (Q-Chart Support System)

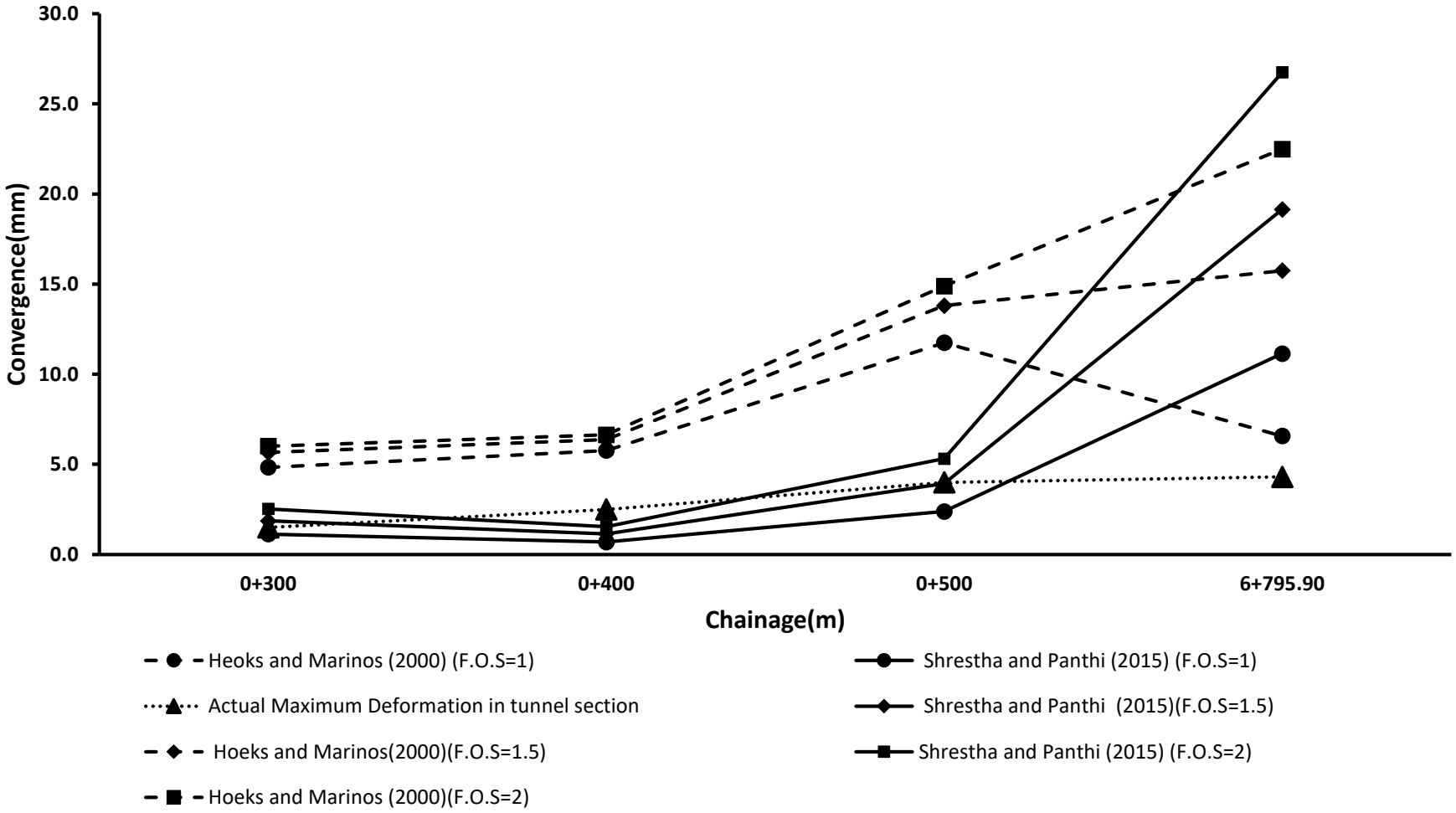


Figure 3-15 Plot of Deformation for Q- Support classification using Shrestha and Panthi. (2015) and Hoek and Marinos (2000)

Comparison of Convergence according to Different approach with actual Observed Deformation (RMR Classification)

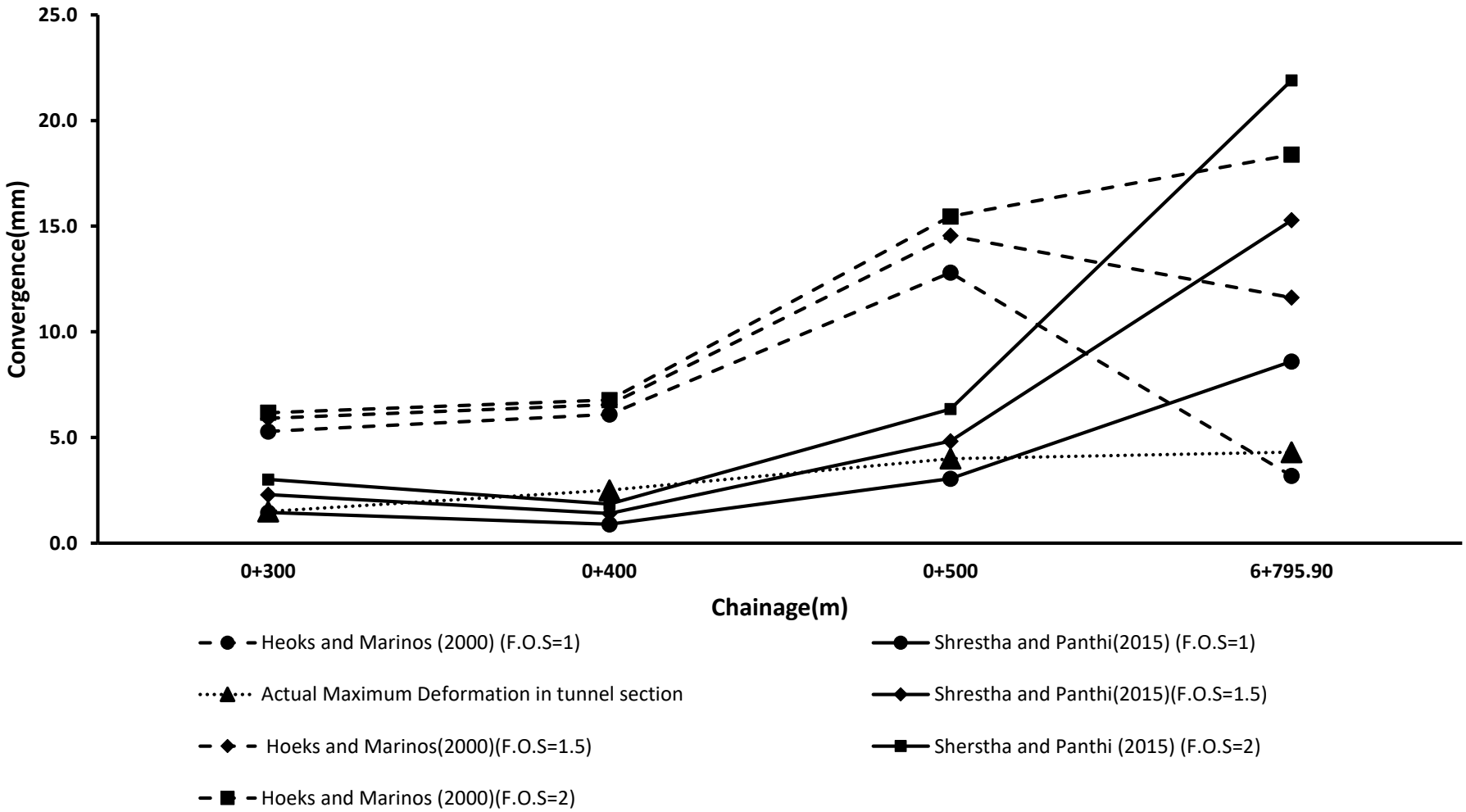


Figure 3-16 Plot of Deformation for RMR- Support classification using Shrestha and Panthi. (2015) and Hoek and Marinos (2000)

Observations made by the analysis of empirical approach of MBKHEP Head Race Tunnel

Among the 29 tunnel sections data that are provided to us, six tunnel sections were studied by comparing the support pressure, deformations. Whilst four more sections of MBKHEP was provided to us by the MBKHEP authority with the convergence value. Having these sections enable us to validate the method and the empirical relations that we used to analyze the support system in more contemplating manner. The following observations are found by manipulating different empirical approaches with lots of "trail-error" methodology to usage of adequate relations.

- Use of the Shrestha et al. (2015) approach for six tunnel section enlisted above gave the amenable result to the squeezing criteria given by Jimenez et al. (2011) and Singh (1992).From this result, it can be inferred that the prediction of squeezing is very important to know how the ground will react to the excavation and what support system should be chosen according to the need. All three support system either designed (at the site) or defined by use of Q-support chart and RMR-Support classification shows the confirmation to the both squeezing criteria.
- When we see the convergence pattern for six sections it can be clearly stated that convergence is the function of the internal support pressure, ground condition, the size of the opening and the stress acting on them.
- Using the graph "Comparison of Maximum support pressure" which compares the maximum support pressure given by different design approaches (*either Q-Support Chart or RMR- Support classification*) and designed one, all of these elucidate that the support system provided in the site is more robust than obtained by the Q-Support chart and RMR Support classification.
- The graph given from Figure 3-8 to Figure 3-13 shows that the support classification using RMR provide more flexible support: high the value of convergence than that of other support condition. This is could be useful in the scenario where more economical support system is searched for.
- Back calculated value of deformation of the four latter tunnel sections shows that the Hoeks and Marino (2000) approach was not applicable to these section as of the large deviation encountered with respect to the measured deformation. This high deviation might be the reason due to the consideration of tunnel shape to be circular and the stress to be hydrostatic.

- The limitation of Hoek and Marino (2000) was addressed by Shrestha et al. (2015) for Himalayan tunnels. Furthermore this addressing can be clearly seen on the back calculated data of deformation with the minimum deviation with the measured deformation.
- The value of the support pressure provide by the designed support system is found to be between support pressure with F.O.S=1.5 and 2 by Shrestha et al. (2015)
- Looking into the Figure 3-15 and Figure 3-16 it can be clearly seen that for weaker ground, the support system defined by Q-Support chart and RMR classification shows more convergence indicating the support system defined to more flexible than that of designed one. Much as the support system of for strong to moderate ground condition (specifically support category 1 to 3) where the deviation of support pressure is low, support pressure is high for weak rock. This suggest us that the support system employed by the MBKHEP does limit to the conventional support definition using Q-Support chart and RMR Support Classification but provides modification to existing support classification as per its need making the designed support system to be robust resulting low convergence which can also be visualized by the graph "Comparison of Support pressure".

The parameters values derived from empirical which are used for preparing CCM is provided in Table 3-11. The support system used in the project sites and classification according to different system are provided from Table 3-4 to Table 3-8.

3.2.1.7 Convergent Confinement Analysis

Using all the parameters mentioned above along with the considerations for Rock mass classification and the support system assigned, LDP, GRC and SCC for different selected sections were computed and the graph showing the respective curves was plotted with the help of excel file prepared. We have included the CCM computation for different sections with highest overburden, deformed sections etc. on the basis of observed conditions, RMR values classification and Q- System Classification.

Table 3-19 Value of Parameters used while construction of CCM for MBKHEP (for section at 2886.6)

Chainage	2+886.6
OverBurden	523.56
Dimension	5.7x5.7
R(m)	3.03
H(m)	523.56
σ_0 (MPa)	9.69
ν	0.30
L(m)	1.00
Q	7.1
Ei(Gpa)	10.35

Erm(Gpa)	5.97
Disturbance Factor	0.8
Friction Angle	25
E_{rm} (GPa)	5.97
G_{rm} (GPa)	2.30
m_i	10.00
GSI	57.77
mb	0.81
s	2.20E-03
a	0.50

σ_{ci} (MPa)	34.51
S_0	3.50E-01
k	0.37
σ_v (MPa)	14.14
σ_h (MPa)	5.25
P_i^{cr} (MPa)	1.54E-01
p_i^{cr} (MPa)	4.2113
$C(\sigma_{ci})$	0.59
Ψ (degree)	0.00
K_Ψ	1.00

For Shotcrete	
σ_{cc}(MPa)	40.00
ν_c	0.30
t_c (mm)	120.00
E_c (GPa)	25.00
P_s^{max} (MPa)	1.55
K_s (MPa/m)	372.64
u_r^{max} (mm)	4.17

Combination of Supports	
Ks(MPa/m)	374.03
urmax(mm)	4.17
Psmax(MPa)	1.559

For Bolts	
bolt provided	yes
l (m)	3.80
d_b (mm)	25.00
E_b (Gpa)	210.00
Q_b (m/MN)	0.14
T_{bf} (MN)	0.20
sc (m)	2.00
sl (m)	2.00
Ps_{max} (MPa)	0.13
k_s (MPa/m)	1.39
u_{rbmax} (mm)	35.07

3.2.1.8 Computation of CCM

a. Longitudinal Deformation Profile

Table 3-20 Calculation of LDP for MBKHEP (for section at 2886.6)

				Observed Condition	**RMR**	**Q-System**
			u_r^0 (mm)	*6.529*	*18.494*	*24.521*
S.N	**x/R**	**x**	**u_r/u_r^m**	**ur**	**ur**	**ur**
1	-4.00	-12.12	0.002	0.03	0.09	0.12
2	-2.91	-8.81	0.010	0.17	0.47	0.62
3	-1.82	-5.51	0.045	0.75	2.12	2.81
4	-0.73	-2.20	0.160	2.68	7.60	10.08
5	0.36	1.10	0.398	6.67	18.90	25.06
6	1.45	4.41	0.669	11.21	31.76	42.11
7	2.55	7.71	0.852	14.27	40.43	53.61
8	3.64	11.02	0.941	15.76	44.64	59.19
9	4.73	14.32	0.977	16.37	46.38	61.50
10	5.82	17.63	0.991	16.61	47.05	62.39
11	6.91	20.93	0.997	16.70	47.31	62.72
12	8.00	24.24	0.999	16.73	47.40	62.85

b. Ground Reaction Curve

Table 3-21 Calculation of GRC for MBKHEP (for section at 2886.6)

	Observed Condition				**RMR**			
S.N	**$_p_i$**	**P_i**	**R_{pl}**	**u_r**	**$_p_i$**	**P_i**	**R_{pl}**	**u_r**
1.00	0.00	3.35E-03	5.917	16.75	0.00	1.51E-03	7.673	47.46
2.00	0.38	1.71E-02	5.116	11.98	0.41	2.44E-02	6.067	28.14
3.00	0.77	3.08E-02	4.678	9.71	0.81	4.73E-02	5.367	21.27
4.00	1.15	4.45E-02	4.358	8.22	1.22	7.02E-02	4.881	17.08
5.00	1.53	5.82E-02	4.101	7.12	1.63	9.31E-02	4.504	14.16
6.00	1.91	7.19E-02	3.886	6.28	2.03	1.16E-01	4.196	11.99
7.00	2.30	8.56E-02	3.701	5.60	2.44	1.39E-01	3.935	10.33
8.00	2.68	9.93E-02	3.538	5.05	2.85	1.62E-01	3.709	9.01
9.00	3.06	1.13E-01	3.392	4.60	3.25	1.85E-01	3.511	7.95
10.00	3.45	1.27E-01	3.260	4.21	3.66	2.08E-01	3.334	7.09
11.00	3.83	1.40E-01	3.140	3.89	4.07	2.30E-01	3.174	6.39
12.00	4.21	1.54E-01	3.030	3.62	4.47	2.53E-01	3.030	5.80
13.00	4.21	1.54E-01	elastic	3.62	4.47	2.53E-01	elastic	5.80
14.00	9.69	3.50E-01	elastic	0.00	8.94	5.05E-01	elastic	0.00

	Q-System			
S.N	**_p_i**	**P_i**	**R_{pl}**	**u_r**
1.00	0.00	2.47E-03	9.075	62.92
2.00	0.45	3.48E-02	6.903	34.66
3.00	0.90	6.71E-02	5.971	25.04
4.00	1.34	9.94E-02	5.336	19.37
5.00	1.79	1.32E-01	4.851	15.54
6.00	2.24	1.64E-01	4.459	12.78
7.00	2.69	1.96E-01	4.132	10.71
8.00	3.14	2.29E-01	3.852	9.10
9.00	3.58	2.61E-01	3.609	7.84
10.00	4.03	2.93E-01	3.394	6.84
11.00	4.48	3.26E-01	3.202	6.04
12.00	4.93	3.58E-01	3.030	5.39
13.00	4.93	3.58E-01	elastic	5.39
14.00	9.08	6.57E-01	elastic	0.00

c. Support Characteristics Curve

Table 3-22 Calculation of SCC for MBKHEP (for section at 2886.6)

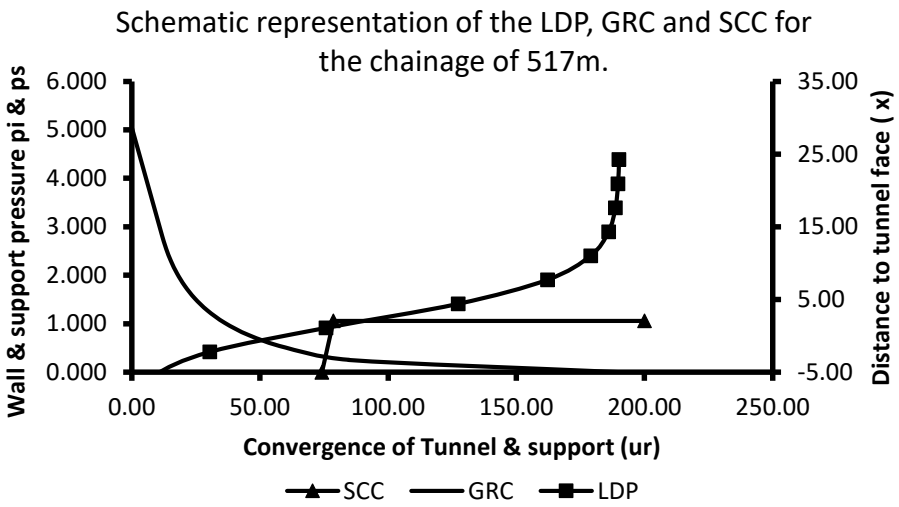
Observed Condition							
ur	**ps**					**For FOS,**	
6.53	0.000		**a**	3.03		P_s^{max}	1.56
10.70	1.559		**u_r^D**	9.00		P_s^d	0.80
20.00	1.559		**u_r^D/a (%)**	0.30%		FOS	1.95

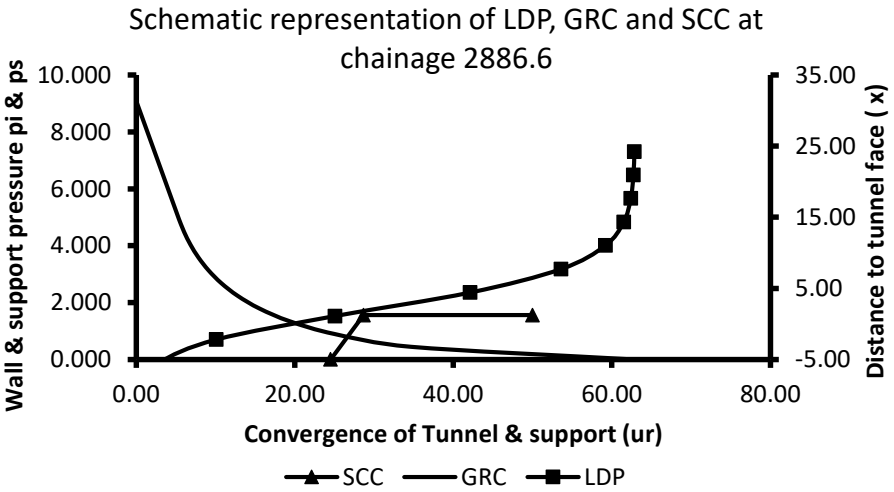
RMR							
ur	**ps**					**For FOS,**	
18.49	0.000		**a**	3.03		P_s^{max}	1.31
22.70	1.308		**u_r^D**	21.00		P_s^d	0.80
50.00	1.308		**u_r^D/a (%)**	0.69%		FOS	1.63

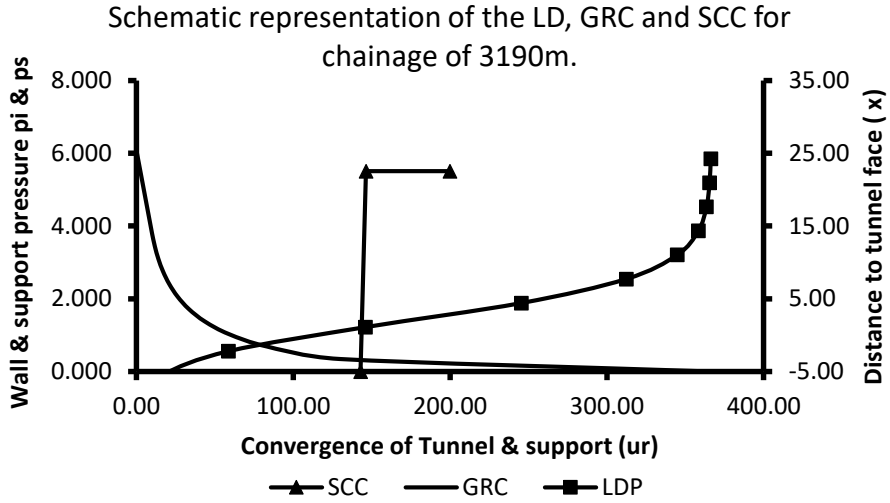
Q-System				
ur	**ps**		**For FOS,**	
24.52	0.000		P_s^{max}	0.66
28.83	0.664		P_s^d	0.60
50.00	0.664		FOS	1.11

3.2.1.8.1 Plot of LDP, GRC and SCC of Some Sections

a. Based on Observed Condition







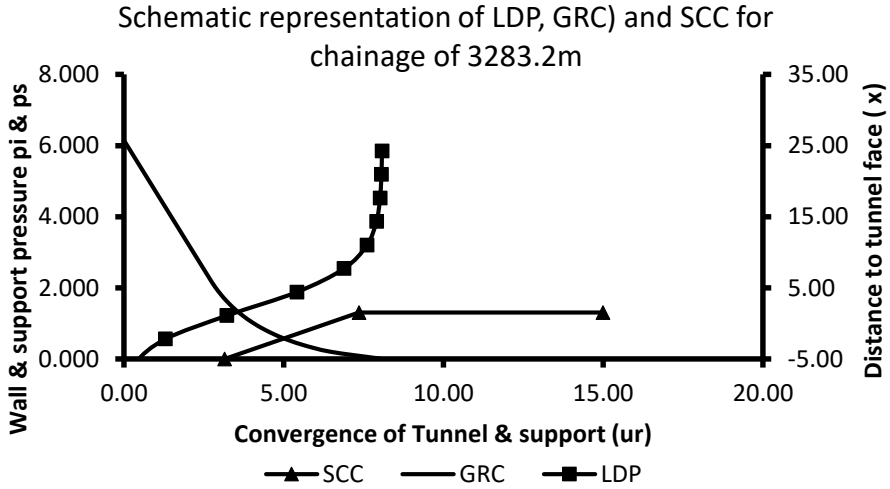
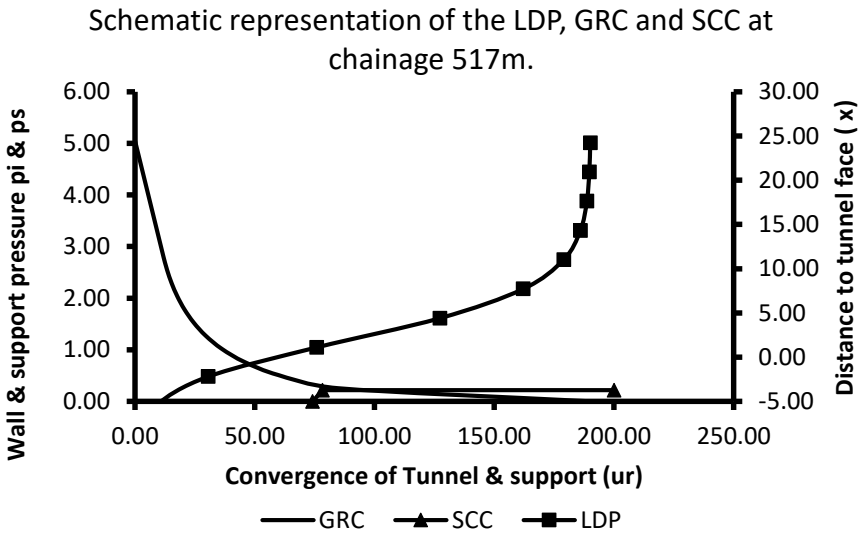
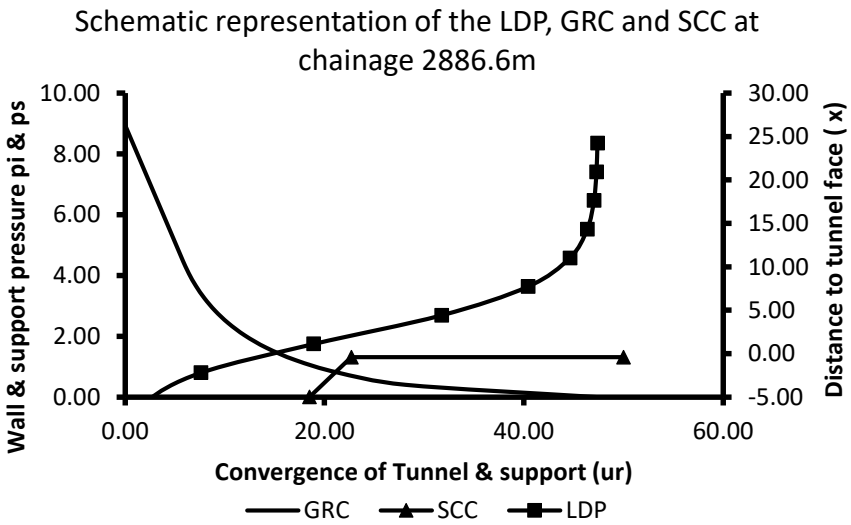
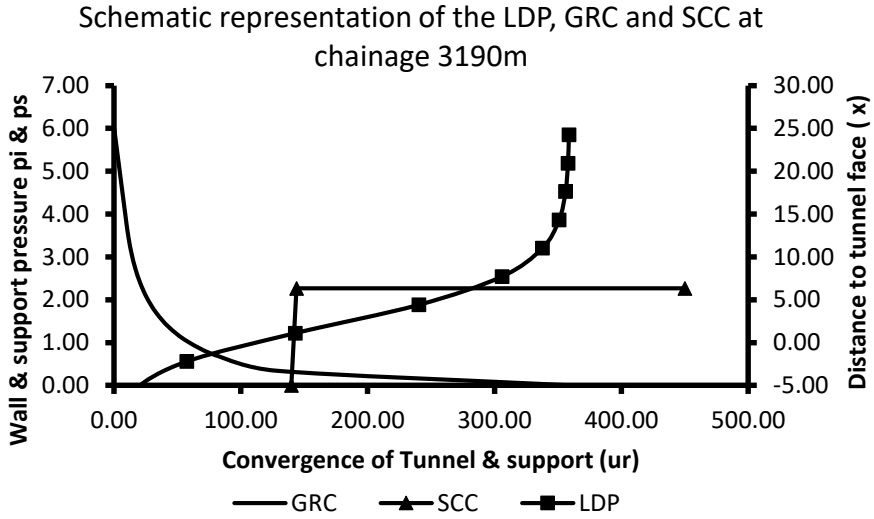


Figure 3-17 Plot of CCM for MBKHEP at Section (a) 517m, (b)2886.6m, (c)3190m and (d) 3283.2m based on Observed Condition

b. Based on RMR







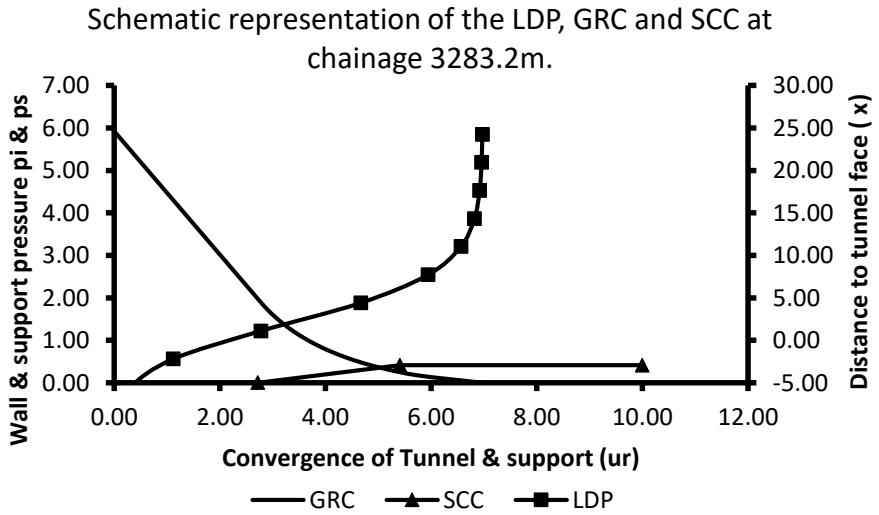
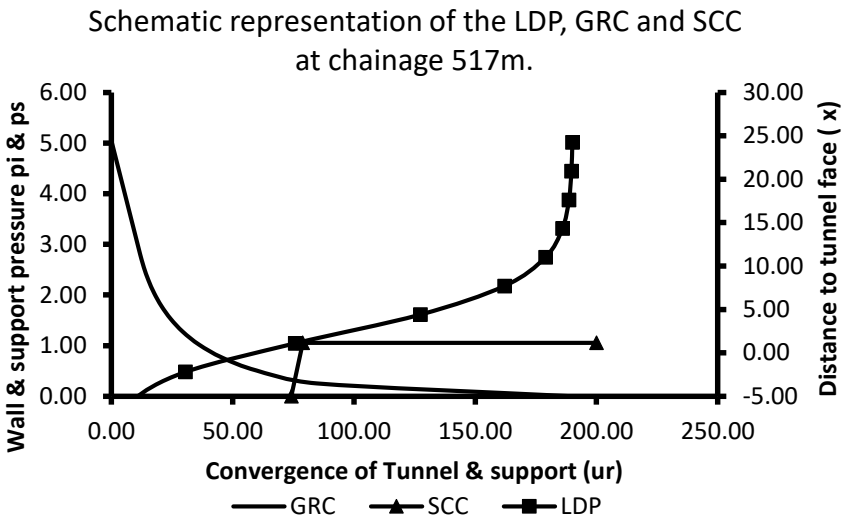
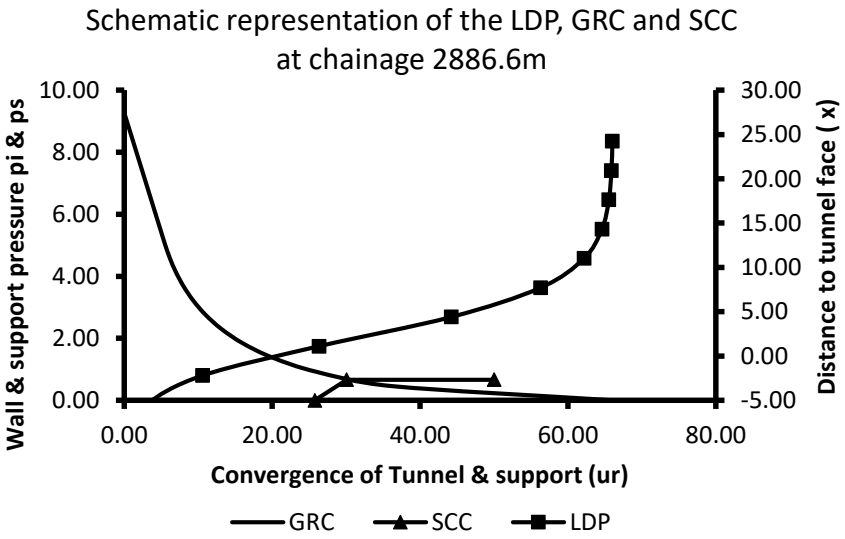
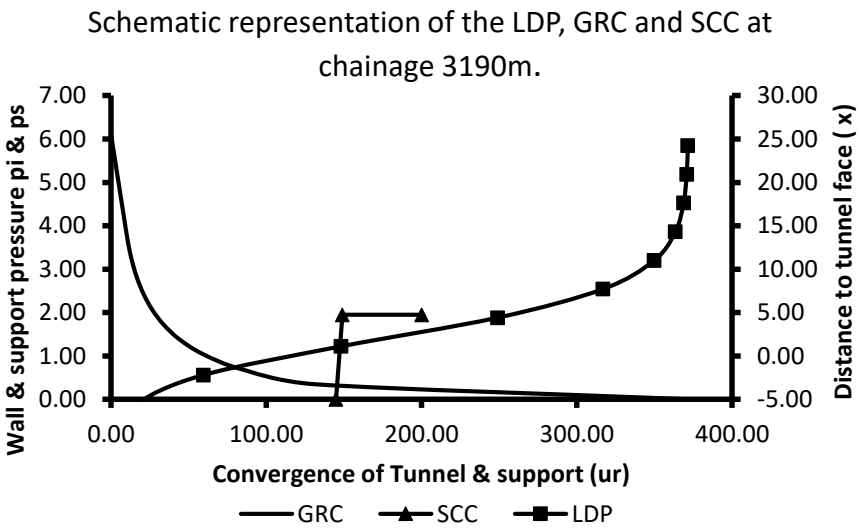


Figure 3-18 Plot of CCM for MBKHEP at Section (a) 517m, (b)2886.6m, (c)3190m and (d) 3283.2m based on RMR System

c. Based on Q-System







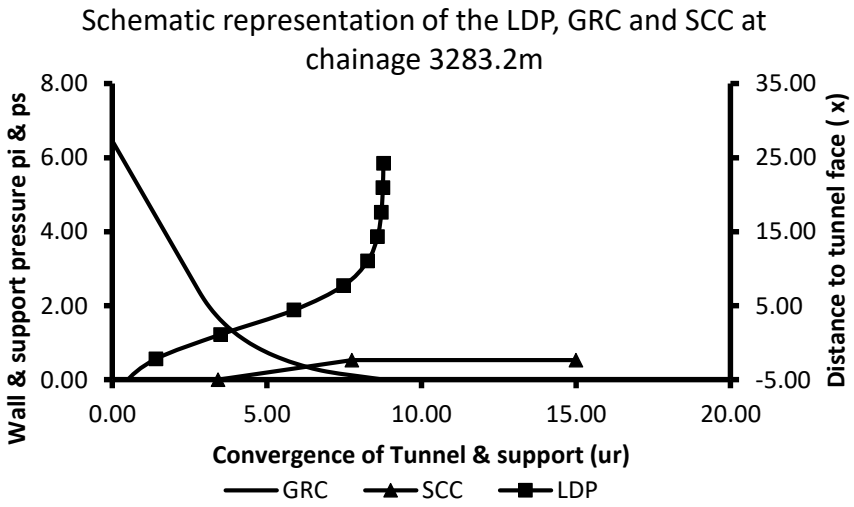


Figure 3-19 Plot of CCM for MBKHEP at Section (a) 517m, (b)2886.6m, (c)3190m and (d) 3283.2m based on Q System

3.2.1.8.2 Comparison of the Results

Table 3-23 Comparative Table for Support Pressure obtained from Observed, RMR and Q-System for MBKHEP

		Observed			RMR			Q-System		
Chainage	**Overburden**	**Maximum Support Pressure**	**Designed Support Pressure**	**Factor of Safety**	**Maximum Support Pressure**	**Designed Support Pressure**	**Factor of Safety**	**Maximum Support Pressure**	**Designed Support Pressure**	**Factor of Safety**
300	181.61	1.56	0.5	3.12	1.31	0.45	2.91	1.57	0.5	3.14
400	246.23	1.56	0.5	3.12	1.31	0.45	2.91	1.57	0.58	2.7
500	286.35	1.56	0.68	2.29	1.31	0.60	2.18	1.57	0.63	2.49
6795.9	147.43	0.67	0.25	2.68	1.31	0.30	4.3	1.95	0.5	3.9
3283.2	339.85	1.56	0.6	2.6	0.41	0.27	1.51	0.53	0.4	1.325
2886.6	523.56	1.56	0.8	1.95	1.31	0.80	1.63	0.66	0.6	1.1
7106.5	52.49	1.56	0.16	9.75	0.82	0.17	4.8	1.57	0.2	7.85
3190	355.13	1.56	0.6	2.6	2.26	0.37	6.1	1.95	0.39	5
6867.5	112.55	1.81	0.3	6.03	2.26	0.12	18.8	0.85	0.18	4.7

Comparison of the sections with known deformations.

Figure 3-20 shows the comparison of the support pressure obtained from the support system of observed site condition, as per RMR values and as per the Q-values. When we compare the majority of the sections, it is seen that the support pressure in the site observed is more than the ones we defined for the particular sections. More support pressure might have been provided because of the different conditions faced in the site. The support pressure design observed in the actual site is more robust than the ones designed using RMR and Q values. More support pressure is provided at the sections with highest overburden and poor rock mass quality. For example, the section with chainage 500m has maximum overburden compared to other sections and more support pressure is provided to the particular section in order to counter that overburden.

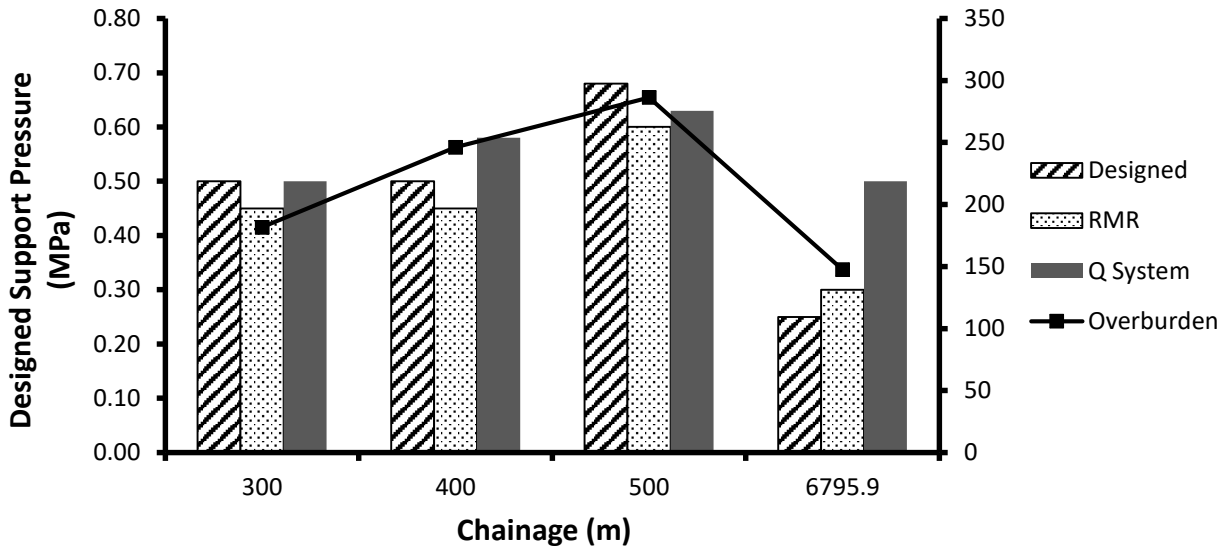


Figure 3-20 Comparison of Sections with known deformations for MBKHEP

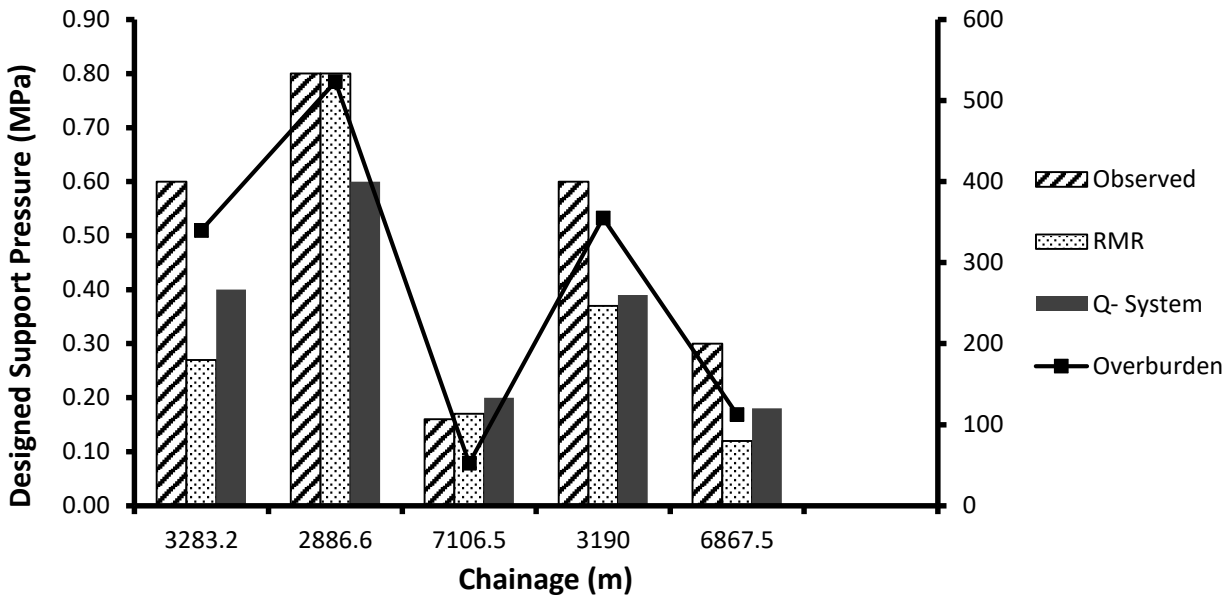


Figure 3-21 Comparison of Support Pressure at different sections of MBKHEP

3.2.1.9 Tunnel closure with and without Support

We observed the closure of the tunnel without support and then observed how the tunnel support limit the extent of the closure in the case of HRT of Middle Bhotekoshi Hydroelectric Project. Using analytical approach, tunnel closure without and without support was determined and compared in the tabulated form as follows:

Table 3-24 Tunnel Closure with and without support for MBKHEP

Chainage	**Closure without support**	**Closure with provided support**	**Closure with defined support**
3283.2	8.2	5	5.3
2886.6	63	27	22
7106.5	1.8	1.2	1.3
3190	365	145	145
6867.5	68	28	27

The table above shows the closure of the tunnel sections at different chainage without support. The support takes the pressure from the deforming rock mass and limits deformation to certain extent. The third column in the table shows the values of closure of different tunnel sections when the support system as provided in the actual site is assigned. The fourth column shows the tunnel closure when the support system defied as per RMR is assigned. The graphical comparison is shown below. The value of closure for the good rock masses is low whereas higher values of closure can be observed at the sections with poor rock masses.

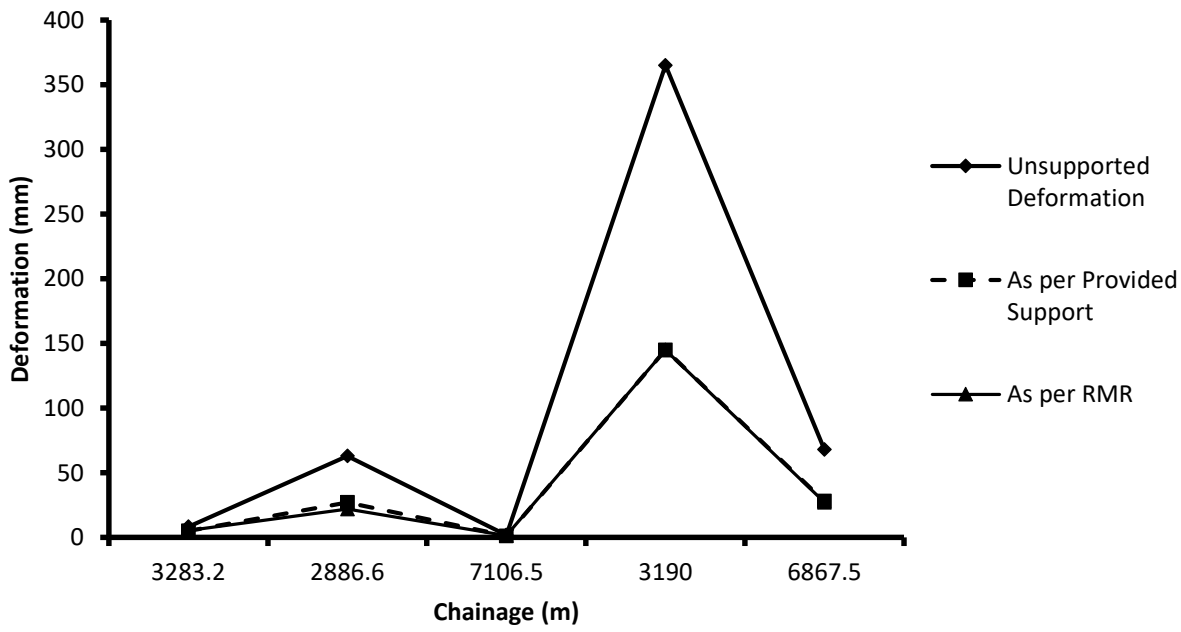


Figure 3-22 Comparison of closure for MBKHEP

3.2.2 Sanjen Hydroelectric Project (SHEP)

3.2.2.1 Project Description

Sanjen Hydroelectric Project is located in Chilime VDC, Rasuwa. It is located in the Upper Himalayan Region. The project area is situated in the Higher Himalaya and the Lesser Himalayan in the Central Nepal separated by Main Central Thrust (MCT). The Higher Himalayan consists of medium to high grade metamorphic rocks such as schist with gneiss and quartzite whereas the lower Himalayan consists of politic to psammitic and calcareous meta sediments, low grade metamorphic and carbonate rocks such as phyllite, slate, quartzite and dolomitic marble. The rock type in the project area can be broadly classified into seven different units which consists of Garnet-Schist with Augen Gneiss, Psammitic Schist with Quartzite, White Quartzite, Graphitic Schist with Crenulated Phyllite and Slate, Psammitic Schist with Crenulated Phyllite and Quartzite, Green-grey Quartzite and Dolomitic Marble.

