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"Assessment of Tunnel Stability and Evaluation of Rock Support in Headrace Tunnel of Super Madi Hydroelectric Project, Nepal"

by

Dev Raj Joshi

A THESIS

SUBMITTED TO THE DEPARTMENT OF CIVIL ENGINEERING

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The undersigned certify that they have read, and recommended to the Institute of Engineering for acceptance, a thesis entitled **"Assessment of Tunnel Stability and Evaluation of Rock Support in Headrace Tunnel of Super Madi Hydroelectric Project, Nepal"** submitted by Mr. Dev Raj Joshi (074/MSHpE/007) in partial fulfillment of the requirements for the degree of Master of Science in Hydropower Engineering.

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ABSTRACT

Underground structure is flexible construction alternative among various other construction. The presence of weathered rocks, difficult slopes and occurrence of frequent hazards in the surface has encouraged the investors and technical personals to work underground for completion of different development related works. Construction of a tunnel, changes in-situ condition and sometime leads to failure if not properly evaluated ground condition. In the most of the case, the risk involved and hazards that may occur is high. Therefore, providing support for tunnel stabilization is important before any failure. Using actual project information of Super Madi Hydroelectric Project the methods to provide tunnel support is discussed in this thesis. Typical site data formed input for the geotechnical engineering design of the tunnel support based on empirical, analytical and finite element modeling. The outcomes of the different approach in the study were unique function of their underlying scientific values.

The rock mass was classified using Q value within 400m of headrace tunnel was studied in this research. The Q value varied from minimum of 0.038 at chainage 1+200m to maximum 1.25 at chainage 1+300m. That can be a class of extremely poor rock class to poor rock class. There are three types of rock class within the selected portion i.e. poor, very poor and extremely poor. The finite element analysis was done using generalized Hoek Brown failure criteria for the support estimated from rock mass classification, analytical approach and support used by the project, all these three set of models was tested for different factor of safety. Such support was further analyzed for the block stability and squeezing problem. This thesis is focused to optimize the estimated support for economic project completion.

The rock support estimated from the rock mass classification is very first estimation for the different class of rock. These supports are optimized at every section using finite element method. This can conclude that the rock support can be optimized significantly while analyzing individual section with the geological condition instead of generalize the support class for the different category of Q values. Therefore, rock mass classification approach only is not adequate to design and estimation of tunnel support. Numerical analysis is very helpful to estimate the tunnel support in such geological region where rock masses are very poor with high rock cover.

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Dev Raj Joshi

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ABBREVIATION

SMHEP	Super Madi Hydro-Electric Project
Eb	Modulus of Elasticity of bolt
Ec	Modulus of Elasticity of concrete/shotcrete
Erm	Modulus of Elasticity of rock mass
Es	Modulus of Elasticity of steel
FEM	Finite Element Method
FS	Factor of Safety
Grm	Shear modulus of rock mass
GPa	Gega Pascal
GSI	Geological Strength Index
h	Height of overburden
ISRM	International Society of Rock Mechanics
m	Meters
MPa	Mega Pascal
MW	Mega Watt
Po	In-situ stress
ri	Internal radius of tunnel
RMR	Rock Mass Rating
RQD	Rock Quality Designation
UCS	Uniaxial Compressive Strength
VDC	Village Development Committee
ε	Strain
γ	Unit Weight
σr	Radial stress
$\sigma\theta$	Tangential Stress
υ	Poisson’s ratio

1 INTRODUCTION

1.1 Background

Tunnels are passageways which can be built to serve different purposes including mobility of people and traffic, underground storage, military fortification and conveyance. An excavation or exposure requires support that may vary depending on the purpose and/or importance of the excavation. Sometimes support is required not necessarily to carry heavy loads, but solely for wisely reasons i.e. to ensure that the acceptable level of safety against personnel and equipment is maintained. The support is then called safety support system.

Tunnel construction has risk associated with it because of lack of knowledge of existing subsurface condition. Although the majority of tunnel construction projects have been completed safely there have been several incidents in various tunneling projects that have resulted in delays, cost overruns, and in a few cases more significant consequences such as injury, and loss of life.

Selection and design of support systems are only two of many inter-related factors in the overall design of serviceable and economical tunnel. The type of support, the method of excavation, and the character of ground are inseparable considerations. If the root is laid out to encounter the worst rather than best geological features, or if the construction method is ill-suited to the geology, no amount of refinement of the lining can appreciably influence the economy of the job. Nevertheless, for each tunnel lay out and each construction and final support during the functional life of the tunnel pose separate requirement, sometimes both are best satisfied by a single support system.

Rational Design presupposes knowledge of the demands on supports system, criteria for successful performance, familiarity with the capabilities of available systems, and method of analysis verified by experiences. The basic function of tunnel support system is to keep the tunnel stable and to make opening useable. The specific purpose of support system, however, depend greatly on the purpose of the tunnel.

1.2 Problem Statement

Fifty-three percent of global tunnel failures are related to ground conditions (Lance et al., 2007) Therefore, correctness of geotechnical and geological ground conditions, herein after referred to as the ground, is critical to ensure safe and stability.

Problematic ground varies comprising hard, abrasive, weak, squeezing and swelling material, rock bursts and discontinuities such as faults, fissures and jointing. These problems can be seen when the tunnel is excavated. Additionally, rock is naturally very diverse and impossible to generalize its properties, behavior, design and suitable construction methods.

The support required for a tunnel in rock is a complex function of the properties and condition of the rock, the geometry and orientation of the tunnel, and the construction procedures. Majority of tunnel in the design phase decision in selecting tunnel alignment and predicting the rock mass quality and rock support requirement has direct influence on the overall cost and time requirement. The past tunneling experience indicated that the majority of the tunnel projects developed have had suffered severe stability problems that made delay in completion and cost overruns.

In the Nepalese context the geological study is limited because of resources, that make change the designed parameter while excavation. This phenomenon also rises in this project. The preliminary design of rock support in this project is done with limited field observation, testing result and improper geological investigation. During the excavation, these supports may be over or less at the various section which affect the project timing and cost. This need to be revise the rock support design at various section with the proper geological investigation. This research is focused on the revision and optimization of rock support for the various geological condition.

1.3 Objective of Study

The overall objective of this research work is to design tunnel support and evaluate the support provided in order to stabilize the headrace tunnel.

The specific objectives of this research work are as follows:

- To assess stability of the tunnel at critical sections.
- To estimate a support in rock for hydro-tunnel using empirical as well as Finite Element Method

1.4 Scope and limitation of the study

- It is limited to relevant literature to tunneling principle of rock mechanics and geotechnical engineering.

- Only geotechnical engineering factors influencing the tunnel stability were considered.

1.5 Structure of Thesis

This thesis has been structured into six chapters. Chapter 1 includes introduction to thesis and include the need of the study, its objective and scope. Chapter 2 provides the review of literature about the rock mass classification, stress analysis, rock mass deformability, rock properties, this chapter also include the failure behavior of the tunnel, support optimization, block stability analysis, squeezing prediction and the numerical modelling. Chapter 3 involves methodology and process adopted to fulfill the objective of study. Chapter 4 describes the study area and the project description chosen in the thesis. Chapter 5 presents the summary of results obtained and its discussion. Final chapter gives the Conclusion of the study and Recommendation for further study.

The References and Annexes are incorporated at the end of this thesis while the Acknowledgements and Abstract are given in the preface portion.

2 LITERATURE REVIEW

2.1 Literature Review Related to the Rock Mass Classification

Different rock mass classification systems have been developing throughout the history of development of rock mechanics. The first attempt to develop a classification of rock mass was by Ritter (1879) and Wickham et al (1972) developed later on multi-parameter classification schemes. One of the earliest use of rock mass classification schemes in tunnel support was used by Terzaghi (1946) in his classification system the rock mass was classified into intact, stratified, moderately jointed, blocky or seamy, crushed, squeezing and swelling rock.

Deere et al (1967) developed quantitative estimation of rock mass quality from drill cores and introduced RQD. When no core is available but discontinuity traces are visible in surface exposures or exploration adits, the Rock Quality Designation (RQD) is estimated from the number of discontinuities per unit volume Palmström (1982).

The RMR or Geomechanical classification developed by Bieniawski in 1973 and later revised in 1974, 1975, 1976, and 1989 gave the rock mass classification for the determination of rock mass quality and support design and suggested five parameters RQD, UCS, condition of discontinuities, orientation of discontinuities and groundwater condition for the classification and a correction factor for drive direction.

Barton et al (1974) of the Norwegian Geotechnical Institute proposed a Rock Quality Index (Q) system. The Q system suggested the six parameters for the rock mass classification and tunnel support estimation: Rock Quality Designation (RQD), joint set number (Jn), joint roughness number (Jr), joint alteration number (Ja) joint water reduction factor (Jw) and stress reduction factor (SRF).

GSI system was introduced by Hoek and According to Cai et al (2004), the Geological Strength Index (GSI) system is the only rock mass classification system that is directly correlated to the Mohr-Coulomb, Hoek-Brown and rock mass modulus engineering parameters. However, as the application of the GSI system is limited by its subjective nature, a quantitative approach, utilizing block volume and joint condition factors as quantitative parameters has been developed by Cai et al (2004).

Kim et. al. (2007) concluded that the significance of block size is higher than that of the angle between joint sets, indicating that it is important to consider joint persistence in a rock mass classification program.

The earliest reference to the use of rock mass classification for the design of tunnel support is in a paper by Terzaghi (1946) in which the rock loads, carried by steel sets, are estimated on the basis of a descriptive classification. While no useful purpose would be served by including details of Terzaghi's classification in this discussion on the design of support, it is interesting to examine the rock mass descriptions included in his original paper, because he draws attention to those characteristics that dominate rock mass behaviour, particularly in situations where gravity constitutes the dominant driving force.

Lauffer (1958) proposed that the stand-up time for an unsupported span is related to the quality of the rock mass in which the span is excavated. In a tunnel, the unsupported span is defined as the span of the tunnel or the distance between the face and the nearest support, if this is greater than the tunnel span. Lauffer's original classification has since been modified by a number of authors, notably Pacher et al (1974), and now forms part of the general tunnelling approach known as the New Austrian Tunnelling Method (NATM).

The New Austrian Tunnelling Method includes a number of techniques for safe tunnelling in rock conditions in which the stand-up time is limited before failure occurs. These techniques include the use of smaller headings and benching or the use of multiple drifts to form a reinforced ring inside which the bulk of the tunnel can be excavated. These techniques are applicable in soft rocks such as shales, phyllites and mudstones in which the squeezing and swelling problems, described by Terzaghi (see previous section), are likely to occur. The techniques are also applicable when tunnelling in excessively broken rock, but great care should be taken in attempting to apply these techniques to excavations in hard rocks in which different failure mechanisms occur.

The Rock Quality Designation index (*RQD*) was developed by Deere (Deere et al 1967) to provide a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact core pieces longer than 100 mm (4 inches) in the total length of core. The core should be at least NW size (54.7 mm or 2.15 inches in diameter) and should be drilled with a double-tube core barrel.

Wickham et al (1972) described a quantitative method for describing the quality of a rock mass and for selecting appropriate support on the basis of their Rock Structure

Rating (RSR) classification. Most of the case histories, used in the development of this system, were for relatively small tunnels supported by means of steel sets, although historically this system was the first to refer to shotcrete support. In spite of this limitation, it is worth examining the RSR system in some detail since it demonstrates the logic involved in developing a quasi-quantitative rock mass classification system.

Bieniawski (1976) published the details of a rock mass classification called the Geomechanics Classification or the Rock Mass Rating (RMR) system. Over the years, this system has been successively refined as more case records have been examined that Bieniawski has made significant changes in the ratings assigned to different parameters. The discussion which follows is based upon the 1989 version of the classification (Bieniawski, 1989). Both this version and the 1976 version deal with estimating the strength of rock masses.

2.2 Literature Review Related to Stress Analysis

Basically, the numerical methods and analytical method are the two methods, highly used in stress analysis of a tunnel opening. In the Nepal Himalaya, where the major rocks are subjected to directional strength and deformability, empirical method of stress analysis can play vital role in prediction of squeezing phenomenon (Shrestha, 2005).

Hooker et al. 1972; Brown and Hoek 1978, indicate that for depth of stress determination of mining engineering interest shows the vertical direction is rarely the principal stress direction.