The project area in the lesser Himalayan region is made complicated by the faults and folds. Since the area mainly consists of the metamorphic rocks, foliation is very common. Apart from foliation, it also includes crenulation cleavage, slaty cleavage, schistosity and gneissocity. Joints are observed in dolomitic marble, slate and quartzite and rare on phyllite, schist and augen gneiss. As a result of tectonic activity i.e. due to influence of the MCT a shear zone passes through the powerhouse of the Sanjen (Upper) Hydroelectric Project with a numbers of thin-bedded shear zones parallel to foliation joint.

The most common rocks that are observed in the tunnel alignment are Psammitic Schist with Quartzite, White Quartzite, Graphitic Schist with Crenulated Phyllite and Slate, Psammitic Schist with Crenulated Phyllite and Quartzite, Green-grey Quartzite and Dolomitic Marble. The overburden seems to lie between 34.36m to 355.33m.

SHEP is a cascade run off river project with the capacity of 42.5MW. The net head of SHEP is 436.19m with the discharge of 11.57m^3/s. The Intake type is the off-take from the tail water of Sanjen Upper Hydro Electric Project.

The headrace tunnel in SHEP is of length 3528m. The HRT is of D shape with the sectional area of 11.99m^2 having height of 3.75m and the width of 3.5m. The crown is of height of 1.75m and wall height of 2m. The HRT starts from 34.180m after the inlet portal. The length of HRT inlet portal is 34.18m with the section diameter of 3.9m.There are two audit tunnels connected to the HRT namely Audit 1 and Audit 3

3.2.2.2 Geology of Tunnel

Common rocks on the tunnel alignment are Psammitic Schist with Quartzite, White Quartzite, Graphitic Schist with Crenulated Phyllite and Slate, Psammitic Schist with Crenulated Phyllite and Quartzite, Green-grey Quartzite and Dolomitic Marble There is a presence of good overburden i.e. between 34.36 and 355.33 m. No major structural disturbances such as fault and major shear zones were observed during mapping in the area except thin bands of shear and fractured zones along foliation plane. Neither any deep nor active and recent landslide were found to intersect the tunnel. Distribution of Rock type along the Headrace tunnel is provided in Table 3-25.

Table 3-25 Distribution of Rock Types along Headrace Tunnel (edited after SHEP Feasibility Report, 2011)

Chainage (m)	Rock Type
0+000 - 0+755.14	Graphitic Schist with Crenulated Phyllite and Slate
0+755.14 - 1+052.04	Dolomitic Marble
1+052.05 - 1+282.82	Graphitic Schist with Crenulated Phyllite and Slate
1+282.82 - 1+498.48	Dolomitic Marble
1+498.48 - 1+644.54	Graphitic Schist with Crenulated Phyllite and Slate
1+644.54 - 2+130.22	Dolomitic Marble
2+130.22 - 2+402.49	Psammitic Schist with Crenulated Phyllite and Quartzite
2+402.49 - 2+581.06	Green-grey Quartzite
2+581.06 - 3+081.23	Psammitic Schist with Crenulated Phyllite and Quartzite
3+081.23 - 3+292.28	Graphitic Schist with Crenulated Phyllite and Slate
3+292.28 - 3+303.06	White Quartzite
3+303.06 - 3+629.67	Psammitic Schist with Quartzite

3.2.2.3 Support Classification

a. <u>Based on Q-Chart Guidelines (NGI (2013)) (after Barton et al (1993)</u>

Table 3-26 Different Parameters used for Support Design

Parameters	Value
Diameter	3.5
ESR	1.6
Equivalent Dimension	2.188
Factor of Safety	1
Effective De	2.188
Length of Bolt (m)	2.352
Diameter of Bolt (mm)	25

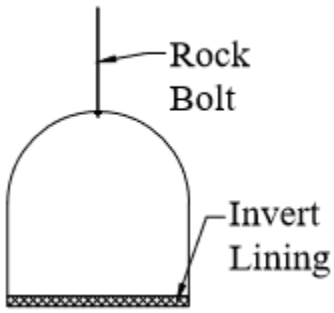
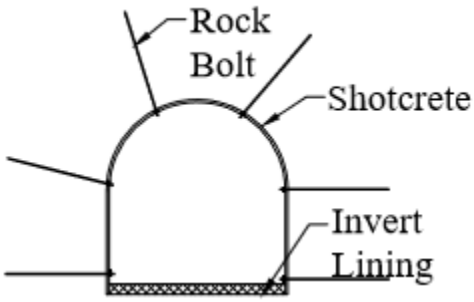
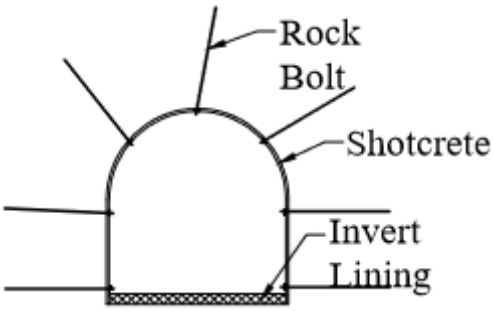
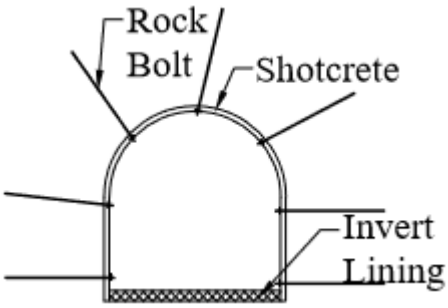
Table 3-27 Range of the support Category

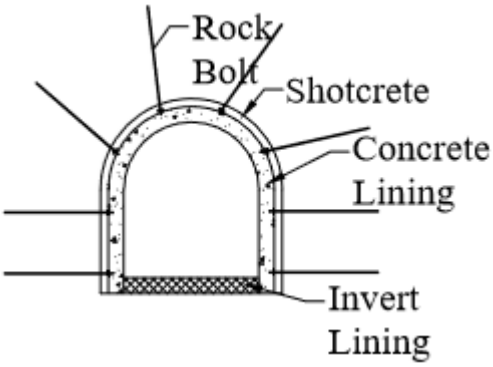
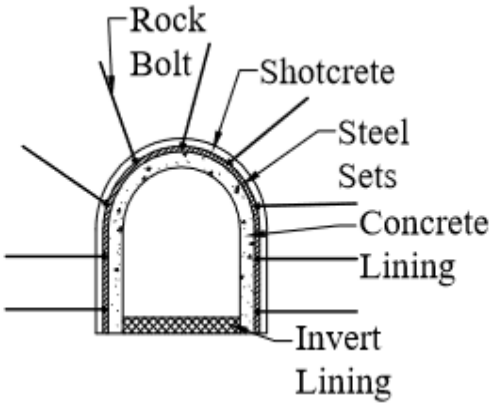
Defining Support Class		
Maximum	**Minimum**	**Category**
1000	1	S1-I
1	0.4	S1-II
0.4	0.1	S1-III
0.1	0.04	S1-IV
0.04	0.01	S1-V
0.01	0.001	S1-VI

Table 3-28 Support Description using Q-Support Chart (NGI (2013)) (after Bashin and Grimstad, 1993)

Support Class	Description of Support
S1-I	Bolting @ 1.3c/c
S1-II	Fiber reinforced shotcrete of 5cm thickness and bolting @ 1.5m c/c
S1-III	Fiber reinforced shotcrete of 9cm thickness and bolting @ 1.3m c/c
S1-IV	Fiber reinforced shotcrete of 12cm thickness and bolting @ 1.2m c/c
S1-V	Fiber reinforced shotcrete of 15cm thickness and bolting @ 1.0m c/c
S1-VI	Fiber reinforced shotcrete of 25 cm thickness and bolting @ 1.0m c/c

Table 3-29 Support Description of rock support provided in HRT of SHEP

Support Category	**Q-value Range**		**Support Description**
S-I	>4		Spot Bolting and Invert lining of 200mm thickness.
S-II	1-4		50mm SFRS with pattern bolting at 1.8m c/c with 2m length, 20mm diameter of bolt and Invert lining of 200mm thickness
S-III	0.5-1		50mm SFRS with pattern bolting at 1.6m c/c with 2m length, 20mm diameter of bolt and Invert lining of 200mm thickness
S-IV	0.1-0.5		100mm and 50mm SFRS at crown and wall respectively with pattern bolting at 1.4m c/c with 2m length, 20mm diameter of bolt and Invert lining of 200mm thickness

S-V	0.01-0.1		150mm and 100mm SFRS at crown and wall respectively with pattern bolting at 1.2m c/c with 2m length, 20mm diameter of bolt and Invert lining of 300mm thickness and concrete lining of 300mm
S-VI	<0.01		150mm SFRS at with pattern bolting at 1m c/c with 2m length, 20mm diameter of bolt and Invert lining of 300mm thickness and concrete lining of 300mm with ISMB 75 X 150 at 1m c/c.

3.2.2.4Empirical Approach for prediction of Squeezing

a. Singh et al. (1992)

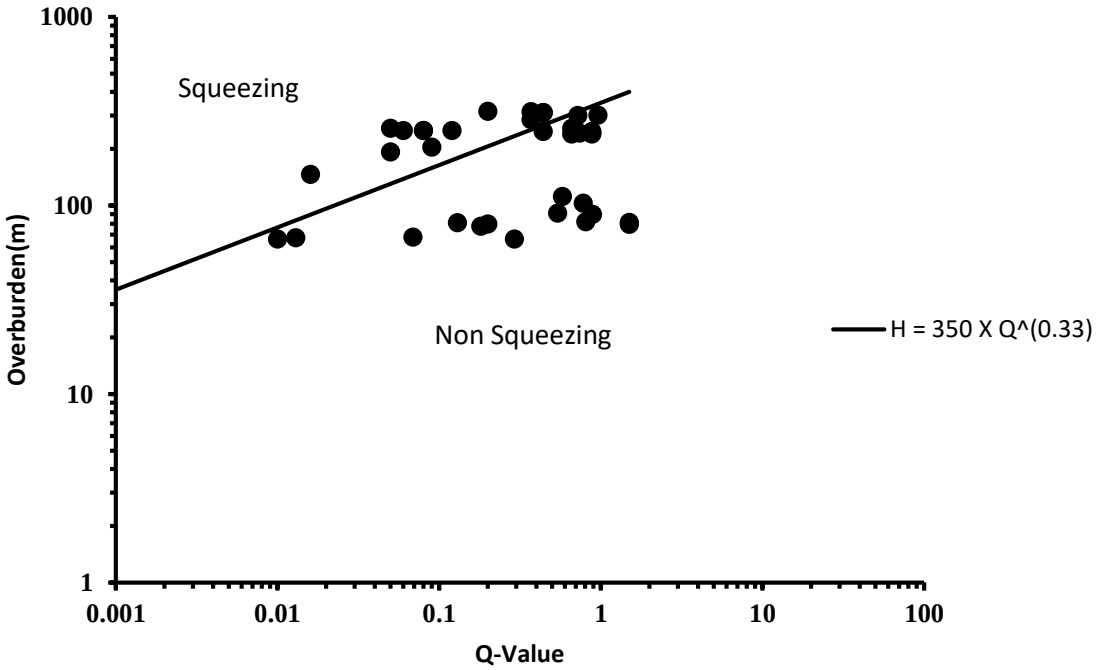


Figure 3-23 Squeezing Analysis using Singh et al. (1992)

b. Goel et al. (1994)

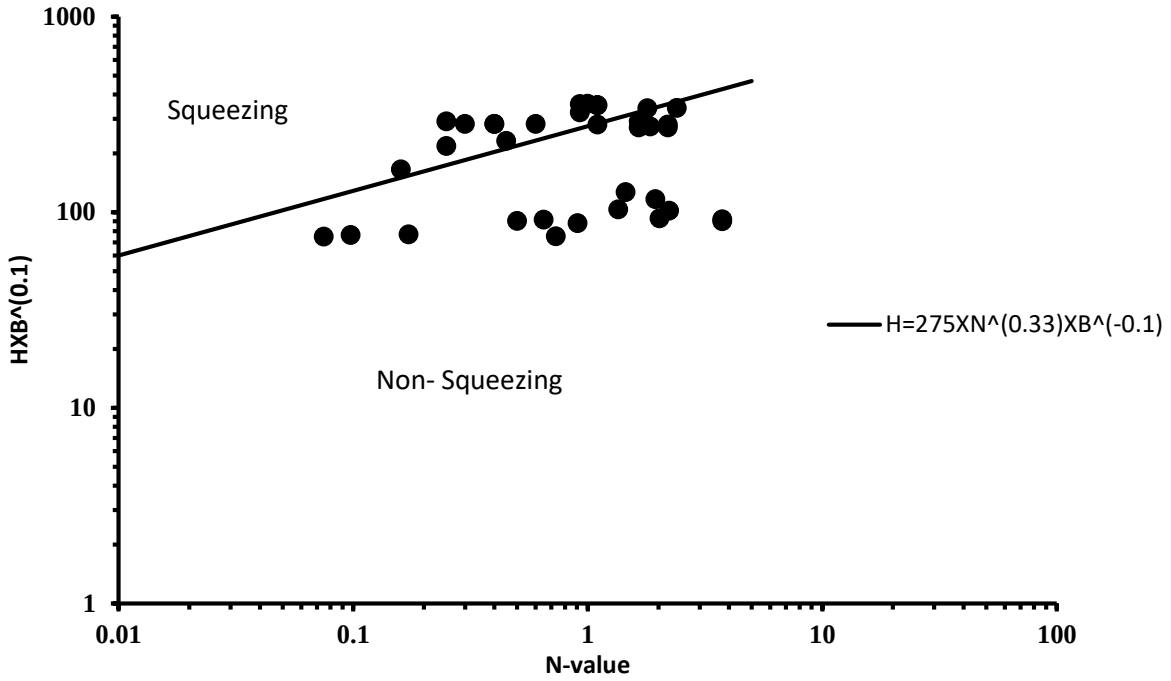


Figure 3-24 Squeezing Analysis Graph using Geol et al. (1994)

c. Jimenez et al. (2011)

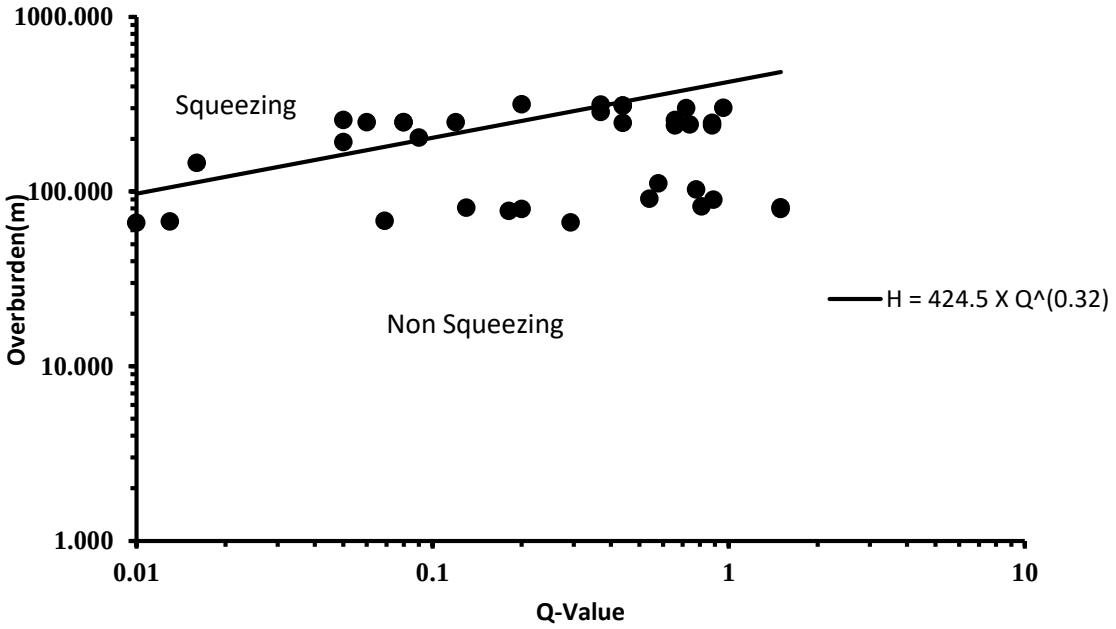


Figure 3-25 Squeezing Analysis using Jimenez et al. (2011)

Table 3-30 Squeezing Assessment and Rock Burst Condition

Chainage (m)		Q - Value	SRF	Overburden (m)	$HxB^{0.1}$	Status of Squeezing(Singh et.al Criteria)	Jr/Ja	Rock Burst condition (Geol et.al)	N (Rock Mass Number) (Geol et. al)	Squeezing Category By Goel et al. (1995)	Squeezing Status Using Jimenez and Recio(2011)
Starting	Ending										
2+266	2+264.6	0.88	2.5	240.00	272.03	No Squeezing	0.33	No	2.20	No squeezing	No Squeezing
2+263.8	2+261.4	0.66	2.5	240.00	272.03	No Squeezing	0.25	No	1.65	No squeezing	No Squeezing
2+259.1	2+256.7	0.74	2.5	242.50	274.86	No Squeezing	0.33	No	1.85	No squeezing	No Squeezing
2+256.7	2+254.0	0.88	2.5	247.50	280.53	No Squeezing	0.33	No	2.20	No squeezing	No Squeezing
2+251.3	2+248.4	0.44	2.5	247.50	280.53	No Squeezing	0.25	No	1.10	No squeezing	No Squeezing
2+248.4	2+245.6	0.12	5	250.00	283.37	Squeezing	0.13	No	0.60	Very Mild Squeezing	Squeezing
2+245.6	2+244.3	0.08	5	250.00	283.37	Squeezing	0.13	No	0.40	Mild Squeezing	Squeezing
2+244.3	2+242.7	0.08	5	250.00	283.37	Squeezing	0.13	No	0.40	Mild Squeezing	Squeezing
2+242.7	2+241.5	0.06	5	250.00	283.37	Squeezing	0.13	No	0.30	Mild Squeezing	Squeezing
2+240.3	2+237.6	0.05	5	257.50	291.87	Squeezing	0.13	No	0.25	Mild to Moderate	Squeezing
2+234.1	2+231.2	0.66	2.5	257.50	291.87	No Squeezing	0.33	No	1.65	No squeezing	No Squeezing
2+160.7	2+157.9	0.37	2.5	315.00	357.04	Squeezing	0.25	No	0.93	Mild Squeezing	Squeezing
2+164.5	2+162.0	0.37	2.5	285.667	323.79	Squeezing	0.25	No	0.93	Very Mild Squeezing	No Squeezing
2+137.7	2+135.3	0.72	2.5	300.401	340.49	No Squeezing	0.25	No	1.80	Very Mild Squeezing	No Squeezing
2+135.3	2+134.0	0.96	2.5	301.72	341.99	No Squeezing	0.33	No	2.40	No squeezing	No Squeezing
2+118.2	2+117.0	0.44	2.5	311.122	352.64	Squeezing	0.25	No	1.10	Very Mild Squeezing	No Squeezing
2+117	2+114.0	0.44	2.5	311.781	353.39	Squeezing	0.33	No	1.10	Very Mild Squeezing	No Squeezing
2+094.6	2+091.8	0.2	5	316.576	358.83	Squeezing	0.25	No	1.00	Very Mild Squeezing	Squeezing

1+957.546	1+955	0.09	5	204.316	231.58	Squeezing	0.33	No	0.45	Very Mild Squeezing	Squeezing
1+938.460	1+937.00	0.05	5	192.453	218.14	Squeezing	0.33	No	0.25	Very Mild Squeezing	Squeezing
1+864.112	1+861.804	0.02	10	146.233	165.75	Squeezing	0.25	No	0.16	Very Mild Squeezing	Squeezing
1+808.768	1+806.963	0.58	2.5	111.813	126.74	No Squeezing	0.33	No	1.45	No squeezing	No Squeezing
1+794.655	1+791.877	0.78	2.5	103.033	116.78	No Squeezing	0.50	No	1.95	No squeezing	No Squeezing
1+759.88	1+756.881	1.5	2.5	81.423	92.29	No Squeezing	0.75	Yes	3.75	No squeezing	No Squeezing
1+756.881	1+754.05	1.5	2.5	79.558	90.18	No Squeezing	0.75	Yes	3.75	No squeezing	No Squeezing
1+645.488	1+643.084	0.29	2.5	66.562	75.45	No Squeezing	0.33	No	0.73	No squeezing	No Squeezing
1+643.084	1+640.48	0.01	7.5	66.384	75.24	No Squeezing	0.08	No	0.08	No squeezing	No Squeezing
1+637.949	1+635.30	0.01	7.5	67.545	76.56	No Squeezing	0.08	No	0.10	No squeezing	No Squeezing
1+635.30	1+632.48	0.07	2.5	68.141	77.24	No Squeezing	0.08	No	0.17	No squeezing	No Squeezing
1+596.0	1+594.33	0.18	5	77.707	88.08	No Squeezing	0.13	No	0.91	No squeezing	No Squeezing
1+590.860	1+589.23	0.2	2.5	79.752	90.40	No Squeezing	0.13	No	0.50	No squeezing	No Squeezing
1+587.730	1+585.730	0.13	5	80.998	91.81	No Squeezing	0.33	No	0.65	No squeezing	No Squeezing
1+584.72	1+583.48	0.81	2.5	82.197	93.17	No Squeezing	0.67	Yes	2.03	No squeezing	No Squeezing
1+565.10	1+563.23	0.89	2.5	89.999	102.01	No Squeezing	0.50	No	2.23	No squeezing	No Squeezing
1+563.23	1+561.98	0.54	2.5	91.247	103.42	No Squeezing	0.67	Yes	1.35	No squeezing	No Squeezing

3.2.2.5 Support System Provided in Project

Table 3-31 Calculation of Support Pressure using Empirical Relations

Chainage (m)		Q-value	Q-value including Wall factor (Qw)	Overburden(m)	UCS (MPa)	Support Pressure Using Barton et. al (1974) (MPa)		Support Pressure Bhasin and Grimstad (1996)(MPa)	f(N)	Support Pressure using Geol(1995a) and Singh et al.(1997)(MPa)
Starting	Ending					Roof Pressure (Pv)	Wall Pressure (Ph)			
2+266	2+264.6	0.88	2.2	240.00	78.77	0.209	0.154	0.146	0.00	0.133
2+263.8	2+261.4	0.66	1.65	240.00	78.77	0.230	0.169	0.161	0.00	0.150
2+259.1	2+256.7	0.74	1.85	242.50	78.77	0.221	0.163	0.155	0.00	0.143
2+256.7	2+254.0	0.88	2.2	247.50	78.77	0.209	0.154	0.146	0.00	0.134
2+251.3	2+248.4	0.44	1.1	247.50	78.77	0.263	0.194	0.184	0.00	0.178
2+248.4	2+245.6	0.12	0.3	250.00	78.77	0.405	0.299	0.284	1.50	0.248
2+245.6	2+244.3	0.08	0.08	250.00	78.77	0.464	0.464	0.325	1.20	0.249
2+244.3	2+242.7	0.08	0.08	250.00	78.77	0.464	0.464	0.325	1.20	0.249
2+242.7	2+241.5	0.06	0.06	250.00	78.77	0.511	0.511	0.357	1.20	0.299
2+240.3	2+237.6	0.05	0.05	257.50	78.77	0.543	0.543	0.380	1.00	0.293
2+234.1	2+231.2	0.66	1.65	257.50	78.77	0.230	0.169	0.161	0.00	0.151
2+160.7	2+157.9	0.37	0.925	315.00	78.77	0.279	0.205	0.195	1.20	0.197
2+164.5	2+162.0	0.37	0.925	285.67	78.77	0.279	0.205	0.195	1.50	0.225
2+137.7	2+135.3	0.72	1.8	300.40	78.77	0.223	0.164	0.156	1.50	0.173
2+135.3	2+134.0	0.96	2.4	301.72	78.77	0.203	0.149	0.142	0.00	0.132
2+118.2	2+117.0	0.44	1.1	311.12	136.84	0.263	0.194	0.184	1.50	0.223
2+117	2+114.0	0.44	1.1	311.78	136.84	0.263	0.194	0.184	1.50	0.223
2+094.6	2+091.8	0.2	0.5	316.58	136.84	0.342	0.252	0.239	1.50	0.237
1+957.5	1+955	0.09	0.09	204.32	136.84	0.446	0.446	0.312	1.50	0.238
1+938.4	1+937.0	0.05	0.05	192.45	136.84	0.543	0.543	0.380	1.50	0.310
1+864.1	1+861.8	0.016	0.016	146.23	136.84	0.794	0.794	0.555	1.50	0.301
1+808.7	1+806.9	0.58	1.45	111.81	136.84	0.240	0.177	0.168	0.00	0.144
1+794.6	1+791.8	0.778	1.945	103.03	136.84	0.145	0.107	0.101	0.00	0.126
1+759.8	1+756.8	1.5	3.75	81.42	136.84	0.116	0.086	0.082	0.00	0.091
1+756.8	1+754.0	1.5	3.75	79.56	136.84	0.116	0.086	0.082	0.00	0.090
1+645.4	1+643.0	0.293	0.732	66.56	136.84	0.301	0.222	0.211	0.00	0.178
1+643	1+640.4	0.01	0.01	66.38	66.69	0.928	0.928	0.649	0.00	0.421
1+637.9	1+635.3	0.013	0.013	67.55	66.69	0.850	0.850	0.595	0.00	0.383
1+635.3	1+632.4	0.069	0.069	68.14	66.69	0.488	0.488	0.341	0.00	0.311
1+596.0	1+594.3	0.181	0.452	77.71	66.69	0.354	0.261	0.247	0.00	0.167
1+590.8	1+589.2	0.2	0.5	79.75	66.69	0.279	0.206	0.239	0.00	0.212

1+587.7	1+585.7	0.13	0.325	81.00	66.69	0.395	0.291	0.276	0.00	0.191
1+584.7	1+583.4	0.81	2.025	82.20	66.69	0.107	0.079	0.075	0.00	0.120
1+565.1	1+563.2	0.89	2.225	90.00	66.69	0.104	0.077	0.073	0.00	0.116
1+563.2	1+561.9	0.54	1.35	91.25	66.69	0.123	0.090	0.086	0.00	0.144

Table 3-32 Calculation of Deformation using Barton (2008)

Chainage(m)		UCS of intact Rock mass (MPa)	Modulus of Intact Rock mass (Ei) (MPa)	Vertival Stress (σv) (MPa)	Stress Ratio(k) (Shorey (1994))	Horizontal Stress(σh)(MPa)	Barton (2008) deformation of Roof and Wall	
Starting	Ending						Roof (mm)	Wall (mm)
2+266	2+264.6	78.77	31508	6.480	1.390	9.004	0.011	0.007
2+263.8	2+261.4	78.77	31508	6.480	1.390	9.004	0.015	0.009
2+259.1	2+256.7	78.77	31508	6.548	1.380	9.036	0.014	0.008
2+256.7	2+254.0	78.77	31508	6.683	1.362	9.100	0.012	0.007
2+251.3	2+248.4	78.77	31508	6.683	1.362	9.100	0.023	0.013
2+248.4	2+245.6	78.77	31508	6.750	1.353	9.131	0.085	0.049
2+245.6	2+244.3	78.77	31508	6.750	1.353	9.131	0.128	0.073
2+244.3	2+242.7	78.77	31508	6.750	1.353	9.131	0.128	0.073
2+242.7	2+241.5	78.77	31508	6.750	1.353	9.131	0.171	0.098
2+240.3	2+237.6	78.77	31508	6.953	1.327	9.227	0.208	0.119
2+234.1	2+231.2	78.77	31508	6.953	1.327	9.227	0.016	0.009
2+160.7	2+157.9	78.77	31508	8.505	1.171	9.957	0.031	0.018
2+164.5	2+162.0	78.77	31508	7.713	1.243	9.584	0.030	0.017
2+137.7	2+135.3	78.77	31508	8.111	1.205	9.772	0.016	0.009
2+135.3	2+134.0	78.77	31508	8.146	1.202	9.788	0.012	0.007
2+118.2	2+117.0	136.84	54736	8.400	1.865	15.664	0.020	0.011
2+117	2+114.0	136.84	54736	8.418	1.862	15.675	0.020	0.011
2+094.6	2+091.8	136.84	54736	8.548	1.843	15.757	0.044	0.025
1+957.5	1+955	136.84	54736	5.517	2.508	13.838	0.078	0.045
1+938.4	1+937	136.84	54736	5.196	2.624	13.635	0.136	0.078
1+864.1	1+861.8	136.84	54736	3.948	3.253	12.845	0.372	0.212
1+808.7	1+806.9	136.84	54736	3.019	4.060	12.257	0.009	0.005
1+794.6	1+791.8	136.84	54736	2.782	4.352	12.106	0.006	0.004
1+759.8	1+756.8	136.84	54736	2.198	5.339	11.737	0.003	0.002
1+756.8	1+754.1	136.84	54736	2.148	5.449	11.705	0.003	0.002
1+645.5	1+643.1	136.84	54736	1.797	6.389	11.483	0.014	0.008
1+643.1	1+640.4	66.69	26676	1.792	3.250	5.825	0.574	0.328

1+637.9	1+635.3	66.69	26676	1.824	3.201	5.838	0.445	0.254
1+635.3	1+632.5	66.69	26676	1.840	3.177	5.845	0.084	0.048
1+596	1+594.3	66.69	26676	2.098	2.840	5.958	0.034	0.020
1+590.8	1+589.2	66.69	26676	2.153	2.778	5.982	0.031	0.018
1+587.7	1+585.7	66.69	26676	2.187	2.742	5.997	0.049	0.028
1+584.7	1+583.5	66.69	26676	2.219	2.708	6.011	0.008	0.005
1+565.1	1+563.2	66.69	26676	2.430	2.512	6.103	0.008	0.004
1+563.2	1+561.9	66.69	26676	2.464	2.483	6.118	0.012	0.007

3.2.2.6 Comparison of SHEP sections using Empirical Approach

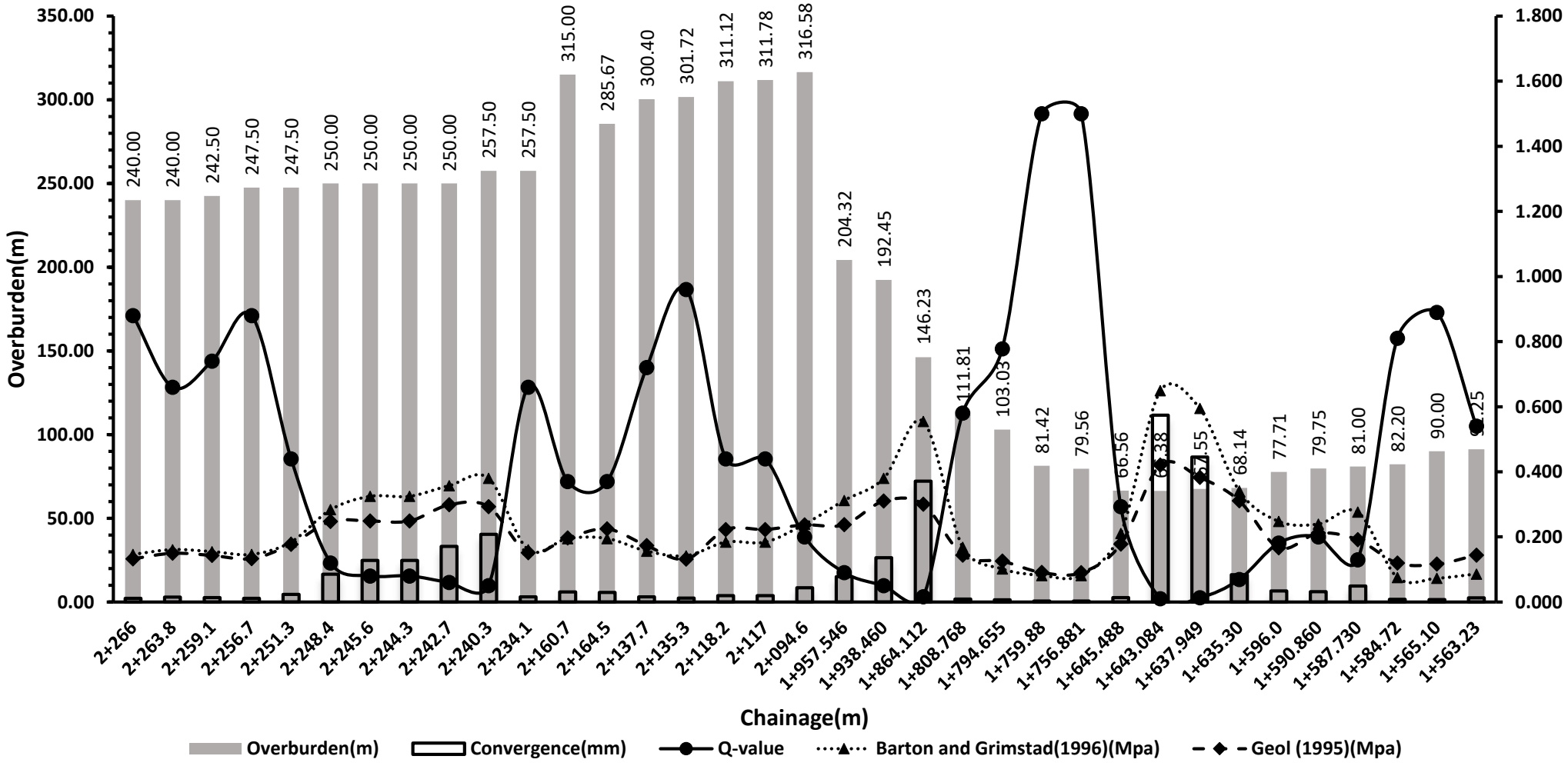


Figure 3-26 Plot Showing relation between different parameters for underground opening of SHEP

Observation made by the analysis if empirical approach of SHEP Head Race Tunnel

Using different empirical relation, head race tunnel sections of Sanjen Hydroelectric Project different parameters were studied and analysis was made accordingly. The observation made for SHEP sections are as follows:

- Stability and support system definition greatly varies with both internal and external factors. Internal factors being the characteristics of rock mass characteristics- its strength, overburden, structural arrangement of joints and external factor comprising of accessibility of material, equipment and manpower.

3.2.2.7 Convergent Confinement Analysis

Using all the parameters mentioned above along with the considerations for Rock mass classification and the support system assigned, LDP, GRC and SCC for different selected sections were computed and the graph showing the respective curves was plotted with the help of excel file prepared. We have included the CCM computation for different sections with highest overburden, deformed sections etc on the basis of observed conditions and Q- System Classification.

Table 3-33 Value of Parameters used while construction of CCM for SHEP (for section at 1643.084)

Chainage	1+643.084
OverBurden	66.384
Dimension	3.5x3.75
R(m)	1.94
H(m)	66.38
σ_0 (MPa)	1.20
ν	0.25
L(m)	1.00
Q	0.01
Ei(Gpa)	33.35

Erm(Gpa)	0.78
Disturbance Factor	0.8
Friction Angle	45.9
E_{rm} (GPa)	0.71
G_{rm} (GPa)	0.28
m_i	7.00
GSI	15.00
mb	0.07
s	0.00E+00
a	0.58

σ_{ci} (MPa)	66.69
S_0	2.74E-01
k	0.34
σ_v (MPa)	1.79
σ_h (MPa)	0.61
P_i^{cr} (MPa)	1.09E-01
p_i^{cr} (MPa)	0.4767
$C(\sigma_{ci})$	0.82
Ψ (degree)	0.00
K_Ψ	1.00

For Steel Sets	
σys(MPa)	245
Es(Gpa)	210.00
As	0.0091

For Shotcrete	
σ_{cc}(MPa)	40.00
ν_c	0.25
t_c (mm)	150.00

For Bolts	
bolt provided	yes
l (m)	2.00
d_b (mm)	20.00

Sl	2
ro	1.94
Pss Max	0.574613
Kss	253.8793
Ur max	2.263333

E_c (GPa)	25.00
P_s^{max} (MPa)	1.86
K_s (MPa/m)	1134.12
u_r^{max} (mm)	1.64

Combination of Supports	
Ks(MPa/m)	207.65
urmax(mm)	1.64
Psmax(MPa)	0.340

E_b (Gpa)	210.00
Q_b (m/MN)	0.14
T_{bf} (MN)	0.13
sc (m)	1.00
sl (m)	1.00
Ps_{max} (MPa)	0.13
k_s (MPa/m)	6.16
u_{rbmax} (mm)	20.63

3.2.2.8 Computation of CCM

a. Longitudinal Deformation Profile

Table 3-34 Calculation of LDP for SHEP (for section at 1643.084)

				Observed	**Q-System**
				u_r^0 (mm) = 5.183	*u_r^0 (mm) = 5.183*
S.N	**x/R**	**x**	**u_r/u_r^m**	**ur**	**ur**
1	-4.00	-7.76	0.002	0.02	0.02
2	-2.91	-5.64	0.010	0.12	0.12
3	-1.82	-3.53	0.045	0.53	0.53
4	-0.73	-1.41	0.160	1.90	1.90
5	0.36	0.71	0.398	4.72	4.72
6	1.45	2.82	0.669	7.93	7.93
7	2.55	4.94	0.852	10.09	10.09
8	3.64	7.05	0.941	11.14	11.14
9	4.73	9.17	0.977	11.57	11.57
10	5.82	11.29	0.991	11.74	11.74
11	6.91	13.40	0.997	11.81	11.81
12	8.00	15.52	0.999	11.83	11.83

b. Ground Reaction Curve

Table 3-35 Calculation of GRC for SHEP (for section at 1643.084)

	Observed				Q			
SN	**_p_i**	**P_i**	**R_{pl}**	**u_r**	**_p_i**	**P_i**	**R_{pl}**	**u_r**
1.00	0.00	0.00E+00	3.755	11.84	0.00	0.00E+00	3.755	11.84
2.00	0.04	9.91E-03	3.077	7.35	0.04	9.91E-03	3.077	7.35
3.00	0.09	1.98E-02	2.833	6.01	0.09	1.98E-02	2.833	6.01
4.00	0.13	2.97E-02	2.660	5.14	0.13	2.97E-02	2.660	5.14
5.00	0.17	3.96E-02	2.521	4.51	0.17	3.96E-02	2.521	4.51
6.00	0.22	4.95E-02	2.406	4.02	0.22	4.95E-02	2.406	4.02
7.00	0.26	5.95E-02	2.306	3.63	0.26	5.95E-02	2.306	3.63
8.00	0.30	6.94E-02	2.217	3.31	0.30	6.94E-02	2.217	3.31
9.00	0.35	7.93E-02	2.138	3.05	0.35	7.93E-02	2.138	3.05
10.00	0.39	8.92E-02	2.066	2.82	0.39	8.92E-02	2.066	2.82
11.00	0.43	9.91E-02	2.001	2.63	0.43	9.91E-02	2.001	2.63
12.00	0.48	1.09E-01	1.940	2.47	0.48	1.09E-01	1.940	2.47
13.00	0.48	1.09E-01	elastic	2.47	0.48	1.09E-01	elastic	2.47
14.00	1.20	2.74E-01	elastic	0.00	1.20	2.74E-01	elastic	0.00

c. Support Characteristic Curve

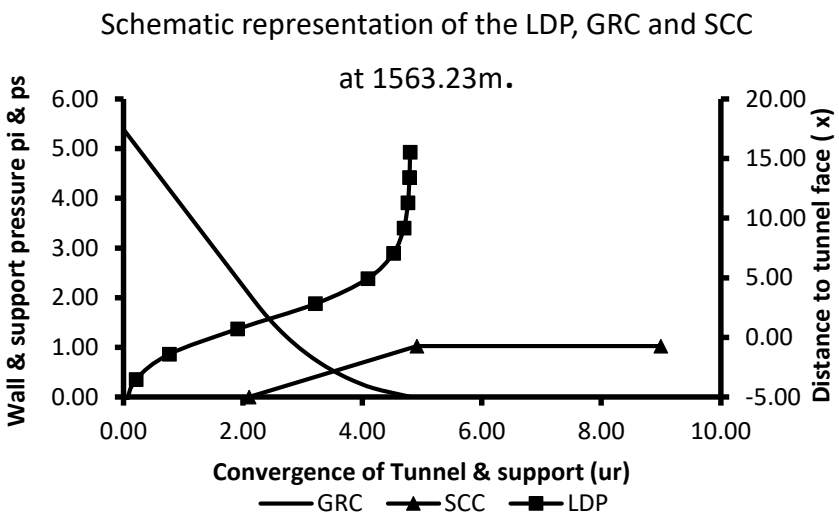
Table 3-36 Calculation of SCC for SHEP (for section at 1643.084)

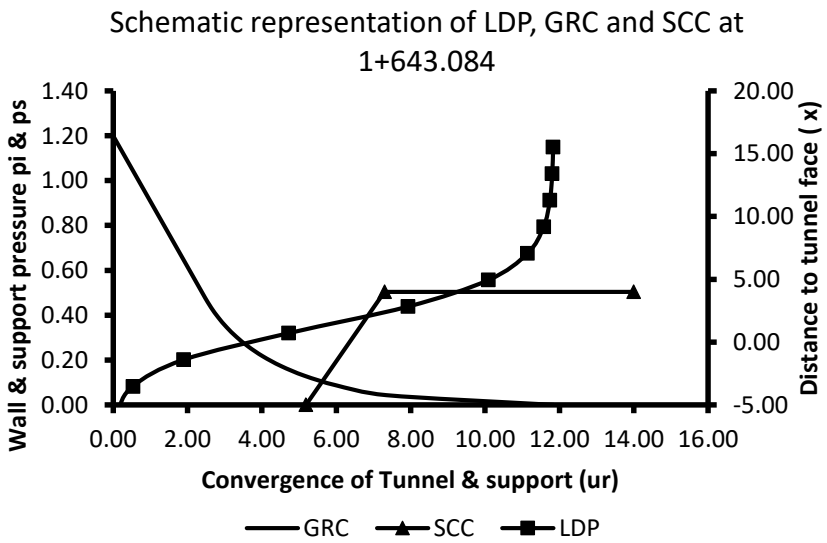
Observed Condition					
ur	**ps**	**For FOS**		**Tunnel Closure**	
5.18	0.000	**P_s^{max}**	0.50	**a**	1.94
6.82	0.504	**P_s^{d}**	0.10	**u_r^D**	5.20
14.00	0.504	**FOS**	5.04	**u_r^D/a (%)**	0.27%

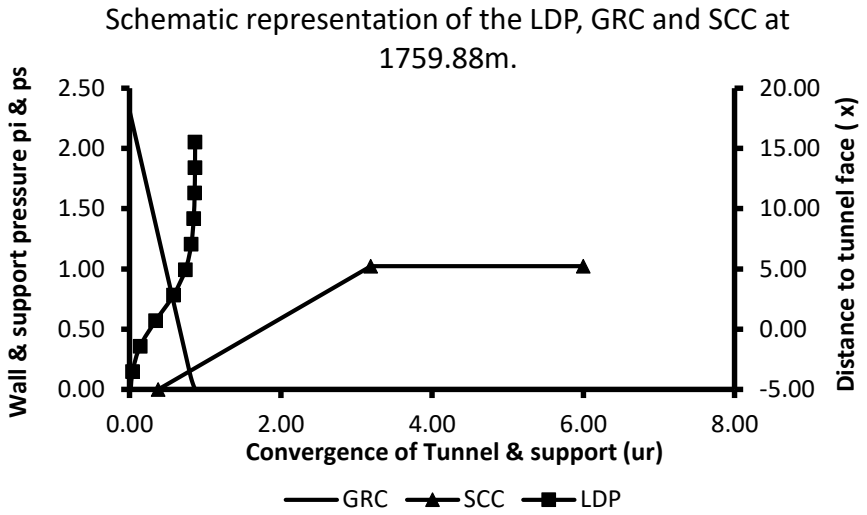
Q-System			
ur	**ps**	**For FOS,**	
5.18	0.000	**P_s^{max}**	4.84
7.63	4.837	**P_s^{d}**	0.2
14.00	4.837	**FOS**	24.19