Stress-induced failure of tunnels in brittle rock is the notched-shape of the failure region and the associated slabbing and spalling that may occur in a stable manner or violently in the form of strain bursts. These slabs can range in thickness from a few millimeters to tens of centimeters and with large openings can be several square meters in surface area (Ortlepp, 1997 and Martin et al. 1997). Fairhurst and Cook (1966) suggested that the formation and thickness of these slabs could be related to strain energy. In rocks with low value of uniaxial compressive strength conditions for rock failure due to concentration of initial stresss may lead to slow compression 'squeeze' and destruction of tunnel support rather than violent collapse (Goodman 1989). The major discontinuities presence or of a number of joint sets does not necessarily imply that the rock mass will behave as a discontinum (Brady and Brown, 1985). According to them highly jointed rock mass also may have as an incompetent massive rock mass. In such a rock types solution for stresses and displacement derived from the theory of plasticity provide useful basis for engineering work. Martin et al. (1997) provided detailed observations of the failure process around a circular test tunnel and concluded that the slab formation is associated with the advancing tunnel face, and that once plane-strain conditions are reached the new notched-tunnel shape is essentially stable.

Palmstrom and Nilsen (2000) defined numerical modeling as descretization of the rock mass in consideration into a large number of individual elements that are analyzed by use of computers for the valuation of rock stresses and deformations.

Tunnel squeezing is commonly a problem in a tunnel with a relatively higher depth, but it may also occur in shallow depth due to tectonic or topographic setting of the area (Shrestha and Broch, 2008). Hoek and Marinos (2000) has developed a relation between rock mass uniaxial strength σ_{cm} and Lithostatic stress $\sigma_0 = \gamma_r H$ to predict the tunnel squeezing. It states that values of $\frac{\sigma_{cm}}{\sigma_0} < 0.35$ are likely to produce squeezing (as defined by normalized convergences of more than 1% in unsupported tunnels).

Solak and Schubert (2004) have studied the Influence of block size and shape on the deformation behavior and stress development around the tunnel. The change in block size and shape influence the mode of failure and stress development in the different parts of the tunnel.

Shrestha (2005) recommended that squeezing phenomenon of tunnel in Himalaya where major rocks are subjected to directional strength and deformability, empirical methods of stress analysis can play vital role in prediction of the squeezing phenomenon.

Kim et al. (2007) has used the non-persistent joints for estimation of block sizes for rock masses. The assumption of the persistent joints the sizes of the blocks tend to be underestimated. Moreover, poor understanding of the rock bridge strength may lead to lower rock mass strengths, and consequently, to excessive expenditure on rock support.

Ganesh (2010), Carried out the stress analysis along the underground structures produced by overburden rock body along the headrace tunnel using RMR, GSI and Q from surface mapping and other value obtained from different empirical methods.

Madirolas and Perucho (2014) studied in situ stress in rock mass for the reason, the stress state is directly introduce into the numerical model used for the design of tunnels. He concluded that stress analysis is necessary to implement a routine procedure of study of civil work project.

González et al. (2014) gives result for high values of stress amplification factor (SAF), deformation in the tunnel sections were much higher than these expected in the design. The stress amplification zone can be identified and evaluated by Tectonic Stress Indices (TSI) and SAF indices.

Khanal (2014), gives the conclusion that the stress analysis by numerical approach shows quite different result compared to empirical approach. Higher strength rock shows the higher principal stress difference and lower strength rock shows the lower difference in the principal stress.
The purpose of FEM is to obtain the optimum condition for the formation of over break similar to ground conditions. It was observed that the size of over break is controlled by joint persistence, spacing and shear strength of rock joints (Panthee et al. 2016).

2.3 Literature Review Related to Rock Mass Deformability

Different rock mass classification systems have been developed throughout the history of development of rock mechanics. The first attempt to develop a classification of rock mass was by Ritter (1879) and later on multi-parameter classification schemes were developed by Wickham et al. (1972). One of the earliest use of rock mass classification schemes in tunnel support was used by Terzaghi (1946) in his classification system the rock mass classified into intact, stratified, moderately jointed, blocky or seamy, crushed, squeezing and swelling rock.

ISRM (1978) gave the quantitative field estimation of the Uniaxial Compressive Strength by visual and sensory description of hardness of rock mass and suggested the strength of the rock can also be judged from the simple hardness tests in the field with a geological hammer by observing the resistance to breaking under impact. Martin et al. (1997) provided detailed observations of the failure process around a circular test tunnel and concluded that the slab formation is associated with the advancing tunnel face and that once plane-strain conditions are reached the new notched-tunnel shape I essentially stable. Hoek and Marions (2000) has developed a relation between rock mass uniaxial strength with overburden to predict the tunnel squeezing.

Solak and Schubert (2004) has studied the Influence of block size and shape on the deformation behavior and stress development around the tunnel. The change of the block size and shape influence the mode of failure and stress development in the different parts of the tunnel. Shrestha (2005) recommended that squeezing phenomenon of tunnel in Himalaya where major rocks are subjected to directional strength and deformability, empirical methods of stress analysis can play vital role in prediction of the squeezing phenomenon. Hoek (2007), developed the rock mass classification system, Geological Strength Index (GSI), which varies from 0-100.

Kim et al. (2007) has used the non-persistent joints for estimation of block sizes for rock masses. The assumption of the persistent joints the sizes of the blocks tend to be underestimated. Moreover, poor understanding of the rock bridge strength may lead to lower rock mass strengths, and consequently, to excessive expenditure on rock support. Ganesh (2010), Carried out the stress analysis along the underground structures produced by overburden rock body along the headrace tunnel using RMR, GSI and Q from surface mapping and other value obtained from different empirical methods.

Panthee et al.(2016) have studied about the deformation modulus and determination. Among the several equations proposed using regression analysis of E_m and rock mass class, equations based on RMR, and Q were selected for the calculation of E_m values for different rock types along the tunnel alignment of the Kulekhani III Hydroelectric Project, Nepal. Finally, he found that higher the rock mass class, the higher the difference of E_m values.

Kayabasi et al.(2003) have done the comparative study on deformation modulus of rock masses. This study includes the assessment of the prediction performance of some existing empirical equations, construction of fuzzy inference system for the estimation of modulus of deformation and making the comparison between the results obtained from them. Among the prediction models, the fuzzy inference system provided the more reliable results than others.

Hoek and Diederichs (2006) have studied about estimating the values of rock mass deformation modulus on the basis of classification schemes by using empirical relationships. Finally, they proposed a new relationship based upon a sigmoid function.

Steiner et al. (1996)have studied about the case histories in tunneling in squeezing rocks and concluded that squeezing ground conditions are influenced by the factors such as rock type, strength of fragmentation if rock mass, orientation of the rock structure, stress state (overburden), water pressure, construction procedures and support system, not all of which contribute to the same degree.

Kavvadas (2003) have studied about the Monitoring and modelling ground deformation during tunneling. He described about the difficulties in obtaining ground measurement and their subsequent evaluation and, the application of these measurements in modelling tunnel excavations and support and in establishing early warning systems against incipient ground collapses or damages.

2.4 Literature Review Related to Properties of Rock

Although rock is naturally stable or slowly changes its chemical composition only under extreme conditions, its material properties influence strength, deformability, permeability and stability of rock masses (Stegner, 1971; Palmström, 1995). Material properties of a rock determines whether it is suitable for construction or not and the precautions required when using it. It is therefore important to understand rock mineralogy, structure and fabric, discontinuities/discontinuity sets, hydrogeology, squeezing and swelling problematic material behavior (Panthi, 2006; Sepp, 2000). Table 2-1 shows the specific material properties which influence discontinuous rock parameters.

Table 2-1 Rock Parameters

Parameter	Specific Material Properties
Rock mass structure	Type, strength, degree of weathering of rock, in-situ stress magnitudes and direction
Rock mass description	Interlocking / wedge spacing, block size and shape, discontinuity sets and persistence

Discontinuities Type,	orientation, roughness, aperture / width, infilling material type
Construction	Excavation method and support sequence
Hydrogeology and voids	Groundwater, seepage / permeability, pore pressures

Textural characteristics of rock materials are influenced by the following factors: mineral composition, size, shape, and spatial distribution of mineral grains, porosity, and inherent micro cracks.

2.5 Literature Review Related to Failure Behavior in Tunnel

Rock mass failure does not always involve discontinuities. There are cases for tunnel where the stress simply exceeds the strength of the rock metrics and hence this later can fail (Table 2-2). Creating an opening in a rock mass modifies the stress distribution in the ground, some stresses would increase and some would decrease. The increase of stress could lead to failure. For opening, failure typically occurs in the vicinity of the excavation wall.

Hoek and Brown (1980) identified four principals of sources of underground instability.

i) High rock stress failure associated with hard rock. This kind failure can occurs e.g. when mining at great depth or for large excavation at shallow depth. Stress conditions for tunneling in steep mountain regions or in weak rock conditions can also result in stress-induced instability problems.

ii) Structurally controlled failure tends to occur in faulted and jointed hard rock, in particular when several joint sets are steeply inclined.

iii) Weathered or swelling rock failure often associated with relatively poor rock. This kind of failure may also occur in isolated seams within as otherwise sound hard rock.

iv) Groundwater pressure or flow induced failure, which can occur in almost any rock mass. If the failure is combined with any of the other types of instability listed above, it could reach serious proportions.

According to Palmstrom and Stille (2007), more than one stability problem can occur simultaneously. This depends on factors such as the composition of the rock mass, stress, groundwater pressure, and size of the excavation.

Table 2-2 Failure Behavior in Rock

Rock Mass	**Failure**	**Behavior**
Hard and Brittle	Spalling	Sudden detachment of thin rock slabs
Anisotropic	Bending	Deflection of column or beam when it is subjected to a force that is applied axial and perpendicular to its axis.

Weak -Altered	Shear	Shearing of the rock mass resulting is a shear surface and shearing along a pre-existing weakness zone/discontinuity in the rock mass.
Hard rock	Wedge	Falling or sliding of block formed pre-existing discontinuities.

Hoek et al, (1995) studied different types of failure and stability problem in underground excavation under high and low in-situ stress condition and as a function of the jointing of the rock mass. Martin et al (2001) incorporated the effect of the intermediate in-situ stress as presented in Figure 2-1.

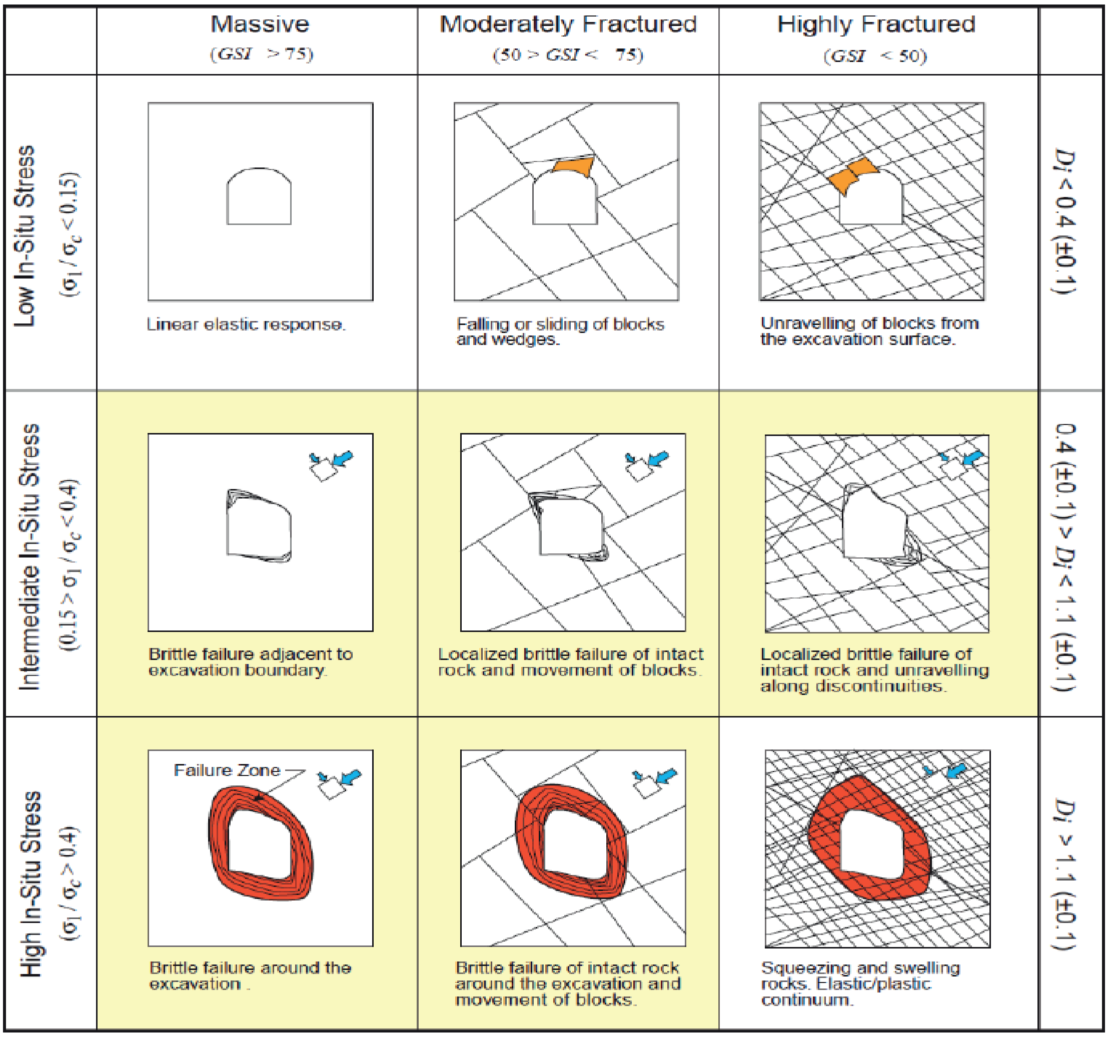


Figure 2-1 Types of Failure in Rock Masses (Martin et al 2001)

2.6 Literature Review Related to Support Optimization

The design and performance of tunnels are usually affected by some uncertainties that can be costly and time-consuming for tunnel construction projects. Traditional empirical and deterministic design approaches do not include uncertainty in tunnel support design, but tend to be based on trial-and error processes that consider safety and cost. Reliability based optimization (RBO) makes provision for the uncertainty of structures by adding probabilistic constraints. This is quite straight forward if the results of the reliability analysis are accurate and precise so that no question arises as to whether a given design satisfies safety requirements. The purpose of RBO is to find a balanced design that is not only economical but also reliable in the presence of uncertainty.