3.2.2.8.1 Plot of LDP, GRC and SCC of some Sections

a. Based on Observed Condition







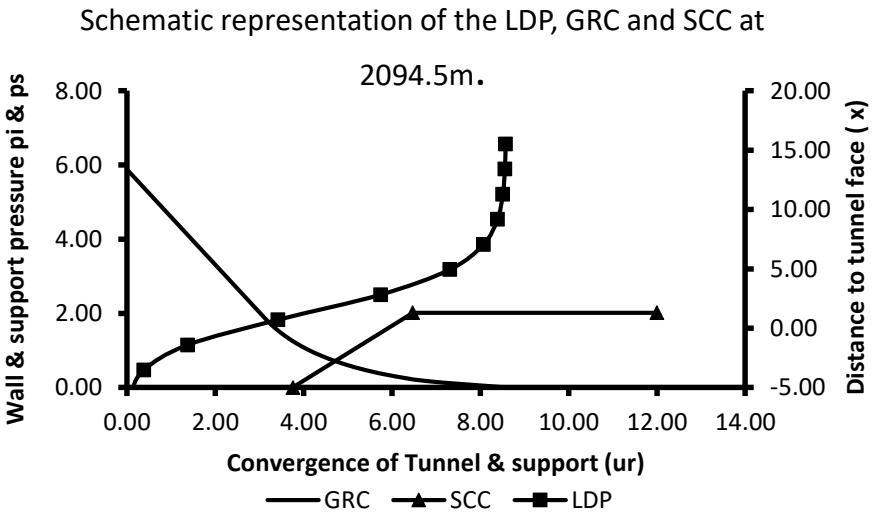
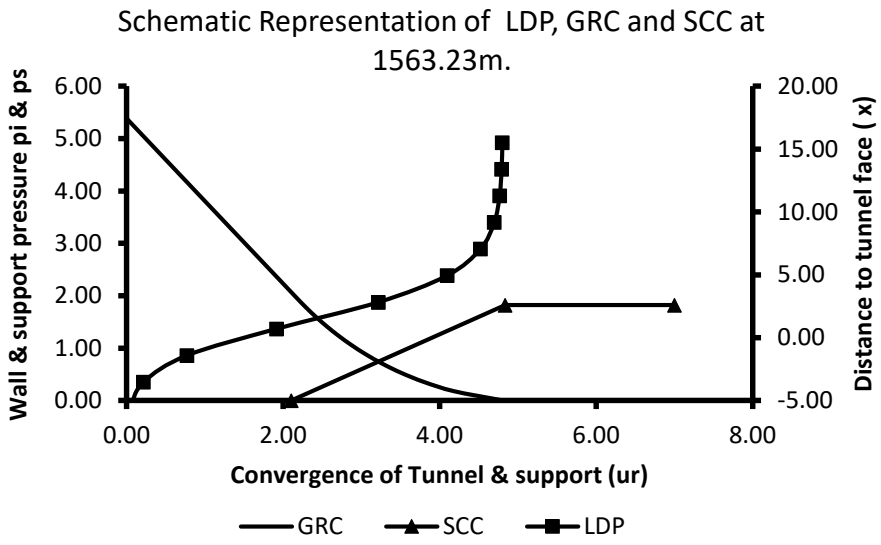
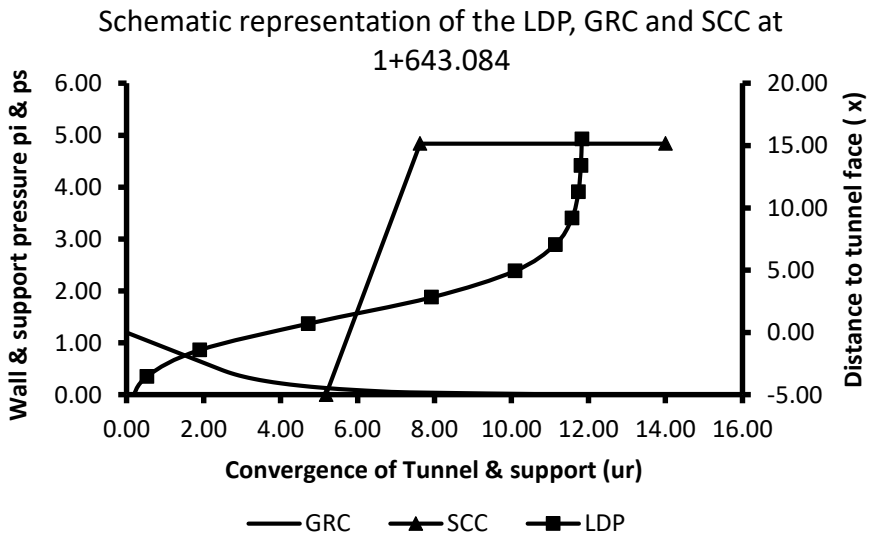
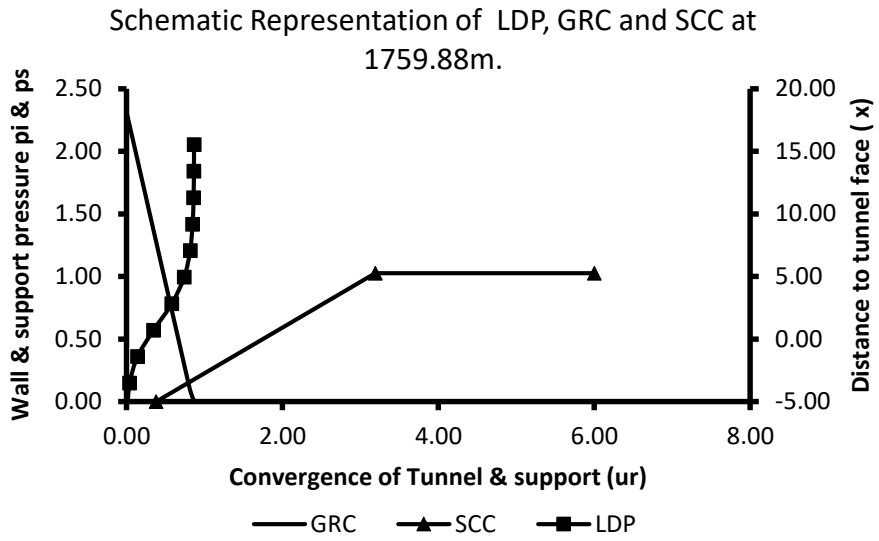


Figure 3-27 Plot of CCM for SHEP at Sections (a) 1563.23m, (b) 1643.084, (c) 1759.88, (d)1938.46 for Observed Condition

b. Based on Q System







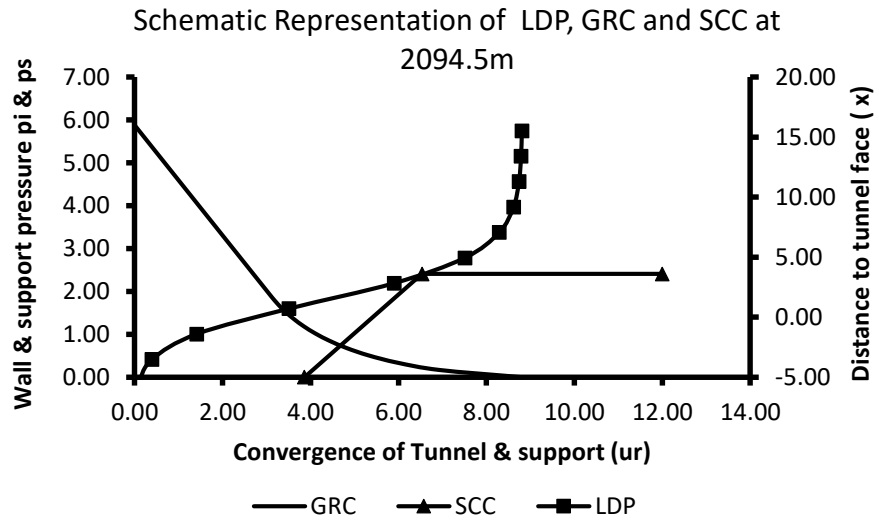


Figure 3-28 Plot of CCM for SHEP at Sections (a) 1563.23m, (b) 1643.084, (c) 1759.88, (d)1938.46 for Q-System

3.2.2.8.2 Comparison of the results

The results obtained from different approaches for the same sections are compared in order to determine the most accurate result which corresponds to the original site conditions. The results are compared and the reasons for the deviations are shown below.

Table 3-37 Comparative Table for Support Pressure obtained from Observed and Q-System for SHEP

Chainage (m)	Overburden (m)	Support Class	Observed Condition		Q-System	
			Maximum Support Pressure	Designed Support Pressure	Maximum Support Pressure	Designed Support Pressure
1759.88	81.42	SII	1.02	0.15	1.03	0.15
1563.23	90	SIII	1.02	0.16	1.82	0.26
2137.7	300.4	SIII	1.02	0.60	1.82	0.26
2094.5	315	SIV	2.02	0.70	2.41	0.7
1938.46	192.45	SV	8.21	0.70	2.99	0.6
1643.084	66.38	SVI	0.50	0.10	4.84	0.2
1864.11	146.23	SVI	0.50	0.35	4.84	0.2

<u>In graphical form:</u>

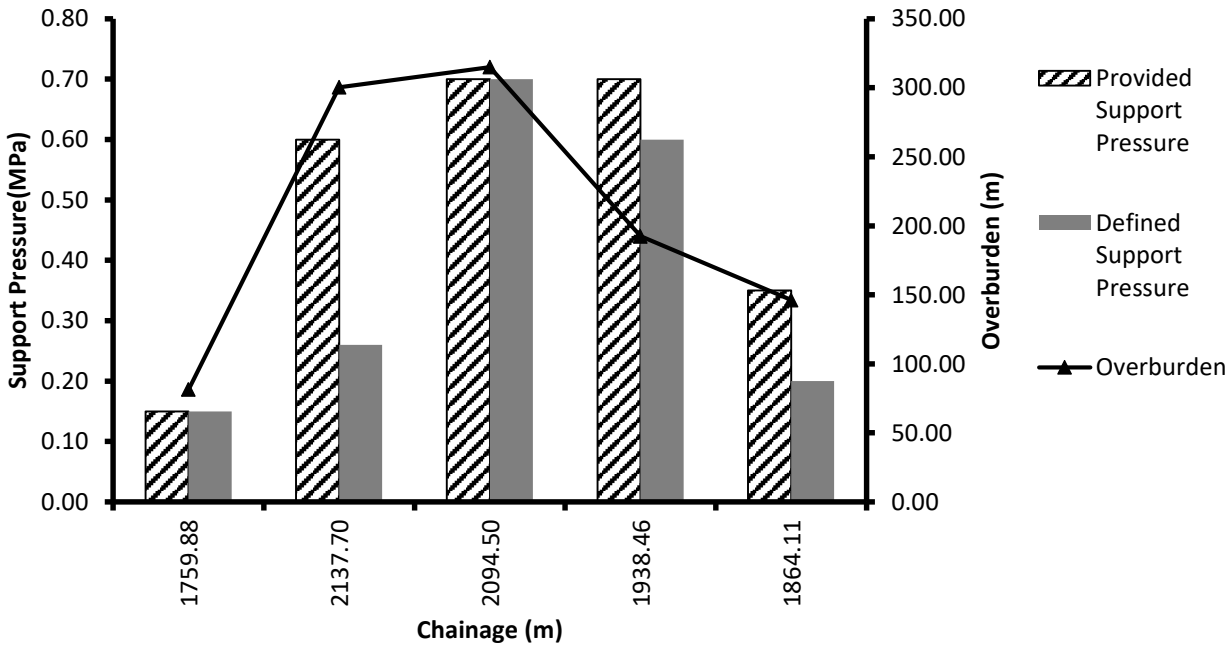


Figure 3-29 Comparison of Support Pressure from different approaches

From the graph above, we can see that the designed and the observed (i.e. Support system provided at SHEP are almost similar). Two bar diagrams are attached to one another in order to compare them. The right one shows the support pressure at different chainage in HRT of SHEP according to the support system provided in the actual site. The left one shows the support pressure at different chainages according to the support system provided as per Q chart (Barton et al. 1993). From the graph, it is clear that the observed support pressure is more in most of the sections and equal in other sections. It suggests that the Support system provided in the actual site is actually more robust than what we determined from another approach. The support we defined are more flexible and provides less support pressure while the support provided at the site is robust.

3.2.2.9 Tunnel closure with and without Support

We observed the closure of the tunnel without support and then observed how the tunnel support limit the extent of the closure in the case of HRT of Sanjen Hydroelectric Project. Using analytical approach, tunnel closure without and without support was determined and compared in the tabulated form as follows:

Table 3-38 Tunnel Closure with and without support for SHEP

Tunnel Closure			
Chainage (m)	**Without Support**	**With Provided Support**	**With Defined Support**
1759.88	0.9	0.79	0.8
1563.23	1.3	1.05	1.1
2137.7	4.8	3.5	3.2
2094.5	8.4	4.8	4.7
1938.46	8.8	4	4.4
1643.084	11.8	5.6	5.8
1864.11	9.4	5.6	4.4

The table above shows the closure of the tunnel sections at different chainage without support in the second column. The support takes the pressure from the deforming rock mass and limits deformation to certain extent. The third column of the table shows closure of tunnel when support system as provided in the site is assigned to respective chainages. The fourth column gives the values of closure when the support system we defined is assigned. The graphical comparison is shown below. The value of closure for the good rock masses is low whereas higher values of closure can be observed at the sections with poor rock masses.

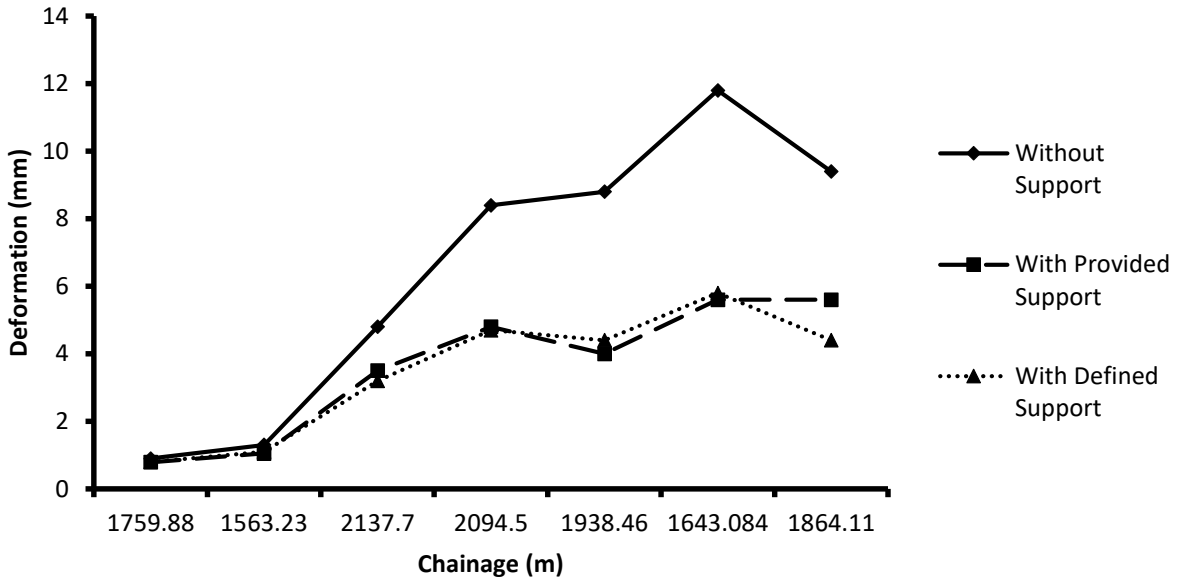


Figure 3-30 Comparison of Closure for SHEP

3.2.3 Comparison of the results from Analytical and Empirical Approaches

We have calculated the Maximum Support Pressure from Empirical Approach. Then we calculated the Maximum Deformation from the Analytical method using CCM for the support system provided at Sanjen Hydroelectric Project and also the support system defined using Q chart. Values of Support Pressure from both the approaches are presented below:

i. <u>Comparison of Maximum Support Pressure for the support system installed at the site</u>

The maximum support pressure values obtained from empirical and analytical approaches for the support system provided are tabulated below:

Table 3-39 Comparative table for Support Pressure for Observed Condition

		Maximum Support Pressure	
Chainage (m)	**Support Class**	**Using CCM**	**Using Empirical**
1759.88	II	1.022985	1.385
1563.23	III	1.024405	1.152
2137.7	III	1.024405	1.152
2094.5	IV	2.017245	2.185
1938.46	V	8.211294	8.442

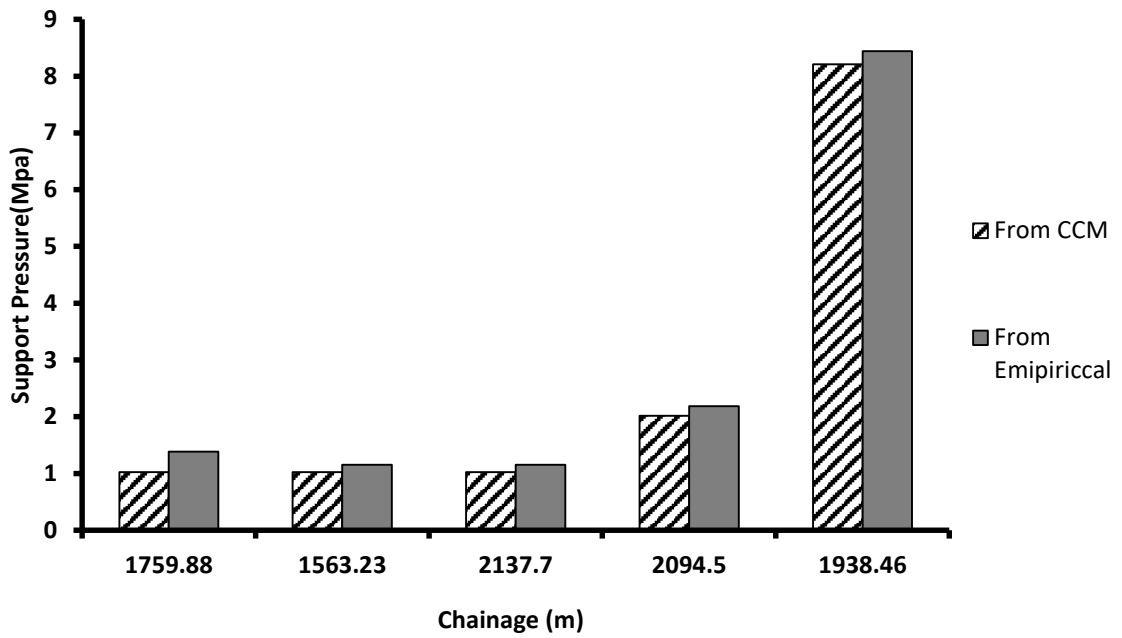


Figure 3-31 Comparative graph for Observed Condition

ii. <u>Comparison of Maximum Support Pressure for the Defined Support System</u>

The maximum support pressure values obtained from the analytical and empirical approaches as per defined support system using Q chart is tabulated as below:

Table 3-40 Comparative table for Support Pressure for Q-System

		Maximum Support Pressure	
Chainage (m)	**Support Class**	**Using CCM**	**Using Empirical**
1759.88	II	1.025179	1.171
1563.23	III	1.822384	2.017
2137.7	III	1.822384	2.017
2094.5	IV	2.408919	2.637
1938.46	V	2.989029	3.138
1643.084	VI	4.837241	5.168
1864.11	VI	4.837241	5.168

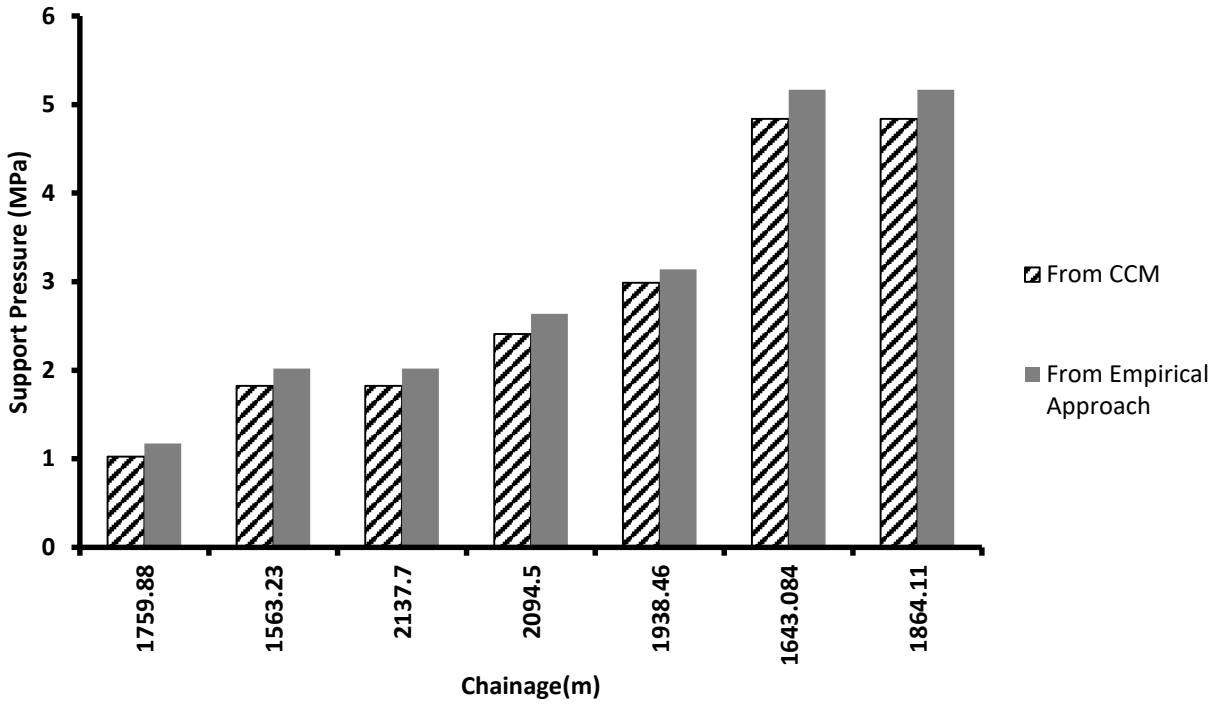


Figure 3-32 Comparative graph for Q-System

CHAPTER 4 EXPERIMENTAL INVESTIGATION

4.1 General

A Direct Shear Test is the Laboratory test conducted by Geotechnical engineers to measure the shear strength properties of the rock mass. The main objective of this test is to determine the shear strength of discontinuity planes or weak horizons in rock as a function of the stress perpendicular to the sheared plane. Peak and residual strength of the rock mass are distinguished.

The normal and shear stresses are calculated as follows:

$$Normal\ Stress = \frac{P_n}{A} \qquad Shear\ Stress = \frac{P_s}{A}$$

Where,

- Pn is the total normal force,
- Ps is the total shear force and
- A is the area of contact of both parts of the secimen.

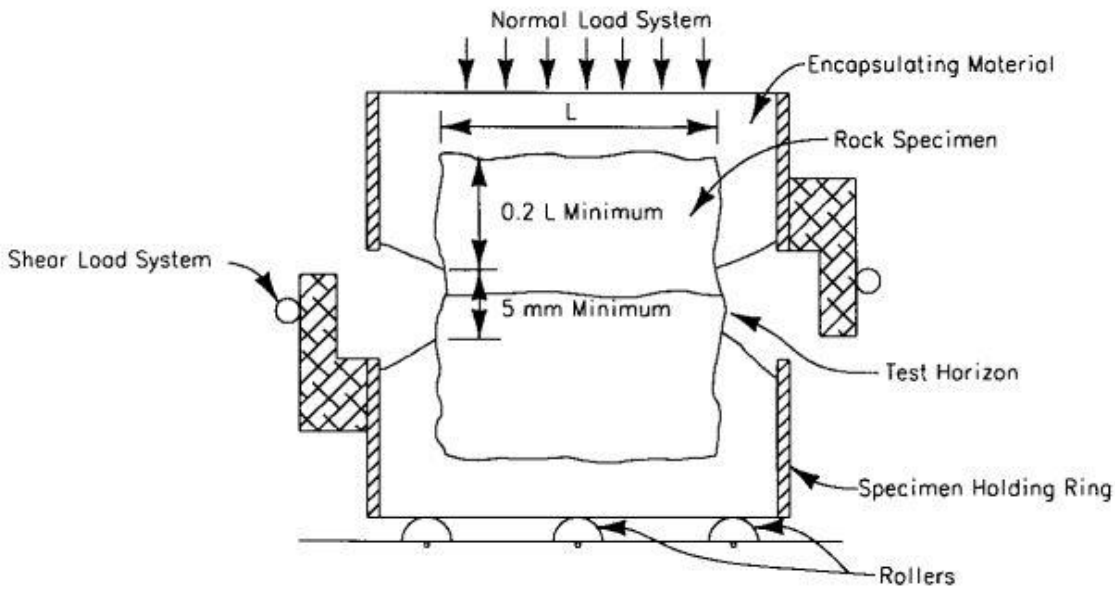


Figure 4-1 Schematic Test Setup- Direct Shear Box with Encapsulated Specimen

Graphs are plotted for shear stress against shear displacement and for normal displacement against shear displacement. In these graphs the value of normal stress should be annotated for the test. The values for peak and residual shear strength and the normal stresses, shear displacements and normal displacements at which these occur can be read from the graphs.

Graphs of peak and residual shear strength versus normal stress are plotted from the combined results for all test specimens. The peak and residual shear strength parameters of cohesion and angle of friction can so be determined.

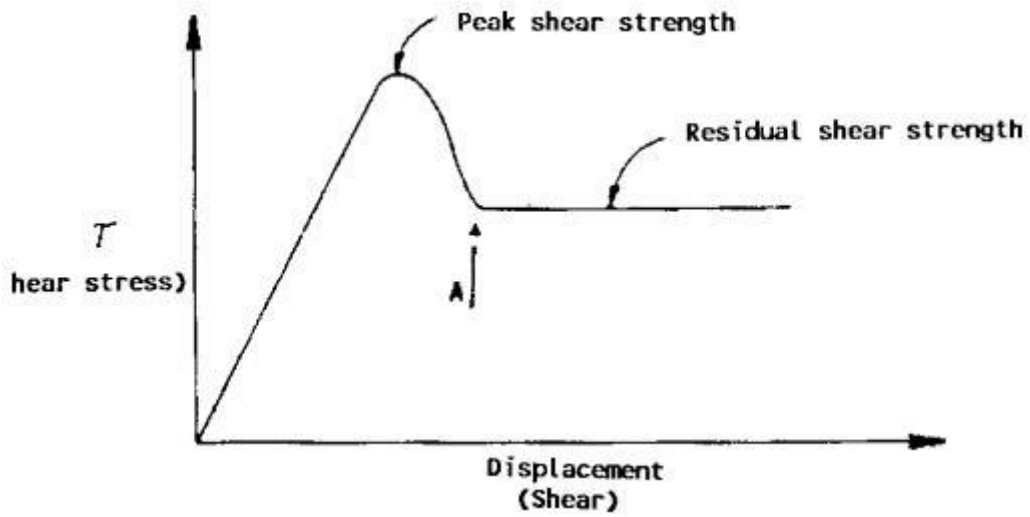


Figure 4-2 Generalized Shear Stress and Shear Displacement Curve

The laboratory testing was done with the standard ISRM suggested method (Muralha et al., 2013) According to the ISRM Method, the shape and the size of the specimen should follow the following standard:

a. Specimens with a regular (rectangular or elliptical) cross-sectional area are preferred. However, specimens may have any shape, such that the cross-sectional areas can be determined with a required accuracy.
b. The height of specimen shall be greater than the thickness of the shear (test) zone and sufficient to encapsulate the specimen in the specimen holder.
c. The length of the test plane (measured along the shear direction) should be at least 10 times the maximum asperity height.
d. The width of the test plane (measured perpendicularly to the shear direction) should have at least 48 mm, corresponding to discontinuities collected from NQ cores.
e. The width of the test plane should not change significantly over the shearing length. Minimum width should be greater than 75 % of the maximum width.
f. The sample half that remains fixed during shear tests should have a greater length than the moving half, so that the joint is always supported and the nominal area in contact remains constant. If this procedure is not feasible due to reduced length of the specimen, the nominal area reduction during shear has to be taken into account in the calculations.

Procedure

- The test sample is prepared in the mould whose dimension is same as that of Shear box. The rock sample is firmly placed in the concrete filler material with not more than 5mm depth of rock sample measured from surface of concrete.
- After placement of the test sample in the shear box, normal and shear displacement measurement gauges are installed. The normal loading due to the vertical hydraulic jack is provided and corresponding deformation is noted at regular intervals.
- While the normal load is applied and steady, the shear load can be applied to allow the various readings to be made. The shear load should be recorded continuously so as not to miss the peak value. If at least four consecutive readings give not more than 5% of variation in shear stress over a displacement of more than 0.5mm, it can be concluded that the residual shear strength value has been reached. After a shear displacement of not more than 10% of the length of the shear plane in shear direction, the shear load can be reduced to zero.
- The shear direction can now be reversed (without decreasing the normal load) if residual values of shear strength for large amounts of shear displacements are requested. If not, or if residual strength has already been reached, the normal load can also be released, the specimen removed from the shear box and the shear plane inspected and described.

4.2 Laboratory Testing

The Sample of the rock used for the Direct Shear Test was obtained from the Headrace Tunnel of Middle Bhotekoshi Hydroelectric Project. The sample obtained was Phylitic Schist. The two samples with the cross sectional areas of 63.5 sq.cm and 86 sq. cm were taken for the Direct Shear Test as shown in the figure 4.5. The observations and results obtained are tabulated below:

Table 4-1 Initial Reading for Peak and Residual Shear Stress

Initial Readings		
Normal Force (kN)	**Shear Reading (mm)**	
	Peak	**Residual**
1.5	1.93	17.13
2.5	0.95	1.24
3.5	1.66	16.79
4	20.13	16.28

Table 4-2 Observation table for Normal Load

Observation Table for Normal Load								
Loading (kN)	**Gauge A**		**Gauge B**		**Gauge C**		**Gauge D**	
	Reading (mm)	**Displacement (mm)**	**Reading (mm)**	**Displacement (mm)**	**Reading (mm)**	**Displacement (mm)**	**Reading (mm)**	**Displacement (mm)**
0	3.91	-	0.65	-	5.32	-	3.71	-
1.5	3.82	0.09	7.75	7.1	5.32	0	4.01	0.3
2.5	9.14	5.32	2.6	5.15	7.84	2.52	3.2	0.81
3.5	4.25	4.89	3.09	0.49	6.42	1.42	1.91	1.29
4	9.05	4.8	2.86	0.23	7.5	1.08	1.43	0.48

Table 4-3 Observation Table for Shear Load

Normal Load = 1.5			
Shear Force (kN)	**Shear Strength (N/mm2)**	**Reading (mm)**	**Displacement (mm)**
0	0.00	0	0
1.5	0.20	1.5	0.43
2	0.27	0.74	1.19
1.5	0.20	19.65	3.71

Normal Load = 2.5			
Shear Force (kN)	**Shear Strength (N/mm2)**	**Reading (mm)**	**Displacement (mm)**
0	0.00	0	0
1.5	0.20	22.73	3.22
2	0.27	18.84	7.11
2.5	0.33	14.54	11.41
2	0.27	3.44	24.76
1.5	0.20	2.01	26.19

Normal Load = 3.5			
Shear Force (kN)	**Shear Strength (N/mm2)**	**Reading (mm)**	**Displacement (mm)**
0	0.00	0	0
1.5	0.20	0.52	1.14
2	0.27	0.13	1.53
3	0.40	24.18	2.48
3.5	0.47	22.69	8.38
2	0.27	18.42	12.65
1.5	0.20	18.36	12.71

Normal Load = 4			
Shear Force (kN)	**Shear Strength (N/mm2)**	**Reading (mm)**	**Displacement (mm)**
0	0.00	0	0
1.5	0.20	19.95	0.18
2	0.27	19.84	0.29
2.5	0.33	19.4	0.73
3	0.40	19.32	0.81
3.5	0.47	23.95	8.48
3	0.40	23.27	9.16
2.5	0.33	19.54	12.89
2	0.27	19.15	13.28
1.5	0.20	19.1	13.33

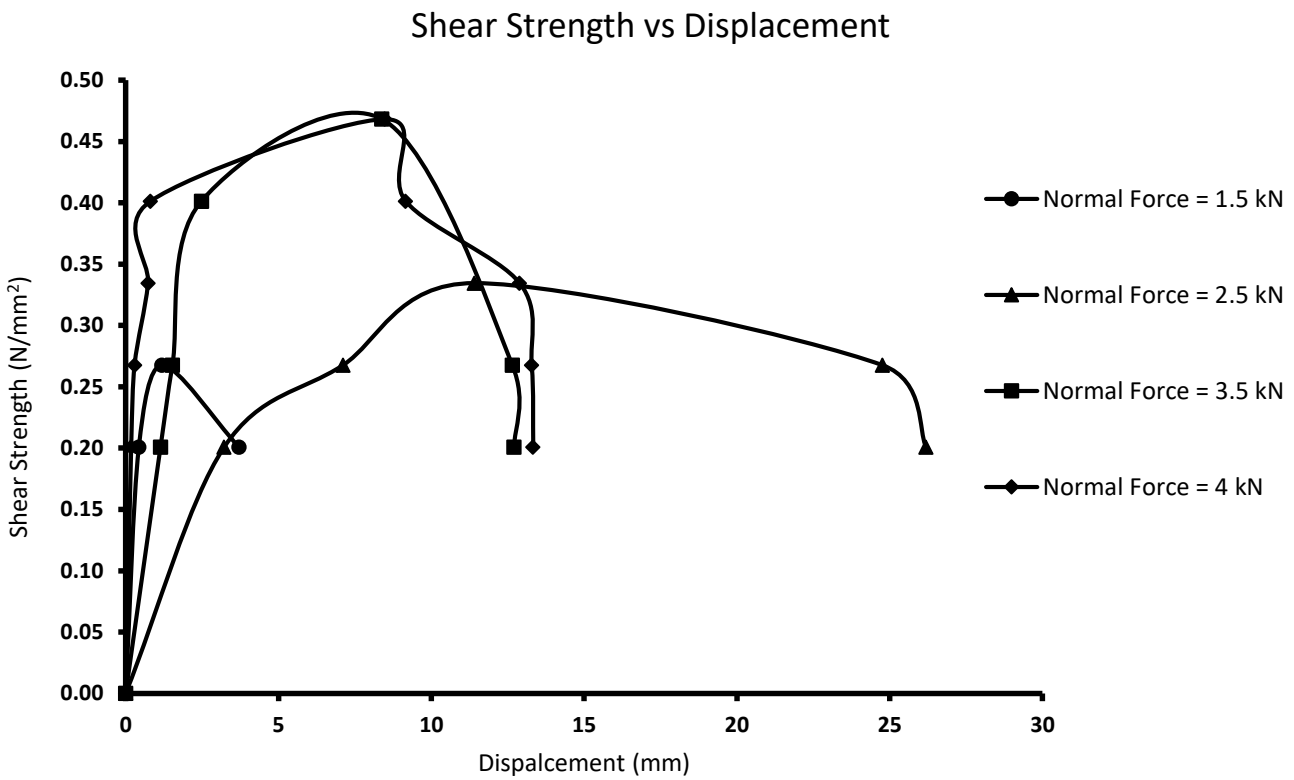


Figure 4-3 Plot between Shear Strength and Displacement for different Normal Loads

As shown in Figure 4-3 obtained above, peak and residual values of the shear stress for corresponding Normal force applied to the test sample can be determined. For example, in the first graph above for Normal load of 1.5kN, the highest point corresponding to 1.19mm deformation which represents the shear strength can be determined (2.7MPa) along with the corresponding residual Shear Strength of 0.2MPa. Similarly, the corresponding values of the peak and residual shear strength with respect to the normal force applied are listed in Table 4-4

Table 4-4 Peak and Residual Shear stress for different Normal Loads

Normal Stress	Peak Shear Stress	Residual Shear Stress	Percent Reduction
0.20	0.27	0.20	25.92
0.33	0.33	0.27	18.18
0.47	0.47	0.27	42.55
0.54	0.47	0.40	14.89

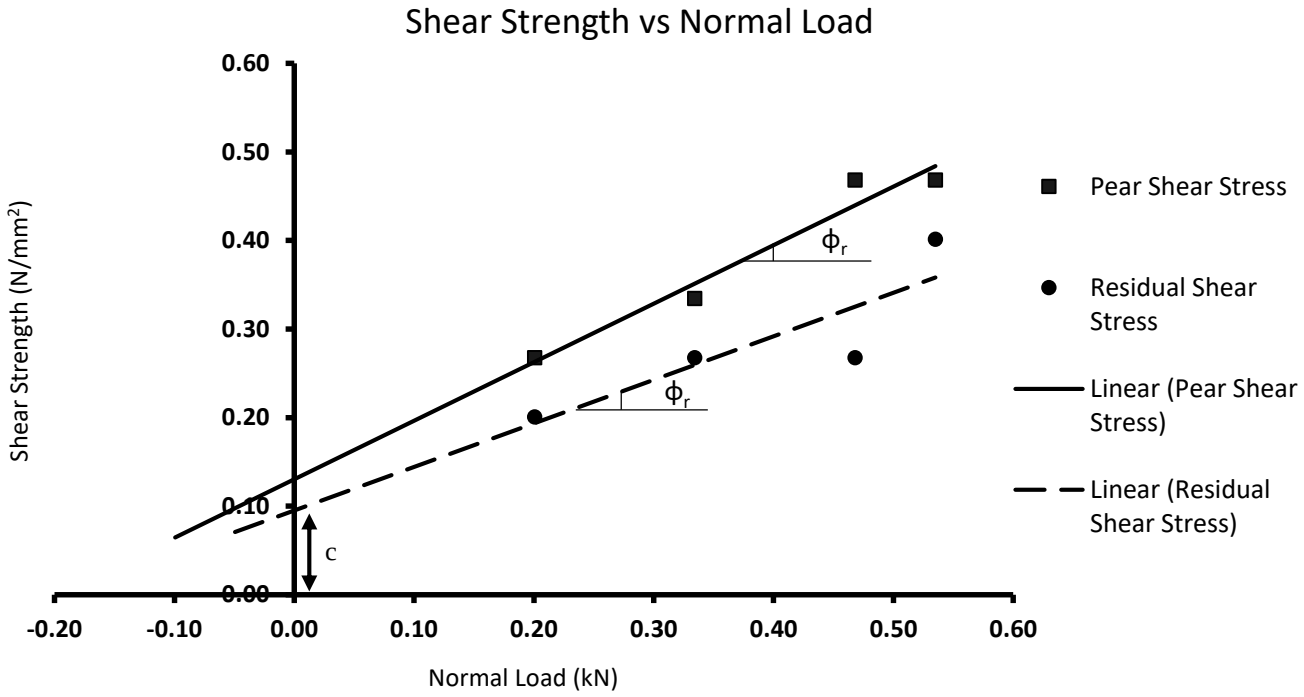


Figure 4-4 Shear Strength vs. Normal Load

The plot between Shear Strength and Normal Load gives the information about the internal angle of friction and the apparent cohesion. The slope of the lines in the graph gives the value of residual internal angle of friction and the point of intersection between the line and the shear stress axis yields the apparent cohesion. The results obtained from the above graph are tabulated below:

Table 4-5 Values obtained from Direct Shear Test

	Peak	**Residual**
Y-Intercept (c)	0.13	0.10
Slope (ϕ_r)	37.87	28.16

From the experiment, the value of peak and residual angle of friction was found to be 37.87^o and 28.16^o respectively whereas the peak and residual values of cohesion was found to be 0.13 and 0.10 respectively.

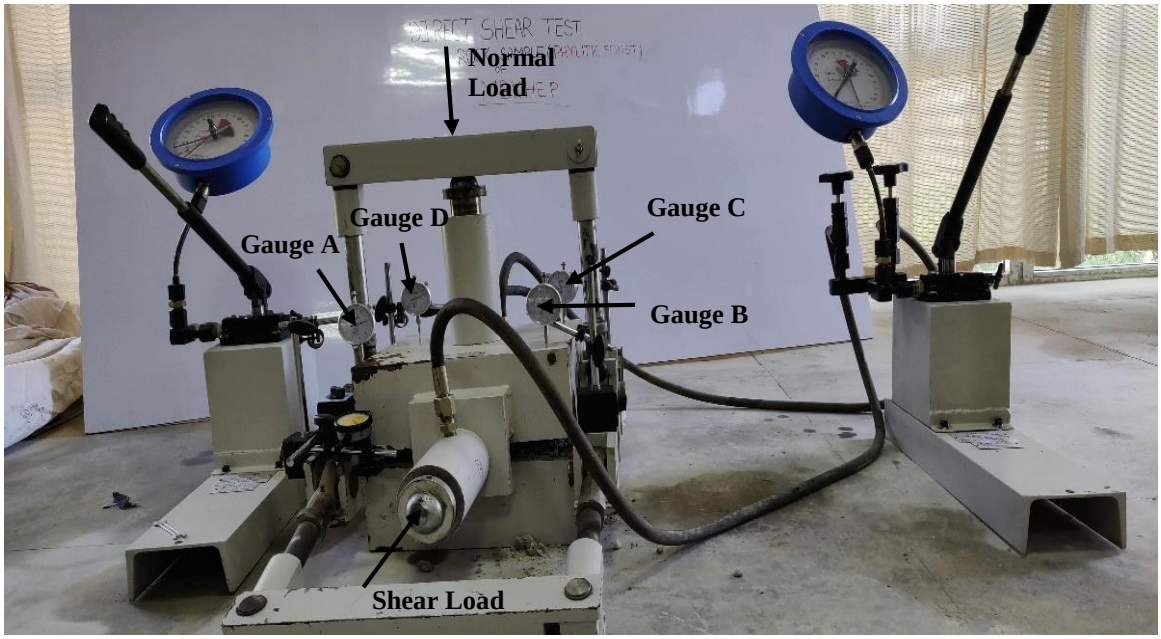


Figure 4-5 Direct Shear Test Machine

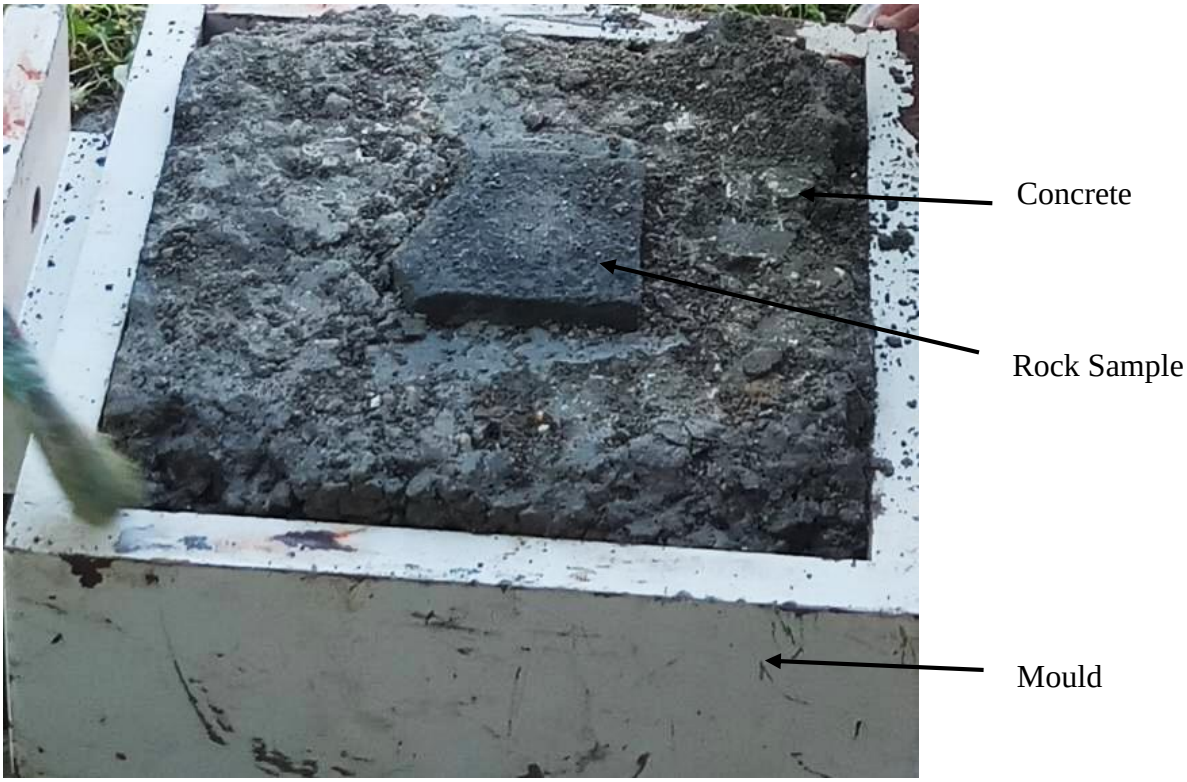


Figure 4-6 Rock Sample after placing in the Mould

CHAPTER 5 NUMERICAL INVESTIGATION

5.1 General

The numerical model is the formulation of real time problem using mathematical models to show the real time behavior of model surrounding and effect of the change in any parameters in the model. The numerical model helps us to provide the inside into the interaction of different entity and their holistic behavior. This helps us to point out the potential flaws or potential improvement of the model and acts as the effective tool to visualize the efficiency of the model. There are various methods of the numerical modeling such as domain method like Finite Element Method (FEM) and Finite Difference Method (FDM) and Boundary Element Method (BEM).(Baral et al. 2000)

Furthermore, moving towards the fundamentals of numerical modeling of underground excavation, one can finding two approaches of modeling.

1. Continuum modeling

This approach is most widespread approach to model the tunnelling problems. It simulates the rock mass interaction towards introduction of underground opening by taking equivalent continuum. In this approach the intact rock mass parameters like UCS of intact rock, material constant(mi), Modulus of intact rock mass (Ei) are scaled down using empirical relations. Using these intact properties the correlated rock parameters like mb, s, a (given by Heoks and Brown (1997)) and cohesion (c) and internal angle of friction (Φ) (Mohr Columb Failure criteria) are used for modeling. In this type of modeling discontinuities like joints are not considered.

2. Discontinuum modeling

This approach of modeling provided more realistic view as it encompasses the discontinuities like rock joints and their structural arrangement. As these discontinuity provide the clear picture of potential local failure around the underground opening. The computer programs like UDEC and 3DEC provides the discontinuum modeling of the underground problem.

5.2 Phase^2 Software

The tunnel sections of two tunnels namely HRT of Sanjen Hydro Electric Project and HRT of Middle Bhotekoshi Hydro Electric Project were modeled using Finite Element Method (FEM). These numerical modeling were used to provide the idea of dynamics of introducing the excavation in the rock mass and effect of support installation. The modeling program Phase2 software developed by Rocscience Inc. is used for numerical assessment. This software provides the two dimensional design and analysis of underground structures and geotechnical problems.

(Khadka et al., 2017) This program uses failure criteria like Generalized Hoek Brown, Mohr's Columb Failure criteria, Cam-clay, modified Cam-Clay and Discrete function. This software provides the elasto-plastic finite element analysis for assessment of the model and different geotechnical problem. Some of the features provided by Phase^2 software are given below (*Rocscience*):

a. Modeling

- Boundary definition (external and internal) using grid/vertex/object snapping.
- Simulation of excavation process using sequential staging (up to 300 stages)
- Analysis method like plain strain and axisymmetric.

b. Element definition and Meshing

- Definition of triangular and quadrilateral element of the boundary where 3 or 6 noded triangles and 4 or 8 noded quadrilateral can be used.
- Number of mesh element can be adjusted along with the checking of mesh quality

c. Material and support definition

- Elastic or non-linear material definition with functionality of staging the material properties.
- Material properties can be defined as isotropic, transversely isotropic or orthotropic elastic.
- Staged support installation with different support types like liner, rock bolt and composite liner with functionality of user adjustability.

d. Load and Stress definition

- User can define the field stress as gravity or constant field stress.
- Loading can be also provided like seismic loading, uniform loading, concentrated loading etc.
- Feature like load splitting is also provided.

e. Data Interpretation

- Use of Contour display option to visualize the model more clearly.
- Features like Query function and graph.
- Provides the support capacity curve for support.
- Exports the final result either graph or contour.

5.3 Numerical assessment using Phase^2 Software

For tunnel numerical assessment of the tunnel sections of both project sites following three steps are employed:

1. Modeling

The rock parameters for the section of study were found out by either laboratory testing or by using the correlated empirical relation that shows compliance to the real rock mass properties at site. Use of the support and excavation and staging is performed.

2. Compute

Model computation is done using the computation tab. The computation time depends upon the degree of sophistication of the model and degree of accuracy required by the user.

3. Interpret/Analyze

The interpret function provided by the phase2 takes user to new window where user are able to query the result, view different parameter and analyze the result using in-built function such graph and query. If the user is the not satisfied with the result or if some modification is to be done, one can directly go to modeling window by click on the model tab on the interpret window.

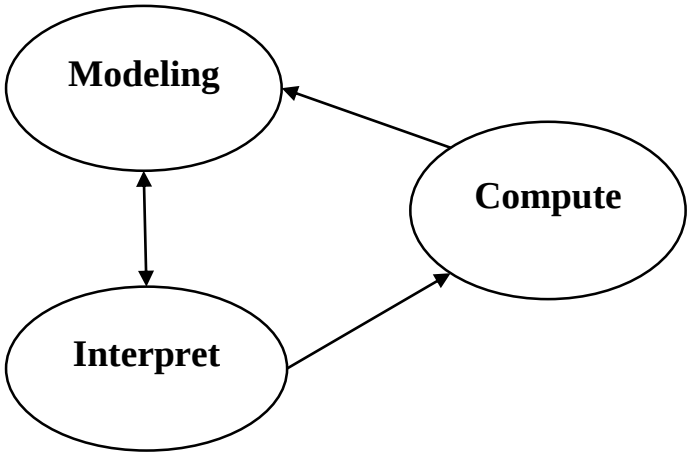


Figure 5-1 Schematic Diagram Showing the process of Analysis

5.4 Procedure of modeling a tunnel section

There are different steps that were undertaken for the modeling of tunnel section for Numerical modeling using Phase2 software. These steps are explained as follows:

- **Project Setting**

Firstly, the analysis method is defined as plain strain and the Gaussian elimination method was taken as the solver type for computing the model. Units were also defined to be metric. To simulate the real steps of excavation and to effectively model the time taken for excavation, number stages were used. The tolerance level and maximum iteration was adjusted according to the need with the convergence computation using "Absolute Energy" method.

- **Excavation of the opening**

For defining the boundary of the opening, "Add excavation" tab is clicked on where the tunnel boundary is provided in term of coordinate input.

- **Defining External Boundary**

The external boundary is defined according need of the section. If the far-field stress or the plastic zone does not included in the external boundary, external boundary is increased. For external boundary for tunnel section of Sanjen Hydro Electric Project expansion factor 7 is taken and for Middle Bhotekoshi section expansion factor 5 is taken.

- **Definition of Material Properties of rock mass**

The rock mass properties is defined using "Define Material Properties" tab where the material is taken as the plastic material and use of generalized Hoek Brown criteria was implemented. Here the intact material constant like UCS of intact rock mass (σci) and Modulus of intact rock mass (Ei) or Modular ratio and rock mass constant (mi) is given as the input parameter.

The correlations used for the parameters numerical analysis are:

$$\frac{Erm}{Ei} = 10^{((RMR-100)\times(100-RMR))/(4000\times e^{\left(\frac{-RMR}{100}\right)}} \qquad \text{(Sonmez et al., 2006)}$$

$$Shear\ Modulus(G) = \frac{Erm}{2\times(1+\upsilon)}\ Mpa \qquad \text{(Sherstha et al., 2015)}$$

$$GSI = RMR - 5$$

$$\upsilon = 0.5 - 0.2 \times \frac{RMR}{RMR + 0.2 \times (100 - RMR)} \qquad \text{(ydan et al. 2018)}$$

$$Residual\ GSI = GSI \times e^{(-0.0134\ GSI)} \qquad \text{(Cai et al., 2007)}$$

$$Ei = Modular\ Ratio \times \sigma ci \qquad \text{(Hoeks and Diederich, 2006)}$$

Where, Ei is the intact rock modulus (MPa)

Erm is the Modulus of Rock Mass (MPa)

υ is the poission ratio

GSI is the Geological Strength Index

RMR is the Rock Mass Rating

σci is the UCS of intact rock mass

- **Defining Stress**

The definition of the stress is done by using the overburden and multiplying it with the rock density for obtaining the vertical stress and for horizontal stress the vertical stress is multiplied by the stress ratio obtained using Shorey (1994).

$$\sigma v = \gamma XH$$

Where, γ is the rock density in MN/m^3 (Taken value 0.027 MN/m^3)

$$K = 0.25 + 7Ei \times (0.001 + 1/H)$$

$$\sigma h = \sigma v \times K$$

Where, H is the overburden

σh is the horizontal stress in MPa

σv is the vertical stress in MPa

K is the stress ratio (Shorey (1994))

- **Definition of support system**

The rock support system is defined using the "Define Liner" or "Define Rock bolt" tab where the properties of the liner and rock bolt is can be altered according to the need. To simulate the combined support "Define Composite liner" tab is used. The flexural strength of Steel Fiber Reinforced Shotcrete (SFRS) is given by:

$$\frac{fsp}{\sqrt{fc}} = 0.614 + 0.4\left(v + \frac{l}{d}\right)^{1.029} \qquad \text{(Musmar, 2013)}$$

Where, v is the volumetric fraction of fiber content

l is the length of the fiber

d is the diameter of fiber

fc is the compressive strength of shotcrete

- **Meshing and Discretizing**

As the definition of parameters for modeling is done, the boundary of the model both external and internal is discretized and the material is divided into smallest entity by meshing the material.

- **Assigning the material properties and installation of support**

After the definition of the material properties, assigning of these properties is done using "Assign properties" tab. The support installation is done by adding the bolt and adding the liner as per designed support or defined support.

- **Compute and Interpret the model**

Finally computing and interpretation of the model is done and the result is analyzed where the deformation of excavation boundary, stress redistribution and support adequacy is assessed for the project sites.

5.5 Case Studies

5.5.1 Middle Bhotekoshi Hydroelectric Project (MBKHEP)

5.5.1.1 Parameters used for numerical modelling

Table 5-1 Parameters used for Numerical Modelling of Sections for MBKHEP

Chainage (m)	RMR value	Overburden (m)	Vertical stress (σv) (MPa)	Stress Ratio (k) (Shorey (1994))	Horizontal Stress (σh)	Correlated GSI value (Ceballos et. al 2014)	Uniaxial Compressive Strength of intact rock (σci) (MPa)	Tunnel Support(As per Q-chart)	Tunnel Support(As per RMR)	Value of mi (Heok (2006)	Value of Modular Ration (MR)
0+300	42	181.62	4.904	1.525	7.48	37	80	S1-III	R-III	9.00	475
0+400	49	246.23	6.648	1.424	9.47	44	80	S1-III	R-III	9.00	475
0+500	41	286.35	7.731	1.13	8.74	36	80	S1-III	R-III	9.00	475
6+795.90	31	147.43	3.981	0.814	3.24	26	80	S1-IV	R-IV	9.00	675

5.5.1.2 Deformation of Tunnel

Table 5-2 Measurement of Maximum Deformations of Sections for MBKHEP

Chainage (m)	Before Support Installation			Support Installation behind the face (m)	Deformation of ground allowed (mm)	After installation of Used support		After installation as per Q-System		After support installation as per RMR	
	Maximum Deformation w/o support installation (mm)	Equivalent tunnel radius (m)	Radius of Plastic Zone (m)			Deformation (mm)	Radius of Plastic Zone (m)	Deformation (mm)	Radius of Plastic Zone((m)	Deformation (mm)	Radius of Plastic Zone (m)
0+300	179	3.03	16.509	1	44	51.6	8.102	34	6.899	32	6.844
0+400	85	3.03	11.132	1	24	25	6.932	23	6.942	23	6.963
0+500	116	3.03	11.321	1	31.57	96	9.636	87	9.553	88	9.562
6+795.90	137	3.03	19.48	1	22	112	15.047	34	8.508	31	7.624

5.5.2 Sanjen Hydroelectric Project

5.5.2.1 Parameters used for Modelling

Table 5-3 Parameter Used in Numerical Modeling of sections for SHEP

Support Class	Chainage (m)		Q-value	Overburden (m)	GSI	Ei (MPa)	Material constant (mi)	φ (Degree)	Cohesion (c) (MPa)	Stress Ratio (k)	Residual GSI	Vertical stress (MPa)	Horizontal Stress (MPa)	Value of Poisson ratio of intact rock (υi)
	Starting	Ending												
S-II	1+759.88	1+756.88	1.5	81.42	47.641	116314	9	45.9	0.55	1.35	25.161	2.198	2.968	0.23
S-III	2+137.7	2+135.3	0.72	300.4	42.86	43324	7	27.46	0.657	1.563	24.134	8.111	12.677	0.236
S-IV	2+094.6	2+091.8	0.2	316.58	34.515	116314	9	28.8	0.708	1.35	21.734	8.548	11.539	0.23
S-VI	1+864.11	1+861.80	0.016	146.23	18.062	116314	9	23.53	0.223	1.2	14.179	3.948	4.738	0.23

5.5.2.2 Deformation of the tunnel

Table 5-4 Deformation measured in the tunnel of SHEP

At Invert Level											At Spring Level		
Chainage (m)	Before Support Installation			Maximum unsupported Span (m)	Support Installation behind the face (m)	Deformation of ground allowed (mm)	After installation of Used support		After installation of Defined support		Maximum Deformation w/o support installation (mm)	Deformation (As per Used Support) (mm)	Deformation (As per Q-chart) (mm)
	Maximum Deformation w/o support installation (mm)	Equivalent tunnel radius(m)	Radius of Plastic Zone (m)				Deformation (mm)	Radius of Plastic Zone (m)	Deformation (mm)	Radius of Plastic Zone((m)			
1+864.1	53	1.94	11.23	0.61	0.5	13	16	5.72	12.84	5.22	42.7	9.2	9.50
2+094.6	73	1.94	9.44	1.68	1	24	40	5.79	30.64	5.82	49.8	20.5	18.4
2+137.7	150	1.94	11.35	2.81	1	36	65	7.23	49.16	5.25	94	31.7	27.5
1+759.8	2	1.94	3.11	3.76	3	0.8	1	3.06	1.34	3.92	1.4	1.2	1.2

Support system used in the site and support defined by Q-Support and RMR guidelines where modeled using this computer aided software. These modeled sections are the representation of different rock mass condition prevalent at the site of MBKHEP and SHEP. The parameters provided in Table 5-1 for MBKHEP and Table 5-3 for SHEP were used during the modelling of various sections of the site. From the result obtained from the modelling, deformation of tunnel was recorded which is provided in Table 5-2 and Table 5-4 for MBKHEP and SHEP respectively. A support capacity plot of the sections were also derived from the numerical modelling. A representative plot of two sections from each site is shown in Figure 5-2 and Figure 5-3 for SHEP and Figure 5-4 and Figure 5-5 for MBKHEP. The plot is made for Factor of Safety (FOS) of 1, 1.2 and 1.4. The points shown in the figure represent various points on the surface of the excavated surface of the tunnel and if those points lie well within the curve, then the provided support is sufficient to withstand the pressures of the ground. But as can be seen in the figure most of the points lie outside the curve indicating the provided support is not sufficient. This indicates that there is a high chance of the support to fail. The points that lie outside of the curve belongs to the bottom of the tunnel walls as shown in Figure 5-6. Similarly for all other section, all the points that lie outside the FOS plot belongs to the bottom of the tunnel wall.

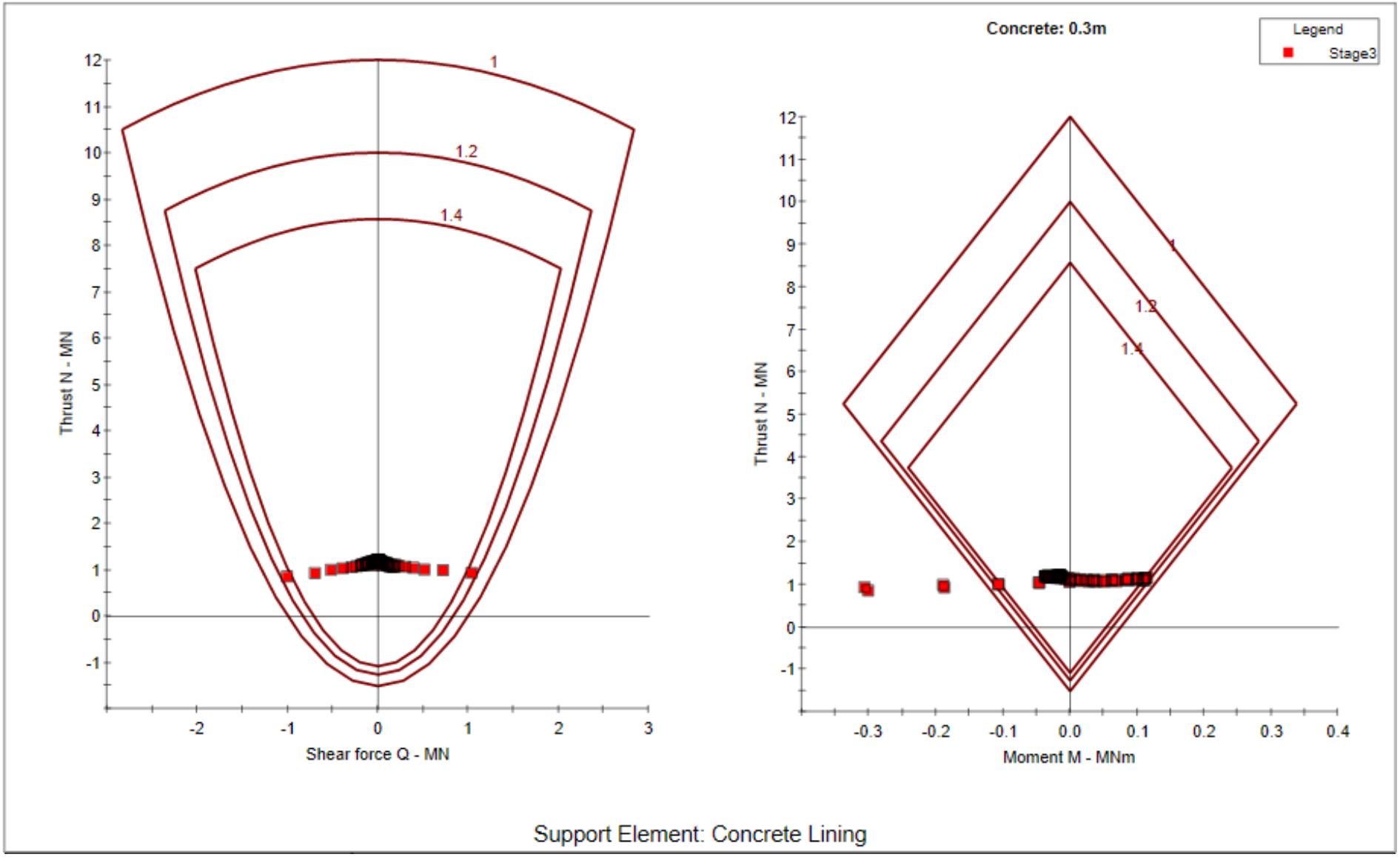


Figure 5-2 Support Capacity Plot for Concrete Lining (designed) for SHEP at chainage 1864.122m

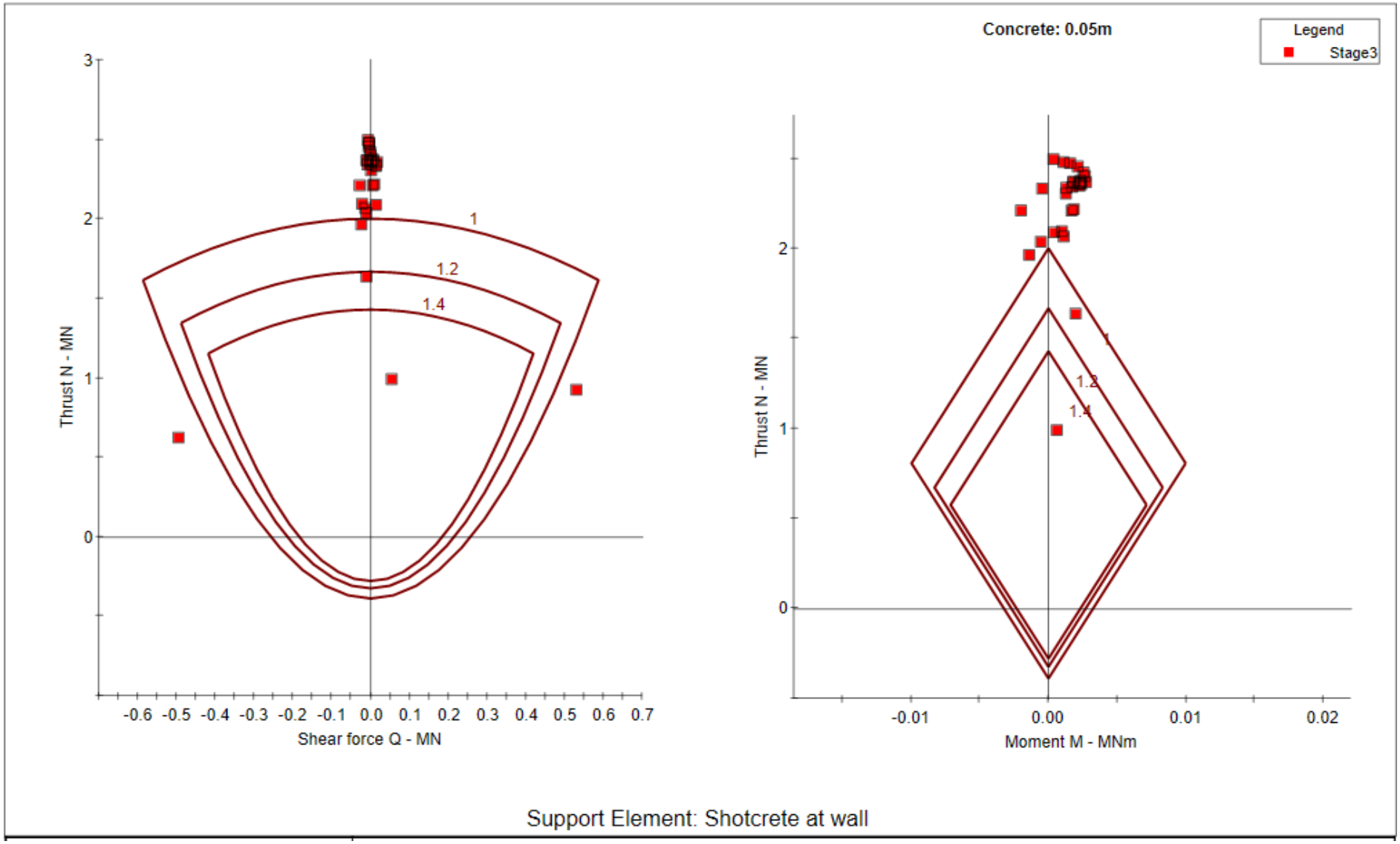


Figure 5-3 Support Capacity Plot for Shotcrete (designed) for SHEP at chainage 2094.6m

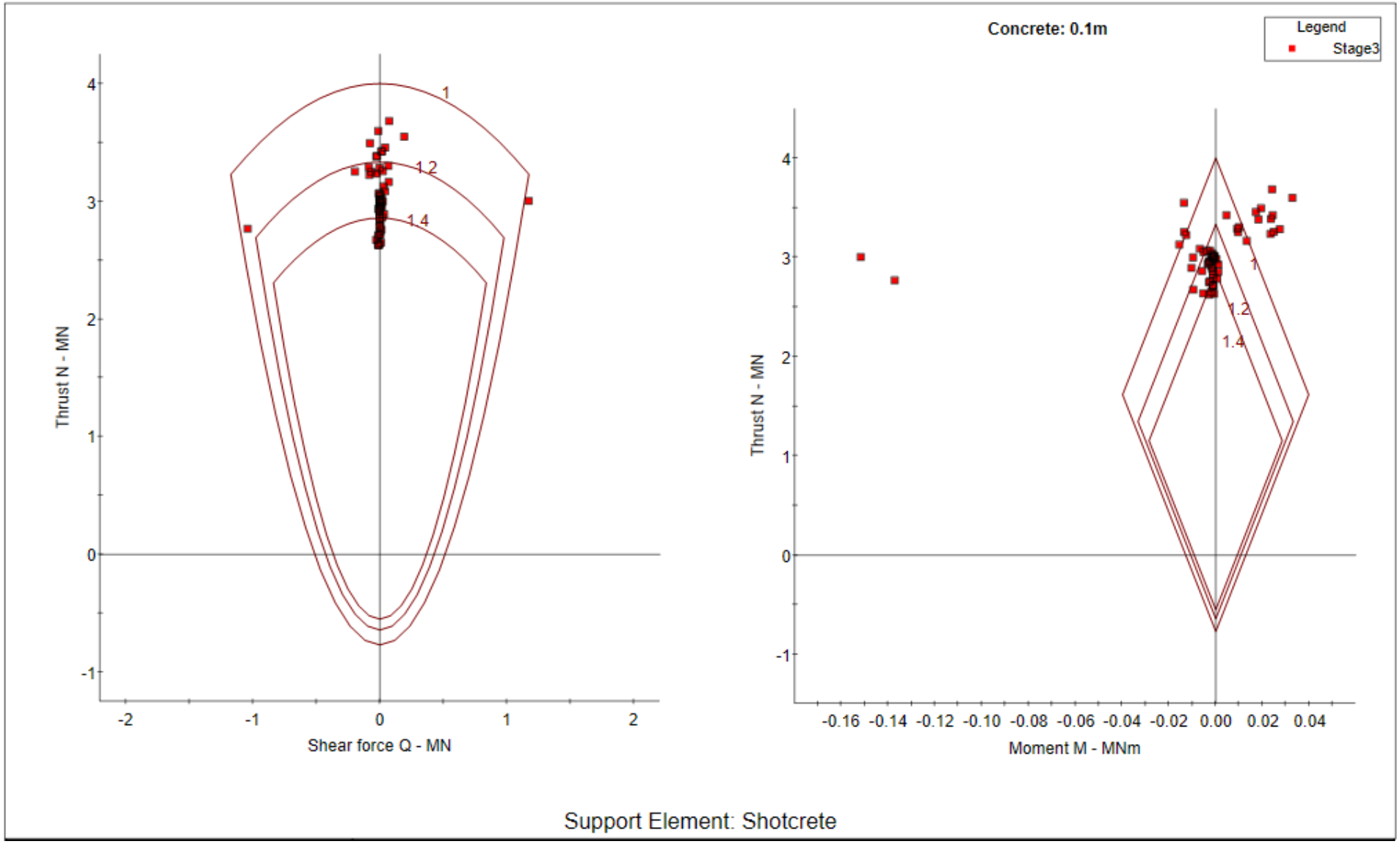


Figure 5-4 Support Capacity Plot for Shotcrete (designed) for MBKHEP at chainage 500m

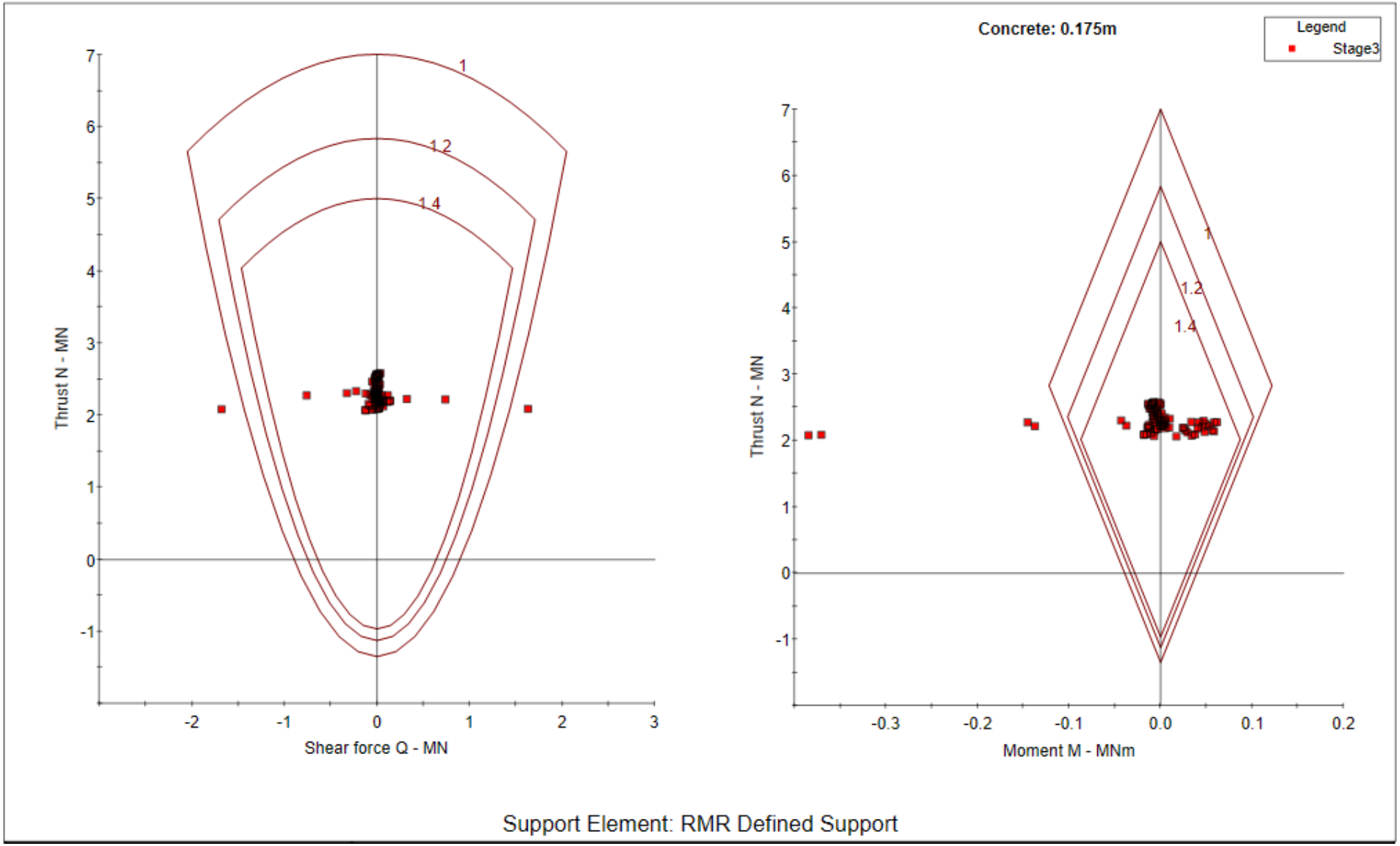


Figure 5-5 Support Capacity Plot for Shotcrete (designed) for MBKHEP at chainage 6795.5m

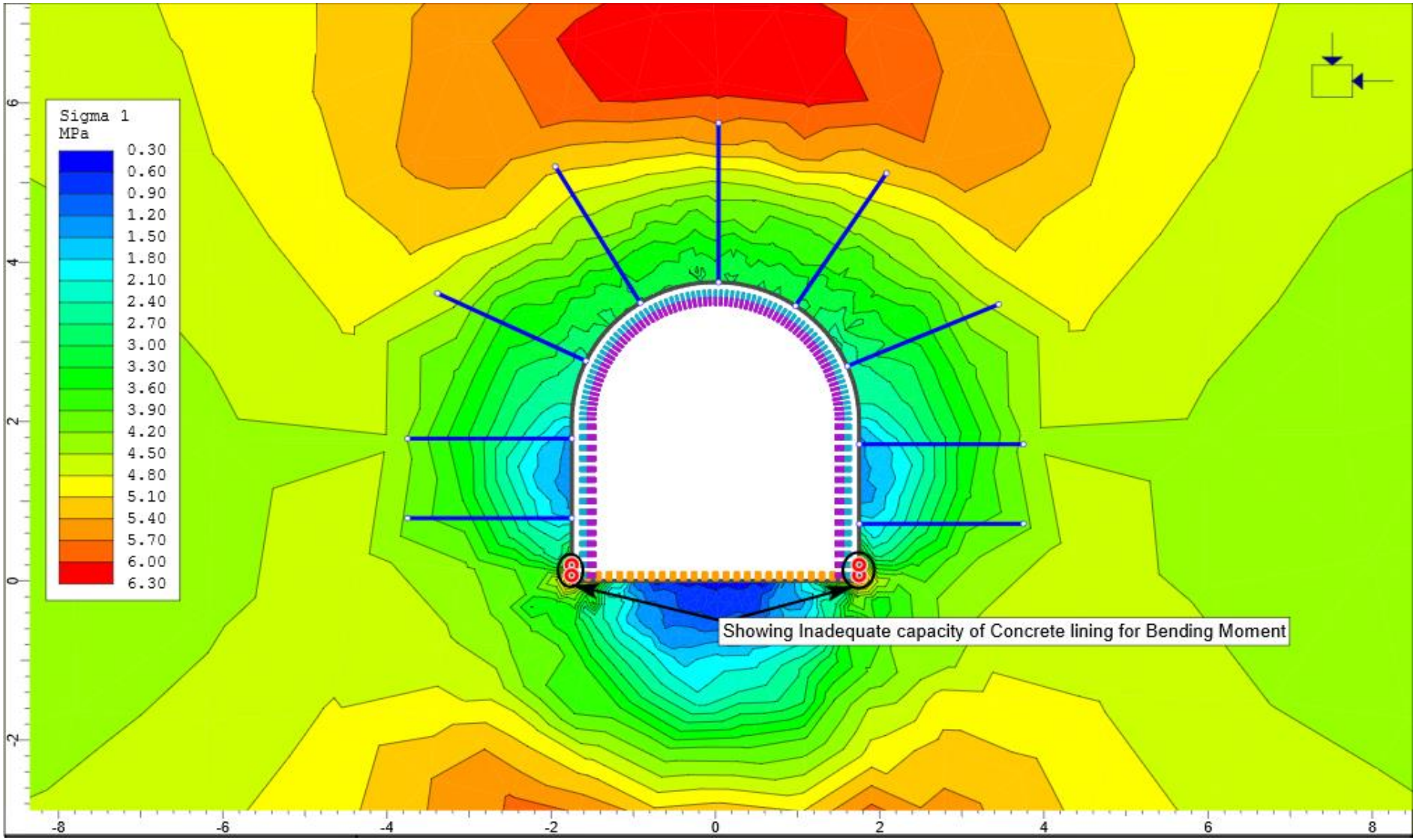


Figure 5-6 Insufficient support for chainage 1864.122m of SHEP

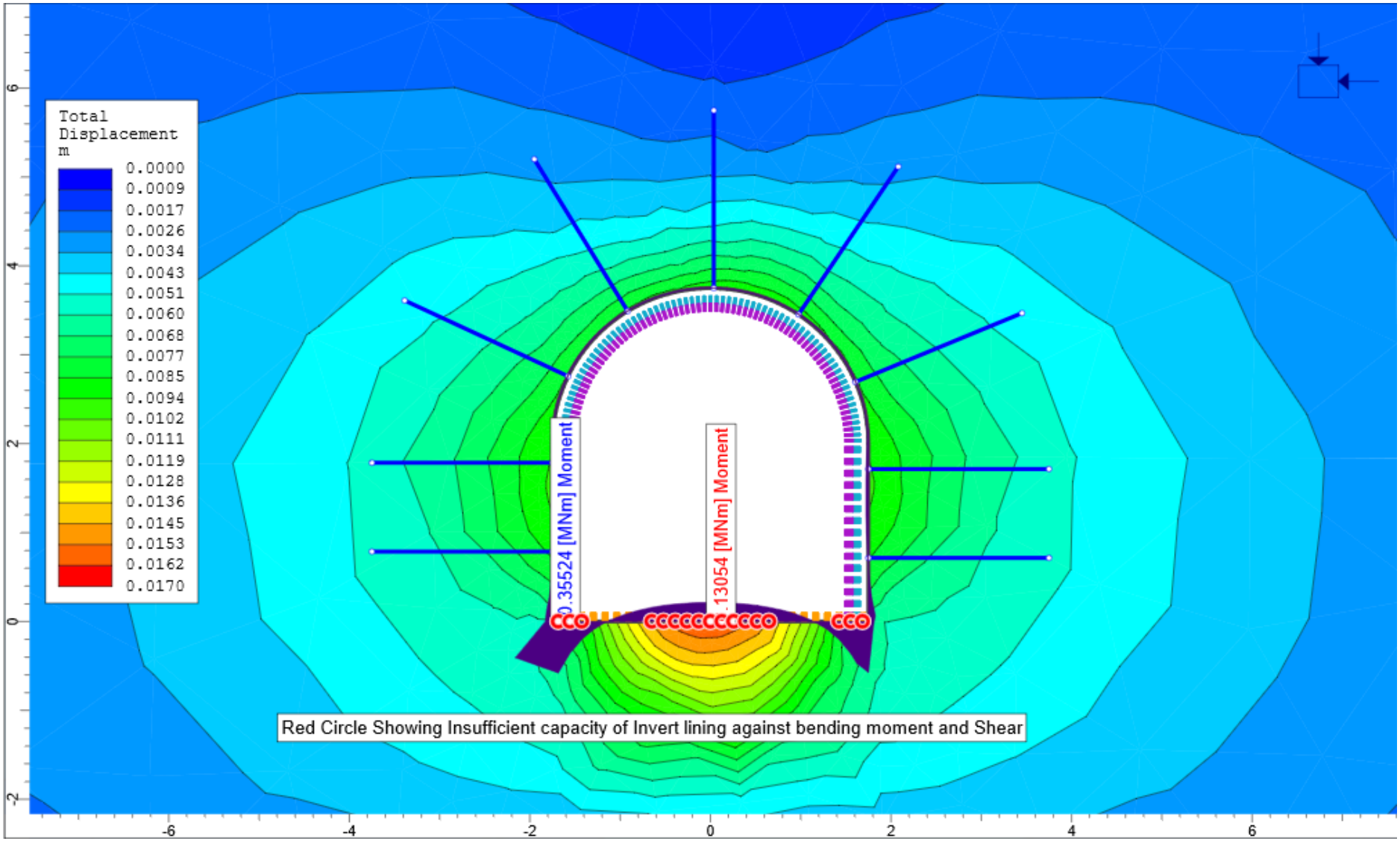


Figure 5-7 Plot of Bending moment with points of failure for section 1864.122m of SHEP

5.6 Result and Discussion

The inadequacy of support on the bottom walls of tunnel is due to high concentration of stress in the region. As can be seen in Figure 5-2, Figure 5-3, Figure 5-4 and Figure 5-5 the failure is mostly caused due to the support being unable to bear the moment. A plot of moment for section 1864.122m of SHEP is provided in Figure 5-7. As can be seen in the figure the point of bottom of tunnel walls lies in the negative moment region and the moment is maximum in this region. So the support structure fails in bending and the failure type is tensile failure. Thus designing of support system requires the use of modelling technique. This helps to provide a clear picture of the adequacy of the support designed according to conventional approaches and bring modifications to it as per requirement. In these case studies, the support modification is done based on numerical assessment. A parabolic profile is found to be suited in these cases with increasing the support as we move down from spring level to bottom of tunnel wall. This increasing thickness of support system counters the high stress in the region hence providing adequate support to the structure. Schematic diagram of the modified support is provided in Figure 5-8 for MBKHEP and Figure 5-9 for SHEP.

5.7 Modified Support

The problem of inadequate support is addressed by increasing the thickness of liner from spring level to the base with maximum thickness at the base. The profile so obtained is parabolic. Support capacity plot for the modified section has been shown in Figure 5-10 and Figure 5-11 for SHEP and Figure 5-12 and Figure 5-13 for MBKHEP respectively. Here, the points lie within the FOS curve which indicate the support to be sufficient. Since the support is provided in increasing order to form the parabolic shape, the support plot of modified section is provided for the lowermost section where maximum failure can be seen The large deformation is also observed at the invert level where the assessment shows inadequate section in bending moment at edges and middle section where there is maximum value of negative and positive bending moment respectively. For this issue, rebars are provided with less spacing at the edges and at the middle part which controls the tensile failure, whereas the strength of the concrete is increased up-to 65 MPa to provide required compressive strength.