Over the past few decades, numerous reliability optimization techniques have been proposed. Younes and Alaa (2009) overviewed the various RBDO approaches using mathematical and finite element models with different levels of difficulties. Marcos and Gerhart (2010) produced a detailed literature review on reliability-based optimization. Although RBO has some evident advantages over deterministic optimization design, it is often computationally inefficient. Response surface methodology has been applied in RBO in attempts to improve its efficiency. Zhang et al. applied the mean first-order reliability method (MFORM) to the optimization of geotechnical systems. Those methods improved the computational efficiency but decreased the accuracy of the reliability analysis, which affects the results of RBO. The selection of an optimization method is critical to RBO applications, especially for complex nonlinear optimization problems. Gen and Yun reviewed the application of soft computing methods in reliability optimization. Genetic algorithms and particle swarm optimization have also been applied to RBO. Lee et al. proposed a methodology to convert an RBO problem requiring very high reliability to an RBDO problem requiring relatively low reliability by appropriately increasing the input standard deviations for efficient computation in sampling-based RBDO.

Zhao et. al. (2018) use the least squares support vector machine (LSSVM) approach was adopted to build a relationship between reliability index and design variables, and the artificial bee colony (ABC) algorithm was employed for the reliability-based optimization. A proposed LSSVM/ABC-based reliability optimization method was applied to the case of a tunnel with rock bolt reinforcement. The mechanical parameters of the rock mass, in-situ stress and internal pressure were considered as the random variables. The reliability index of tunnel was analyzed. The length, distance out of plane and the number of rock bolts were determined and optimized considering the uncertainty based on RBO. The proposed method improved the efficiency of RBO while maintaining high accuracy.

Arbanas et al. (2010) conclude that, it is then possible to make the geotechnical model and perform the stability and stress-strain analyses. It is first assumed that the rock mass slope is unsupported during excavation. If a low initial factor of safety is obtained, the stability analysis should include a support system. The interactive design is applied throughout construction of the support system and includes extensive monitoring. A re-design of the excavation and additional support measures are required when monitoring imply on significant deviations of defined conditions.

2.7 Literature Review Related to Block Stability Analysis

Gautam (2012), studied about wedge analysis on headrace tunnel for the various section on hard rock. He recommended Numerical approaches have many benefits over empirical and analytical approaches, specifically in complex geometry like settling basin cavern. Rocscience software for numerical analysis such as and Un-wedge has been recommended. He used Generalized Hoek and Brown failure criterion are used to determine the state of stresses, strength factors, and deformations around the periphery of the caverns in. His study used to analyze the wedge failure due to low shear strength of joints, empirical approach suggested by Barton and Bandis is used in the numerical analysis through rocscience software-Unwedge.

Ajender (2016) did the Probabilistic Unwedge analysis to assess the structurally induced stability problems. Analytical and empirical studies involve Kirsch's equations and Hoek and Brown (1980) methods to assess the redistribution and concentration of stresses in the cavern contour. He determines the spalling potential and depth of brittle failure is estimated based on cavern span, rock mass spalling strength and tangential stresses. His results are compared to estimation of failure depth from numerical analysis using analysis of strength factor with Hoek-Brown brittle parameters in 2D finite element program, Phase2. He further did the 3D finite element analysis (RS3) is carried out for the final selected cavern alignment. Reasonable difference between analytical/empirical and numerical approach is found considering caverns location in low rock cover and near tow slope.

2.8 Literature Review Related to Tunnel Squeezing

Steiner (1996) studied about the case histories in tunneling in squeezing rocks and concluded that squeezing ground conditions are influenced by the factors such as rock type, strength of fragmentation if rock mass, orientation of the rock structure, stress state (overburden), water pressure, construction procedures and support system, not all of which contribute to the same degree.

Shrestha (2005) recommended that squeezing phenomenon of tunnel in Himalaya where major rocks are subjected to directional strength and deformability, empirical methods of stress analysis can play vital role in prediction of the squeezing phenomenon.

Basnet et. al. (2013) assesses the squeezing phenomenon along headrace of Chameliya Project in which tunnel stretch through evaluation of rock mass properties and support pressure. He approaches three different methods (two analytical and one 2D finite element numerical modeling program) for the analysis. His finding is that it is possible to predict extent of squeezing in tunnel if more than one method is used to verify rock mass mechanical properties.

Singh's (1992), squeezing prediction method is based on the rock mass classification approach. Singh et al. (1992) developed an empirical relationship from the log-log plot between the tunnel depth (H) and the logarithmic mean of the rock mass quality, Q. A clear line of demarcation is in between the elastic and squeezing condition. Goel (1994) developed an empirical approach based on the rock mass number N. Rock mass number N is equal to Q-value with SRF = 1. 'N' was used to avoid the problems and uncertainties in obtaining the correct rating of parameter SRF in Q method.

Martin et al. (1997) provided detailed observations of the failure process around a circular test tunnel and concluded that the slab formation is associated with the advancing tunnel face and that once plane-strain conditions are reached the new notched-tunnel shape I essentially stable.

Ayden et al. (1993) studied about the squeezing potential of rocks around Tunnels. They proposed general method to predict the squeezing potential of rocks around the tunnel and its degree and a specific application of the method to circular tunnels under hydrostatic state of stress is described.

2.9 Literature Review Related to Numerical Modelling

Various researchers have used FEM (Finite Element method) showing versatility of the method towards successful implementation in various rock engineering problems (Eberhardt 2001; Vermeer et al. 2003; Lee 2009; Kainthola et al. 2012; Singh et al. 2013, 2015).

Equivalent Mohr-Coulomb shear strength parameters (c and f) have been obtained from linear curve fitting method, using generalized Hoek-Brown failure criterion. This criterion allows incorporation of GSI into the model, and the benefit of which is the fact that GSI includes rock mass deformation parameters in addition to disturbance factor (Sonmez and Ulusay 1999).

The low stress condition; a case of Kulekhani III hydroelectric project was analysed using Phase2 for the study of rock joint parameters on deformation of tunnel opening (Panthee et al. 2016).

In Phase^2, field stress can be constant or gravity stress. The gravity field stress option is used to define a gravity stress field which varies linearly with depth from a user-specified ground surface elevation. Gravity field stress is typically used for surface or near surface at shallow depth elevations and the areas where the effect of topography stress magnitudes and directions. Stress ratio is calculated with the help of Poisson's ratio. In addition, the material parameters such as unconfined compressive strength of intact rock(σ_{ci}), HoekBrown constant (m_i), Geological Strength Index (GSI), Young's Modulus of Intact Rock (E_i), Poisson's ratio (ν), density of rock mass are the inputs to the material property.

The principle stress can be displayed and the results can be seen. The stress level could be checked in particular location of the analysis. The major and minor principle stress and angle between stresses with horizontal can be used to calculate the vertical and horizontal stress at that point and the result can be compared with the gravity and tectonic stress.

The strength factor of the rock mass around the tunnel can be displayed with contours. With the elastic analysis if the strength factor is greater than 1 everywhere around the tunnel, the result will be same even if the plastic analysis has been done. Hence there is necessity of plastic analysis if the strength factor is less than one around the tunnel with elastic analysis.

The value of vertical, horizontal, total displacement can be displayed with the contour around the tunnel. The value can be compared with the result obtained from analytical, semi-analytical method and also with the value of measured convergence.

Li et al (2019) study the mechanical response of a hard rock tunnel excavated by double shield tunnel boring machine (DS-TBM), a numerical method was introduced to simulate the TBM excavating process. The failure modes of surrounding rock mass described based on the cohesion weakening and frictional strengthening (CWFS) Mohr-Coulomb strain-softening criterion. The characteristics of stress field and plastic zone on the cross and longitudinal section of the tunnel were analyzed in detail, and the results were compared with those in the intrinsic condition (when TBM model is not activated). The simulation results indicate that the stress paths at the vault are relatively simple, and the stress concentration caused by excavation unloading is obviously reduced by lining and backfill grouting, while the sidewall is less disturbed by the excavation of TBM. The invert experiences three unloading processes, due to excavation, the contact between the rear shield and the bottom of surrounding rock, as well as backfill grouting at gap between the lining and the rock mass. The vault has a larger plastic zone than the invert, attributing to the geometrical difference between the cutter-head and the front shield, as well as the conicity of the front and rear shields.

Khadka et al. (2019) conduct numerical analysis of hydropower tunnel of the Lesser Himalayan Region of Nepal. This region lies between two major faults namely Main Boundary Thrust (MBT) and the Main Central Thrust (MCT) with weak rock mass like phyllite, schist, gneiss, phyllitic schist, etc. Thus, to overcome the stability problems during underground construction, proper rock support system must be installed. Rock mass classification systems are commonly used for estimating the rock support system in this region, but this approach is inadequate to address the underground stability problems. In his study, numerical analysis is done to define the requirement of support and the result compared to actual support provided in selected case study. Analytical approach is used along with two-dimensional Finite Element Analysis using the software, RS2 provided by RocScience for the study. Finally, required modification of the provided support has been suggested to overcome the problem faced in the selected tunnel.

3 METHODOLOGY

3.1 General Flowchart

Limited research has been conducted in context to the stability analysis of underground structures in the Nepal Himalaya. Stress-induced failure is the common failure in the Himalayan geology having high overburden and poor rock mass quality. One hydropower tunnels are selected for the numerical modeling. The output from numerical modeling is used for the verification, namely Super Madi Hydro-Electric Project, taken as case studies in this MSc work.

The research question of this research is to design the reliable support system in rock underground excavation for the Super Madi Hydroelectric Project Headrace Tunnel. This is the primary objective of study by collecting various geological data either by site surveying or by the detailed project report of the project. Also, this study is focused on the economical point of view for the support system. How can we minimize the project cost? This is another big question for this study. This will be taken by analyzing various alternative of support system designed. And finally, we can recommend various support system for further excavation in the same Headrace tunnel based on the geological condition. At the end of this study we summarize the Q-value and rock support based on that Q-value for the recommendation.

To meet the above-mentioned objective, the following research methodology is applied in this study as presented in Figure 3-1. The flow chart clearly explains the overall methodology for this study from the beginning to the report writing.

As we know every research work starts from the Literature review related to the study. Lots of research paper on rock support, tunneling, support optimization, geotechnical field, Geology of Nepal and worlds, Support failure cases, Support System in rock, excavation methodology etc. this process have been continued to the end of this research.

Geological parameter of the study area was collected by surveying at site and from the detailed project report of Super Madi Hydroelectric Project. On the basis of these data rock support was designed by using Q-Chart, furthermore the finite element analysis was done using Rock science software provided for the study purpose. The detail of the research methodology taken for this thesis was described here. The methodology taken in this study was of from various research published and previous thesis.