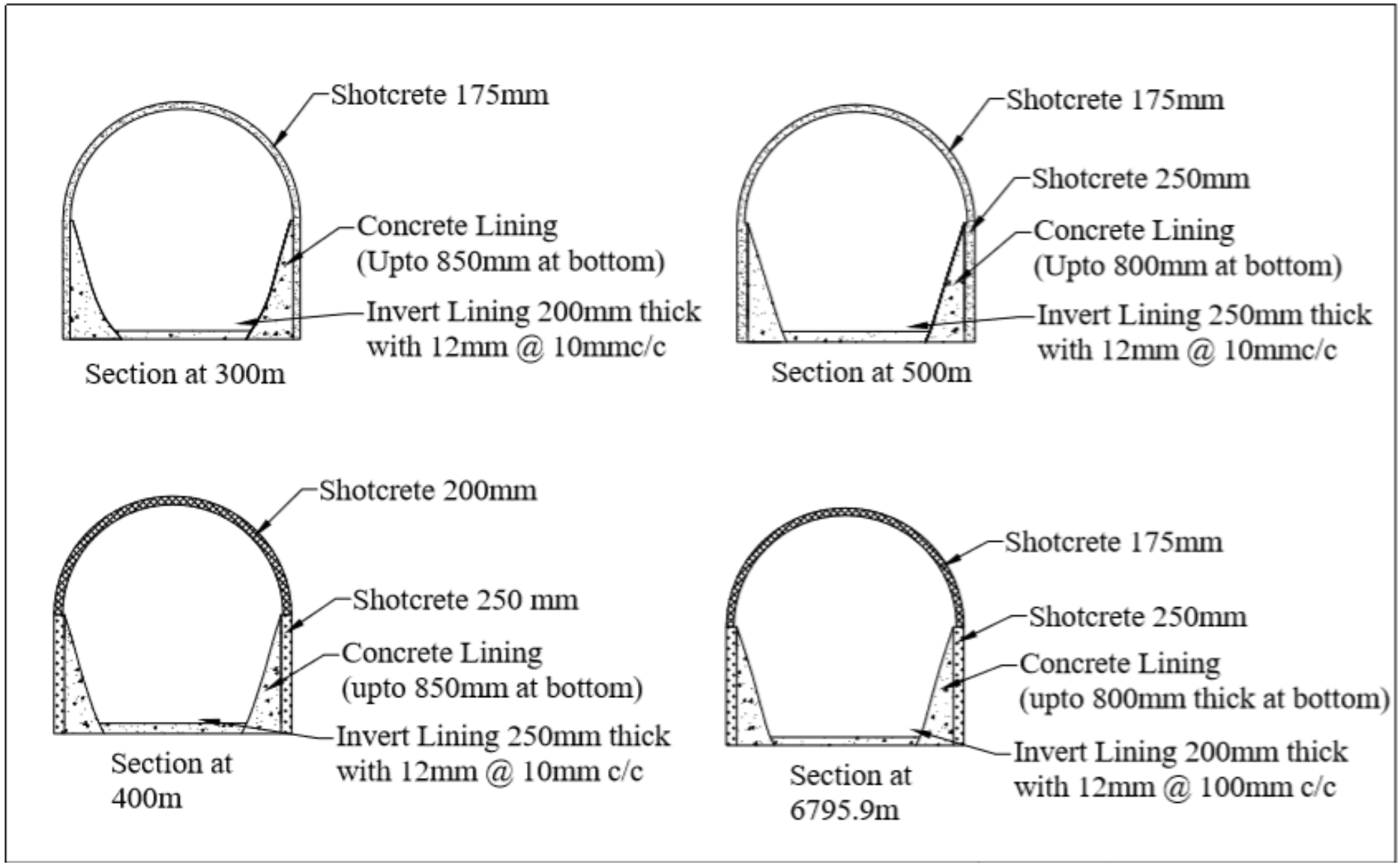


Figure 5-8 Schematic Diagram of Modified Section for MBKHEP

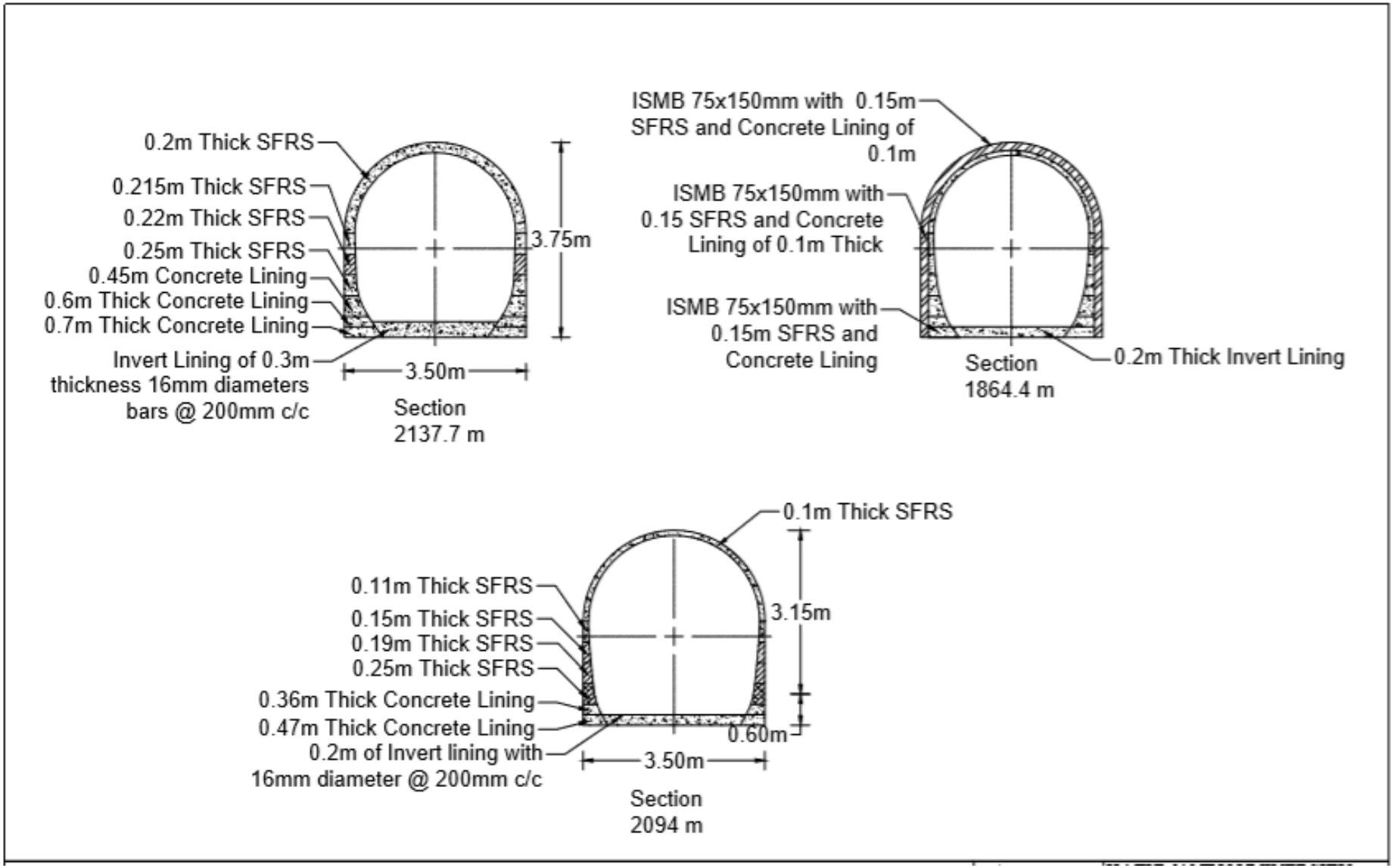


Figure 5-9 Schematic Diagram of Modified Section for SHEP

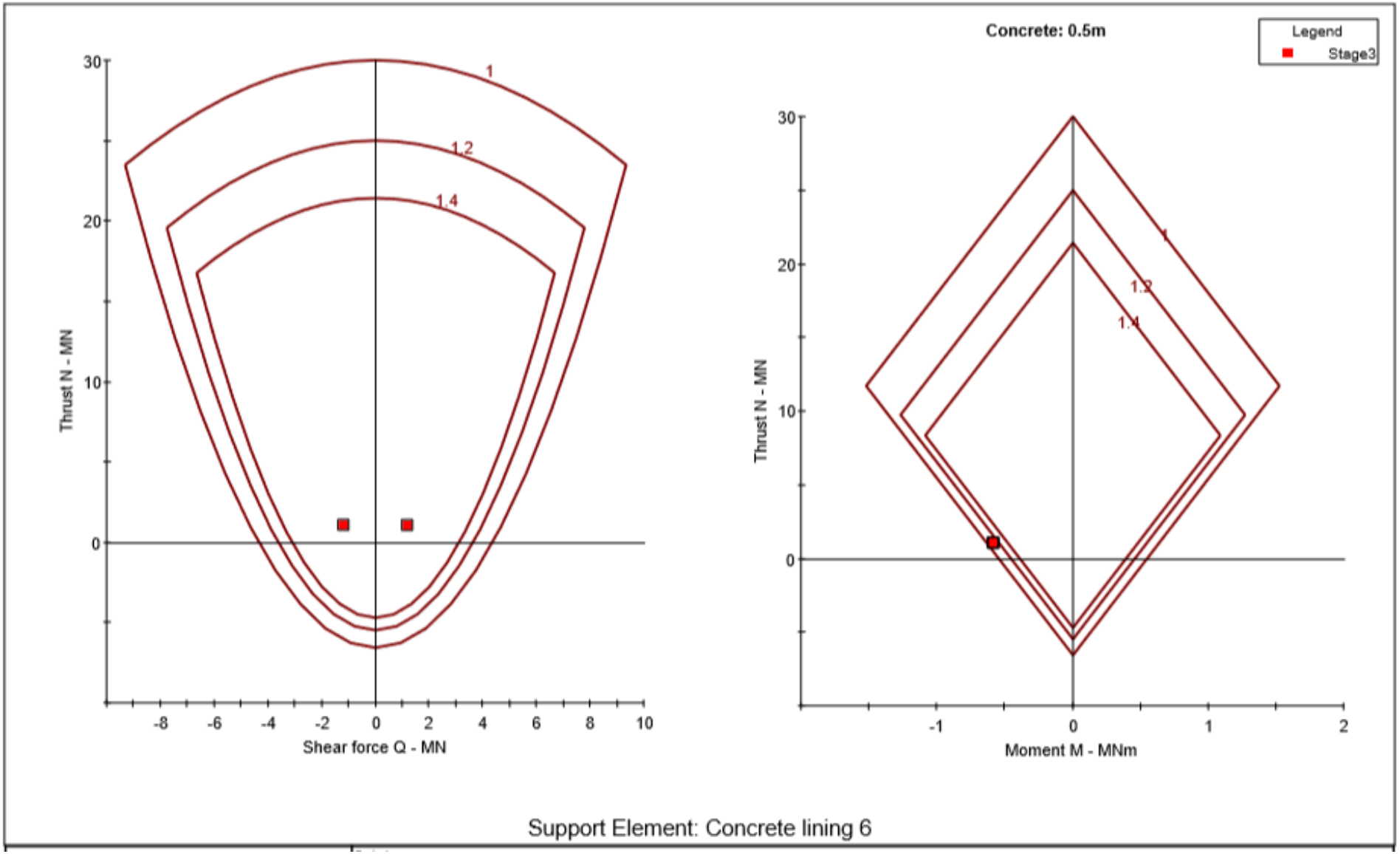


Figure 5-10 Support capacity plot for Modified Support for SHEP at chainage 1864.122m

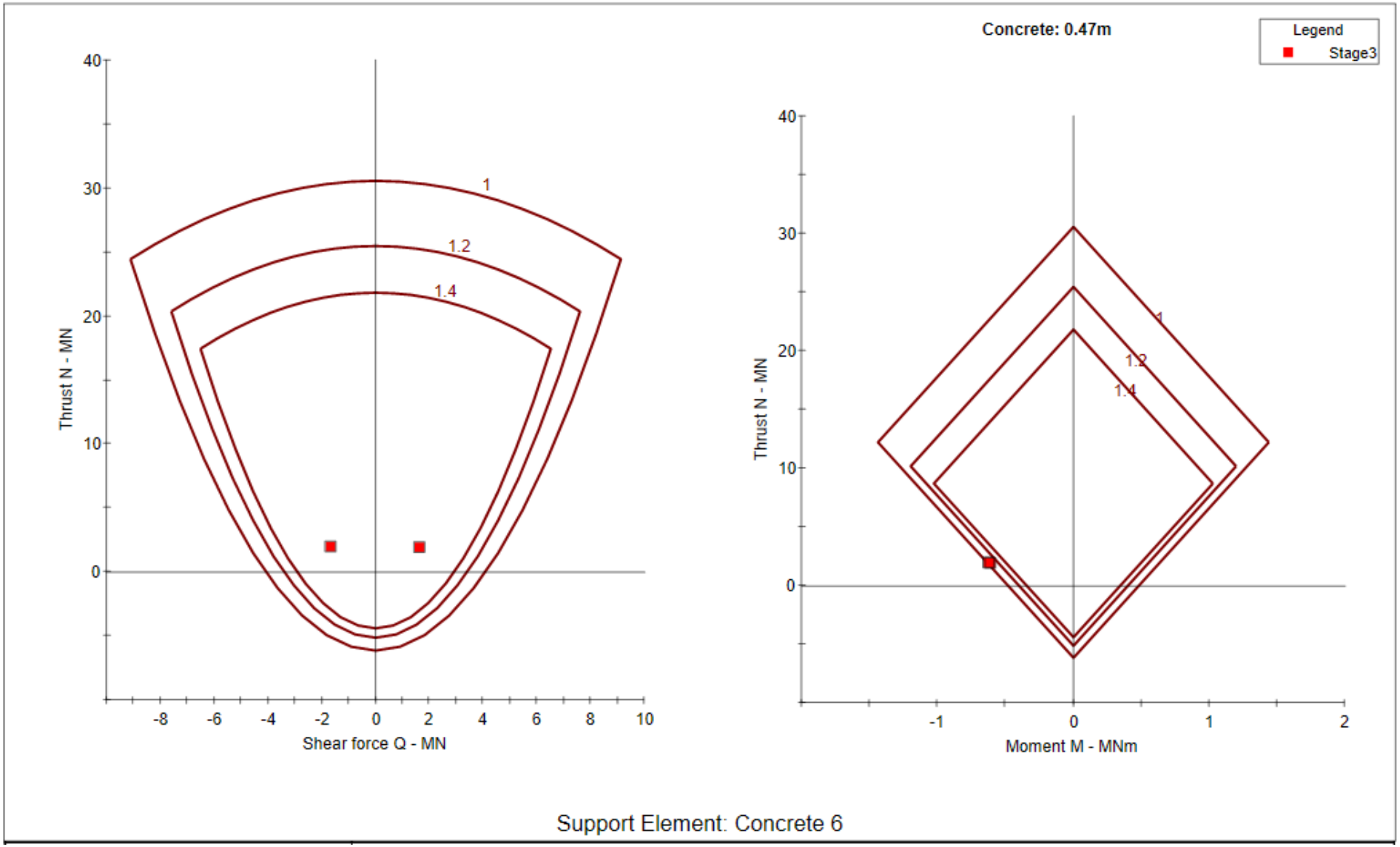


Figure 5-11 Support capacity plot for Modified Support for SHEP at chainage 2094.6m

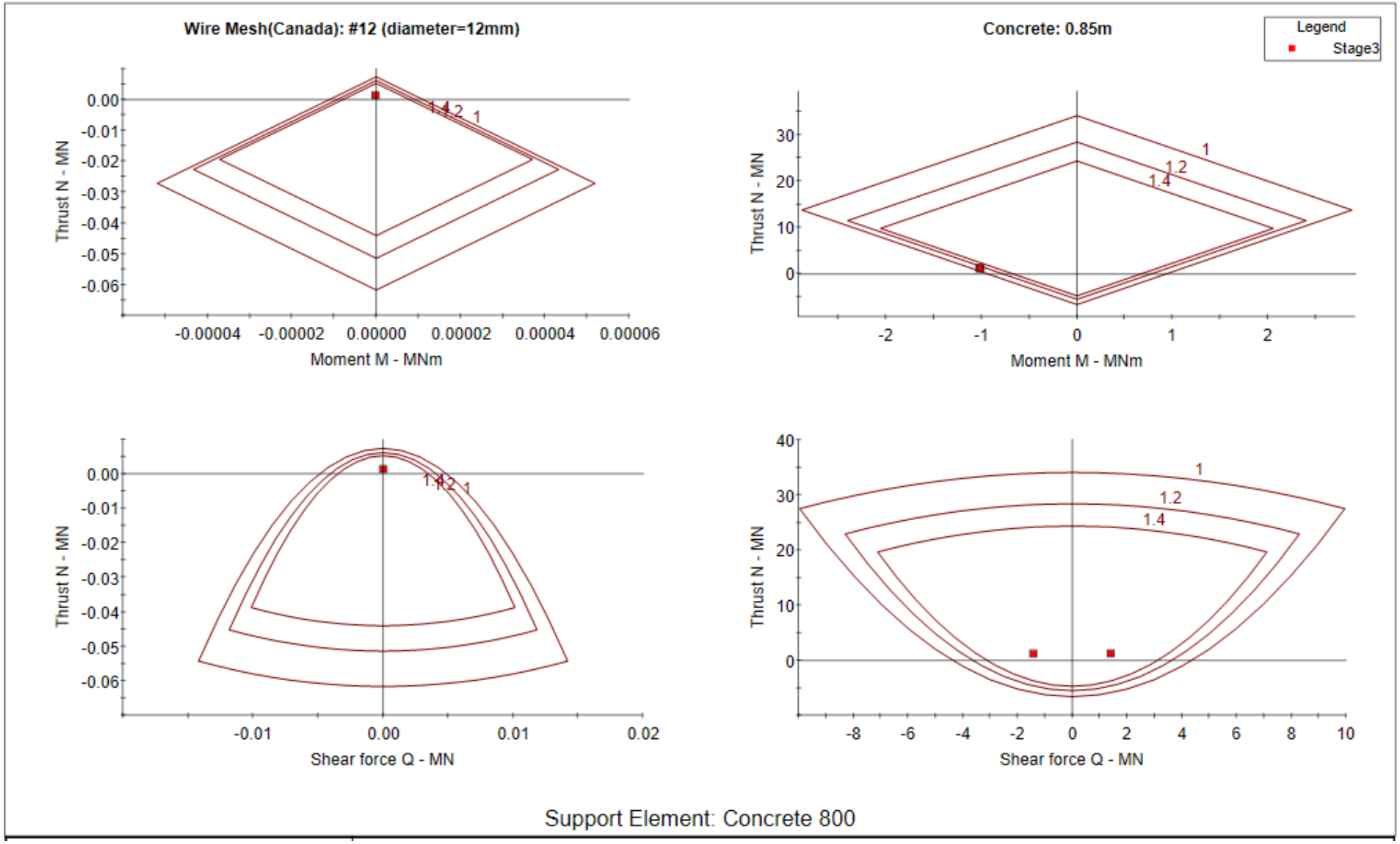


Figure 5-12 Support capacity plot for Modified Support for MBKHEP at chainage 500m

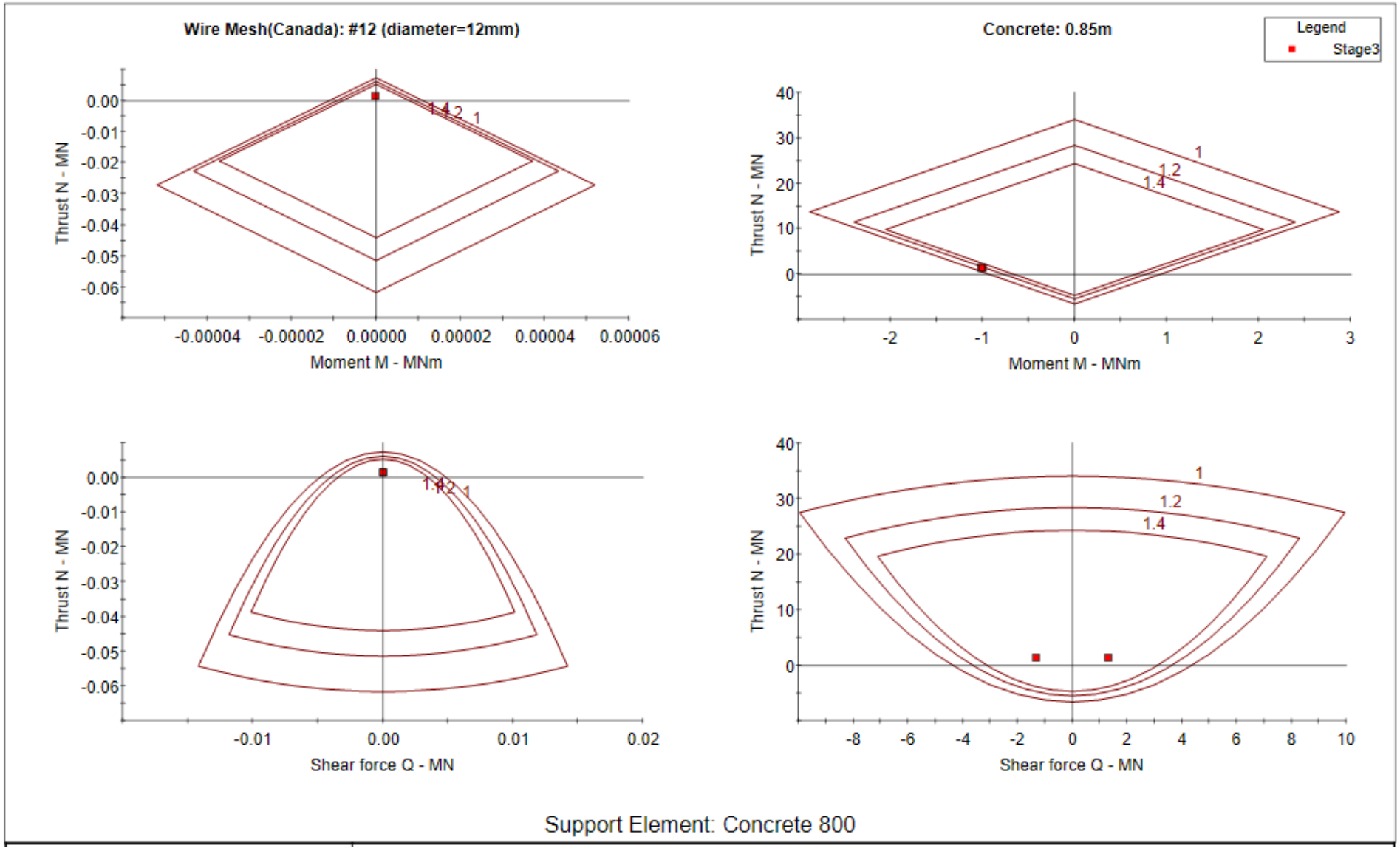


Figure 5-13 Support capacity plot for Modified Support for MBKHEP at chainage 6795.5m

The deformation of tunnel, after modification of support is measured and the value are provided in Table 5-5 and Table 5-6. As can be seen the deformation of the tunnel has also been greatly reduced hence proving the efficiency of the support.

Table 5-5 Maximum Deformation after Support Modification from Modelling for MBKHEP

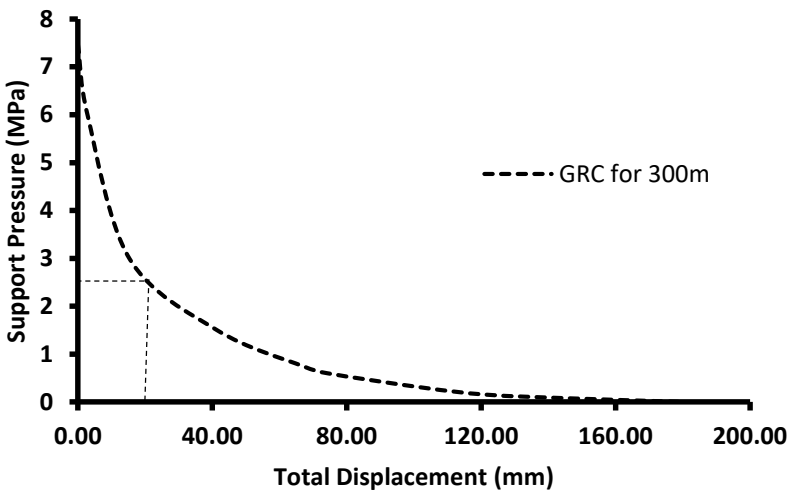
Chainage (m)	**Deformation (mm)**	**Radius of Plastic Zone (m)**
0+300	20	6.204
0+400	13	4.685
0+500	17	5.083
6+795.90	15	4.938

Table 5-6 Deformation after support modification from Modelling for Tunnel of SHEP

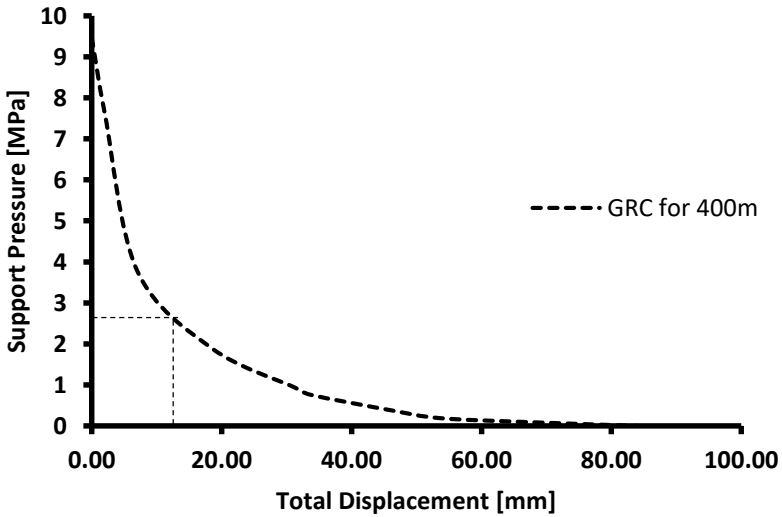
Chainage (m)	**At Invert Level**		**At Spring Level**
	Deformation (mm)	**Radius of Plastic Zone((m)**	**Deformation (Modified by using Numerical Modeling)(mm)**
1+864.1	13.10	5.08	9.80
2+094.6	33.80	6.33	21.90
2+137.7	38.30	5.55	26.90
1+759.8	No Modification Required		

Similarly a plot of GRC obtained from numerical modelling of the modified support can be used for determining the support pressure for several values of convergence. Figure 5-14 and Figure 5-15 proves the GRC for MBKHEP and SHEP respectively. The corresponding values of pressure for the deformation obtained after support modification is provided in Table 5-7 and Table 5-8 for MBKHEP and SHEP respectively. The graph can also be used to determine the support pressure at various points along the tunnel if the deformation at that particular point is known.

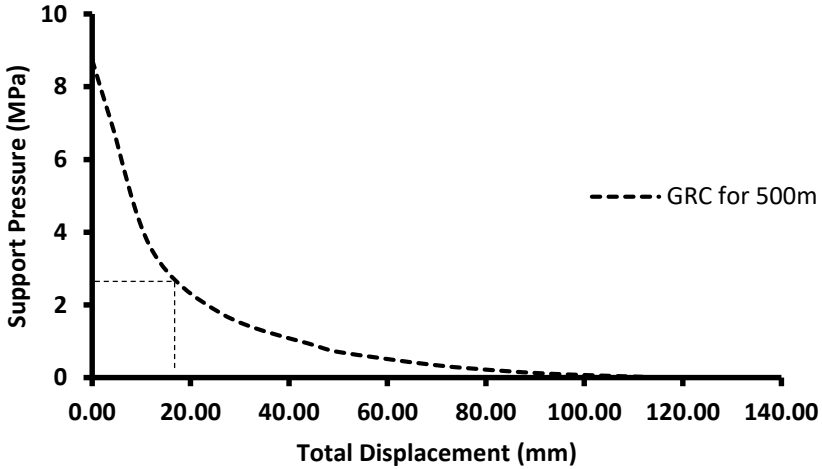
a. Chainage 0+300m



b. Chainage 0+400m



c. Chainage 0+500m



d. Chainage 6+795.5m

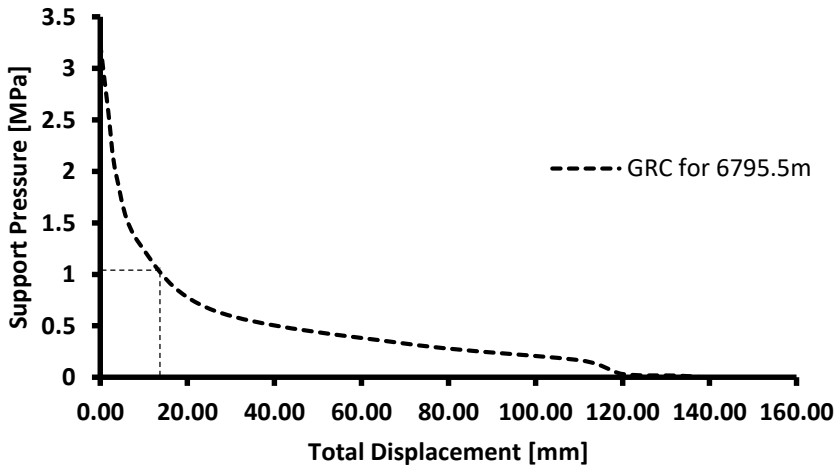
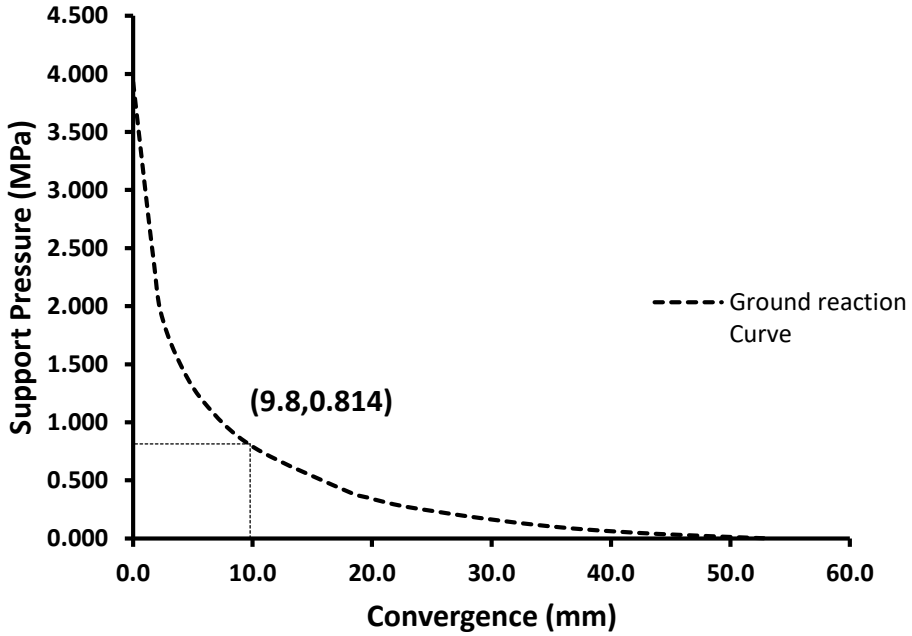
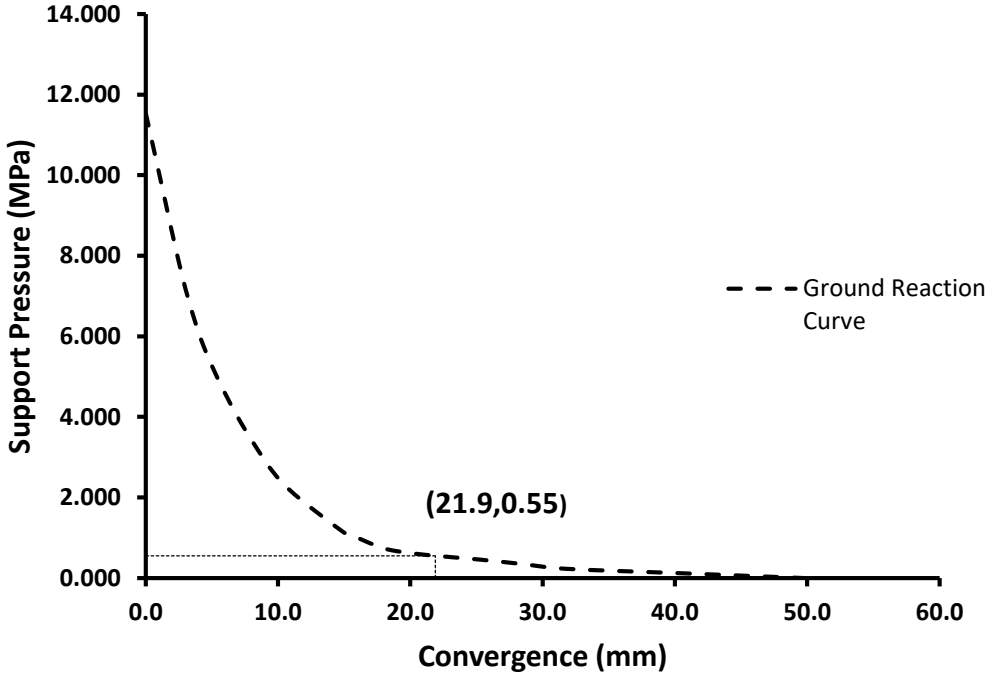


Figure 5-14 Plot of GRC obtained from Numerical Modelling for MBHEP at sections (a) 300m, (b) 400m, (c) 500m and (d) 6795.5m

a. Chainage 1864.22 m



b. Chainage 2094m



c. Chainage 2137.7m

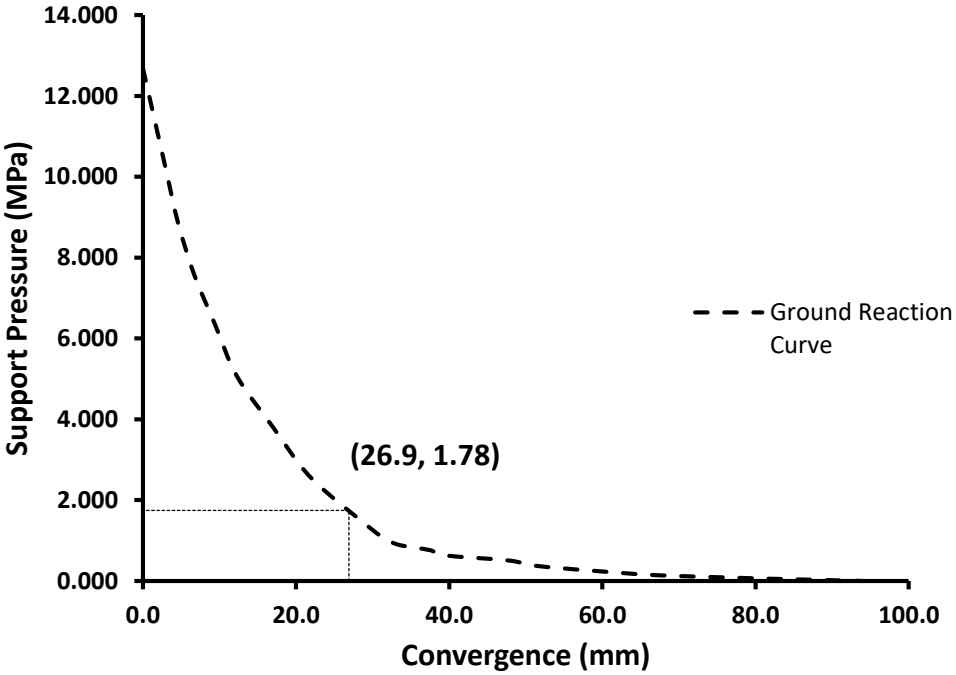


Figure 5-15 Plot of GRC obtained from Numerical Modelling for SHEP at sections (a) 1864.22m, (b) 2094m, (c) 2137.7m

Table 5-7 Support Pressure of Modified section from Numerical Modelling of MBKHEP

	From Numerical Modeling	
Chainage (m)	**Deformation (mm)**	**Support Pressure (MPa)**
0+300	20	2.52
0+400	13	2.797
0+500	17	2.9
6+795.90	15	1.07

Table 5-8 Support Pressure of Modified section from Numerical Modelling of SHEP

Chainage (m)	**Support Pressure using Numerical modeling (MPa)**
1+864.1	0.814
2+094.6	0.54
2+137.7	1.78
1+759.8	No modification required

CHAPTER 6 DESIGN OF HYDROPOWER COMPONENT

6.1 Hydraulic Design

6.1.1 Intake

The structure which allows a water of desired quantity to be conveyed into a conduit from a river or stream in a controlled manner is termed as intake. Intake is the structure to obtain the required quantity of water from the river or the reservoir for the different engineering purpose such as irrigation, power generation, water supply etc.

Intake is a structure to divert water into a conduit leading to the power plant or simply defined as a structure to divert water to a waterway. The intake serves as a transition between a stream that can vary from a trickle to a raging torrent, and a controlled flow of water both in quality and quantity. Its design, based on geological, hydraulic, structural and economic considerations, requires special care to avoid unnecessary maintenance and operational problems that cannot be easily remedied and would have to be tolerated for the life of the project.

Types of intake

Functionally, intake also can be divided as free-flow or surface type intake and pressure orifice or sub surface type depending on type of operation required for the intake.

- **Surface intake:** Canal intake, side intake, frontal intake and bottom rack intake are surface intake type.
- **Sub surface intake:** It is also called pressure intake, the intake from reservoir- side slope intake, tower intake, dam intake, power or forebay intake are called sub surface intake.

Generally, one of the following types of intake shall be used for run-of-river hydropower projects:

- Side (or Lateral) intake.
- Frontal intake.
- Drop (or Trench) intake.

But most commonly used type of intake are:

- Side Intake
- Bottom Intake

(a) Side Intake

A structure built along a river bank and in front of a canal/conduit end for diverting the required water safely. Side intakes are simple, less expensive, easy to build and maintain.

A side intake shall be used to draw water from the river through an intake structure located on the riverside, its longitudinal axis shall be aligned perpendicular to the axis of the river. It shall be sited immediately upstream of the diversion structure.

Side intakes shall typically consist of the following components:

1. A trash rack supporting structure.
2. Intake opening for permitting entry of water from the river.
3. Gate slot for closing intake openings/ stop log grooves.
4. Breast walls for control of flow during flood season.
5. Piers for diving intake with large horizontal spans into two or more sections.
6. Service platform for operation of gates and stop logs, trash handling and general maintenance.

(b) Bottom Intake

Bottom intakes are suitable where there is no significant sediment movement along the river bed, the longitudinal river slope is relatively steep (e.g. at least 1:10), and excess flow is available for flushing even during the dry season. Usually, bottom intakes are designed as trench so that the direction of diverted flow and river flow are perpendicular to each other.

<u>Selection of type of intake</u>

The most suitable type of intake for a particular site shall be selected considering the following factors:

1. Nature of river.
2. Nature and scale of hydropower development.
3. Sediment, trash and debris content.
4. Construction and maintenance considerations.

The type of intake selected based on the above considerations should generally be verified through model studies.

Components of an Intake

1. Trashrack

The water conduit system (e.g. headrace canal) needs to be prevented from floating logs and sediments from entering through the intake. For this purpose, a trash rack is placed at the intake mouth. The bars in the trash rack should be spaced should be such that any gravel that enters the system can be transported by the headrace to the downstream flushing structure, such as a gravel trap. As the bar spacing of trashrack at intake is wider as compared to the trashrack at the forebay, the trashrack of intake is also known as "Coarse Trashrack".

The hydraulic design of Trashrack for the project is in compliance with IS 11388:2012. The design consists of the shape of the trashrack, inclination of racks, rack velocity, spacing between bars and geometry of rack bars.

2. Orifice for Side Intake

An orifice is an opening in the intake from which the river water is conveyed towards the headrace. It allows passage of design flow through it but obstruct higher flows during flood. The discharge through an orifice for submerged condition is:

$$Q = AC\sqrt{2g(h_r - h_h)} \qquad 6.1$$

Where,

Q = discharge through orifice

A = orifice area in m2

C = coefficient of discharge of orifice (depends upon the shape of the orifice)

h_r = water level in the river next to the orifice relative to a datum

h_h = water level in the headrace canal measured from the same datum as h_r

Generally, the velocity through the orifice is kept within 1.0 – 1.5 m/s. However, if the orifice is placed directly at the river (without a trash rack), the velocity must be limited to 1.0 m/s so to avoid the passage of bed load into the intake. The orifice shall be preferably of rectangular shape so to draw design flow into the water conduit system and limit excess flow during floods. The orifice must be kept fully submerged during dry season so to ensure full design flow.

3. Gate

There is a provision of placing a control/sluice gate at the intake so that the plant can be easily closed for repair and maintenance. The gate should be able to resist damage caused by floods and boulders.

6.1.2 Approach Canal

The approach canal connecting the intake structure to the settling basin shall have an optimum length chosen with due consideration to the topography of site. However, the canal shall be economically effective and hydraulically efficient for transporting the specified size of sediments passing through the gravel trap.

Design Assumption

The design of the approach canal shall be performed assuming steady (sub-critical) flow regime in the canal. Under this assumption, the canal shall be designed using Chezy's or Manning's formula.

Hydraulic Characteristics

Based on the principle of non-silting and non-scouring velocity with efficient transporting capacity of flow with regard to sediments, either Manning's or Chezy's formula may be applied to determine the hydraulic characteristic of the approach canal. Manning's equation shall be given by

$$V = \frac{1}{n}(S^{1/2}R^{2/3}) \qquad 6.2$$

Where, V is the mean flow velocity in the canal in m/s,

n is Manning's rugosity coefficient,

S is the longitudinal bed slope and

R is the hydraulic mean radius in m, given by

R =A/P

Where, A is the flow area of cross-section in m^2 and

P is the wetted perimeter in m.

Likewise, Chezy's formula shall be given by

$$V = C * (RS)^{1/2} \qquad 6.3$$

Where, C is Chezy's coefficient

As Manning's and Chezy's coefficients are sensitive to increase or decrease in flow velocity and can directly affect the size of an approach canal, the coefficients shall be chosen as to reliable values as per soil/rock characteristics of the site.

Head loss and seepage

Head loss and seepage in the canal should be minimum as far as possible. Head loss is governed by the canal slope while the seepage is governed by the lining material. To achieve minimum head loss, the canal slope should be selected as gentle as possible and lining material should be chosen on the basis of the nature of soil available

6.1.3 Settling basin

Settling basin is the structure to remove suspended sediments from conveyance water for power plant. Settling basin shall be designed to ensure that water entering the water conveyance system is free of sediments that can damage penstock and turbine runners due to allow suspended sediment particle to settle out from water body and deposit on the bottom of basin.

The main principle of design of settling basin is to reduce mean velocity of the flow, by increasing the cross sectional area (widening its width and lowering the floor). Settling basin can remove the suspended inorganic particles ranging from sand (2mm in diameter) to silt (0.002mm in diameter).

Design principle and criteria

a. The basic principle of settling is that the greater the basin surface area and the lower the through velocity, the smaller the particle that can settle.
b. The settling basin shall be designed to be functional, easily operable and economical, both for construction and operation.
c. The settling basin shall be designed to remove as much of the sediment load in the water as is economically and hydraulically possible. As removal of all suspended sediments is physically not possible, the design shall attempt to remove as much of the coarser fraction of the suspended load as possible so that the hydraulic transport capacity of the water conveyance system can be maintained and the sediment load to the turbines, valves, etc., is reduced to acceptable limits.
d. The settling basin shall be designed to ensure efficient flushing of settled sediments so that frequent flushing during floods, when the sediment content of river is at its peak, is

not required. The settling basin shall be planned and designed such that power generation is not interrupted, or reduced, during flushing operations.

A settling basin must satisfy the following design considerations:

1. Settling capacity

The size of basin must be large enough to allow a large percentage of the fine sediment to fall out of suspension and be deposited on the bottom.

The geometry of the inlet and outlet transitions and any other curvatures must be such as to cause minimum turbulence, which might increase the trapping efficiency of the basin.

2. Storage capacity

The basin should be able to store the settled particles for some times unless it is flush out.

<u>Components of settling basin</u>

Settling basins shall consist of the following components:

a. An inlet transition including gated arrangement,
b. Settling chamber/s to settle the sediments in the incoming water,
c. An emergency spillway to spill the settling basin discharge into the parent river, if required, but generally it is not preferred as it creates higher turbulence.
d. An outlet transition with the regulator to control the flow of water from settling chamber,
e. A flushing channel with gates for flushing the settled sediment.

1. Inlet transition

a. The inlet transition for settling basins shall be designed to prevent turbulent flow at the entrance to the settling basin chamber.
b. The approach canal to the settling basin shall have a straight alignment for a stretch equal to about 10 times the width of the canal upstream of its junction to maintain an even flow distribution at the start of the inlet transition.
c. The hydraulic design of the canal shall eliminate secondary currents in its flow caused by rotational flow.it shall also ensure flow velocities in the range of 1.1 to 1.3 m/s.
d. To maintain gradual expansion of inlet channel, set the horizontal expansion ratio at about 1:5(or $\alpha = 7°$ to $15°$). The vertical expansion ratio can be set at about 1:2 (or $\alpha = 20°$ to $30°$).

e. If space constraints do not permit the long transition may be shortened through guide walls in the transition.
f. If space constraints do not permit a straight section of approach canal, even flow in the settling basin, may be achieved using a tranquilizer or baffle blocks.

2. Settling chamber

The hydraulic design of the chamber should be carried out to secure:

a. An even flow distribution between parallel settling basins for various flows.
b. An even flow distribution internally inside each basin for various flows.
c. Efficient removal of deposits during flushing of the basin.

<u>Flow Design parameters</u>:

a. Mean depth of flow in the basin
b. Mean longitudinal velocity
c. Fall velocity
d. Flushing discharge
e. Sediment concentration of flow

3. Outlet zone

The operation water level of the settling basin is controlled at the outlet zone. The outlet zone is designed for decanting the outflow from the settling basin over a small weir/ spillway or simple transition structure. The major parts included in the outlet zone are: transitions to headrace/ forebay inlet with or without control gate mechanism, sediment flushing gate/valves arrangements. The outlet transition can be more abrupt than the inlet expansion. The horizontal and vertical expansion ratio can be set at about 1:2 and 1:1 respectively.

4. Sediment flushing system

The flushing arrangement should be provided to flush the sediment deposited in the basin. It should ensure that when the flushing gate or valve is opened, the water level in the basin decreases and the sediment deposited in the bed starts flushing out. To facilitate the flushing of sediment, the bed slope at the bottom should be 1:20 to 1:50 in longitudinal direction. The bed can be provided with a flat or under sluicing manifold system with purging channel. The under sluicing of flushing system is designed as a manifold system to withdraw sediment laden water uniformly along the length of the purging channel. The design should be based on pressurized flow. The flow velocity should be more than 1.75m/s and not higher than 2.5 m/s.

6.1.4 Surge Tank

Surge tank is generally a cylindrical storage reservoir which is connected to a penstock pipe as close as possible to the penstock pipe. The surge tank, also called expansion chamber, is located between the headrace pressure conduit and the steeply sloping penstock pipe and is designed either as chamber excavated in the mountain or as a tower raising high above the surrounding terrain. Primary purpose of surge tank is to protect the long pressure tunnel and penstock pipe from high water hammer effect caused due to the sudden acceptance and rejection of loads.

The surge tank converts these fast (water hammer) pressure oscillation into much slower and lower pressure fluctuation due to mass oscillation in the surge chamber. The surge chamber divides the pressure tunnel into a short high pressure penstock downstream and low pressure tunnel upstream thus function as reservoir for the absorption and delivery of the water to meet the requirements of the load.

The height of the surge tank is governed by the highest possible water level that can be anticipated during operation. The variations in the demand initiated by the rapid opening or closure of the terminal valve or turbine are followed with a time lag by water masses moving in the tunnel.

Major functions

a. Reduce the effect of water hammer.
b. Intercept or, at least, to radically reduce the pressure surges due to water hammer and to exempt thereby the pressure tunnel from excessive internal loads.
c. The surge tank provides protection to the penstock against the detrimental effects of the water hammer if no bypass valve is installed or if the bypass valve fails to operate.
d. Use, receive or store water when the turbine is stopped (it acts as a quick storage device)
e. Temporary supply of the water when the load on the turbine is suddenly increased.
f. Provide water supply to the turbines in case of the starting up (load demand) until the conduit velocity is accelerated to the new steady state value. When the turbine is started, water demand suddenly increases the flow of the water in the penstock is suddenly initiated and respectively accelerated.

Design consideration of surge tank

Design of surge tank is associated with the transient analysis of the waterway and water hammer therefore; it is recommended to use a specific literature devoted to the design of the surge tank.

Some of the basic surging conditions that are to be examined for the design of the surge tank are listed below:

1. Stability Condition

Small water level fluctuation during the operation should be damped and water in the surge tank should be made stable.

2. Up-surging condition

The top elevation of surge tank should be higher than the up-surging water level due to rapid interception of full load.

3. Down-surge condition

Surge tank should be above the penstock inlet sufficiently to avoid air traps.

4. Damping condition

A surge tank should be damped even if the succeeding load fluctuation arises.

6.2 Optimization of Tunnel

Tunnel optimization is an important aspect of tunnel design. The factor considered while working out the economical diameter are the velocity requirement, head loss in tunnel, annual operation and maintenance charge. (Baral, 2073)

6.2.1 Geometry of Tunnel

The geometry of the tunnel used is the D-shape. For this shape, it necessary for finding the diameter that is the width of the tunnel and wall height relationship. To find the relationship between these parameters and intending to establish the economic relationship, supposing the width of tunnel be D and the wall height be H, then Area of Cross Section (A) and Perimeter of Cross Section (P) can be found by:

$$A = \frac{\pi}{8} \times D^2 + D \times H \qquad 6.4$$

$$P = \left(\frac{\pi}{2} + 1\right) \times D + 2 \times H \rightarrow 2.57 \times D + 2 \times H \qquad 6.5$$

Solving these equations, we finally obtain a final relation of Area and Perimeter with the Diameter of the tunnel. The relation is as follows:

$$A = 0.9786\ D^2 \qquad 6.6$$

$$P = 3.792D \qquad 6.7$$

The entire calculation for obtaining the given relation is provided in Appendix A.

6.2.2 Support Condition

Support Condition was designated by using Q system developed by Barton, Lien and Lunde. The Q-value varies between 0.001 and 1000. Q value is given by:

$$Q = \frac{RQD}{Jn} * \frac{Jr}{Ja} * \frac{Jw}{SRF} \qquad 6.8$$

Where, RQD is the Rock Quality Designation

Jn is the joint set number

Jr is the joint roughness number

Ja is the joint alteration number

Jw is the joint water reduction factor

SRF is the stress reduction factor

6.2.3 Unsupported Span

Unsupported span can be defined as the distance between the excavated face of the tunnel and the nearest rock support. Based on the value of ESR and Q value we can determine the maximum unsupported span that can be used for the excavation before installation of the support. If the rock mass is left unsupported even after the maximum unsupported span has exceeded, it is considered to be dangerous.

For Good Rock the unsupported length that can be provided is 20m.

For Fair Rock, the unsupported length that can be used is 10m. However minimum support should be provided after each blasting.

For Poor Rock, the unsupported length that can be used is 10m (rarely). However the support shall be provided concurrently with the excavation.

For very poor rock, the unsupported length should not be considered. Support should be installed after each excavation work. Shotcrete should be applied after each blasting.

Calculation for unsupported span we use formula:

$$Maximum\ unsuported\ span = 2 \times ESR \times (Q - value)^{0.4} \qquad 6.9$$

6.2.4 Calculation of head loss

Head loss refers to the measurement of energy dissipated in a system due to friction. It accounts for the totality of energy losses due to the length of a pipe and those due to the function of fittings, valves and other system structures. The loss in head due to friction is known as the major head loss whereas the head loss due to expansion, contraction, Bends etc are considered as the minor head loss.

Calculation of the Major Head Loss due to Friction

The major head loss due to friction (for non-circular tunnels) is given by Darcy Weisbach Formula:

$$\mathrm{Hf} = \frac{fLv^2}{2g(4R)} \qquad 6.10$$

Where f =friction coefficient

L=Length of Tunnel

V= Velocity of water

g=acceleration due to gravity

R= Hydraulic Radius

The bend loss can be calculated using the formula given by:

$$H_b = k_b\left(\frac{v^2}{2g}\right) \qquad 6.11$$

Where, K_b=Coefficient for the bend

v=velocity of water

g=acceleration due to gravity

In order to determine the value of the coefficient of the bend loss formula given by Lysne et al. has been used. The bend loss is one of the significant parts of singular losses along the waterway system and it is even more pronounced in sharp bends with small radius. Figure 6-1 shows the bend loss coefficient as the function of ratio between bend radius (R_b) and width of the tunnel at bend (D) for 90° bend (Lysne et al., 2003). For bends with other angles, a reduction factor has also been proposed. The bend loss coefficient for different bend angles can be calculated by multiplying bend loss coefficient of 90° bend with a reduction factor.

To determine the value of the coefficient of bend for 90°, we can even use the formula given by:

$$y = 0.2476x^{-0.874} \qquad 6.12$$

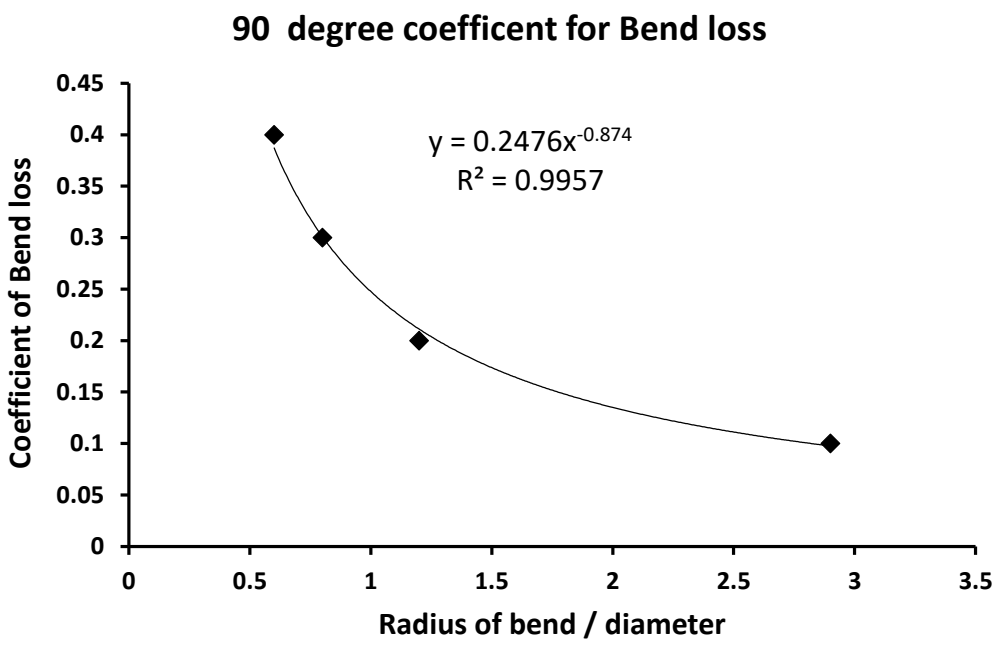


Figure 6-1 Coefficient of Bend for 90 degree bend (Panthi K., Basnet C., 2018)

Now, after determining the value of the coefficient of bend for 90 degree angle, we can determine the coefficient for the actual bend. For that, we can use formula:

$$K_b = \text{Reduction factor} * \text{coefficient of bend for } 90^0 \qquad 6.13$$

The reduction factor can be determined from the graph below once we have the bend angle.

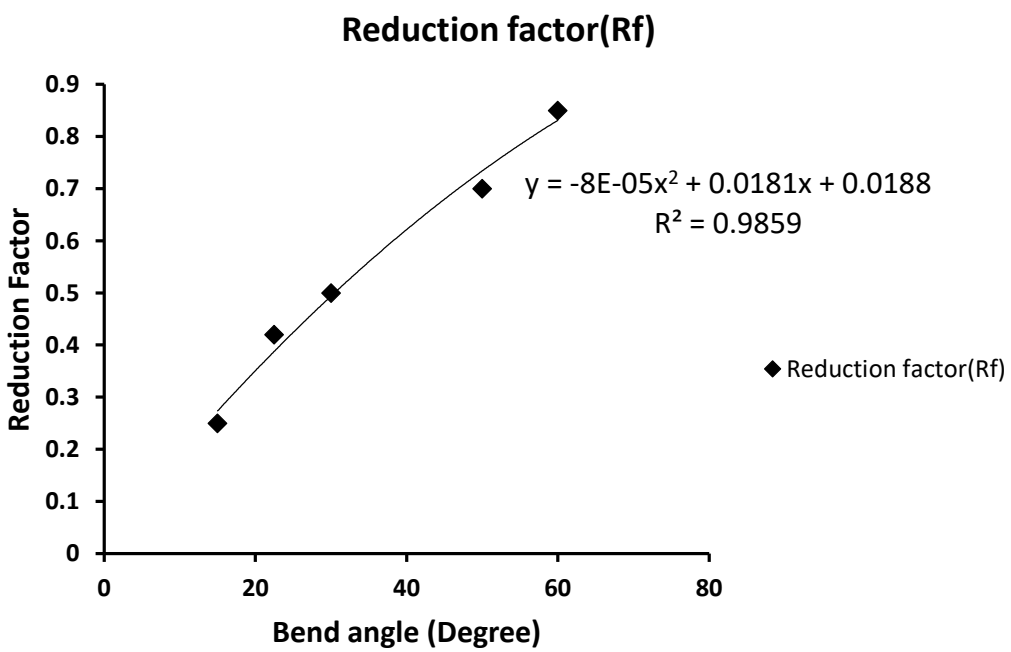


Figure 6-2 Reduction Factor (Panthi K., Basnet C., 2018)

6.2.5 Reinforcement of Invert concrete in HRT

The reinforcement of the invert concrete in HRT is done according to as-built drawing. According to which the invert concrete is of 25 cm thick with the main bar of 20mm diameter having the length of 3.5m, while the distribution bar is of 16mm diameter running along the axis of tunnel. 200mm center to center is provided for both main and distribution bars.

For which calculation is done to find out the number of bars:

a. For main bars

$$Number\ of\ bars\ per\ meter = \left(\frac{1000mm}{Spacing\ of\ bars\ c/c}\right) + 1 \qquad 6.14$$

b. For distribution bars

$$Number\ of\ bars\ per\ meter = \left(\frac{1000mm}{Spacing\ of\ bars\ c/c}\right) + 1 \qquad 6.15$$

Then total number of in HRT is given by $= Number\ of\ bars\ per\ meter\ \times length$

6.2.6 Case Studies

6.2.6.1 Sanjen Hydroelectric Project

<u>Geological investigation data</u>

Table 6-1 Rock Type along the chainage with its Q-Value (after SHEP Feasibilty Report)

Chainage	Rock Type	Q
0+000 -0+755.14	Graphictic Schist with Crenulated Phylite and Slate	1.43
0+755.14 -1+052.04	Dolomitic Marble	10.44
1+052.04 - 1+282.82	Graphictic Schist with Crenulated Phylite and Slate	2.69
1+282.82 - 1+498.48	Dolomitic Marble	14.59
1+498.48 - 1+644.54	Graphictic Schist with Crenulated Phylite and Slate	2.95
1+644.54 - 2+130.22	Dolomitic Marble	13.13
2+130.22 - 2+402.49	Psammitic Schist with Crenulated Phylite and Quartzite	10.61
2+402.49 - 2+581.06	Green-grey Quartzite	13.13
2+581.06 - 3+081.23	Psammitic Schist with Crenulated Phylite and Quartzite	11.97
3+081.23 - 3+292.28	Graphitic Schist with Crenulated Phylite and Quartzite	6.57
3+292.28 - 3+303.06	White Quartzite	2.74
3+303.06 - 3+629.67	Psammitic Schist with Quartzite	5.64

Detailed Rate for Construction used in optimization has been provided in Appendix A

Total Construction Cost

Table 6-2 Total Construction Cost for SHEP

Diameter (m)	Other price (Rs)	Total Price of excavation (Rs)	Total support cost (Rs)	Cost of rock trap (Rs)	Total cost (m)
3.00	137960578.26	155194503.61	69235971.70	16711740.18	379102793.75
3.10	137960578.26	165713242.19	72619608.30	16711740.18	393005168.93
3.20	137960578.26	176576857.44	71966547.34	16711740.18	403215723.22
3.30	137960578.26	187785349.37	74020998.11	16711740.18	416478665.92
3.40	137960578.26	199338717.97	76075448.89	16711740.18	430086485.29
3.50	137960578.26	211236963.25	78129899.66	16711740.18	444039181.34
3.60	137960578.26	223480085.20	80184350.43	16711740.18	458336754.07
3.70	137960578.26	236068083.83	82238801.21	16711740.18	472979203.47
3.80	137960578.26	249000959.13	84293251.98	16711740.18	487966529.54
3.90	137960578.26	262278711.10	86347702.75	16711740.18	503298732.29
4.00	137960578.26	275901339.75	88402153.53	16711740.18	518975811.71

Head Loss Cost

Table 6-3 Head Loss Cost for SHEP

Diameter (m)	Wall Height (m)	Total Height (m)	Area (m^2)	Perimeter (m)	Hydraulic Radius (m)	Roughness Value	Value of Friction Factor (f)	Head loss (Darcy-weisbach) (m)	Bend Loss (m)	Power lost (kW)	Energy Loss (kWh)	Cost of the power loss (Rs.)
3	1.76	3.26	8.81	11.23	0.785	32.19	0.241	24.49	2.06 E-03	2390.80	20943432	120075676.81
3.1	1.82	3.37	9.40	11.60	0.811	33.26	0.239	20.65	4.42 E-03	2015.72	17657698.80	101237473.10
3.2	1.88	3.48	10.02	11.97	0.837	34.33	0.238	17.50	4.00 E-03	1708.60	14967335.57	85812723.95
3.3	1.93	3.58	10.66	12.35	0.863	35.41	0.236	14.91	3.63 E-03	1455.72	12752118.87	73112148.20
3.4	1.99	3.69	11.31	12.72	0.889	36.48	0.235	12.76	3.31 E-03	1246.26	10917267.20	62592331.95
3.5	2.05	3.80	11.99	13.10	0.915	37.55	0.233	10.98	3.02 E-03	1071.80	9388953.91	53830002.43
3.6	2.11	3.91	12.68	13.47	0.941	38.62	0.232	9.48	2.77 E-03	925.71	8109245.83	46493009.42
3.7	2.17	4.02	13.40	13.85	0.968	39.70	0.231	8.22	2.54 E-03	802.78	7032362.50	40318878.36
3.8	2.23	4.13	14.13	14.22	0.994	40.77	0.229	7.16	2.34 E-03	698.85	6121887.23	35098820.13
3.9	2.29	4.24	14.88	14.59	1.020	41.84	0.228	6.25	2.16 E-03	610.58	5348670.40	30665710.28
4	2.34	4.34	15.66	14.97	1.046	42.92	0.227	5.48	1.99 E-03	535.30	4689240.96	26884981.52

Remarks: For Dry season the cost of power generation is Rs.4.40 and Wet season Rs. 8.40

Optimization

Table 6-4 Parameters for Optimization for SHEP

Diameter (m)	Total construction cost (Million Rs)	Head Loss cost (Million Rs)	Total cost (Million Rs)
3.00	379.10	120.08	499.18
3.10	393.01	101.24	494.24
3.20	403.22	85.81	489.03
3.30	416.48	73.11	489.59
3.40	430.09	62.59	492.68
3.50	444.04	53.83	497.87
3.60	458.34	46.49	504.83
3.70	472.98	40.32	513.30
3.80	487.97	35.10	523.07
3.90	503.30	30.67	533.96
4.00	518.98	26.88	545.86

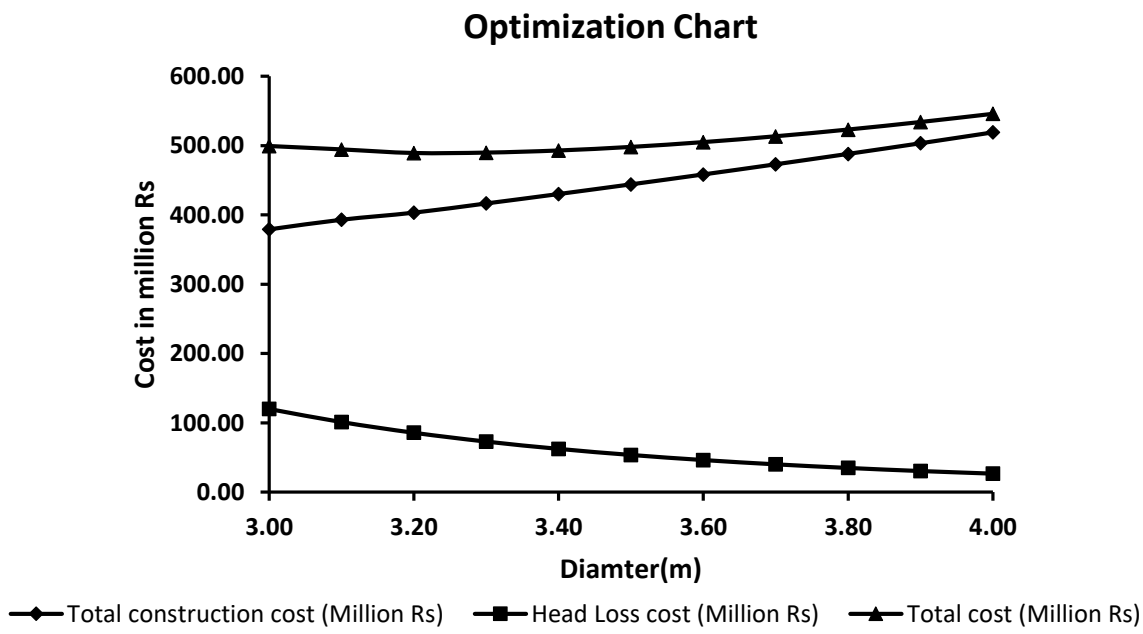


Figure 6-3 Optimization Chart

Using the values obtained from the table above, Plotting diameter of the tunnel from 3m to 4m on X axis and the total cost of the tunnel construction, Head loss cost and the total cost (in million) in the Y axis, we obtain the optimization chart where the minimum value of cost from the total cost was found to be 489.03 Million Rs at the optimum diameter of 3.2m.

6.2.6.2 Middle Bhotekoshi Hydroelectric Project

Geological Investigation Data

Table 6-5 Geological Instigation Data for MBKHEP (after MBKHEP Feasibility Report)

Chainage From - to (m)	Length (m)	Bieniawski's RMR Value	Barton's Q	Classification	Description
0-50	50	45-57	5.9	III	Fair
50-150	100	55	2.5	III-IV	Fair-Good
150-880	730	64	18.8	II	Poor
880-1300	420	29	1.4	IV	Good-Poor
1300-2350	1050	65	9.36	II-III	Very Poor
2350-2550	200	17	0.8	V	Fair
2550-3900	1350	58	7.1	III	Fair
3900-5300	1400	44	3.1	III-IV	Fair-Poor
5300-5560	260	29	1.3	IV	Poor
5560-7000	1440	54	6.6	III	Fair
7000-7124	124	44	6	III	Fair

Total Construction Cost

Table 6-6 Total Construction Cost for MBKHEP

Diameter (m)	Other price (Rs)	Total Price of excavation (Rs)	Total support cost (Rs)	Cost of rock trap (Rs)	Total cost (m)
5	41179853.90	846117357.26	92285488.88	16711740.18	996294440.22
5.1	41179853.90	880300498.49	137190258.65	16711740.18	1075382351.23
5.2	41179853.90	915160533.61	139564628.43	16711740.18	1112616756.12
5.3	41179853.90	950697462.62	141938998.21	16711740.18	1150528054.91
5.4	41179853.90	986911285.51	144313367.99	16711740.18	1189116247.57
5.5	41179853.90	1023802002.28	146687737.76	16711740.18	1228381334.13
5.6	41179853.90	1061369612.95	149062107.54	16711740.18	1268323314.57
5.7	41179853.90	1099614117.50	151436477.32	16711740.18	1308942188.89
5.8	41179853.90	1138535515.93	153810847.10	16711740.18	1350237957.10
5.9	41179853.90	1178133808.25	156185216.87	16711740.18	1392210619.20
6	41179853.90	1218408994.45	158559586.65	16711740.18	1434860175.18
6.1	41179853.90	1259361074.55	160933956.43	16711740.18	1478186625.05
6.2	41179853.90	1300990048.52	163308326.21	16711740.18	1522189968.81
6.3	41179853.90	1343295916.39	165682695.98	16711740.18	1566870206.45

Head Loss Cost

Table 6-7 Head Loss Cost for MBKHEP

Diameter (m)	Wall Height (m)	Total Height (m)	Area (m2)	Perimeter (m)	Hydralic Radius (R)	Roughness Value	Value of Friction Factor(f)	Head loss (Darcy-weisbach) (m)	Power lost (kW)	Energy Loss (kWh)	Cost of the power loss (Rs)
5	2.93	5.43	24.47	18.71	1.308	53.64	0.218	65.11	27905.37	244451014.4	1401519149.17
5.1	2.99	5.54	25.45	19.08	1.334	54.72	0.217	58.75	25180.53	220581476.43	1264667131.56
5.2	3.05	5.65	26.46	19.46	1.360	55.79	0.216	53.12	22767.46	199442914.76	1143472711.31
5.3	3.11	5.76	27.49	19.83	1.386	56.86	0.215	48.13	20625.43	180678790.84	1035891734.14
5.4	3.16	5.86	28.54	20.21	1.412	57.94	0.214	43.68	18719.71	163984658.28	940178707.46
5.5	3.22	5.97	29.60	20.58	1.438	59.01	0.214	39.71	17020.52	149099770.74	854838685.57
5.6	3.28	6.08	30.69	20.96	1.465	60.08	0.213	36.17	15502.30	135800163.24	778587602.56
5.7	3.34	6.19	31.79	21.33	1.491	61.15	0.212	33.00	14143.03	123892927.58	710319451.47
5.8	3.40	6.30	32.92	21.70	1.517	62.23	0.212	30.15	12923.68	113211459.40	649079033.88
5.9	3.46	6.41	34.07	22.08	1.543	63.30	0.211	27.60	11827.80	103611498.58	594039258.54
6	3.52	6.52	35.23	22.45	1.569	64.37	0.210	25.30	10841.08	94967820.10	544482168.56
6.1	3.57	6.62	36.41	22.83	1.595	65.45	0.210	23.22	9951.08	87171459.64	499783035.26
6.2	3.63	6.73	37.62	23.20	1.621	66.52	0.209	21.34	9146.96	80127380.73	459396982.84
6.3	3.69	6.84	38.84	23.57	1.648	67.59	0.208	19.64	8419.24	73752507.29	422847708.44

Remarks: For Dry season the cost of power generation is Rs.4.40 and Wet season Rs. 8.40

Optimization

Table 6-8 Parameters for Optimization for MBKHEP

Diameter (m)	Total construction cost (Million Rs)	Head Loss cost (Million Rs)	Total cost (Million Rs)
5	996.29	1401.52	2397.81
5.1	1075.38	1264.67	2340.05
5.2	1112.62	1143.47	2256.09
5.3	1150.53	1035.89	2186.42
5.4	1189.12	940.18	2129.29
5.5	1228.38	854.84	2083.22

5.6	1268.32	778.59	2046.91
5.7	1308.94	710.32	2019.26
5.8	1350.24	649.08	1999.32
5.9	1392.21	594.04	1986.25
6	1434.86	544.48	1979.34
6.1	1478.19	499.78	1977.97
6.2	1522.19	459.40	1981.59
6.3	1566.87	422.85	1989.72

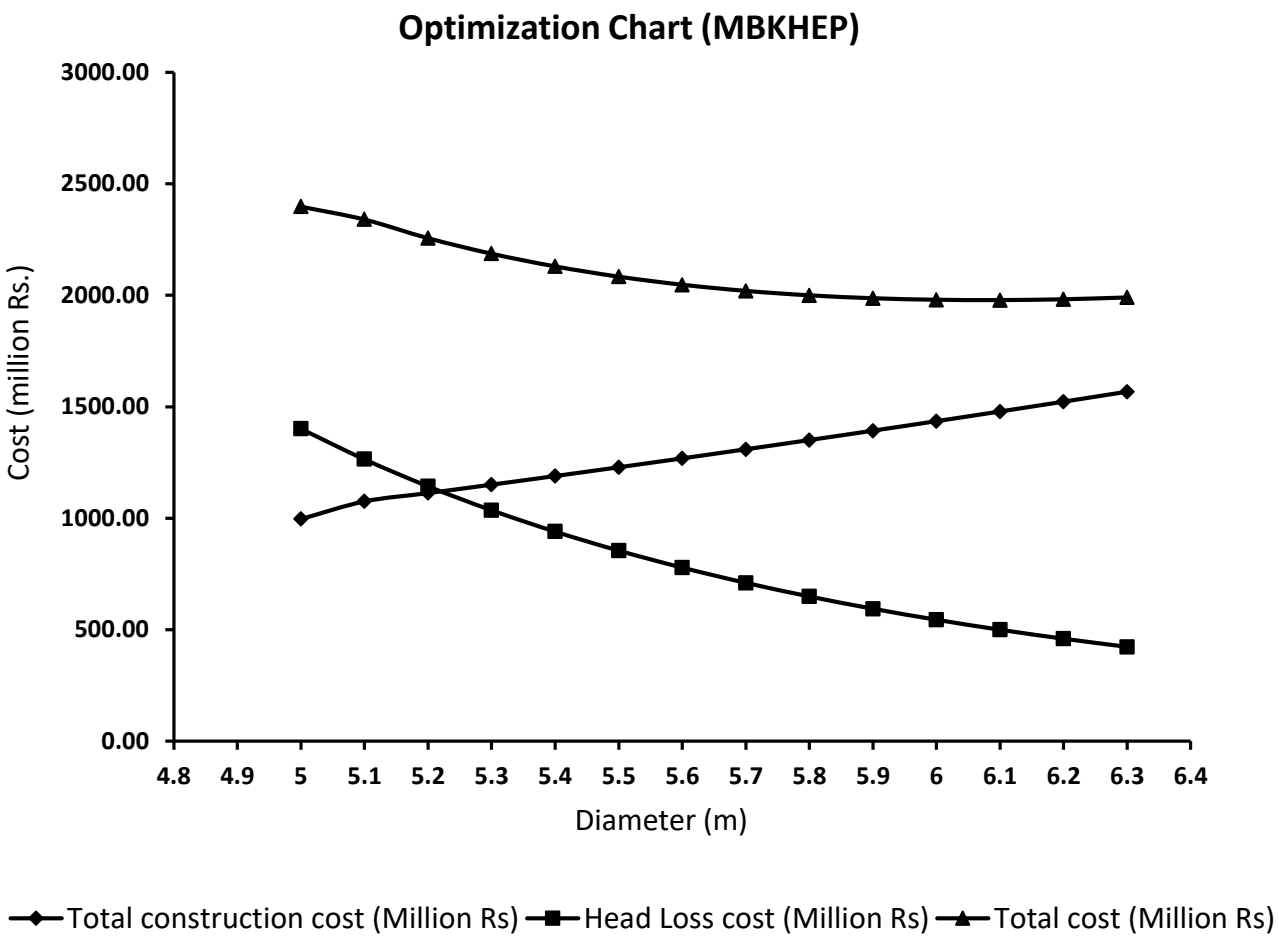


Figure 6-4 Optimization Chart for MBKHEP

From the optimization chart the minimum value of cost from the total cost was found to be 1977.97 Million Rs at the optimum diameter of 6.1m.

CHAPTER 7 CONCLUSION

Tunnels of two hydroelectric projects namely, Middle Bhotekoshi Hydro Electric Project and Sanjen Hydro Electric project were taken under study, ten and six tunnel sections of each HEP respectively where emphasized under the study. The detail analysis of these sections were performed under empirical, analytical and numerical assessment. Various aspects of stability assessment of designed support system- used at the project site itself and defined support system using the Rock Mass Classification(Q and RMR) were analyzed. Accordance with the study undertaken following points are concluded:

- The support systems provided at the site were modified rather than being a conventional support system provided according to the guidelines of Rock Mass Classification.
- The designed support system (Provided at the site) are stiffer than the support system provided by the Rock Mass Classification approach which can be clearly observed in Figure 3-6, Figure 3-7, Figure 3-20, Figure 3-21 and Figure 3-29.
- The comparative study of deformation obtained from support systems according to guidelines and the actual measured value of four tunnel sections of MBKHEP infers high deviation for poor to extremely poor rock mass for Q-Support and RMR support guidelines to be 3 and 2 times respectively with respect to actual value of the deformation as shown in the graph Figure 3-14, Figure 3-15 and Figure 3-16.
- Numerical analysis shows that support system, both used in site and defined using guidelines, for fair to extremely poor rock mass seem to be not adequate. There is high concentration of stress at the bottom end of the tunnel walls where tensile failures were observed.
- Numerical Modeling of the failed tunnel section suggested the thickness of the shotcrete or concrete should vary .The thickness of the shotcrete or concrete increases from spring level to required maximum thickness at the bottom and the profile obtained was more or less parabolic in nature as shown in Figure 5-8 and Figure 5-9.

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APPENDIX A

Values for Optimization

A.1 Rates for Optimization

S.N.	Particulars	Unit	Rate
	Inlet Portal		
1	Plain Shotcrete (50 mm)	*m2*	1362.00
2	Plain Shotcrete (75 mm)	*m2*	2043.00
3	Rock Bolt ordinary type (Dia 20mm)	*m*	1080.00
4	Welded Wire Fabric (6mm Dia, 150x150 Opening)	*m2*	1200.00
5	Rock Anchor (Dia 20mm)	*m*	1368.00
6	Portal Concrete Lining (C20/25:XC1)	*m3*	15936.00
7	Formwork Finish Class F2	*m2*	1195.00
8	Reinforcement	*ton*	113491.00

	Headrace Tunnel		
1	Plain Shotcrete (75mm)	*m2*	2043.00
2	Fibre Shotcrete (50mm)	*m2*	1450.00
3	Fibre Shotcrete (75mm)	*m2*	2175.00
4	Fibre Shotcrete (100mm)	*m2*	2900.00
5	Fibre Shotcrete (150mm)	*m2*	4350.00
6	Rock Bolts, Cement grouted Ordinary (Dia 20mm)	*m*	1080.00
7	Rock Bolts, Resin end-anchored (Dia 20mm)	*m*	1200.00
8	Rock Bolts, Cement grouted End Expansion shell-anchored (Dia 20mm)	*m*	1300.00
9	Welded Wire Fabric (6mm Dia, 150x150 Opening)	*m2*	1200.00
10	Portal Concrete Lining (C20/25:XC1)	*m3*	12791.00
11	Concrete Lining (C20/25:XC1)	*m3*	15936.00
12	Backfill Concrete (C12/15:X0)	*m3*	10080.00
13	Formwork for Lining (F3 Type Plane/Curved)	*m2*	1434.00
14	Reinforcement Steel all Dimentions	*ton*	117209.00
15	Concrete for Invert (C20/25:XC1)	*m*	12906.00
16	Cement mortar grouting	*ton*	23191.00
17	Consolidation Grouting	*ton*	12432.00
18	Drilling for Consolidation Grouting	*m*	967.00
19	Structutral Rib Steel Support	*ton*	125000.00
20	Lattice Girders for ribs of sprayed concrete	*ton*	206181.00
21	Spiles/Forepoling (Dia 25mm)	*m*	2975.00
22	Water Stop, Type A	*m*	2656.00
23	Anchor Bars (Dia 20mm)	*m*	1368.00

Rock Trap			
1	Fibre Shotcrete (50mm)	*m2*	1450.00
2	Fibre Shotcrete (100mm)	*m2*	2900.00
3	Rock Bolts, Cement grouted Ordinary (Dia 20mm)	*m*	1080.00
4	Rock Bolts, Resin end-anchored (Dia 20mm)	*m*	1200.00
5	Rock Bolts, Cement grouted End Expansion shell-anchored (Dia 20mm)	*m*	1300.00
6	Portal Concrete Lining (C20/25:XC1)	*m3*	12791.00
7	Backfill Concrete (C12/15:X0)	*m3*	10080.00
8	Formwork for Lining (F3 Type Plane/Curved)	*m2*	1434.00
9	Reinforcement Steel	*ton*	113491.00
10	Cement Mortar Grouting	*ton*	23191.00
11	Consolidation Grouting	*ton*	12432.00
12	Drilling for Consolidation Grouting	*m*	967.00
13	Anchor Bars (Dia 20mm)	*m*	1368.00

S.N	**Description**	**Units**	**Qunatity**	**Rates**	**Amounts**
1	Removal of unstable timber	No.	25.00	1328.00	33200.00
2	Cleaning and grubbing	*m3*	4100.00	133.00	545300.00
3	Topsoil excavation	*m3*	815.28	210.00	171208.80
4	Loose excavation	*m3*	34850.42	210.00	7318588.20
5	Rock excavation	*m3*	6562.07	837.00	5492452.59
6	50mm thick plain shotcrete	*m2*	869.43	1362.00	1184163.66
7	75mm thick plain shotcrete	*m2*	289.81	2043.00	592081.83
8	Rock bolts ordinary type(Dia 20mm)	*m*	724.53	1080.00	782492.40
9	Welded wore mesh(Dia 6 mm, opening of 150mm x 150mm)	*m*	289.61	1200.00	347532.00
10	Rock Anchor (Dia 20mm)	*m3*	72.45	1368.00	99111.60
11	Portal Concrete lining (C 20/25 : XC1)	*m3*	8.85	15936.00	141033.60
12	Formwork Finish Class F2, curve	*m2*	30.50	1195.00	36447.50
13	Reinforcement	*ton*	0.69	113491.00	78308.79
				Total Cost	16821920.97

S.N	**Description**	**Units**	**Qunatity**	**Rates**	**Amounts**
1	Care of water and Dewatering	lumpsum	1.00	1328.00	1328.00
2	Underground excavation of rock trap of cross section area 17.06 m2	*m3*	4100.00	133.00	545300.00
3	Underground excavation of rock trap fro abnormal overbreak	*m3*	815.28	210.00	171208.80

4	Fiber shotcrete 50mm thickness	*m3*	34850.42	210.00	7318588.20
5	Fiber shotcrete 100mm thickness	*m3*	6562.07	837.00	5492452.59
6	Rock bolts, Cement-grouted ordinary, diameter 20mm	*m2*	869.43	1362.00	1184163.66
7	Form work for lining	*m2*	289.81	2043.00	592081.83
8	Reinforcement steel	*m*	724.53	1080.00	782492.40
9	Cement mortar grouting	*m*	289.61	1200.00	347532.00
10	Consolidation grouting	*m3*	72.45	1368.00	99111.60
11	Drilling for consolidation grouting	*m3*	8.85	15936.00	141033.60
12	Anchour bars(Dia 20 mm)	*m2*	30.50	1195.00	36447.50
				Total Cost	16711740.18

S.N.	Particulars	Unit	Rate (NRs.)
	Inlet Portal		
1	Removal of usable timber	-	1328.00
2	Cleaning and grubbing	*m2*	133.00
3	Topsoil Excavation	*m3*	210.00
4	Loose Excavation	*m3*	210.00
5	Rock expansion	*m3*	837.00

	Headrace Tunnel		
1	Underground Excavation of Tunnel (min net CSA = 11.81 m2)	*m*	53100
2	Underground Excavation of Tunnel (Abnormal Outbreak)	*m3*	2390.00

	Rock Trap		
1	Underground Excavation of Rock Trap (CSA = 17.06 m2)	*m*	45
2	Underground Excavation of Rock Trap (Abnormal Outbreak)	*m3*	38.39

A.2 Relation of Area and Perimeter with the Diameter of Tunnel

Supposing the width of tunnel be D and the wall height be H.

Then,

$$Area\ of\ the\ cross-section(A) = \frac{\pi}{8} \times D^2 + D \times H \qquad \text{A.16}$$

$$Perimeter\ of\ the\ cross-section(P) = \left(\frac{\pi}{2}+1\right)\times D + 2\times H$$
$$= 2.57\times D + 2\times H \qquad \text{A.17}$$

Using Equation (A.16)

$$0 = \frac{\pi}{8}\times D^2 + D\times H - A$$
$$0 = D^2 + D\times\left(\frac{H\times 8}{\pi}\right) - A\times\left(\frac{8}{\pi}\right) \qquad \text{A.18}$$

Solving the equation (3), we get

$$D = -1.27\times H \pm 0.5\times\sqrt{6.48\times H^2 + 10.19\times A}$$
$$P = 2.57 * D + 2H$$

So,
$$P = \sqrt{-1.365H \pm 1.285\sqrt{6.48H^2 + 10.19A}}$$

For Economical Section, $\frac{dP}{dH} = 0$

Thus, solving we get,

$$\pm\sqrt{6.48H^2 + 10.19A} = 5.95H$$

Squaring both sides,

$$10.19A = 29.02H^2$$

$$\text{i.e. } A = \frac{29.02}{1.0.19}H^2$$

$$\text{or, } A = 2.85H^2$$

Replacing the value of A from equation (A.16),

$$\frac{\pi}{8}D^2 + DH = 2.85H^2$$

$$\text{Or,} \qquad H^2 - (o.351D)H - (0.138D^2) = 0$$

Which is quadratic in H. so solving for H, we get

$$H = \frac{0.351}{2}D \pm \frac{\sqrt{0.6752}}{2}D$$

Then we the values of H as,

$$H = 0.568D$$

Or,

$$H = -0.235D$$

Which is not possible for its negative value.

So, using the value of H as 0.586D we get,

$$A = 0.9786\ D^2$$

And,

$$P = 3.792D$$

APPENDIX B

Hydraulic Design

B.1 Design of Side Intake

S.N	Description	Output	References
A	**1. Provided** a. River Bed Level = 2190 m b. Q_{design} = 0.5 m^3/s c. Q_{divert} = 1.2 × 0.5 m^3/s = 0.6 m^3/s (10 % increment for flushing purpose & 10% for the losses) d. Discharge coefficient of orifice (C) = 0.6	Height of opening **= 1 m** Length of opening = **0.8 m** Provide 1 opening of dimension 0.8m X 1m	References: Baral,S.(2013) *Fundamentals of Hydropower Engineering*. Page no. 282
	2. Assumptions a. Velocity through orifice v = 0.75 m/s b. The orifice invert level is placed at 0.5m above river bed level.		
	3. Calculations: a. Cross Section Area required for Orifice (A) = $\frac{\text{Q orifice}}{v} = \frac{0.6}{0.75}$ = 0.8 m^2 Provide 1 orifice opening, hence Area of one opening = 0.8 m^2. Assume the height of orifice (H) = 1 m b. Width of orifice(B) $= \frac{A}{H} = \frac{0.8\ m2}{1\ m}$ = 0.8 m Hence adopt 0.8m width by 1m depth of intake opening. Set the bottom of orifice = 0.5 m above the river bed level. c. Depth of water at intake canal: 1.62 m at NWL (2191.70 amsl). d. Invert Level of intake canal is fixed at R.L of 2190.50 amsl. So, Canal Water Level at normal water level = 2191.38 amsl.		
	4. Check **At Normal Water Level** Q = $AC\sqrt{2g(h_r - h_c)}$ =0.8x1x 0.6$\sqrt{2x9.81x0.12}$ =0.74 m^3/sec > 0.6 m^3/sec (OK)		

B.2 Design of Trash rack

S.N.	Description	Output	Reference
B	**1. Assumptions** a. Approach velocity (v): 0.32 m/s b. Thickness of the bar (t): 16 mm c. Clear spacing between the bars (a): 100 mm d. Flow deviation (β): 120° e. Angle of inclination with horizontal (α): 75° f. Shape factor for circular bar (k): 1.8 g. Surface area coefficient (k_1): 0.55	Trash rack of size 1.04 m x 0.70 m	References: Baral,S.(2013) *Fundamentals of Hydropower Engineering*. Page no. 282
	2. Calculations a. Submerged Depth of trash rack = 1 m (height of orifice opening is 1 m) b. Submerged width = 1×0.8+0.70+0.3×2 = 2.1 m (2×width of intake opening+ gap between 2 opening+2xedge distance) c. Gross submerged area of trash rack = $\frac{1\times2.1}{sin75°}$ = 2.17 m^2. d. Percentage opening of trash rack = $\frac{100}{100+16}$ = 86.21 % e. Effective opening of trash rack = 0.8621×2.17 m^2 = 1.87 m^2.		
	3. Check Approach Velocity = $\frac{Discharge(Q)}{Effective\ opening\ area\ of\ trashrack}$ $= \frac{0.6\ m^3/s}{0.73\ m^2} = 0.82\ m/s$ (OK) (range 0.6-1.5 m/s)		
	No. of bars, $N = \frac{b_T^{provided}}{t+b} - 1 = \frac{800}{100+16} - 1 = 5.89$ $= 6$ (approx)		
	4. Head Loss in Trash rack Head loss due to friction ($h_{friction}$) $= k(^t/_a)^{4/3}\times v^2/(2g)\sin\beta$ $= 1.8\times(16/100)^{4/3}\times0.75^2/(2\times9.81)\times\sin75°$ = 0.0044 m		

B.3 Design of approach Canal

S.N	Description	Outputs	References
C	**1. Provided** a. $Q_{diverted}$= 0.55m^3/s b. Roughness coefficient (n) = 0.07 c. Free Board (F.B) = 0.5 m **2. Assumptions** a. Canal type: Concrete b. Slope of the canal as 0.03765 i.e. 1 in 28 **3. Calculation** **For Normal Water Level** For economical section, b=2y where, b=width of the canal y=water depth at the canal A = b*y = 2y^2 P = b+2y = 4y R = A/P = y/2 Now, $Q = \frac{1}{n} * R^{\frac{2}{3}} * S^{\frac{1}{2}}$ $So, y = \left(\frac{Q * n}{B * S^{\frac{1}{2}} * R^{\frac{2}{3}}}\right) = 0.5$ Hence, width = 2x0.5m = 1.0 m Height = y + freeboard = 1.0 m Take width, B = 1.0 m	Height of canal =**1.0 m** Width of canal =**1.0 m**	Baral,S.(2013) *Fundamentals of Hydropower Engineering*. Page no. 410 IS: 10430: 2000 *Criteria for design of lined canal sand Guidance for selection of type of lining.* Bureau of Indian Standard. New Delhi
	4. Check Velocity in canal (V) = Q/A = $\frac{0.55}{0.50}$ = 1.1 m/s If V < 0.8 V_c $V_c = \sqrt{9.81 * y} = \sqrt{9.81 * 0.5}$ = 2.2147 m/s Also, 0.8 V_c = 0.8 × 2.2147 = 1.77 m/s	Total Depth =**1.0 m**	

	Here, V= 1.1 m/s So, V< 0.8 V_c (Hence OK)		
	Total depth of Canal (HT) = H + free board (F.B) = 0.5+ 0.5 = 1.0 m		

B.4 Design of Settling Basin

S.N	Description	Outputs	References
D	**Design of Settling Zone** **1. Given** a. Design discharge (Q design) = 0.5 m^3/s b. Diverted flow, Q = 1.2*0.5 (10% increment for flushing and 10% for spillway) = 0.6 m^3/s Design 1 chambered basin, then discharge for 1 basin = 0..6 m^3/s		References: Baral,S.(2013) *Fundamentals of Hydropower Engineering*. Page no. 335
	2. Assumptions a. Limiting diameter of particle (d_{limit}) = 0.15 mm b. Kinematic viscosity, v = $1*10^{-6}$ m^2/s c. Specific gravity of particle, ρ_s = 2.6 d. Specific gravity of water, ρ = 1 e. Acceleration due to gravity, g = 9.81 m/s^2 f. Temperature of water, T = 20^0C g. Fall velocity, ω_t = 0.0135 m/s		

	3. Calculations a. Limiting flow velocity, V_{lf} = a*√(d) = 0.44*√(0.15) (since a=0.44 for 1mm>d>0.1mm) = 0.17 m/s b. Plain area of settling basin, $A_s = \frac{K*Q}{\omega_t}$, where K = 1.2 to 1.5 $= \frac{1.4*0.6}{0.0135} =$ 62.22 m^2 c. Calculation of length and breadth: Use L/B = 6 Or, B = L/6 Since A_s = 62.22 m^2 Or, L*B = 62.22 m^2 Or, L*L/6 = 62.22 m^2 Or, L = √62.22 *6 Thus, L = 19.2 m ≈ 19.2 m And B=19.2/6 = 3.2 m ≈ 3.2m	Length of settling basin = 19.2 m Width of settling zone = 3.2 m Total depth = 2 m	
	4. Check Checking the basin width by the following equation: B=4.75*Q^0.5 =4.75*0.6^0.5 = 3.67 m Adopt smaller dimension for easement of the flushing purpose. So, basin width, B= 3.2 m Depth of settling basin, $H=\frac{Q}{B*V_{lf}} = \frac{0.6}{3.2*0.17}$ =1.11 m ≈ 2 m **Check for length** **(i) By efficiency check from Vetter's method** $\eta = 1 - e^{\frac{-\omega A_S}{Q}} = 1 - e^{\frac{-0.0135*62.22}{0.6}} = 75.34\%$ **Check for efficiency** **(i) Vetter's method** $\eta = 1 - e^{\frac{-\omega A_S}{Q}} = 1 - e^{\frac{-0.0135*62.22}{0.6}} = 75.34\%$ **(ii) Hazen's method** $\eta = 1 - \left(1 + m\frac{\omega}{V_o}\right)^{-1/m}$ m = 0.17 for good performance	η from vetters eqn = **75.34 %** η from Hazen's eqn = **71.52 %**	