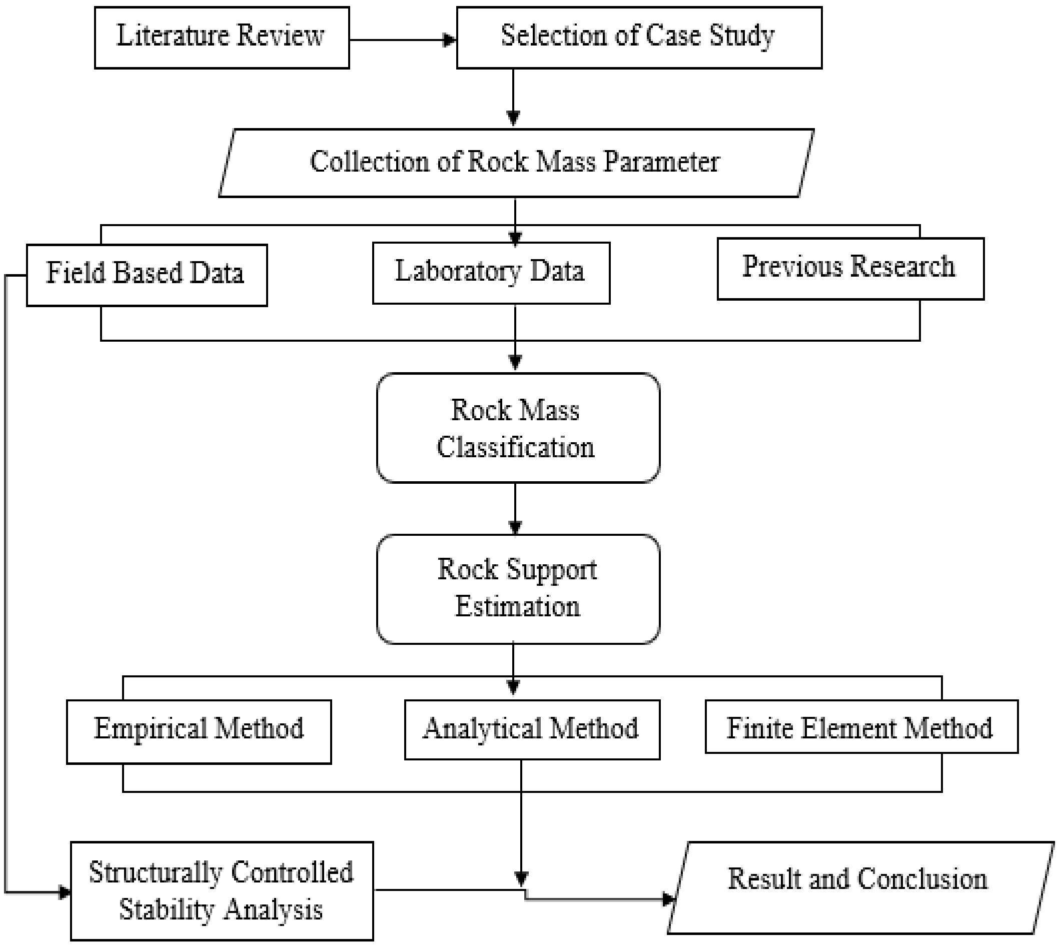


Figure 3-1 General Methodology Flow Chart

3.2 Description of Methodology

Research methodology is the specific procedures or techniques used to identify, select, process, and analyze information about a topic. In a research, the methodology section allows the reader to critically evaluate a study's overall validity and reliability. The methodology of this thesis was summarized in Figure 3-1.

The methodology of the research was described individually in this section. The research starts with the literature review which help to identify the research question and selection of case study in that basis. The collection of data from the filed visit, observation, previous study, previous published research for the calculation of rock mass properties.

This followed to the estimation of rock support with various methods i.e. Empirical, Analytical, Numerical method and that compared to the project support. The tunnel section was also analyzed for the block failure.

3.2.1 Preliminary Study

Preliminary study consists of literature review, study of report, journals, papers, articles and old thesis documents. All the available previous study reports, data/information including maps and drawings and other information related to the study area was studied and analyzed in depth in the context of the objectives of the study.

Literature review related:

- To geology of the Himalaya regarding stress regime, rock types, weathering effect on rock mass and tectonic influence.
- To rock mass classification and support design methods.
- To excavation methods and their effect on stability of tunnel.
- To failure behavior in underground excavation specially for hard rock.
- To the effect of support installation time gap on stability.
- To the study of properties of rock on the study area.
- To used and available support material and their properties.
- To the existing empirical, semi empirical and analytical methods to evaluate the squeezing potential and design of undergrounds structures.
- To numerical modeling and analysis of underground structures.
- To support optimization for economic and safety purpose.
- To in-situ testing for stability test of rock support

The case study was selected of Super Madi HEP headrace tunnel of 400m section which was at the construction stage at the time of field visit. The site need revision on the rock support on the basis of detailed geological investigation. Geological as well as project data was collected from the site office and verified these site condition by the field visit.

Study of Project report:

- To project salient features
- To study the topographic map of the project area.
- To engineering geological condition of project components
- To climatic condition of study area
- To hydrology of the study area including flood hydrology, design flood etc.
- To design and dimension of the various structure unit.
- To construction method and equipment’s

3.2.2 Data Collection

It includes the project details and engineering geological information on the rock mass condition. Geological properties of rock masses will be referred as provided. General data such as unit weight, modulus of elasticity, Poisson's ratio, uniaxial compressive strength and other properties of rock mass were obtained from empirical relations. Where the specific data of the tunnel i.e. dimension, plan and ground profile of tunnel alignment, rock type and rock mass classification were obtained from Geological Baseline Report of SMHEP.

Study site visit was conducted several times to collect various geological data, design data and their field verification. Site visit was focused on various activities discussed below.

- For Geological mapping at the various chainage.
- To study surface geology of the study area.
- To study various support system.
- Rock identification and verification as provided by the project report.
- Water table of the study area.
- Rock mass classification in tunnel

3.2.3 Rock Mass Classification

Calculation of Q-Value

Nepal Himal consists of various types of rocks and geological structures. The Khimti-1 Hydropower Project (KHP) is the first project applying Q-system in Nepali Himalayan Rock. The method was found to be appropriate for drill and blast tunnels in jointed, fractured and sheared rock, which tend to overbreak. Barton et al. (1974) at the Norwegian Geotechnical Institute (NGI) originally proposed Q system of rock mass classification based on 200 case studies of tunnels and caverns. The Q system was updated to include 1000 cases (Grimstad and Barton, 1993). RQD, joint set number (Jn), joint roughness (Jr), joint alteration (Ja), joint water reduction factor (Jw) and stress reduction factor (SRF) are utilized to calculate Q value. Q system was compiled again in 2002 and some changes on support recommendations were made.

The Q system chart for rock support estimate was developed by the Norwegian Geotechnical Institute (NGI), (based on www.ngi.no, 2014). The Q_{wall} values have been introduced in that chart. On the basis of calculated value of Q using above mentioned parameter. The rock class was classified from exceptionally poor to exceptionally good rock. The Q value from 0.001 to 0.01 is classified as exceptionally poor rock mass, 0.01 -0.1 is extremely poor rock, 0.1 -1 is very poor, 1- 4 poor rock class. Q value from 4 to 10 is fair rock mass. If the Q value is greater than 10 and less than 40 is classified as good rock mass. Above 40 and up to 100 is classified as very good rock mass. Q value of 40 – 400 range classify the rock as extremely good. And above 400 is exceptionally good class. These rock classes are categorized as G, F, E, D, C, B and A class respectively.

Calculation of RMR Value

The *rock* mass rating (*RMR*) is a geo-mechanical classification system for *rocks*, developed by the sum of the six parameters is the "*RMR value*", which lies between 0 and 100. Some of the complex *mechanics* of actual *rocks* into *engineering* design. Moreover, the system was the first to enable *estimation* of *rock* mass. RMR combines the most significant geologic parameters of influence and represents them with one overall comprehensive index of rock mass quality, which is used for the design and construction of excavations in rock, such as tunnels, mines, slopes, and foundations.

RMR rock mass classification system was initially developed at the South African Council of Scientific and Industrial Research (CSIR) by Bieniawski (1974) on the basis of his experiences in shallow tunnels in sedimentary rocks. Classification parameters were reduced from 8 to 6 in 1974 and recommended support requirements and adjustment of rating were reduced in 1975. Class boundaries were modified in 1976 and ISRM rock mass descriptions were adopted in 1979. Uniaxial compressive strength (UCS), Rock Quality Designation (RQD), joint or discontinuity spacing, joint conditions, ground water condition and joint orientation are utilized parameters. In order to apply RMR, the site should be divided into a number of geological structural units in such a way that each type of rock mass is represented by a separate geotechnical structural unit. In this paper the 1989 version of RMR (Bieniawski, 1989) ratings for limestone, sandstone and diabase formations were used.

The RMR value was calculated using the following six parameters.

- Uniaxial compressive strength of intact rock material.
- Rock quality designation (RQD).
- Spacing of discontinuities.

- Condition of discontinuities, given as
    - Length, persistence
    - Separation
    - Smoothness
    - Infilling
    - Alteration / weathering
- Groundwater conditions.
- Orientation of discontinuities.

The rock mass is finally classified based on the RMR value using above mentioned parameter. Based on RMR value rock is classified from very poor to very good rock class as RMR value increases. If the RMR value is less than 20 than the rock class is very poor category. The value lies between 21 – 40 the rock class is known as poor rock. While value increases further up to 60 is a fair rock class. RMR value if lies of 61 – 80 the rock class is of good quality and finally if the rock class lies between 81 to 100 is of very good rock class. These rock masses are classified into five class from I to V as decreases the RMR value. RMR 100-81 is I class rock (Very Good Rock), 61-80 is II class rock, 41-60 is III class rock, 21- 40 is IV class rock and the RMR value is less than 20 is classified as V class rock i.e. very poor.

3.2.4 Determination of Rock Mass Parameter

The modulus of deformation of rock mass (E_m) may be defined as the ratio of stress to corresponding strain during loading of rock mass, including elastic and inelastic behavior whereas the modulus of elasticity of intact rock (E_i) is the ratio of applied stress and corresponding strain within the elasticity limit. The jointed rock mass does not behave elastically. Hence, the term modulus of deformation is used instead of modulus of elasticity. The deformation modulus of jointed rock mass is very low compared to the elasticity modulus of intact rock.

Estimation of rock mass characteristics is required to design an underground excavation. Methods such as the generalized Hoek-Brown criterion and Mohr-Coulomb failure criterion can be used to describe the characteristic behaviour of rock mass like strength and deformations. Measured data from core samples are often used to estimate the properties of intact rock (no weakness planes) and from that point through empirical approach to estimate the behaviour of the overall characteristics of the rock mass surrounding an underground opening. Strength of intact rock sample is usually higher than the overall strength of the rock mass and method are therefore needed to convert measured data from core samples to the rock mass (Hoek, 2000).

<u>Generalized Hoek-Brown criterion</u>

Hoek and Brown proposed in the 1980's a method to estimate the strength and properties of a jointed rock mass called Hoek-Brown failure criterion. The method is based on estimation of interlocking between rock blocks and shear conditions in the joints. This method was derived to be used to estimate strength of jointed rock mass where rock blocks are small relative to the excavation considered. The method has been modified over the years but the version introduced in this chapter is a modified version from 2002 (Hoek, Carranza, & al, 2002). The following series of equations represents the criteria:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a \qquad \text{Eq. 3-1}$$

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)} \qquad \text{Eq. 3-2}$$

$$S = e^{\left(\frac{GSI-100}{9-3D}\right)} \qquad \text{Eq. 3-3}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI}/15 - e^{-20}/3\right) \qquad \text{Eq. 3-4}$$

Where σ'_1 and σ'_3 are the maximum and minimum effective principal stresses at failure, m_b is the Hoek-Brown constant for the rock mass and m_i is the Hoek-Brown constant for the intact rock samples, *s* and *a* are constants related to the rock mass characteristics and σ_{ci} is the uniaxial compressive strength of intact rock sample.

GSI is the geological strength index introduced by Hoek in 1994 to simplify the conversion between the intact rock strength and the rock mass strength.

A disturbance factor D is used to consider the disturbance from blasting and stress relaxations in the rock mass.

To estimate the value of mi the Equation 3-1 is used with S=1 and a=0,5 and becomes:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_i \frac{\sigma_3'}{\sigma_{ci}} + 1\right)^{0,5} \qquad \text{Eq. 3-5}$$

A series of triaxial test on core samples can therefore be used to determine the value of m_i *and* σ_{ci}. Authors of the method recommends that series of at least five triaxial tests should be used (Hoek, Carranza, & al, 2002).

Authors also recommend that the range of σ'_3 should be equally distributed between zero and 0.5 time the intact compression strength.

σ'_3 can be set to zero in Equation 3-5 to reveal the uniaxial compression strength of the rock mass and becomes:

$$\sigma_c = \sigma_{ci} s^a \qquad \text{Eq. 3-6}$$

The tensile strength of the rock mass can be found in a similar way by setting σ'_1 equal to zero and the tensile strength becomes:

$$\sigma_t = -\frac{s\sigma_{ci}}{m_b} \qquad \text{Eq. 3-7}$$

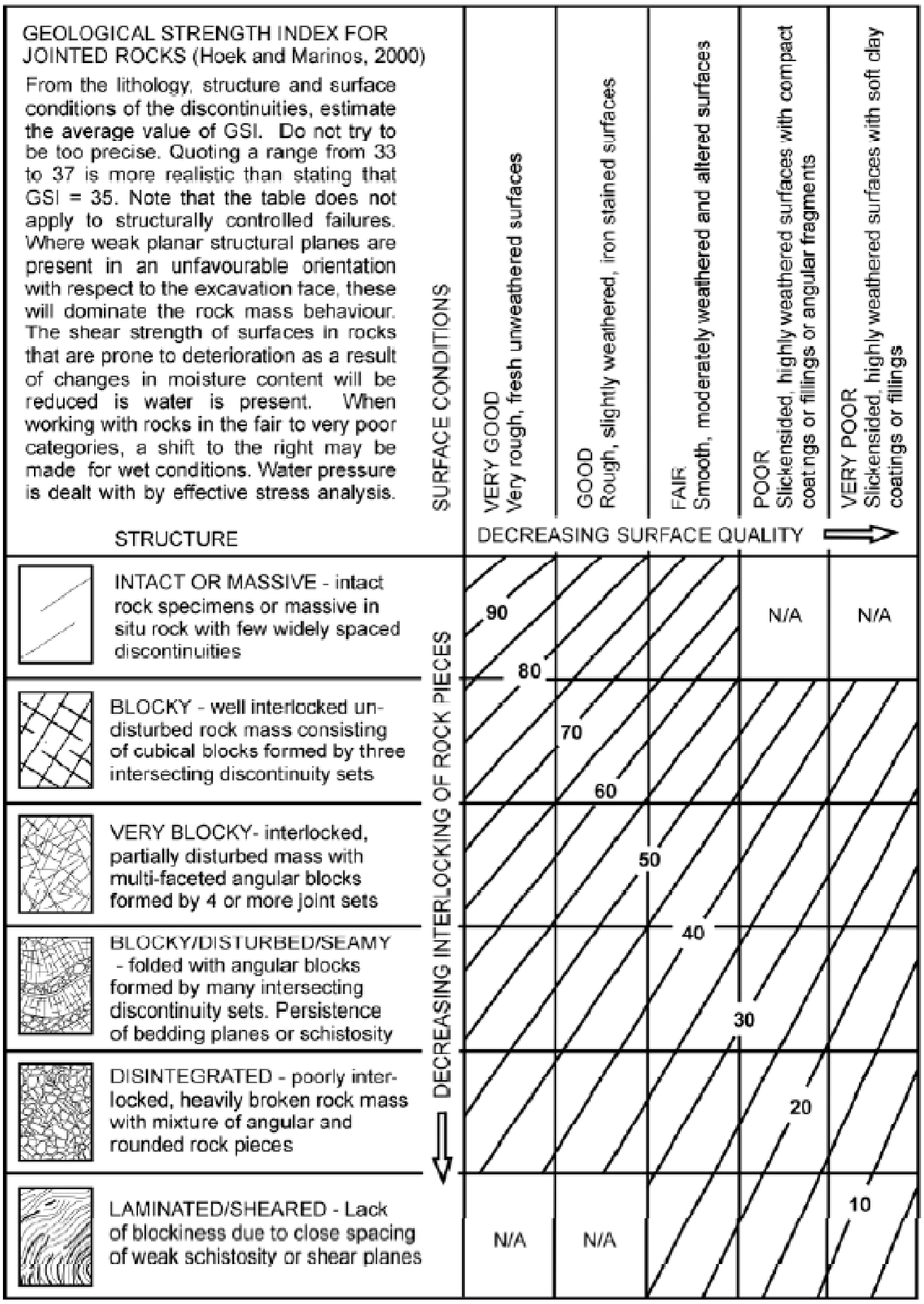


Figure 3-2 Estimation of GSI value. (Hoek, 2000)

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

Figure 3-3 Estimation of Disturbance Factor (Hoek, 2000)

Mohr-Coulomb failure criterion

The Hoek-Brown failure criteria is well suited for jointed or heavily jointed igneous rock types like basalt. But for other rock types like sedimentary or metamorphic rock the Mohr- Coulomb failure criterion can be a better choice (Hoek, 2000). Estimation of shear strength can be made by the Mohr-Coulumb eqution:

$$\tau_f = C + \sigma_n' tan(\theta) \qquad \text{Eq. 3-8}$$

where θ is the internal friction angle of the intact rock sample, C is the cohesion and σ'_n is the normal stress acting on the plain of failure (Erlingsson, 2009). The Mohr-Coulomb equation can also be written as:

$$\sigma_1 = \frac{1+sin(\theta)}{1-sin(\theta)}\sigma_3 + \frac{2C\,cos(\theta)}{1-sin(\theta)} = C^* + tan(\varphi)\,\sigma_3 \qquad \text{Eq. 3-9}$$

Uniaxial compression strength and tensile strength can be derived from Equation 3-9 by putting σ_1 and σ_3 to zero respectively, thus

$$\sigma_1 = \frac{2C\,cos(\theta)}{1-sin(\theta)} = \sigma_c \qquad \text{Eq. 3-10}$$

$$\sigma_3 = \frac{2C\,cos(\theta)}{1+sin(\theta)} = -\sigma_T \qquad \text{Eq. 3-11}$$

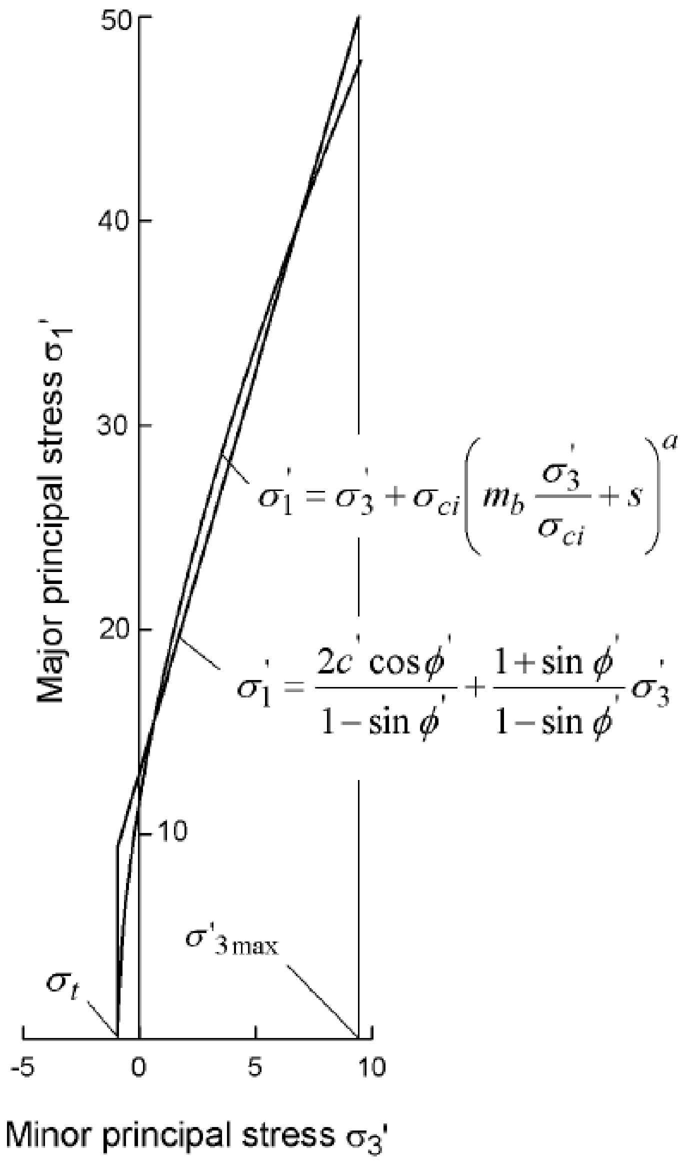


Figure 3-4 Mohr-Coulomb and Hoek-Brown failure criterions (Hoek, 2000)

Rock mass properties such as Hoek–Brown constants, deformation modulus of rock masses and uniaxial compressive strength of rock mass were calculated in accordance with Q_c, Q_N, Q, RMR, RMi.

The modulus of deformation of rock mass (Em) may be defined as the ratio of stress to Corresponding strain during loading of rock mass, including elastic and inelastic behavior whereas the modulus of elasticity of intact rock (Ei) is the ratio of applied stress and corresponding strain within the elasticity limit. The jointed rock mass does not behave elastically. Hence, the term modulus of deformation is used instead of modulus of elasticity. The deformation modulus of jointed rock mass is very low compared to the elasticity modulus of intact rock.

Deformation modulus of rock masses In-situ determination of E_{mass} is costly and often very difficult. Thus, the utilization of empirical methods is inevitable. By means of the empirical methods, E_{mass} can easily be acquired. These values are shown in table 4-7.

For RMRN50, Bieniawski (1978) defines Emass as below:

$$E_{mass} = 2RMR-100 \qquad \text{Eq. 3-12}$$

For RMRb50, Serafim and Pereira (1983) proposed the following formula

$$E_{mass} = 10^{\frac{RMR-10}{40}} \qquad \text{Eq. 3-13}$$

Grimstad and Barton (1993) suggested the following equation for calculating E_{mass} for Q>1 and generally for hard rock:

$$E_{\ mass} = 25 \log Q \text{ (GPa)} \qquad \text{Eq. 3-14}$$

For poor rock rcib100, MPa Hoek and Brown (1998) found a correlation between Emass and GSI:

$$E_{mass} = \sqrt{\frac{\sigma ci}{100}}\ 10^{\frac{GSI-10}{40}} \quad \text{(GPa)} \qquad \text{Eq. 3-15}$$

Read et al. (1999) proposed the below equation for calculating E_{mass} based on RMR value of rock mass:

$$E_{mass} = 0.1\left(\frac{RMR}{10}\right)^3 \quad \text{(GPa)} \qquad \text{Eq. 3-16}$$

Strength of rock masses was also calculated using various empirical formulas and that was shown in table 4-6.

Hoek and Brown (1980b):

$$\sigma_{cm} = \sigma ci.exp^{\left[\frac{RMR-100}{18}\right]} \qquad \text{Eq. 3-17}$$

Yudhbir et. al. (1983):

$$\sigma_{cm} = \sigma ci.exp^{[\frac{7.65(RMR-100)}{100}]} \quad \text{Eq. 3-18}$$

Ramamurthy (1986):

$$\sigma_{cm} = \sigma ci.exp^{[\frac{RMR-100}{18.75}]} \quad \text{Eq. 3-19}$$

Kalamaris and Bieniawski (1995):

$$\sigma_{cm} = \sigma ci.exp^{[\frac{RMR-100}{24}]} \quad \text{Eq. 3-20}$$

Shoery (1997):

$$\sigma_{cm} = \sigma ci.exp^{[\frac{RMR-100}{20}]} \quad \text{Eq. 3-21}$$

Barton (2002):

$$\sigma_{cm} = 5\gamma\ [Q\ \left(\frac{\sigma ci}{100}\right)]^{1/3} \quad \text{Eq. 3-22}$$

Hoek et. al. (1980):

$$\sigma_{cm} = \sigma ci.s^{a} \quad \text{Eq. 3-23}$$

Hoek–Brown failure criterion for rock masses uses mm and sm constants. Some suggested equations based on empirical methods are used to calculate these constants. Hoek et al. (2002) suggested some relationships between mm, sm and GSI as:

$$\frac{mm}{mi} = e^{\frac{GSI-100}{28-14D}} \quad \text{Eq. 3-24}$$

$$s = e^{\frac{GSI-100}{9-3D}} \quad \text{Eq. 3-25}$$

$$a = \frac{1}{2} + \frac{1}{6}(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}) \quad \text{Eq. 3-26}$$

Ramamurthy (1985) used RMR for calculating s constant as follows:

$$S_m = e^{(\frac{1}{40}(0.0564RMR-5.64))} \quad \text{Eq. 3-27}$$

Palmstro¨m (1995) suggested the following equations for calculating Hoek–Brown constants as:

$$S = J\ P^2 \quad \text{Eq. 3-28}$$

$$m_m = m_i\ J\ P^{0.64} \quad \text{Eq. 3-29}$$

3.2.5 Estimation of Rock Support

Estimations of required rock support is usually based on observation, experience and personal judgment of those involved. Engineers generally uses support guidelines or methods to back up their estimation of required rock support. Three types of methods are mostly used for this purpose (Palmstöm & Nilsen, 2000):

- Analytical methods, involving analysis of stress distributions and deformations using methods like numerical analysis, analogue simulation.
- Observational methods, like the New Australian Tunnelling Method which uses measurements of movements in the rock mass during excavation. Observations are off course also used to check if the chosen installed rock support was the right way to go or not and adjustment made if required.
- Empirical methods, often illustrated in table or graphs that connect classification of rock mass to curtain rock support. Number of empirical methods has been derived such as the RMR system end the Q-system.

General overview of the most commonly used empirical methods, some analytical methods and finite element method for the estimation of rock support will be given in this chapter.