	$= 1 - \left(1 + 0.17\frac{62.22*0.0135}{0.6}\right)^{-1/0.17} = 71.52\%$		
ii	**Design of Inlet Transition** **1. Provided** a. Width of settling basin (B) = 3.2 m b. Width of approach canal (B') = 1 m c. Inlet horizontal expansion angle (α) = 11^0 d. Inlet vertical expansion angle (β) = 20^0	Length of inlet transition = **5.7 m** Length of outlet transition = **3.3 m**	
	2. Calculation a. Length of inlet transition, $L_1 = \frac{B-B'}{2tan\propto} = \frac{3.2-1}{2tan11°} = 5.7$ m b. Length of inlet adopted, L_1 = 5.7 m.		
	Design of outlet Transition a. Outlet Horizontal expansion angle(α') = 20^0 b. Outlet vertical transition angle = 45^0 c. Width of headrace canal = 0.8 m d. Length of outlet transition, $L_2 = \frac{B-B'}{2tan\propto} = \frac{3.2-0.8}{2tan20°} = 3.3$ m e. Length of outlet transition adopted = 3.3 m		
	Total length of settling basin, **L_T = 19.2+5.7+3.3 = 28.2 m**		
iii	**Design of Sediment Storage Volume** **1. Given** a. Sediment concentration, C=2 kg/s b. Density of sediment, ρ_s =2600 kg/m^3	Height of sediment storage = **0.5 m**	
	2. Assumption a. Freeboard, FB = 0.5m b. Detention time, T=6 hours=6*60*60= 21600 sec **c.** Packing factor of sediment in submergence, P_{factor}=0.5		

	3. Calculation a. Sediment load, (S_{load}) = Q*T*C = 0.6 m^3/s *21600 s *2 kg/s = 25920 kg b. Sediment volume, (S_V) = $\frac{S_{load}}{\rho_s * P_{factor}} = \frac{25920\ kg}{2600\ kg/m^3 * 0.5}$ = 19.94 m^3 c. Storage Depth, $\Upsilon_{storage} = \frac{S_V}{A_S} = \frac{19.94\ m^3}{62.22}$ = **0.3 m = 0.5 m (adopt)** d. Total depth of settling basin = FB + H+ Υ storage = 0.5+2+0.5 = 3.0 m		
iv	**Design of Flushing System** Side Slope = 40° Width of flushing (Df) = 1 m (assume) Depth of hooper (Dh) = 0.9 m Area of hooper (A) = 0.5*(B+Df) * Dh = 2.34 m^2 Take height, h = **0.5 m** and b = **0.8 m** (assume)	Flushing system = **0.5m x0.8m** Total depth of settling basin = **2 m +0.9 m + 0.5 m = 3.4 m**	
	Check **Check for Hooper section:** Storage Volume = A*Uniform length + b*h*Uniform length = 2.34*19.2 + 0.8*0.5*19.2 = 52.608 m^3 > 19.94 m^3 (sediment volume). **Hence, OK.**		

B.5 Design of Surge Tank

S.N	Description	Outputs	References
D	**1. Given** a. Acceleration due to gravity(g) = 9.81 m/s^2 b. Discharge(Q) = 11.57 m^3/s c. Length of headrace tunnel (L) = 3629 m d. Diameter of headrace tunnel (D) = 3.5 m e. Gross head(h_g) = 442 m f. velocity of flow in headrace tunnel = 1 m/s g. Total headloss = 9.20 m h. Net Head(Ho) = 432.80 m		References: Baral,S.(2013) *Fundamentals of Hydropower Engineering*. Page no. 335

i. Frictional coefficient (f) = 0.013

2. Calculation

a. Area of headrace pipe (A_t) = $\pi * D^2 / 4$ = 11.99 m^2

b. Area of Surge Tank (A_{st}) = $45D_t^{10/3}/H_o$ = 6.77 m^2

c. Diameter of surge tank (D_{st}) = $(4 \times Ast/\pi)^{0.5}$ = 2.94 m

d. Actual area of surge tank = 13.54m^2

e. Actual diameter of surge tank = 4.15 m

Using thoma formula,

Area of surge tank required = $A_t \times L \times vt^2 / (2gh_f (H_g - h_f))$ = 0.56 m^2

f. Maximum upsurge level in case of instantaneous rejection of full load(Z_{max}) = $Q/Ast \times (L \times Ast/(g \times At))^{1/2}$ = 17.47 m

g. Headloss correction factor(Po) = h_f/z_{max} = 0.53

h. Maximum upsurge upon sudden 100% load rejection ($z_{max,up}$) = z_{max} $(1 - 2/3 \times P_o + 1/9 \times P^2)$ = 11.87 m

Dst	Ast	Zmax	Po	Zmax,up	Zmax,do	Hst
3	7.07	24.18	0.38	18.43	(5.78)	24.21
3.5	9.62	20.72	0.44	15.04	(2.32)	17.37
4	12.56	18.13	0.51	12.52	0.27	12.25
4.5	15.90	16.12	0.57	10.57	2.28	8.29
5	19.63	14.51	0.63	9.02	3.89	5.13
5.5	23.75	13.19	0.70	7.77	5.21	2.56
6	28.26	12.09	0.76	6.73	6.31	0.42
6.5	33.17	11.16	0.82	5.87	7.24	(1.37)
7	38.47	10.36	0.89	5.14	8.04	(2.90)
7.5	44.16	9.67	0.95	4.51	8.73	(4.22)
8	50.24	9.07	1.01	3.97	9.33	(5.36)
11	94.99	6.59	1.40	1.89	11.81	(9.92)

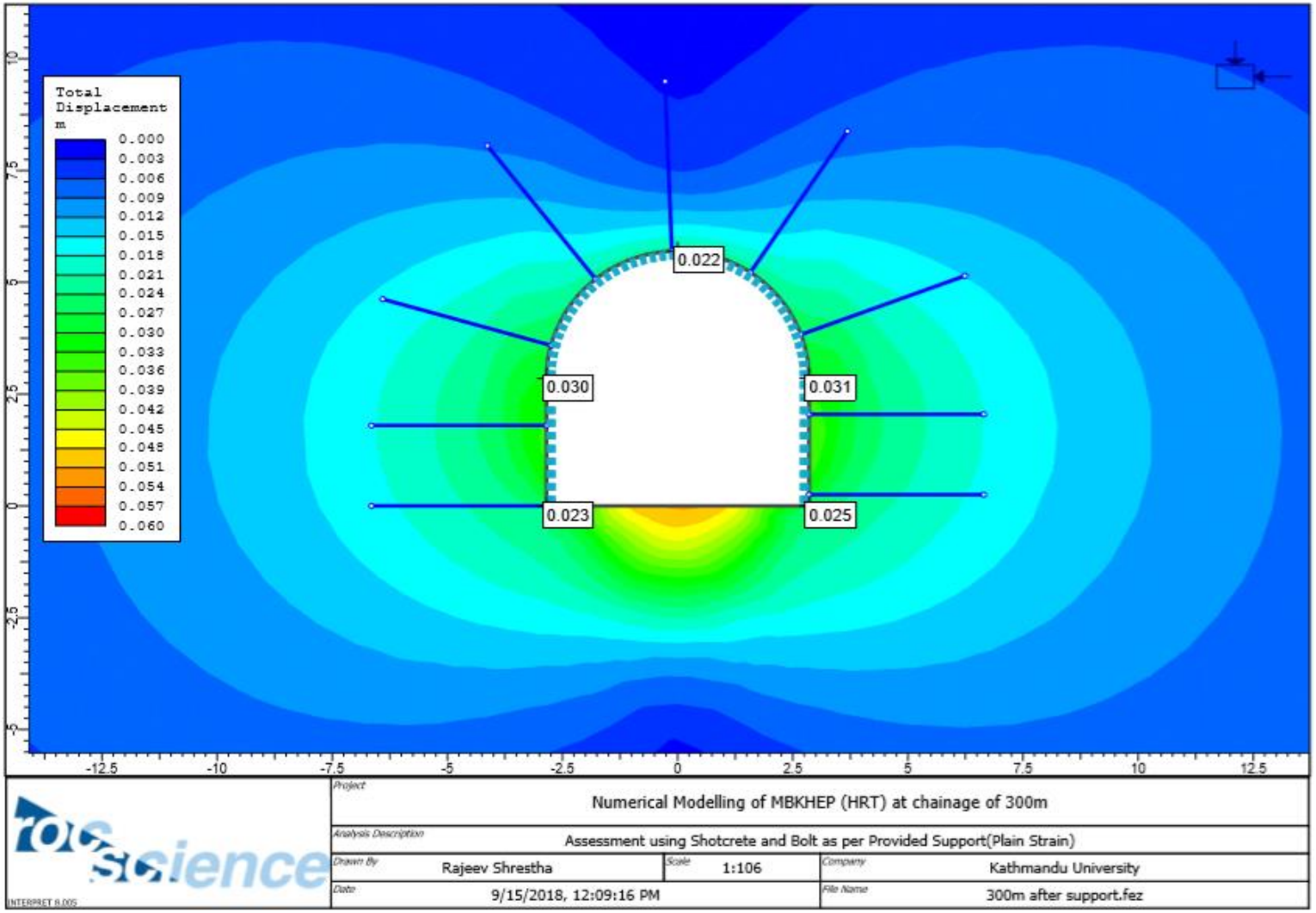
From Iteration Table

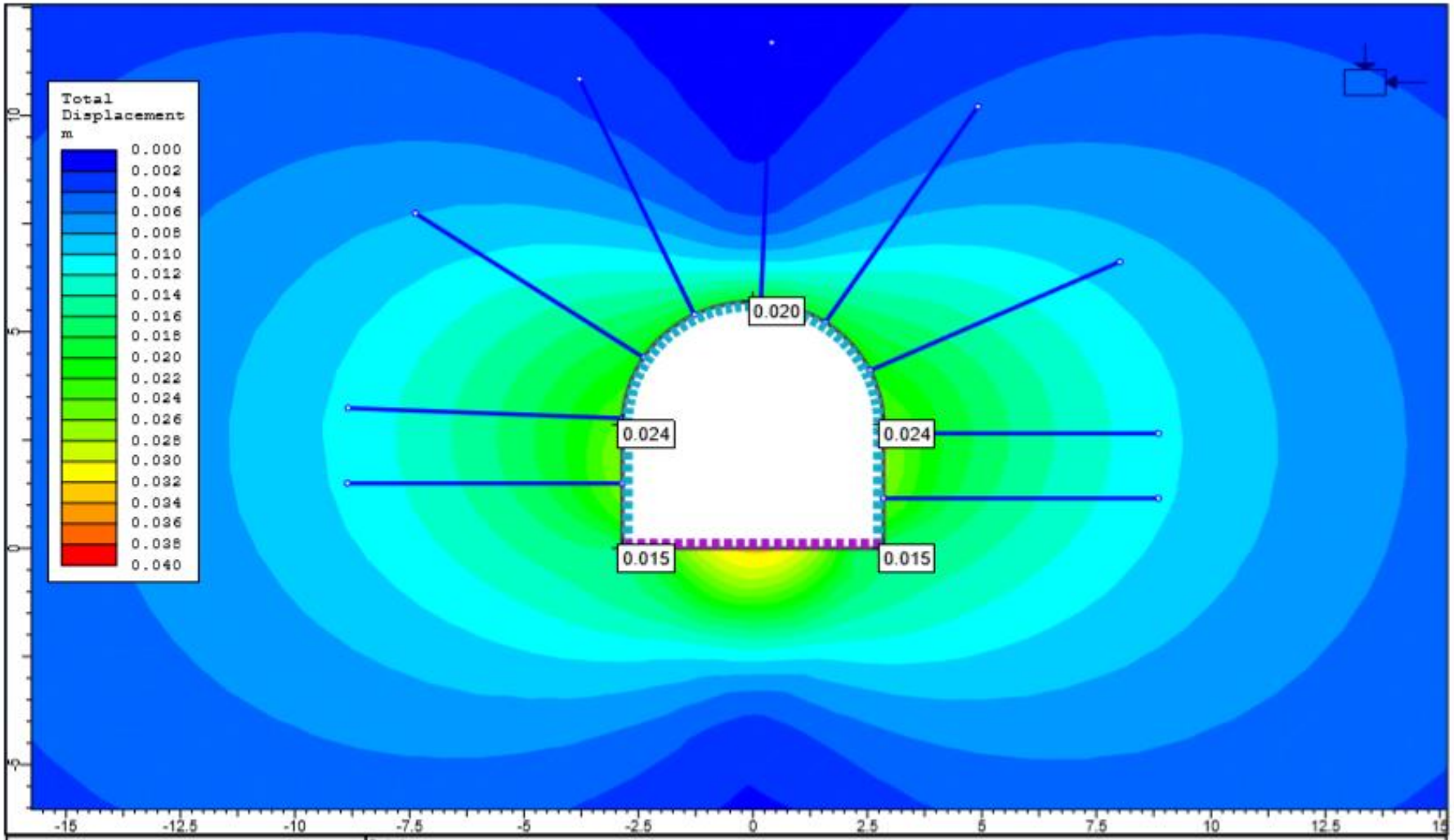
Upsurge Head from static Level = 10.57 m Downsurge Head from static Level = 2.28 m Normal level at Intake = 2191.7 m Head loss correction = 2182.50 m Crown level of HRP = 2,181.78 m Diameter of penstock (Dp) = 3.2 m Area of penstock (Ap) = 8.04 m^2 Velocity of flow at penstock (v_p) = Q/Ap = 1.44 m/s Minimum submergence head 1(h_{sub1}) = $1.5v_p^2/2g$ = 0.158387078 m Minimum submergence head 2(h_{sub1}) = $1.5D_p$ = 4.8 m		

APPENDIX C

Numerical Modelling

C 1: Numerical Analysis Result of Middle Bhotekoshi Hydroelectric Project [Ch 0+300m]





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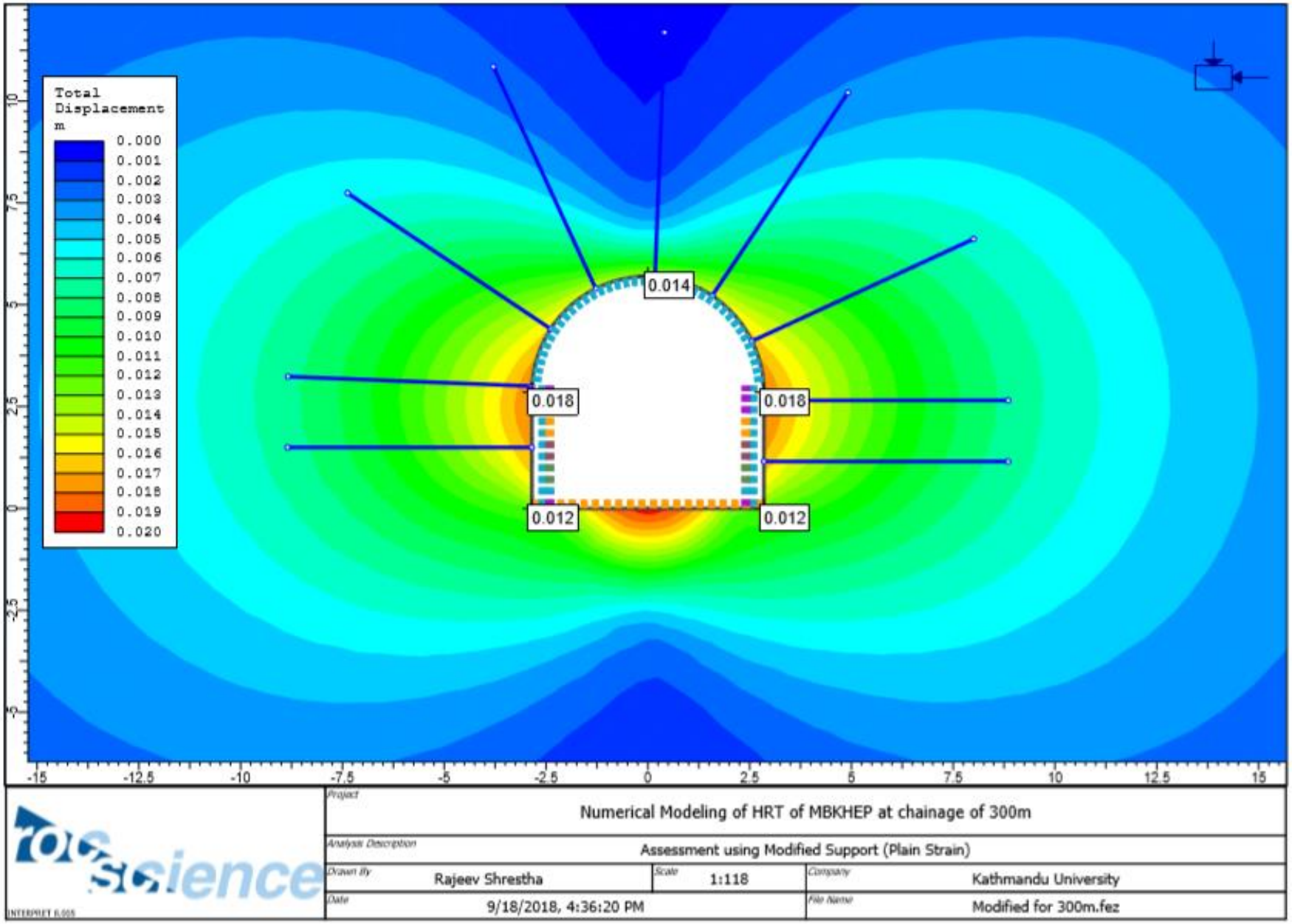
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Analysis Description
Assessment using Modified Support (Plain Strain)
Drawn By
Rajeev Shrestha
Scale
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Company
Kathmandu University
Date
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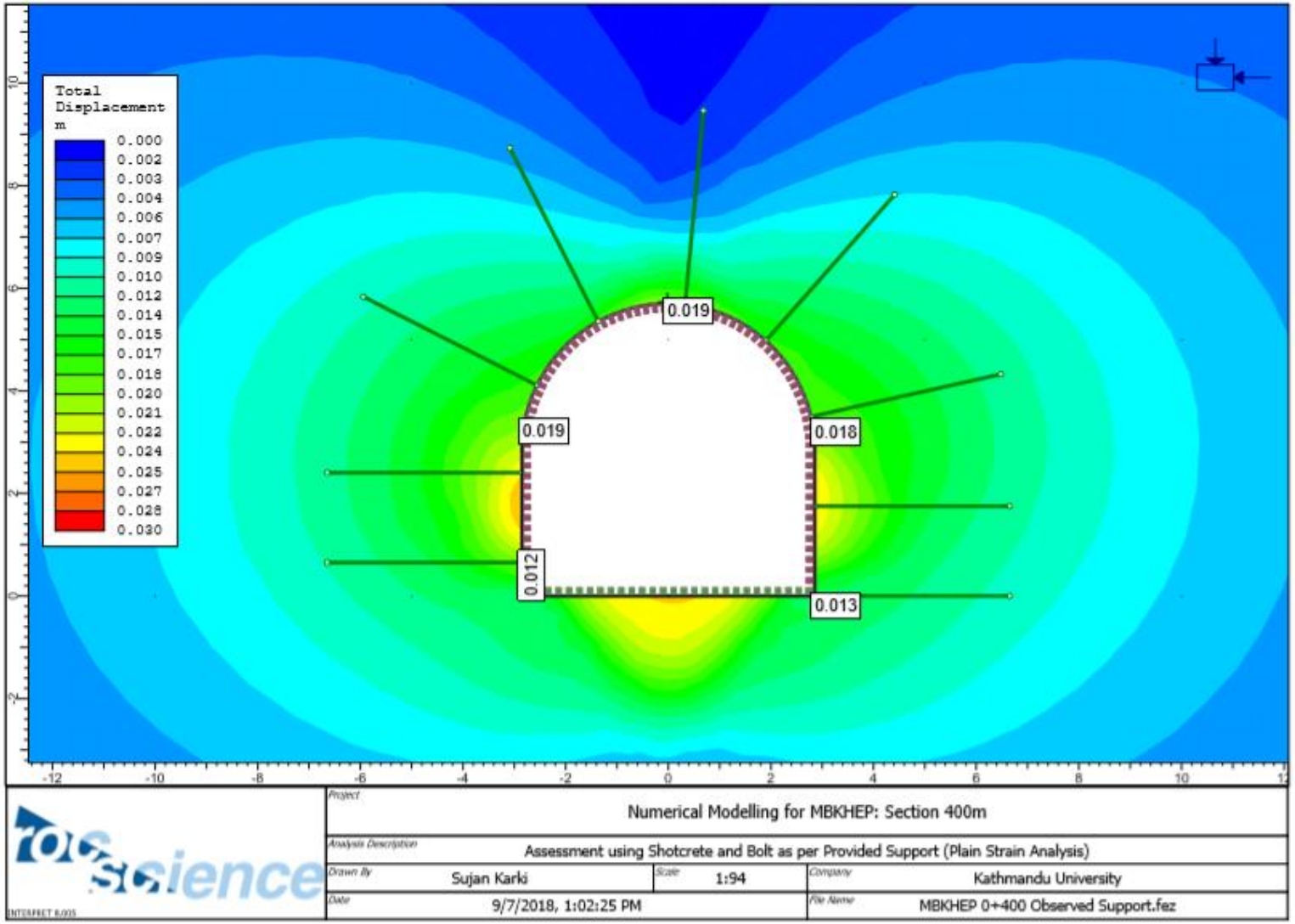
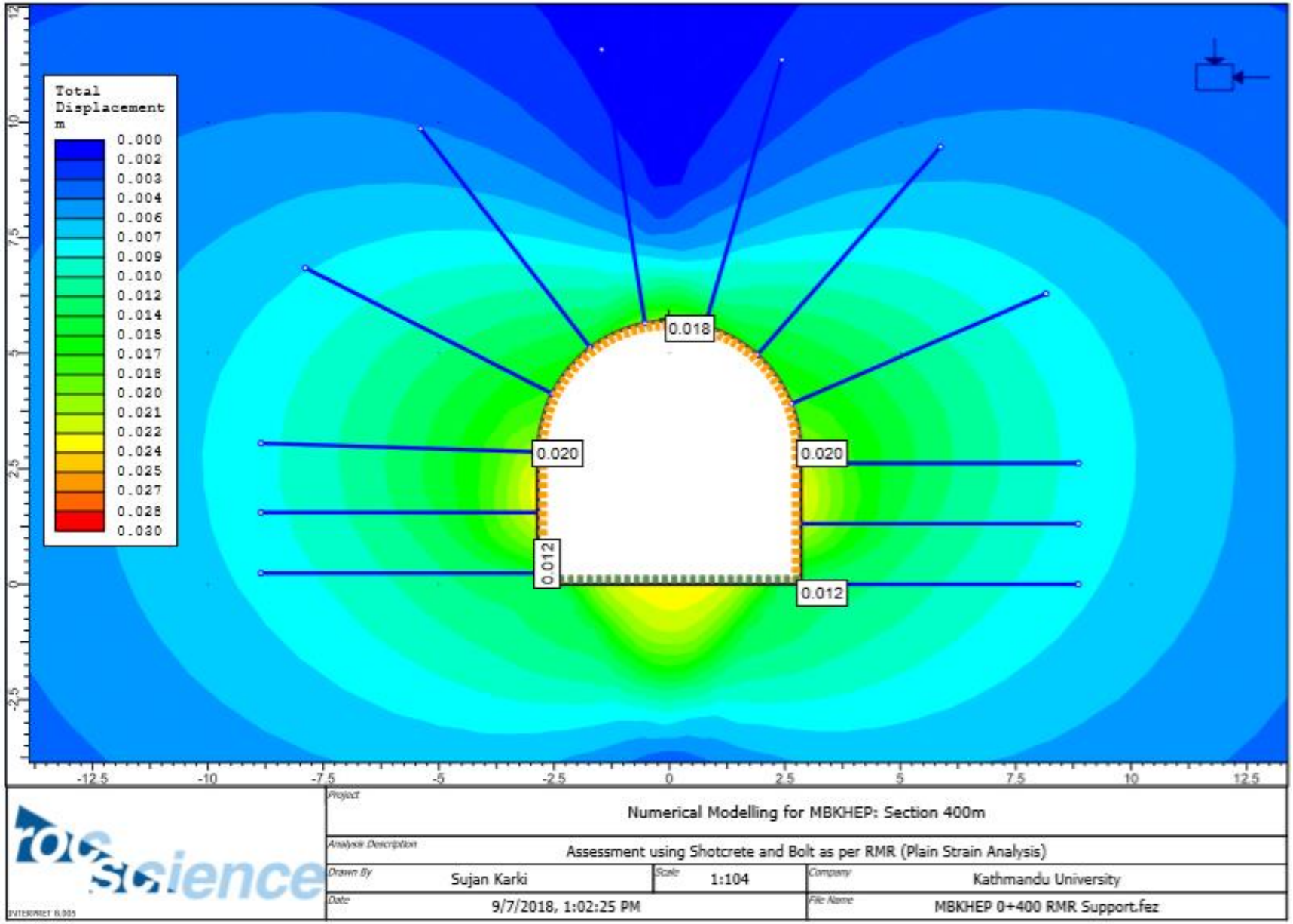
INTERPRET 6.005

C 2: Numerical Analysis Result of Middle Bhotekoshi Hydroelectric Project [Ch 0+400m]





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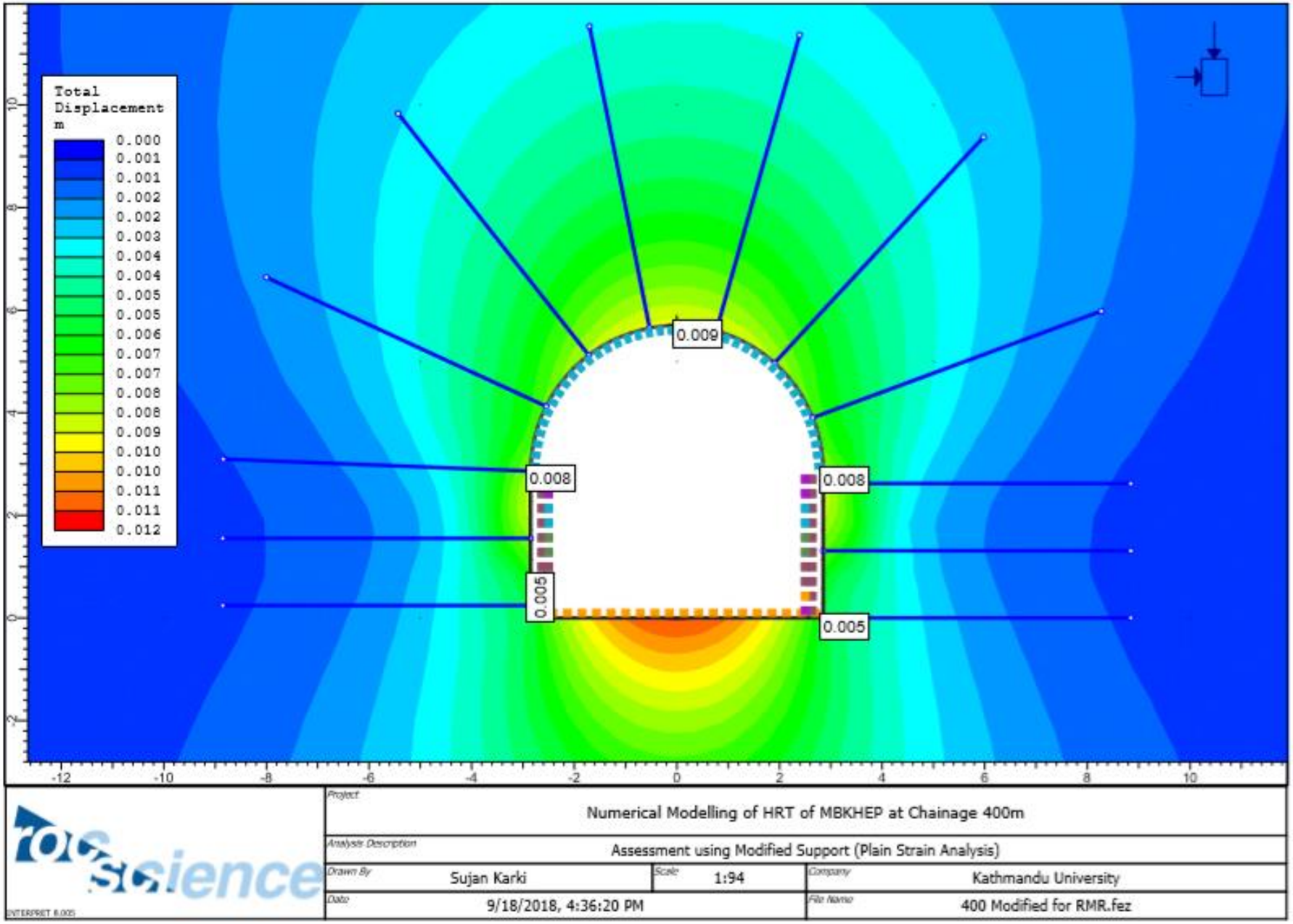
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rocscience
INTERPRET 8.005
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Numerical Modelling of HRT of MBKHEP at Chainage 400m
Analysis Description
Assessment using Modified Support (Plain Strain Analysis)
Drawn By
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Scale
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Company
Kathmandu University
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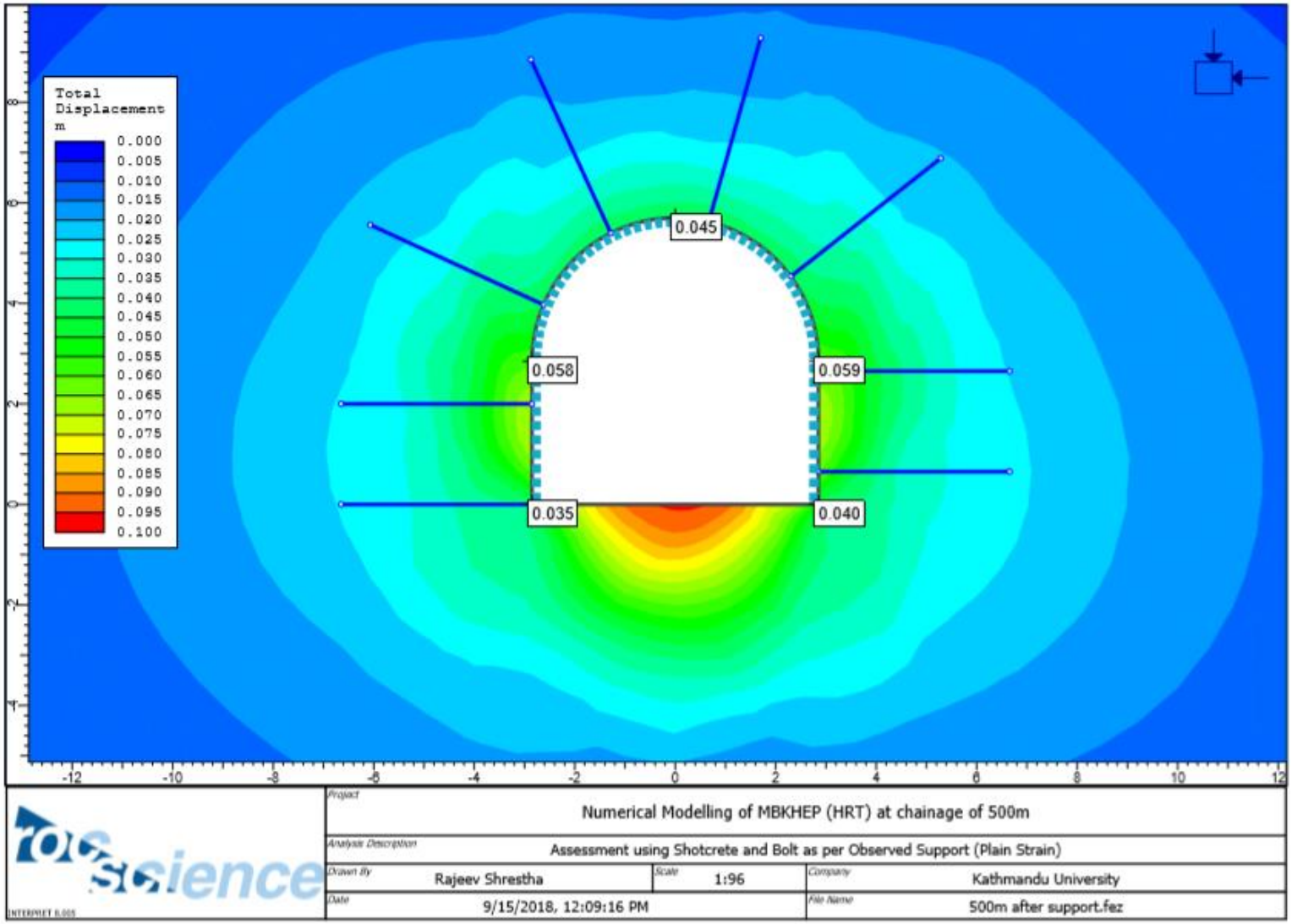

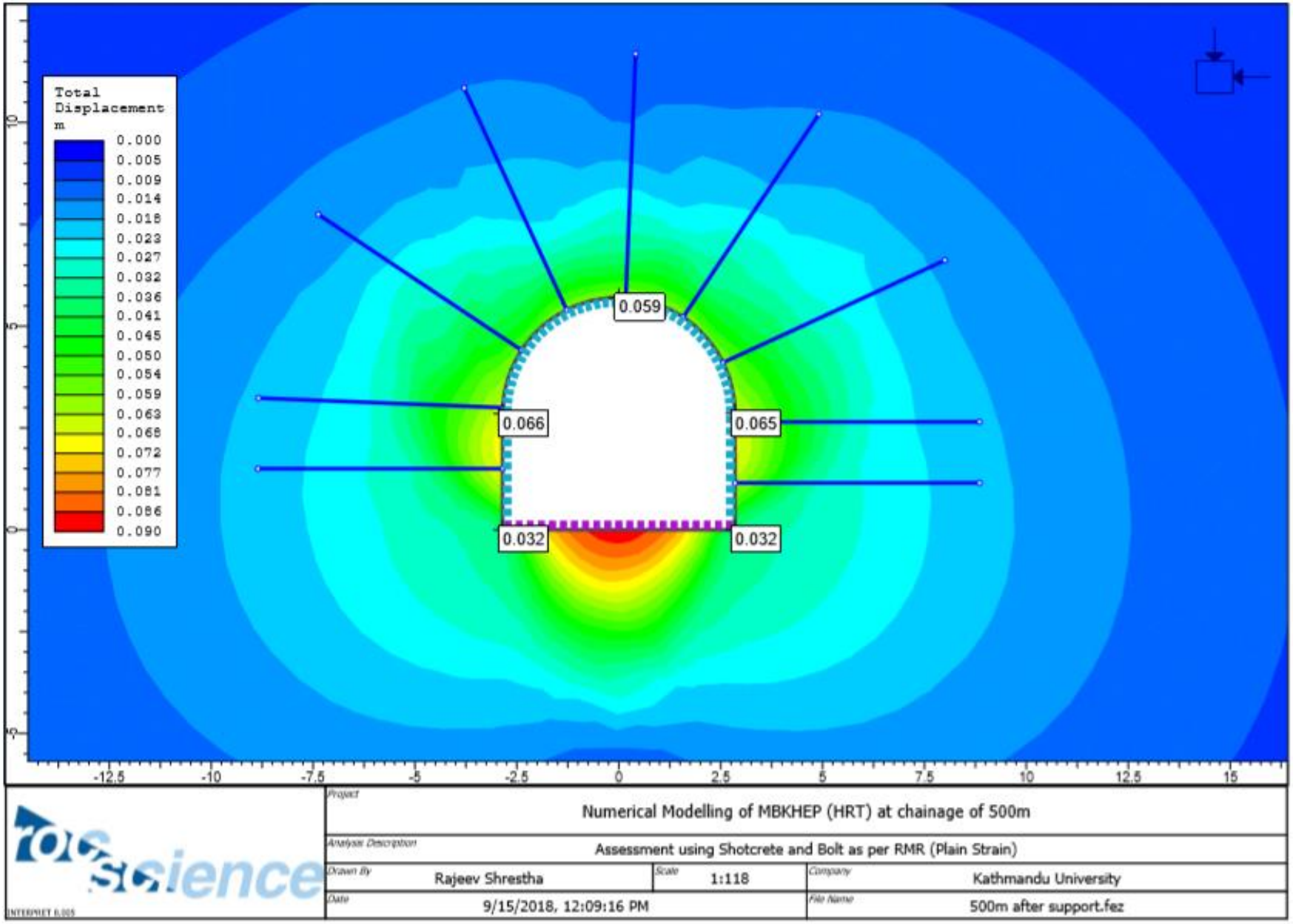
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Analysis Description
Assessment using Shotcrete and Bolt as per RMR (Plain Strain)
Drawn By
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Company
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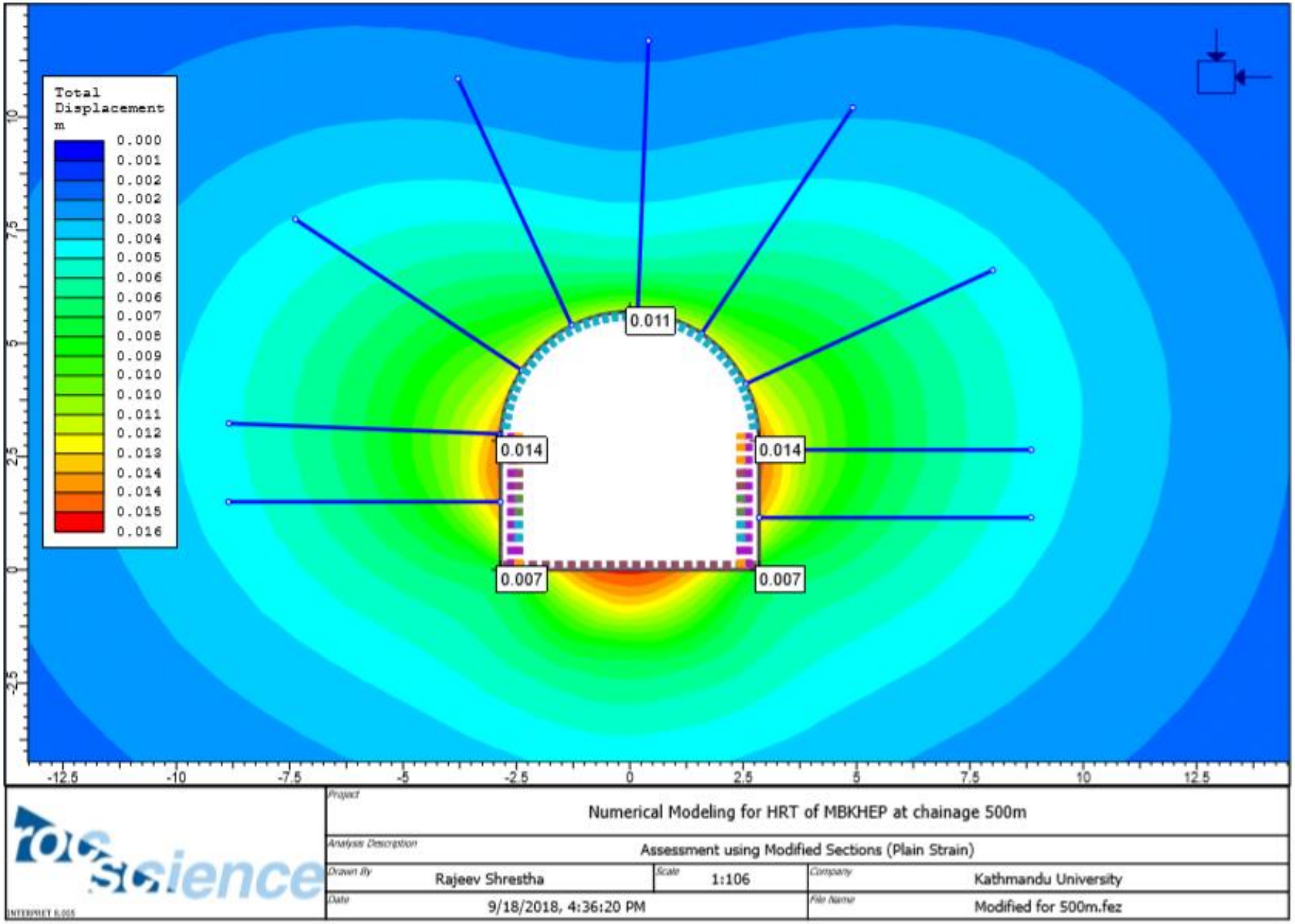
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Analysis Description
Assessment using Modified Sections (Plain Strain)
Drawn By
Rajeev Shrestha
Scale
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Company
Kathmandu University
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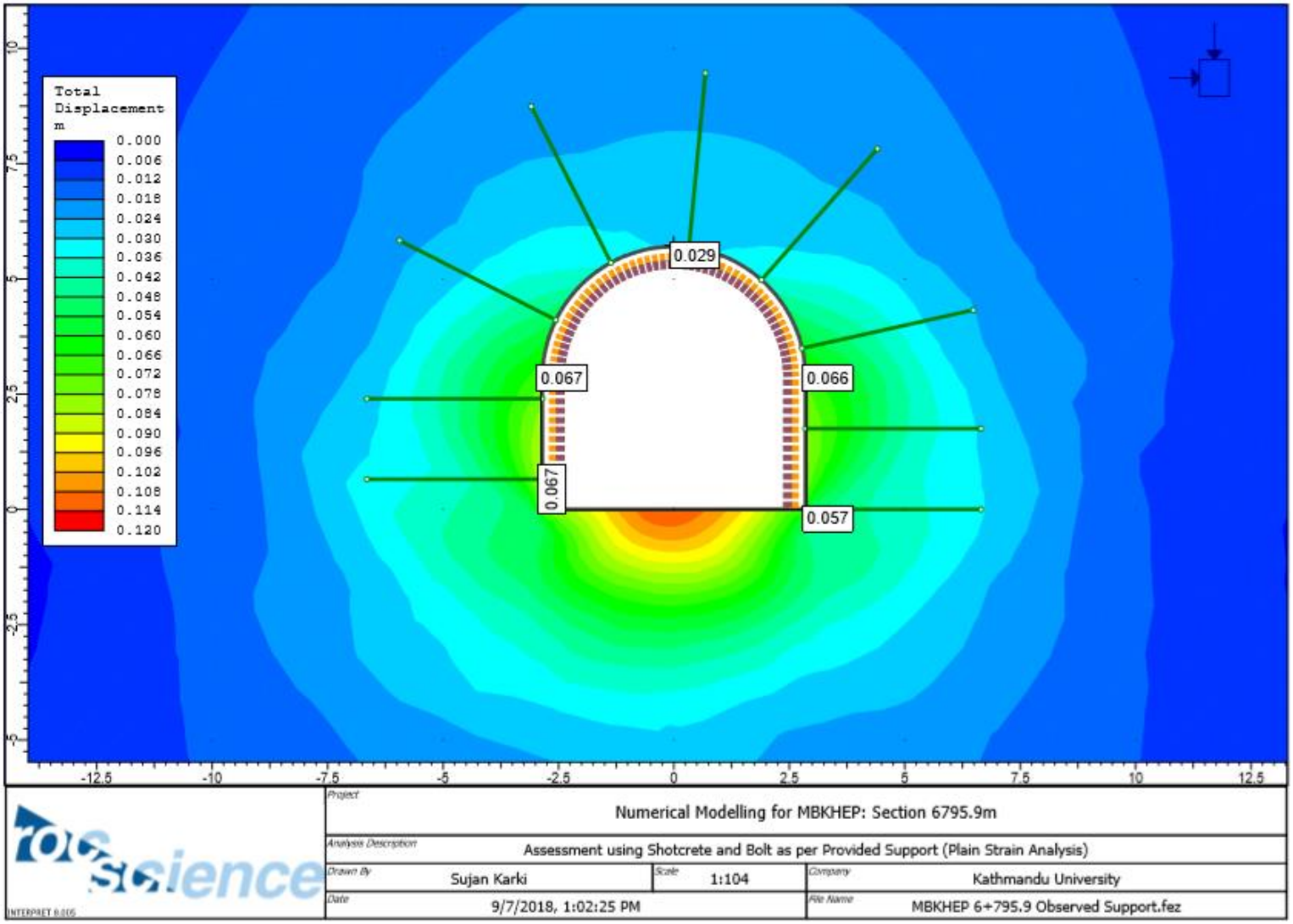
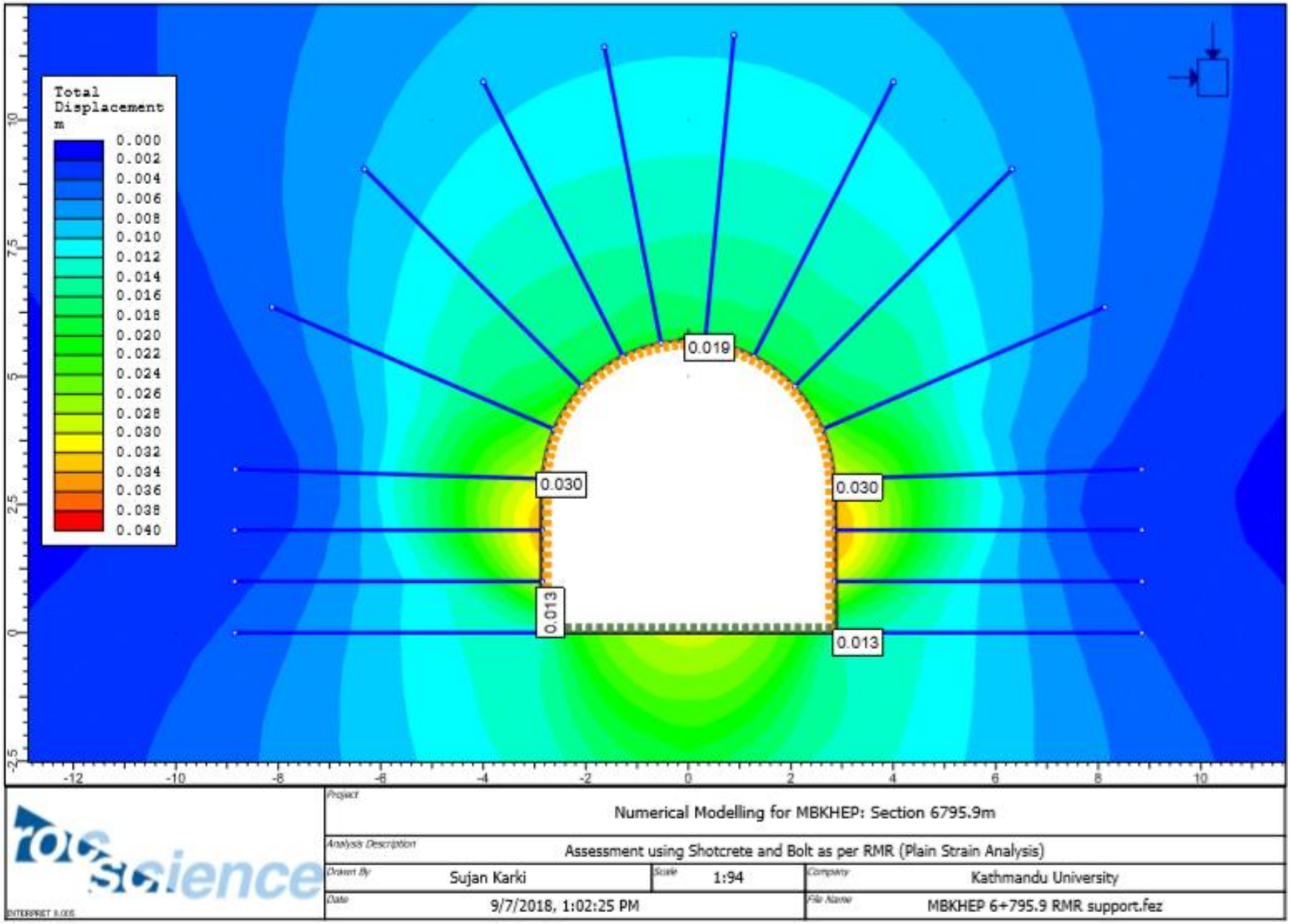
C 4: Numerical Analysis Result of Middle Bhotekoshi Hydroelectric Project [Ch 6+795.5m]