3.2.5.1 Empirical Method

Q-System

The *Q-system* for rock mass classification, developed at the Norwegian Geotechnical Institute (NGI) in 1974, originally included a little more than 200 tunnel case histories, mainly from Scandinavia (Barton et al., 1974). In 1993 the system was updated to include more than 1000 cases (Grimstad and Barton, 1993). It is a quantitative classification system for estimates of tunnel support, based on a numerical assessment of the rock mass quality using the following six parameters:

- Rock quality designation (RQD).
- Number of joint sets (J_n).
- Roughness of the most unfavourable joint or discontinuity (J_r).
- Degree of alteration or filling along the weakest joint (J_a).
- Water inflow (J_w).
- Stress condition given as the stress reduction factor (SRF); composed of
    - Loosening load in the case of shear zones and clay bearing rock,

- Rock stress in competent rock, and
- Squeezing and swelling loads in plastic, incompetent rock.

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

$$Q = \frac{RQD}{Jn} * \frac{Jr}{Ja} * \frac{Jw}{SRF} \qquad \textit{Eq. 3-30}$$

The first two parameters represent the overall structure of the rock mass, and their quotient is a relative measure of the block size.

• The second quotient is described as an indicator of the inter-block shear strength.

• The third quotient is described as the "active stresses".

The ratings of the various input parameters to the Q-value are given in Table 1.

The Q-value is related to tunnel support requirement by defining the equivalent dimensions of the underground opening. This equivalent dimension, which is a function of the size and type of the excavation, is obtained by dividing the span, diameter or wall height of the excavation (Dt) by a quantity called the *excavation support ratio (ESR)*, given as:

$$De = \frac{Dt}{ESR} \qquad \textit{Eq. 3-31}$$

Ratings of ESR are shown in Table 3-7

The Q-value in Figure 1 is related to the total amount of support (temporary and permanent) in the roof. The diagram is based on numerous tunnel support cases. Wall support can also be found using the same figure by applying the wall height and the following adjustments to Q:

For $Q > 10$	use $Q_{wall} = 5Q$
For $0.1 < Q < 10$	use $Q_{wall} = 2.5Q$
For $Q < 0.1$	use $Q_{wall} = Q$

The first quotient (RQD/J*n*), representing the structure of the rock mass, is a crude measure of the block or particle size, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400. If the quotient is interpreted in units of centimetres, the extreme 'particle sizes' of 200 to 0.5 cm are seen to be crude but fairly realistic approximations. Probably the largest blocks should be several times this size and the smallest fragments less than half the size. (Clay particles are of course excluded).

The second quotient (*Jr/Ja*) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favour of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favorable to tunnel stability.

When rock joints have thin clay mineral coatings and fillings, the strength is reduced significantly. Nevertheless, rock wall contacts after small shear displacements have occurred may be a very important factor for preserving the excavation from ultimate failure. Where no rock wall contact exists, the conditions are extremely unfavorable to tunnel stability. The 'friction angles' are a little below the residual strength values for most clays, and are possibly down-graded by the fact that these clay bands or fillings may tend to consolidate during shear, at least if normal consolidation or if softening and swelling has occurred. The swelling pressure of montmorillonite may also be a factor here.

The third quotient (*Jw/SRF*) consists of two stress parameters. *SRF* is a measure of:

- Loosening load in the case of an excavation through shear zones and clay bearing rock,
- Rock stress in competent rock, and
- Squeezing loads in plastic incompetent rocks. It can be regarded as a total stress parameter.

The parameter *Jw* is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. Water may, in addition, cause softening and possible out-wash in the case of clay-filled joints. It has proved impossible to combine these two parameters in terms of inter-block effective stress, because paradoxically a high value of effective normal stress may sometimes signify fewer stable conditions than a low value, despite the higher shear strength. The quotient (*Jw/SRF*) is a complicated empirical factor describing the 'active stress'.

RMR System

In RMR classification, the rock mass along a tunnel route is divided into a number of structural regions, i.e. zones in which certain geological feature are more or less uniform. The above six classification parameters are determined for each structural region from measurements in the field. Once the classification parameters are determined, the ratings are assigned to each parameter. In this respect the typical, rather than the worst conditions, are evaluated. Furthermore, it should be noted that the ratings, which are given for discontinuity spacings, apply to rock masses having three sets of discontinuities. Thus, when only two sets of discontinuities are present, a conservative assessment is obtained.

The ratings of six parameters of the RMR system are provided by Bieniawski in 1973. For reducing doubts due to subjective judgments, the ratings for different parameters should be given a range rather than a single value.

3.2.5.2 Analytical Method

The rock support is estimated from E Hoek et. al. (1993) stress analysis. This hypothesis is highly recommended for the overstressed tunnel excavation. In this approach pressure were further more used to calculate total inward displacement at the respective section. The calculation of critical support pressure p_{cr} at each section is done using hydrostatic pressure and internal support pressure. If the internal support pressure is greater than the critical support pressure, no failure occurs and the behavior of the rock mass surrounding the tunnel is elastic. The radius of plastic zone was calculated and further more total inward radial elastic displacement of the tunnel wall is calculated and compared with the displacement from numerical modeling. The closer value in these comparisons is taken for the conclusion.

These pressures were further more used to calculate total inward displacement at the respective section. Now we have to calculate critical support pressure p_{cr} at each section using hydrostatic pressure and internal support pressure calculated above.

The critical support pressure was calculated using the relation:

$$P_{cr} = \frac{2Po - \sigma cm}{1+k} \qquad \text{Eq. 3-32}$$

If the internal support pressure is greater than the critical support pressure, no failure occurs and the behavior of the rock mass surrounding the tunnel is elastic. The inward radial elastic displacement of the tunnel wall is given by:

$$u_{ie} = \frac{ro\,(1+\nu)}{E}\,(Po - Pi) \qquad \text{Eq. 3-33}$$

Where E is the young's modulus or deformation modulus and ν is the Poisson's ratio.

When the internal support pressure is less than the critical support pressure failure occurs and the radius of the plastic zone around the tunnel is given by:

$$r_p = r_o \left[\frac{2(Po(k-1)+\sigma cm)}{(1+k)((k-1)Pi+\sigma cm)}\right]^{\frac{1}{k-1}} \qquad \text{Eq. 3-34}$$

The total inward radial displacement of the walls of the tunnel is given by:

$$u_{ip} = \frac{ro\,(1+\nu)}{E}\,[2(1-\nu)(Po-Pcr)(\frac{rp}{ro})^2-(1-2\nu)(Po-Pi)] \qquad \text{Eq. 3-35}$$

3.2.6 Numerical Modeling

Finite Element Methods (FEMs) comprising numerical approaches are useful for analysis of underground stresses and loading situations. The methods are used to calculate approximate ranges of a solution and generate two-dimensional (2D) or three-dimensional (3D) models (Kim & Yoo, 2002). The software models simulate ground-support interactions including tunnel deformation and construction sequences (RTM, 2009; Mohammed, 2015). This study used Rocscience Phase^2 software FEM package because of its ability to replicate existing geomechanical properties, inbuilt tutorials and examples and its suitability to model rock mass of various discontinuous excavated in 13 m wide spans (Crouch & Starfield, 1984). Furthermore, Phase^2 is recommended by the British Tunneling Society and the Institution of Civil Engineers and was used by Panda et al. (2014) to investigate the stability of tunnels at an operational hydropower plant in India (Tunnel Lining Design Guide, 2004) and many other projects. Phase^2 models are solutions for the progressive rock mass failure based on the Hoek-Brown failure criterion (Hoek, 2016).

There are two method of modeling under Phase^2 i.e. Core Replacement Method and Load Factor Method. Stress, deformation and stability of tunnel was determined using Phase^2. The tunnel alignment was divided into ten sections on the basis of change of rock type pattern and overburden. Field stress can be entered as gravity or constant value in the software. In this study, field stress is in the form of gravitational stress and calculated as product of unit weight of rock mass and overburden depth for σ_1 and σ_1 times active pressure coefficient of rock mass to estimate σ_3. The head race tunnel was designed using load factor method. Load factor method was used for analysis so internal pressure was applied in the model. Internal pressure is applied normal to the boundary and varies with stage with maximum value equal to in-situ stress and minimum value equals to zero. Maximum iteration is kept 500 with tolerance level of 0.01 which is expected to give significantly reliable results. Analysis of failure was performed using Generalized Hoek-Brown criterion.

The finite element analysis is carried out in three stages using Phase^2. The first stage is a consolidation stage in which the model, with no excavation present, is allowed to deform while being loaded by the in-situ stress field. In the second stage, after excavation of the tunnel of radius (R_t) of 2.1m, a uniform support pressure is applied to the tunnel boundary to control the closure of the tunnel. In the third step the internal pressure is removed a support is proposed. The following three steps is performed for the analysis of underground excavation:

a. The amount of tunnel wall deformation prior to support installation is determined.

In this method, a model of tunnel is built to determine the deformation far from the tunnel face using a simple plane strain analysis and to determine radius of plastic zone.

b. The internal pressure that yields the amount of tunnel wall deformation at the point of and prior to support installation is determined.

A uniform distributed load to the tunnel is added such that the magnitude and direction of the load will be equal and opposite to the in-situ stressed. There will be no deformation since the pressure is equal and opposite to the in-situ stress. Afterward, with suitable factor, the magnitude of the pressure is gradually reduced such that tunnel deformation will increase as the pressure is lowered to zero. Ten stages are considered in this analysis and the factor for each stage are diminished by 50% such that each stage has 1, 0.8, 0.4, 0.2, 0.1, 0.08, 0.04, 0.02 and 0.01 which will decrease to 0 at the final stage.

c. The support is assessed and it is checked whether i) the tunnel is stable, ii) tunnel wall deformation meets the specified requirements, and iii) the tunnel lining meets certain factor of safety requirements.

Care is taken that tunnel closure is not more that 4% of the tunnel span after installation of support. Support capacity diagram is generated for determining the factor of safety of the shotcrete and steel rib support. For a given factor of safety, capacity envelopes are plotted in axial force versus moment space and axial force versus shear force space. Values of axial force, moment and shear force for the liners are then compared to the capacity envelopes. The computed liners values must fall inside an envelope so that they have a factor of safety greater than envelop value.

Some steps are listed below to prepare model in phase^2 rock science software.

Project Setting

Project setting is very first and essential part of Project Modeling, this take analysis type, solver types, unit system, stage of excavations, ground water condition, project description.

Excavation Boundaries formation

After project setting, we need to create tunnel excavation boundary with tunnel cross-sections. We must add external boundary three times bigger than excavation boundary.

Material identifying

Now the important is to define material as per available data from the site. We define the ground material/rock type first and then assign to the model as same as field condition. Defined rock material properties such as unit weight, strength, specific gravity, poisons ration etc. are taken from project detailed design report.

Defining discontinuities

Various discontinuities such as fault, fold, joints etc. are defined on the model as per geological survey data i.e. face map at different chainage.

Loading condition

Before applying loading at the model, we divide the model in various meshes, mesh setup is done with graded mesh and three Nodded Triangle's support on all the sites after discretizing. Than field load is applying through gravity.

Calibration

Calibration is done through verifying of displacement and stress concentrated at the opening with field testing data.

Applying Support System

We should apply support system design before using various empirical methods. First, we should define a support material, properties of the material such as diameter of rock bolt, length of rock bolt, modulus of elasticity of bolt material, tensile strength of rock bolt, spacing of rock bolt etc. were taken from the detailed design report of the SMHEP. Some mechanical properties may be taken from the manufacturing company for the references.

Analysis: Now finally computation of the model was done and various result should be interpreted. On the basic of various result from the analysis we can recommend final support system for the tunnel.

3.2.7 Block Stability Analysis

The wedge stability analysis was done using rock science software UNWEDGE. The analysis of wedges formed at every section are very important for the stability of excavation. Most existing algorithms for underground wedge stability analysis assume that stresses are sufficiently low and can therefore be ignored. If the in-situ stress is low than we have to neglect the wedge but for the deep excavation it needs importantly to analyze the wedge analysis. In this research work, wedge analysis was done for each section and analyze whether they are stable by applying the minimum support as designed.

3.2.8 Squeezing prediction

Squeezing ground conditions refer to large convergence of excavations occurring during excavation and that may continue over time. These conditions are encountered in tunneling drives in poor quality or weak rock but also in structurally defined rock masses. In this portion the tunnel section from 1+000m to 1+400m was analyzed for the squeezing and that was study for safe support system which are minimum as designed earlier.

Squeezing problem seen in tunnel alignment were analyzed by Empirical, Semi-Analytical and Numerical methods. In Empirical methods Singh et al. (1992) and Goel (1994) approaches were used. It was also analyzed by Semi Analytical Method, Hoek and Morinas (2000) method. Which was found quite more acceptable and is recommended by different scientists. Finite Element Method (FEM) using RS2 computer software was used for the Numerical analysis.

3.2.9 Report Preparation

The required support system selected should be minimum results from these independent solutions. The support system designed should give safe and economical project completion.

4 STUDY AREA AND PROJECT DESCRIPTION

4.1 Location of the Project

Super Madi Hydroelectric Project is located in Kaski Districts (Figure 4-1), Gandaki Provience of Nepal. The project lies in the Namarjun and Parche Village of Kaski District. The headworks is located at the foothill of Sikles village and the powerhouse is located just opposite of Sodha village. The project lies about 23 km north-east of Pokhara. At present there is about 16km long earthen road access towards the project from Pokhara City.

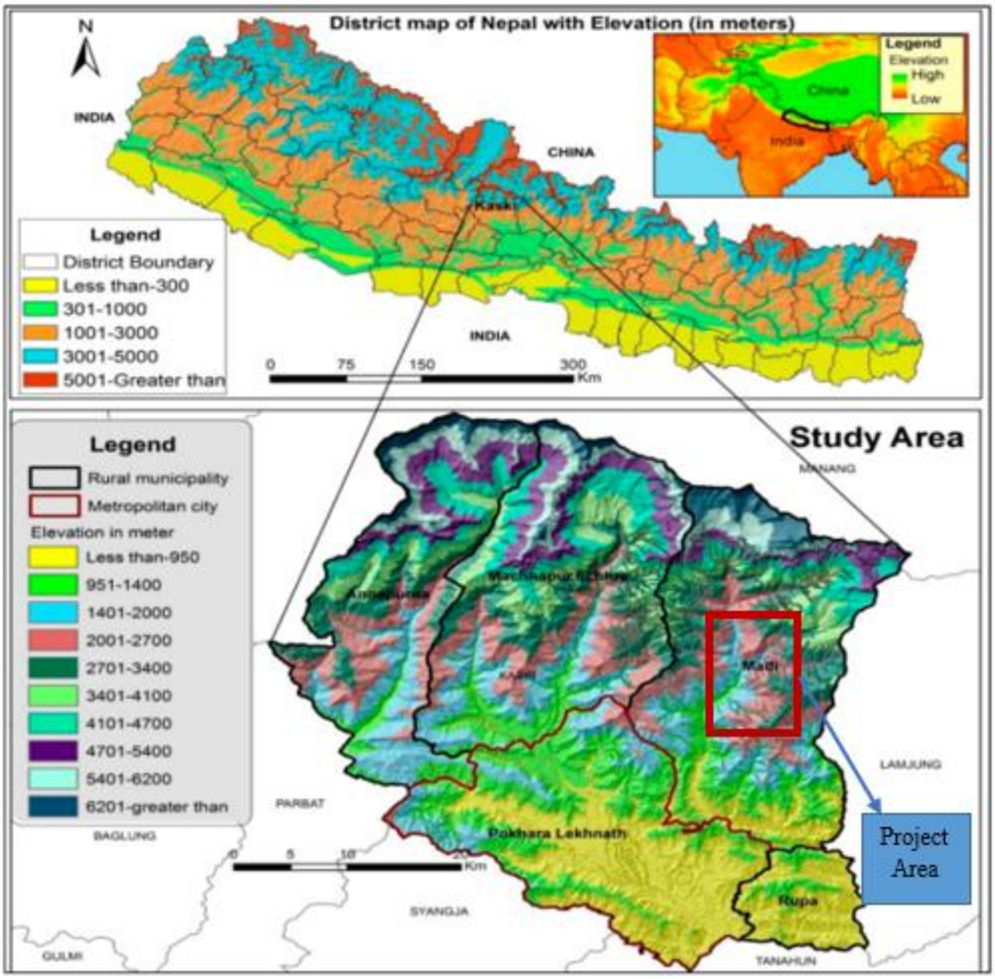


Figure 4-1 Location Map of Project Area

4.2 Hydrology

Madi River is a perennial river which is one of the major tributaries of Seti river and ultimately Trishuli River. Madi River originates from Annapurna II, IV and Lamjung Himal with an elevation from about 7937 m amsl and meets Seti river after flowing about 52 km southwards from the proposed powerhouse location. The entire catchment area of Madi River is of fern shape type with dendritic drainage pattern. The catchment area includes dense forest, cultivated area, barren land and rocky and snowy mountain including Himalayas.

The total catchment area at proposed headwork's location is 278.136 sq. km. (Figure 4-2) Out of the total area 63.742 sq.km. lies above 5000 amsl, 135.51 sq.km. lies between 3000-5000m amsl and 78.884 sq.km. below 3000 m amsl. The average elevation of the catchment area is 4648 m amsl. The annual precipitation of the catchment area has been calculated as 3194 mm.

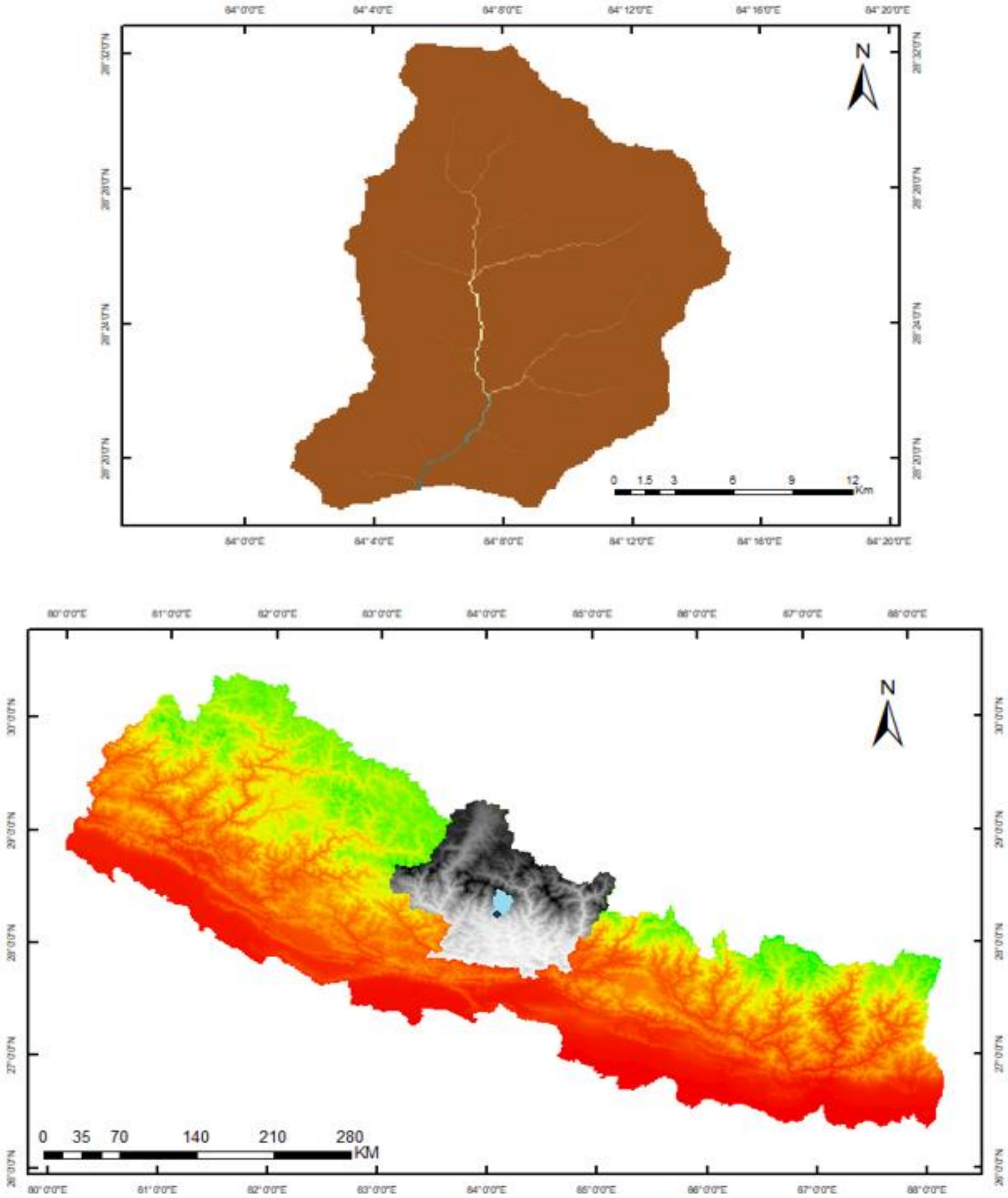


Figure 4-2 Catchment Area of Study Project

Flows at Madi River is strongly influenced by the monsoon climate with high flows during summer monsoon period of 4 months (80% of total annual rainfall) and low flows during off monsoon period of about 8 months when there will be less rainfall (20% of total annual rainfall). The period from October to November is medium flow period. The lowest monthly flow occurs in February or March. The daily flows are dominated by groundwater during the driest season. Snowmelt is usually the major source of flows in April through June.

4.3 Regional Geology

Geologically, project area lies in the Higher Himalayan succession. The Higher Himalayan is sandwiched between the Southern Tibetan Detachment System (STDS) in north and the Main Central Thrust (MCT) in south. The MCT is the major regional thrust in Himalayan which lies in about 2 km (aerial distance) south from the powerhouse area. This zone comprises mainly high-grade metamorphic rocks such as Kyanite-silliminitae bearing gneiss, schist and quartzite. Geologically the project location belongs to higher Higher Himalayan Crystalline Zone consisting of percambrian gneiss. (Figure 4-3) The main lithology of the project area is banded gneiss with the various infilling material. The banded gneisses are fresh to moderately weathered, whereas the mica gneiss is moderately to highly weathered. The overall rock mass condition of the project area is fair to good which is thickly to massively foliated, slightly fractured to highly fractured with intercalation of quartzite and schist. The geology of Nepal (Figure 4-4) is much variable from Terai region to Sewalik, Lesser Himalayan, Higher Himalayan.

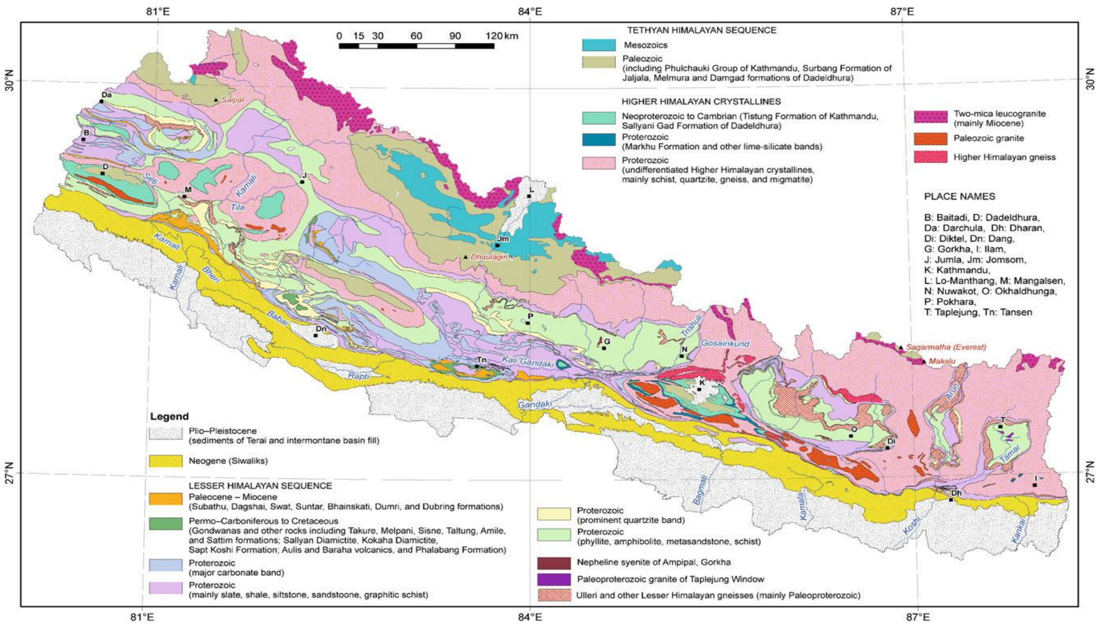


Figure 4-3 Simplified Geological map of Nepal (Dhital M.R., 2015)