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rocscience
INTERPRET 8.005
Project
Numerical Modelling for MBKHEP: Section 6795.9m
Analysis Description
Assessment using Shotcrete and Bolt as per RMR (Plain Strain Analysis)
Drawn By
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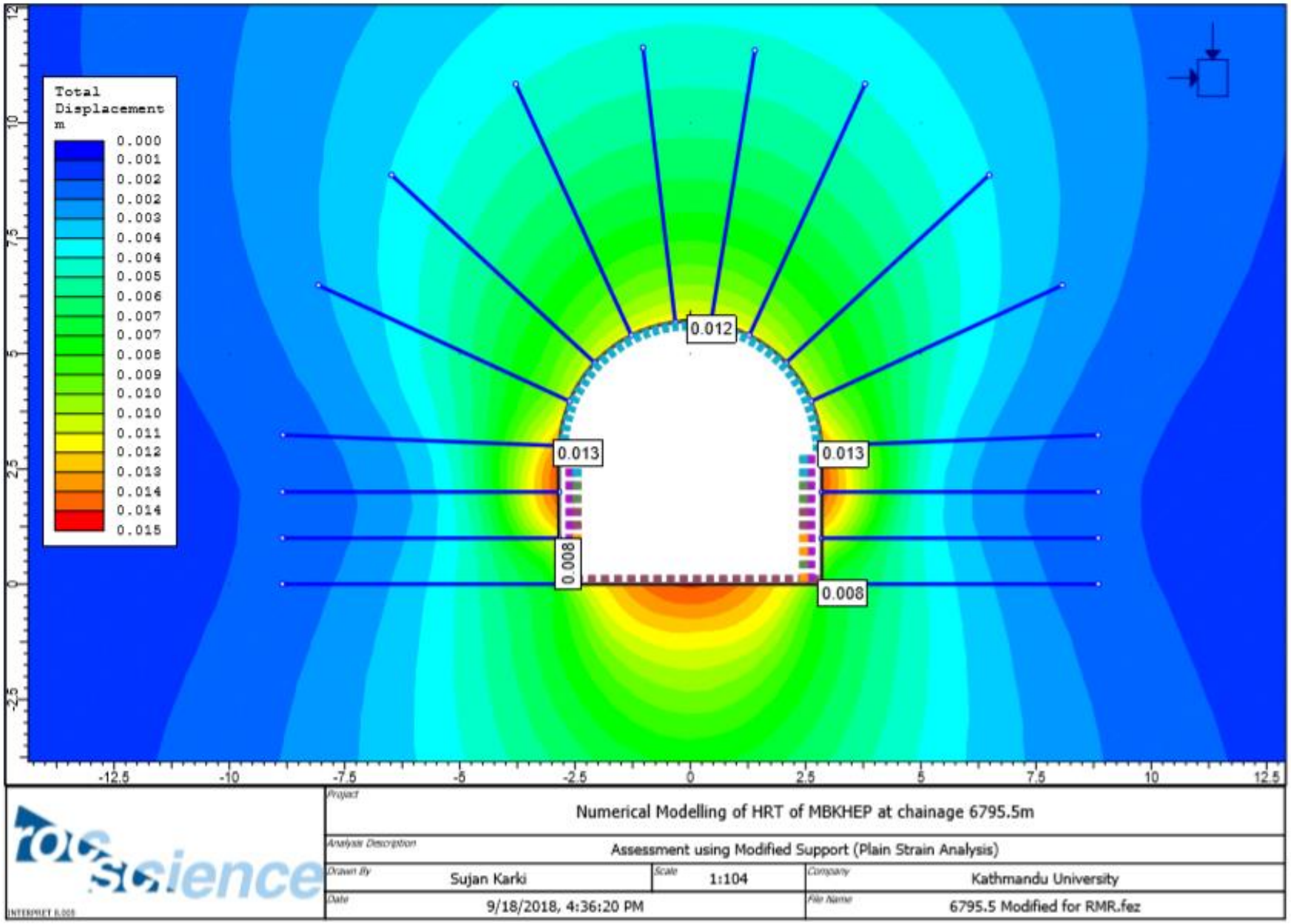

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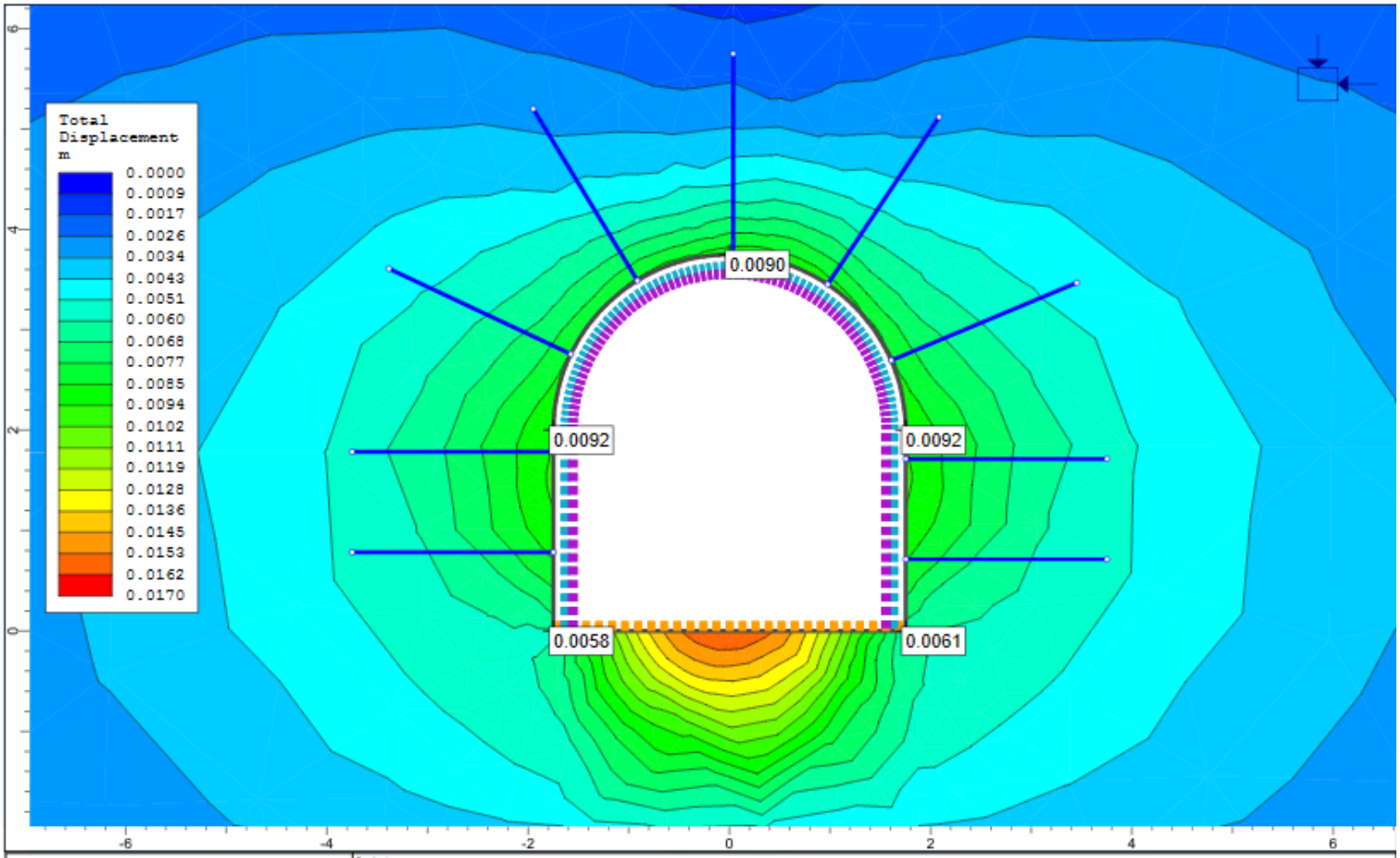

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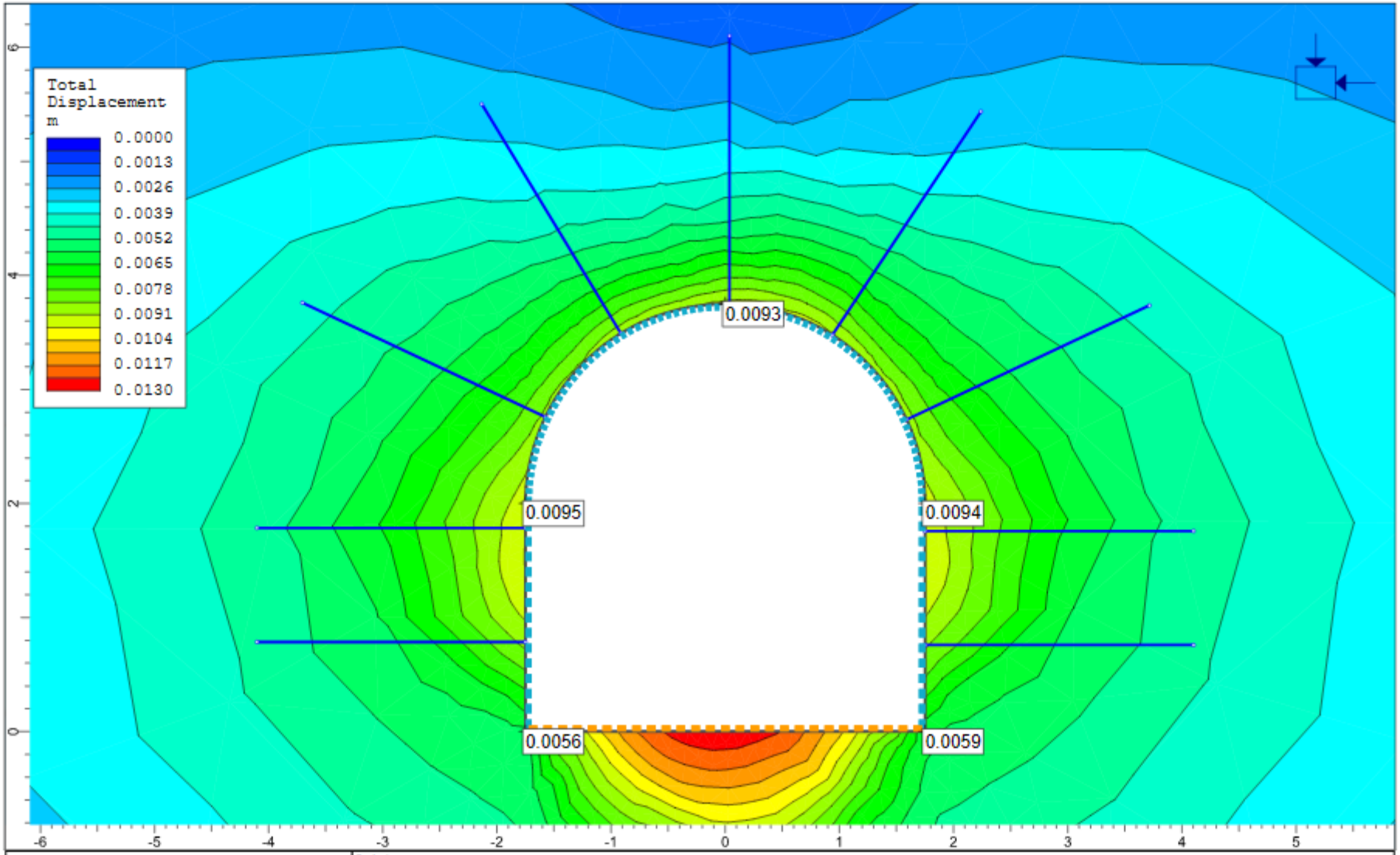
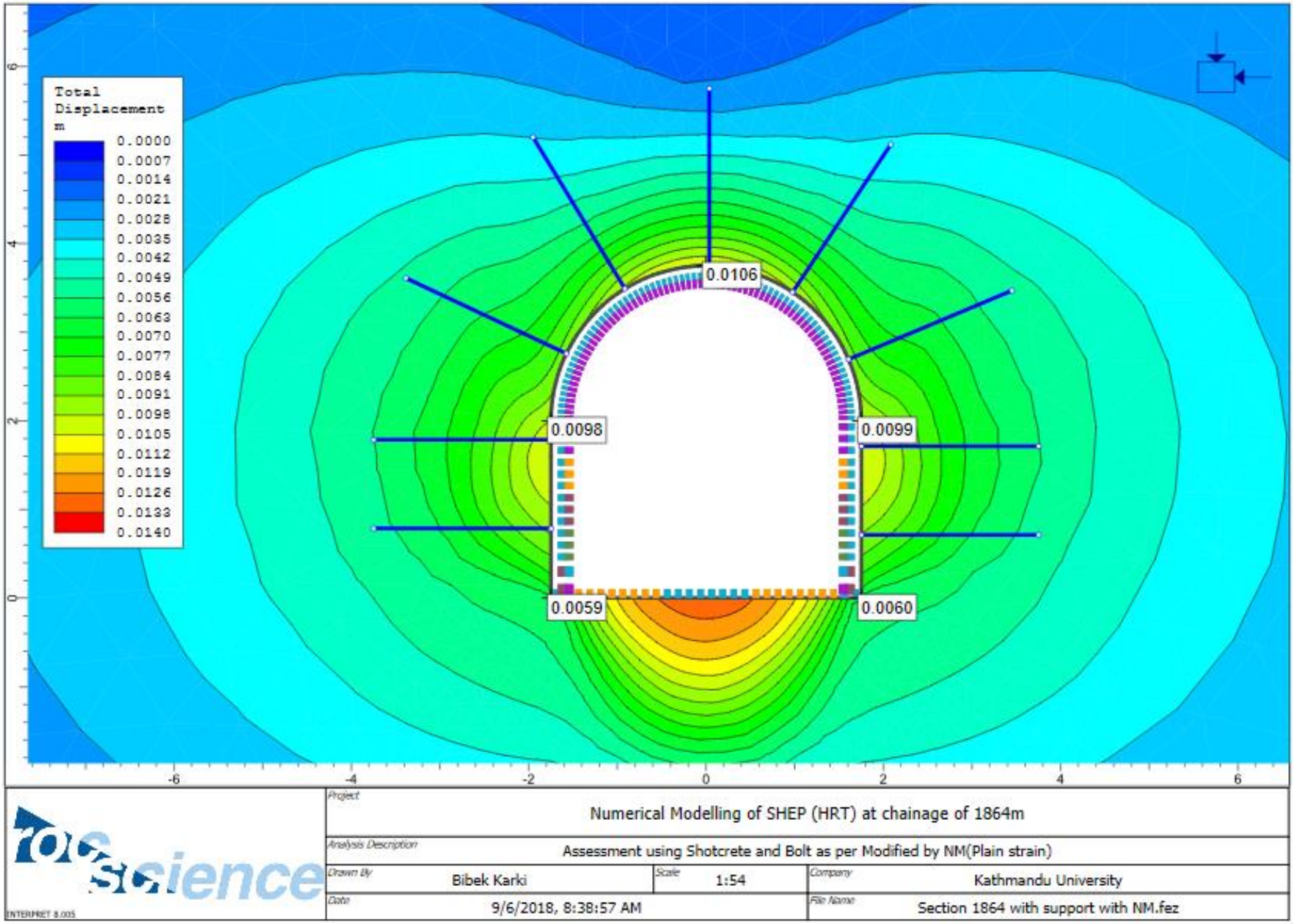
Date: 9/6/2018, 8:38:57 AM		File Name: Section 1864 with support(new).fez	

ENTERPRET 8.005

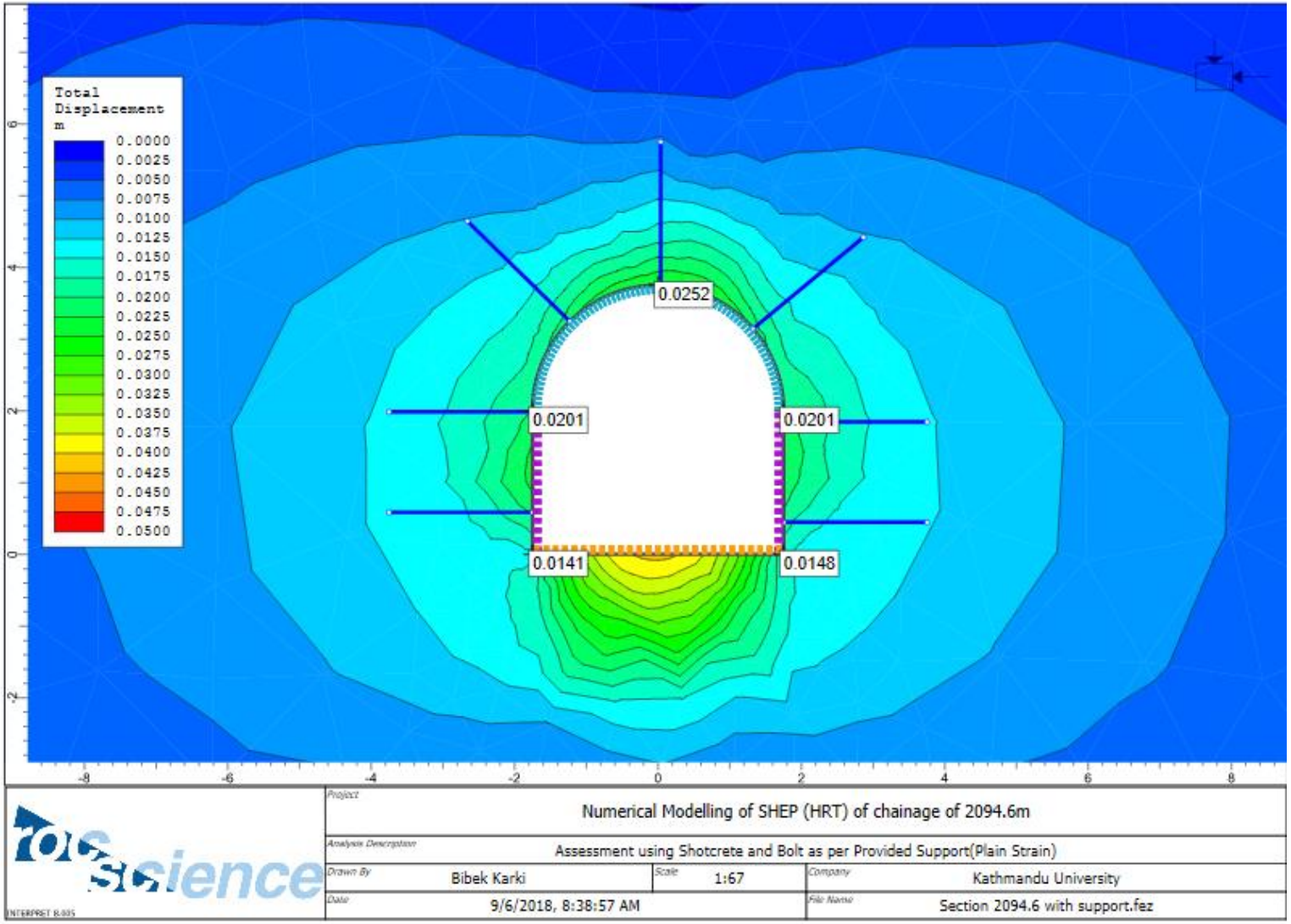


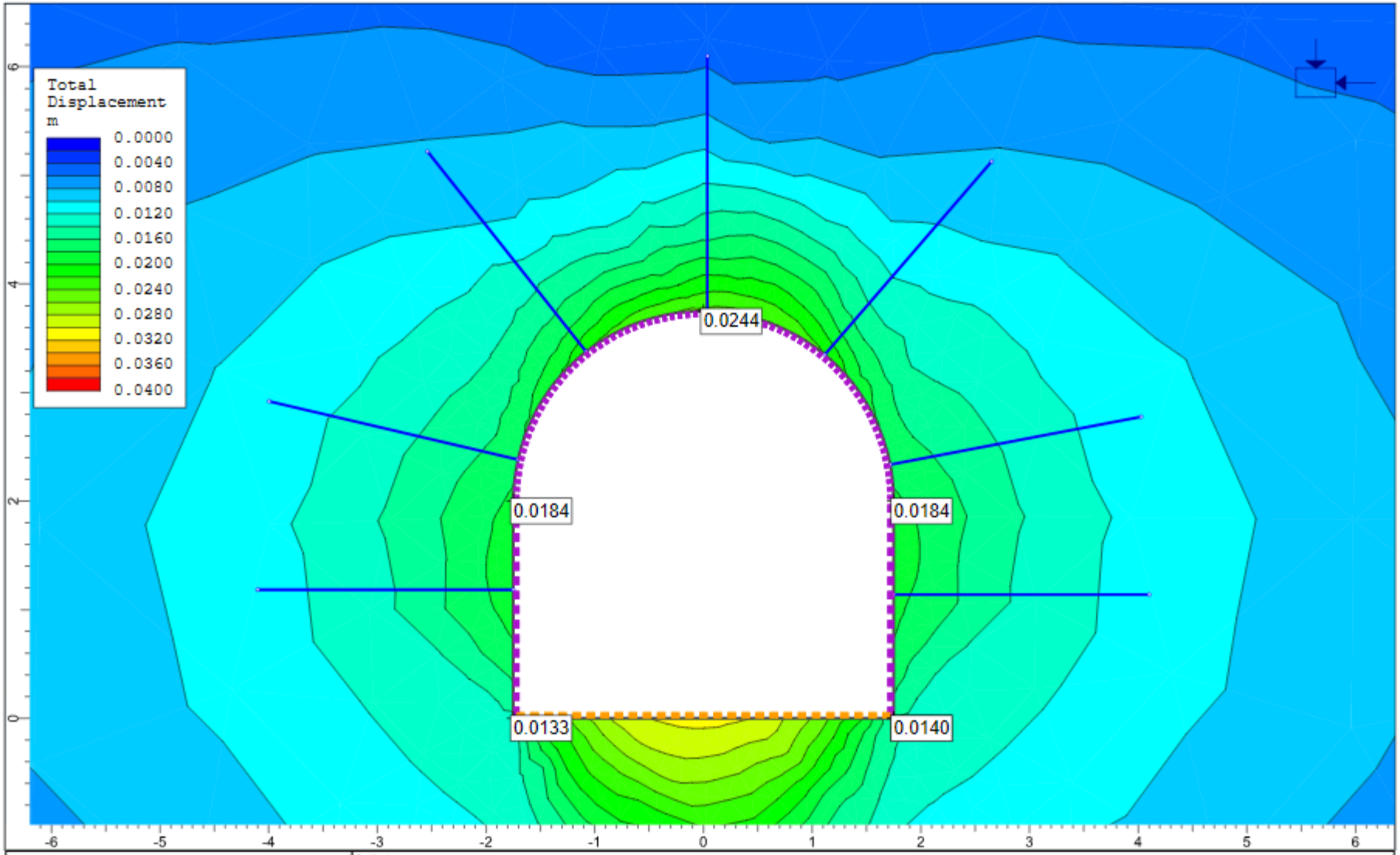
rocscience	Project	Numerical Modelling of SHEP (HRT) at chainage of 1864m		
	Analysis Description	Assessment using Shotcrete and Bolt as per Q chart (Plain Strain)		
	Drawn By: Prajwol Bhattarai	Scale: 1:46	Company: Kathmandu University	
INTERPRET 8.005	Date: 9/6/2018, 8:38:57 AM		File Name: Section 1864 with support.fez	



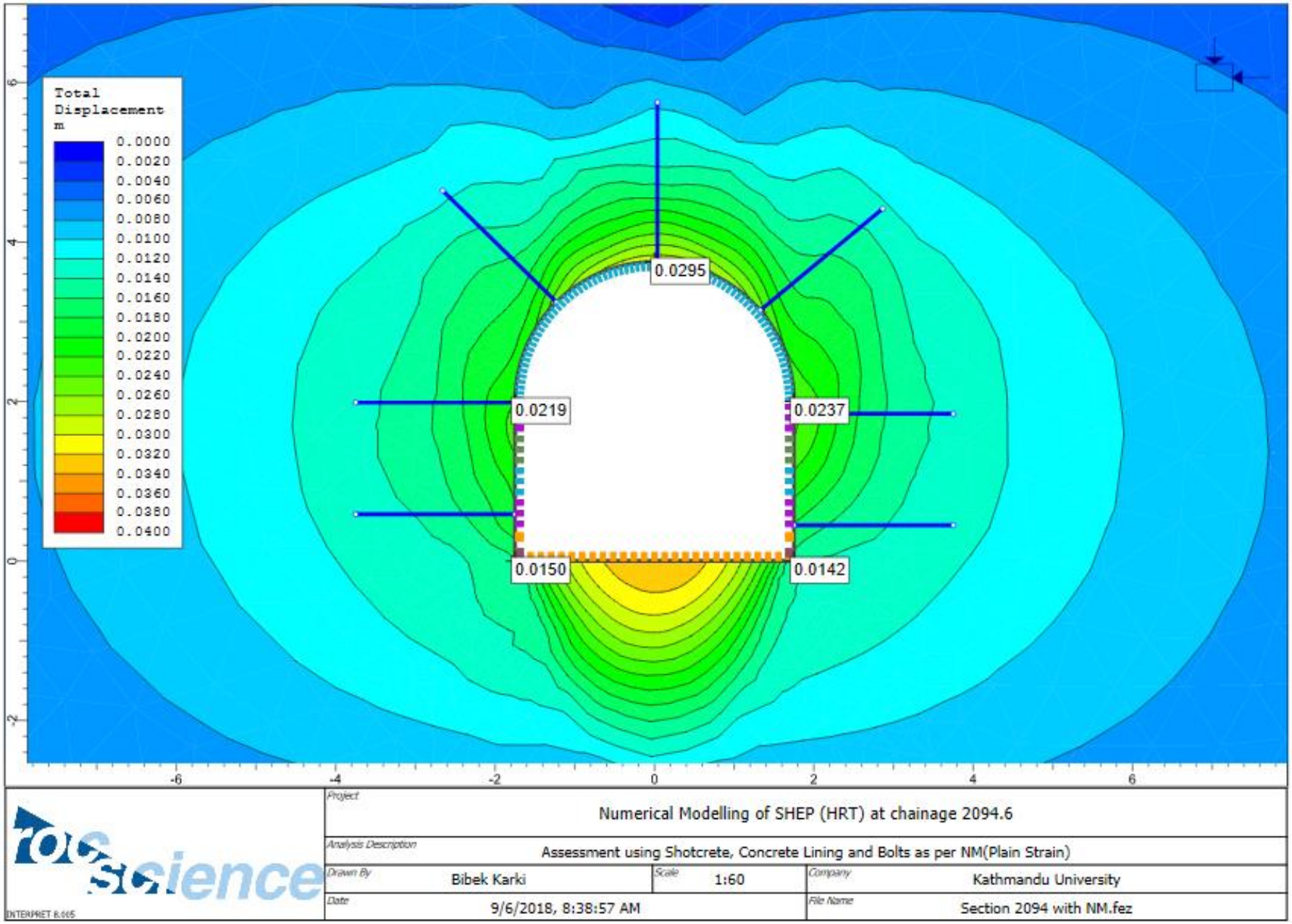
rocscience	Project: Numerical Modelling of SHEP (HRT) at chainage of 1864m		
	Analysis Description: Assessment using Shotcrete and Bolt as per Modified by NM(Plain strain)		
	Drawn By: Bibek Karki	Scale: 1:54	Company: Kathmandu University
INTERPRET 8.005	Date: 9/6/2018, 8:38:57 AM		File Name: Section 1864 with support with NM.fez

C 6: Numerical Analysis Result of Sanjen Hydroelectric Project [Ch 2+094.6m]

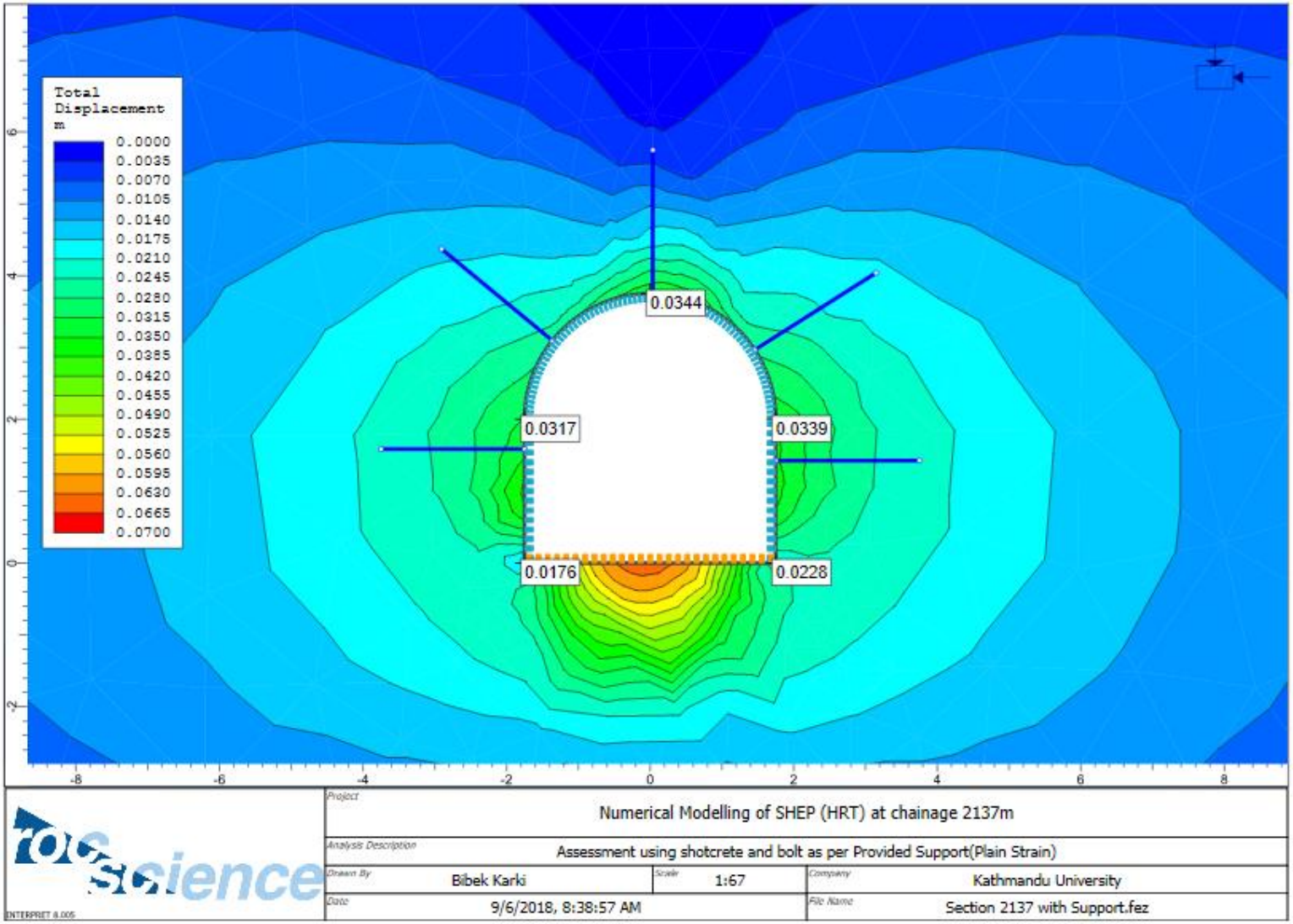


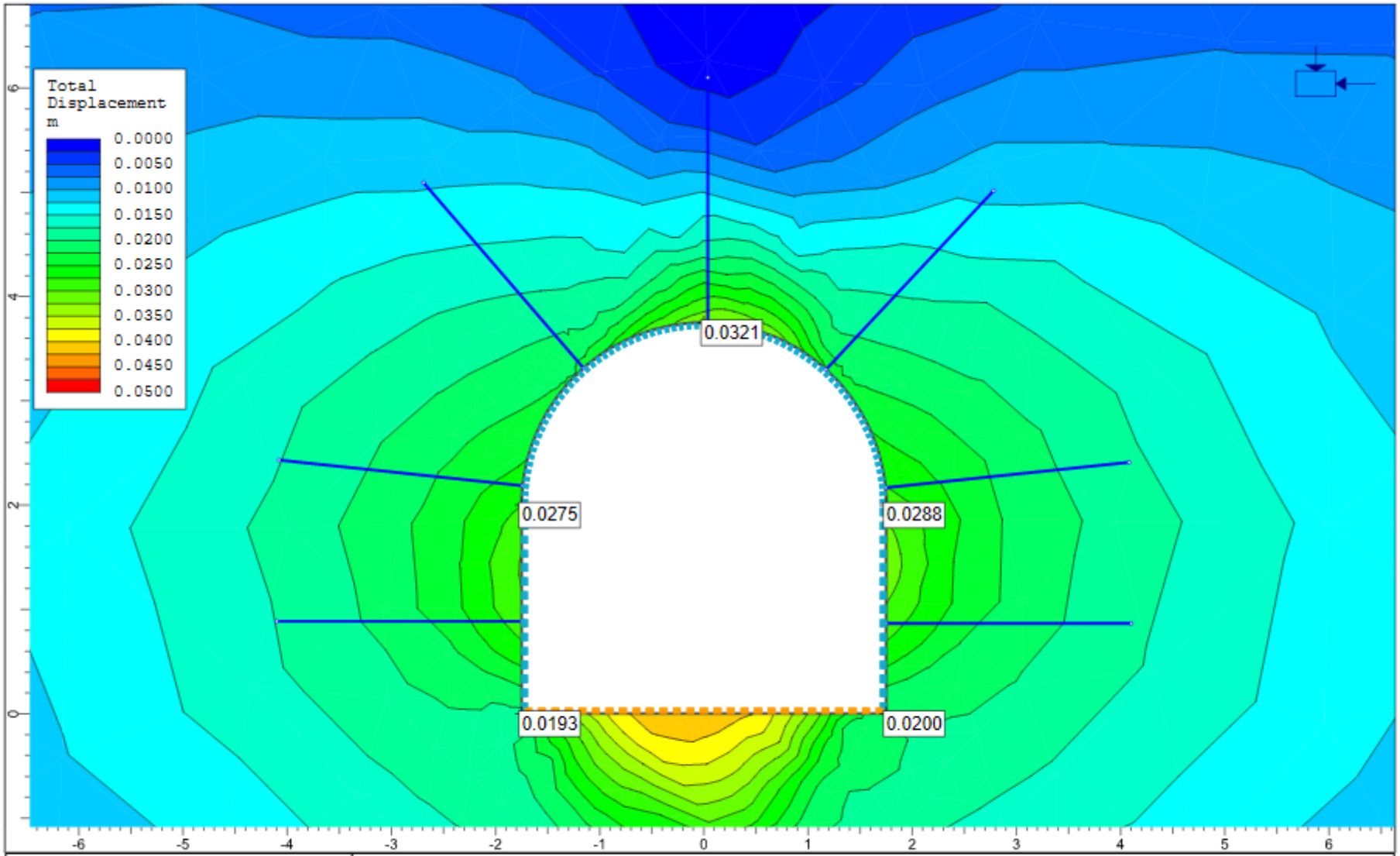


rocscience	Project	Numerical Modelling of SHEP (HRT) at chainage of 2094.6m		
	Analysis Description	Assessment using Shotcrete and Bolt as per Q Chart (Plain Strain)		
	Drawn By: Prajwol Bhattarai	Scale: 1:48	Company: Kathmandu University	
INTERPRET 8.005	Date: 9/6/2018, 8:38:57 AM		File Name: Section 2094.6 with support.fez	

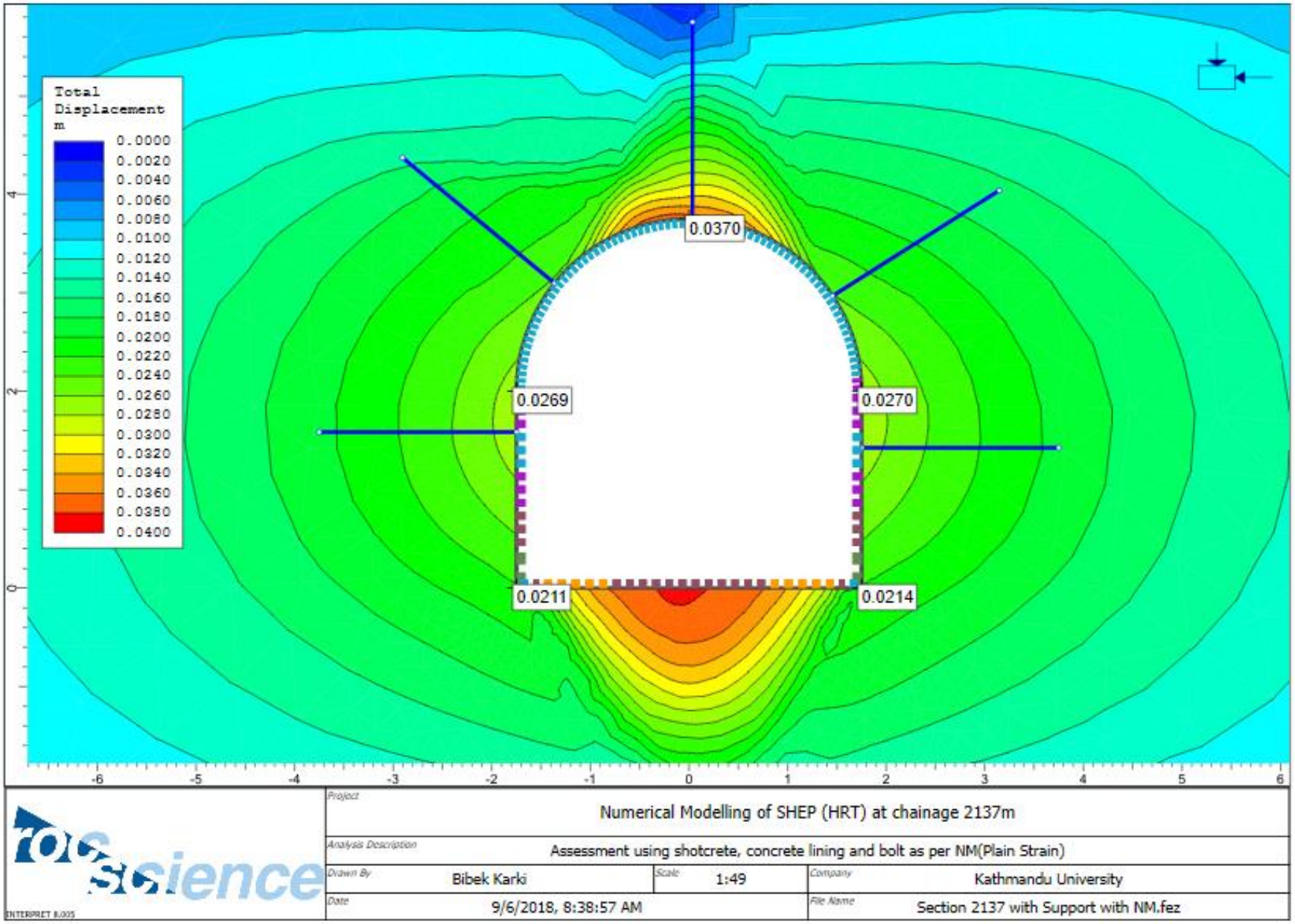

Total Displacement
m
0.0000
0.0020
0.0040
0.0060
0.0080
0.0100
0.0120
0.0140
0.0160
0.0180
0.0200
0.0220
0.0240
0.0260
0.0280
0.0300
0.0320
0.0340
0.0360
0.0380
0.0400
0.0295
0.0219
0.0237
0.0150
0.0142
-6
-4
-2
0
2
4
6
Project
Numerical Modelling of SHEP (HRT) at chainage 2094.6
Analysis Description
Assessment using Shotcrete, Concrete Lining and Bolts as per NM(Plain Strain)
Drawn By
Bibek Karki
Scale
1:60
Company
Kathmandu University
Date
9/6/2018, 8:38:57 AM
File Name
Section 2094 with NM.fez
INTERPRET 8.005

C 7: Numerical Analysis Result of Sanjen Hydroelectric Project [Ch 2+137m]





rocscience INTERPRET 8.005	Project	Numerical Modelling of SHEP (HRT) at chainage of 2137m	
	Analysis Description	Assessment using Shotcrete and Bolt as per Q chart (Plain Strain)	
	Drawn By: Prajwol Bhattarai	Scale: 1:50	Company: Kathmandu University
	Date: 9/6/2018, 8:38:57 AM		File Name: Section 2137 with Support.fez



rocscience	Project	Numerical Modelling of SHEP (HRT) at chainage 2137m	
	Analysis Description	Assessment using shotcrete, concrete lining and bolt as per NM(Plain Strain)	
	Drawn By: Bibek Karki	Scale: 1:49	Company: Kathmandu University
INTERPRET 8.005	Date: 9/6/2018, 8:38:57 AM		File Name: Section 2137 with Support with NM.fez