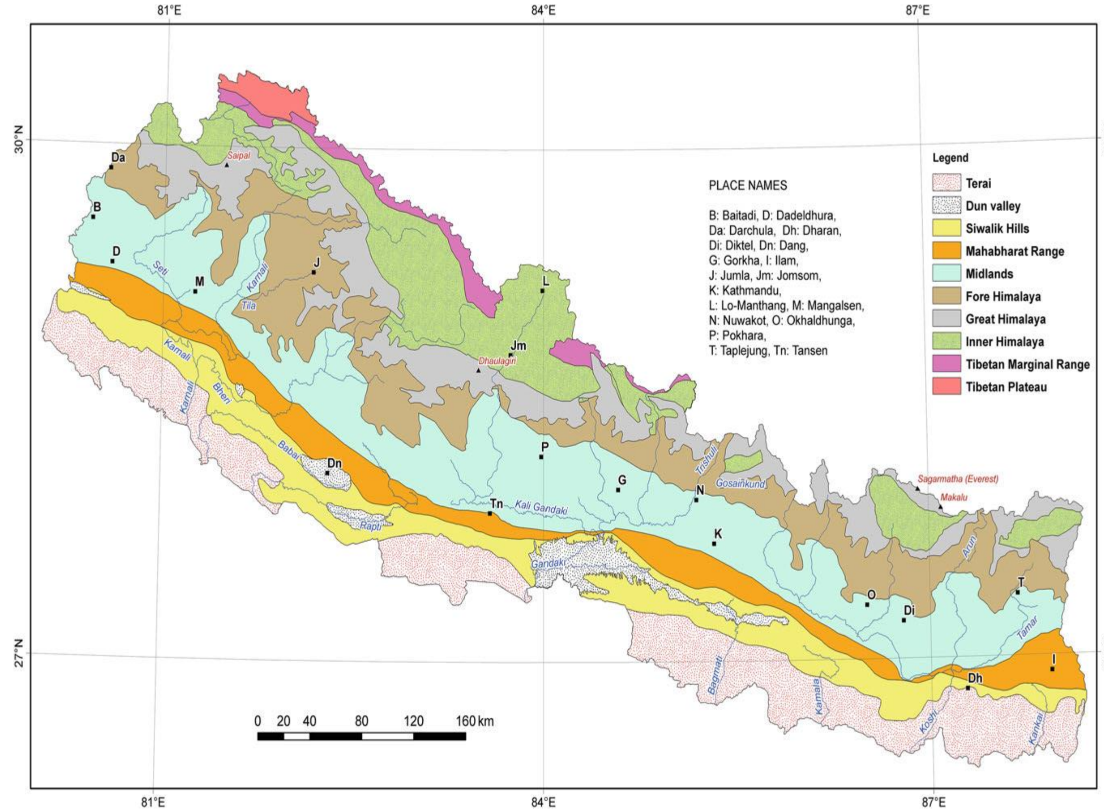


Figure 4-4 Physiographic divisions of Nepal (Dhital M.R. 2015)

4.4 Geomorphology

In the Himalayas, geo-disasters are recurrent features that generally result from the combination of several hazards (gullying, landsliding, flooding), affecting vulnerable, poorly prepared, human communities. Madi River is a snow fed perennial river, originating from Annapurna II, IV and Lamjung Himal which is one of the major tributaries of Seti River and ultimately Trishuli River. Madkyu khola is one of the main tributaries of Madi River that joins at about 800 m upstream of the proposed intake location. Madi River is about 6.7 km long from its confluence with madkyu khola at Palche to the confluence of Chipli khola at sodha. The average gradient of the river is this particular reach is about 2.5%.

Steel cliff of more than 150m are observed in major stretch of Madi River at both the banks. The project area consists of steep and rugged terrain. The occurrence of rigid gneiss in the project area has contributed to the steep and rugged terrain. The elevation of the project area 1345masl in the north to 1040m in south along the river thalweg. The major stream and spurs show east-west trend. Along most of the madi river, the hill slope is controlled by the foliation (12˚/15) and two other joint set (35˚/75 and 28˚/71), one dipping west and the other dipping south respectively.

Table 4-1 Physiographical division of the Nepal Himalaya (modified after Upreti, 1999)

SN	Geomorphic Unit	Width (km)	Altitudes (m)	Main Rock Types	landform development
1	Terai (Northern edge of the Gangetic Plain)	20-50	100-200	Alluvium: coarse gravels in the north near the foot of the mountains, gradually becoming finer southward	River deposition, erosion and tectonic upliftment
2	Churia Range (Siwaliks)	10-50	200-1300	Sandstone, mudstone, shale and conglomerate.	Tectonic upliftment, erosion, and slope failure
3	Dun Valleys	5-30	200-300	Valleys within the Churia Hills filled up by coarse to fine alluvial sediments	River deposition, erosion and tectonic upliftment
4	Mahabharat Range	10-35	1000-3000	Schist, phyllite, gneiss, quartzite, granite and limestone belonging to the Lesser Himalayan Zone	Tectonic upliftment, Weathering, erosion, and slope failure
5	Midlands	40-60	300-2000	Schist, phyllite, gneiss, quartzite, granite, limestone geologically belonging to the Lesser Himalayan Zone	Tectonic upliftment, Weathering, erosion, and slope failure
6	Fore Himalaya	20-70	2000-5000	Gneisses, schists, phyllites and marbles mostly belonging to the northern edge of the Lesser Himalayan Zone	Tectonic upliftment, Weathering, erosion, and slope failure
7	Higher Himalaya	10-60	>5000	Gneisses, schists, migmatites and marbles belonging to the Higher Himalayan Zone	Tectonic upliftment, Weathering, erosion (rivers and glaciers), and slope failure

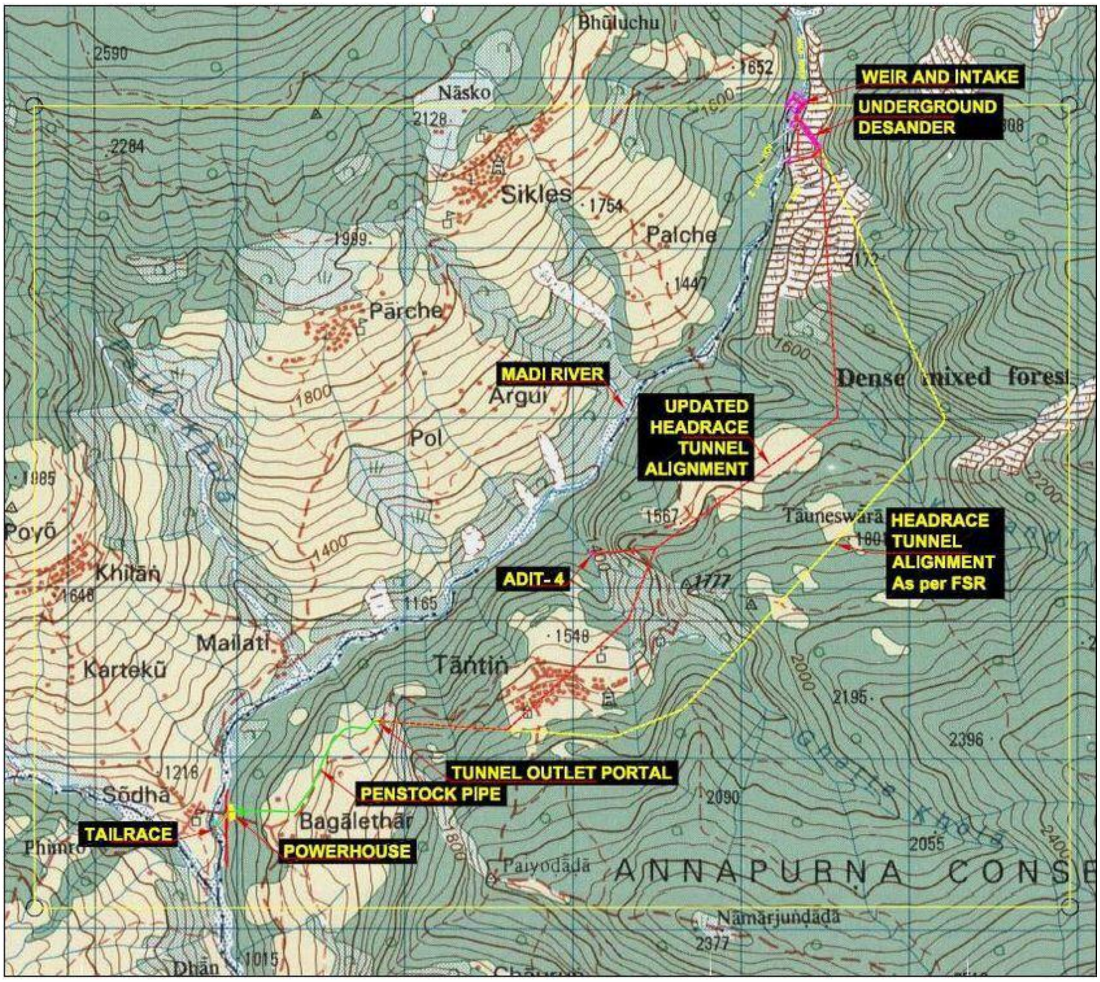


Figure 4-5 Layout Showing Tunnel Alignment

4.5 Geological Condition of Project Components

Weir

The headworks site is selected considering the favorable location available in the area from various aspects like foundation across the weir axis, stability of both sides hill slope and sediment deposit pattern. The proposed weir location is at an elevation of 1344m and the river is about 50m wide with steep tock slope (about > 70◦) at left bank and a bit less steep slope at right bank. The exposed rock type at the right and left banks of the headwork site is predominantly banded gneiss and thin layer of schist intercalated with weak thin bands of micaceous gneiss. The rock at both banks are slightly to moderately weathered, massive to medium foliated with medium to high persistency having three sets of planners to undulating joints (including foliation plane) filled with silt and clay. The RMR and Q values of the exposed rock is about 75 to 80 and 10 to 14 respectively which is defined as good rock. No major slope instability problems are anticipated in the area.

Intake, Gravel Trap and Head pond

The side intake, gravel trap, and headpond lie partially in the recent alluvial deposit and in bedrock. The alluvial deposit comprises rounded boulders, cobles and pebbles of banded gneiss augen gneisses, quartzite and limestone. ERT result have shown that bed rock in this area can be found within the depth of 5 m to 30 m and the average hydraulic conductivity calculated by mise-a'-la-masse method is 134.45 m/day.

The exposed rock in the area comprises slightly weathered, medium to massively foliated, medium to coarse-grained, strong to very strong banded gneiss with few partings of slightly weathered medium grained, thin to thickly foliated schist. The bedrock consists of three plus random joint sets. The joints comprise medium to high persistency, planer- undulated, joint contact is slightly weathered, slightly open to tight with sandy silty clay infilling.

Settling Basin and Tunnel

The geological and topographical site conditions of headworks do not allow exposed settling basin and other required flushing arrangements. Therefore, the settling basin and all other flushing structures are proposed underground along left bank hill side.

The geological condition of the inlet tunnel area, underground settling basin, flushing tunnel and adit-1 are fair to good and favorable. The rock mass of the area is slightly deformed and foliated banded gneiss with few partings of schist. The less deformed massive rocky mountain with sufficient vertical and horizontal rock cover above and valley side is a perfect location for the underground settling basin cavern. The geological condition of the area is favorable even for bigger size caverns.

The rock is slightly weathered, massive to medium foliated having three sets of rough and irregular, undulating, tight to moderately open joints (including foliation plane) with medium to high persistency filled with silt. The RMR and Q values of the exposed rock are about 80 to 85 and 10 to 14 respectively which is defined as good rock.

Headrace Tunnel

The head race tunnel is aligned along the left hill side of Madi River. The total length of headrace tunnel is approximately 5.28km excluding settling basin. The tunnel has exposed at both banks of kalbandi kholsi which is being used as adits during construction and shall be connected by penstock pipe at the final stage with river crossing structure. The rock condition along the tunnel are interpreted by extrapolation of rock mass condition from the tributaries and foot trails in the vicinity of tunnel alignment. The rock types and their estimated attitudes that will be encountered in tunnel is summarized in table below.

Table 4-2 Description of rock mass along headrace tunnel (source: Super Madi HEP)

Chainage (meter)	Description	Major joints
(0+240) To (0+450)	The rock mass is thickly to massively foliated, light grey, strong to very strong, fresh to slightly weathered banded gneiss. Three major joints are observed with random joint. Thin to thick schist parting is also present, which will be associated as weakness zone. The rock mass forms steep cliff. The tunnel alignment is oblique with the strike of major discontinuity (foliation). The rock overburden within this stretch is between 200m to 250m.	F:120◦/13◦ J1:351◦/75◦ J2:280◦/71◦
(0+450) To (0+650)	The rock mass is thickly to massively foliated, light grey, strong to very strong, fresh to slightly weathered banded gneiss of kyanite grade. The tunnel alignment is oblique to the major discontinuity (foliation plane) with the excavation driving against dip. The rock overburden within this stretch is between 250m to 325m.	F:75◦/15◦ J1:188◦/75◦ J2:278◦/68◦
(0+650) To (0+900)	The rock mass is thickly to massively foliated, light grey, strong to very strong, fresh to slightly weathered banded gneiss with three plus random joint sets. The tunnel alignment makes oblique angle to the major discontinuity with the excavation driving against dip. The rock overburden within this stretch is between 300m to 325m.	F:095◦/17◦ J1:340◦/70◦ J2:260◦/65◦
(0+900) To (1+300)	The tunnel alignment makes less angle with the strike of major discontinuity with the excavation driving against dip. The rock overburden within this stretch is between 200m to 300m.	F:082◦/15◦ J1:181◦/75◦ J2:278◦/68◦
(1+300) To (1+600)	The rock mass is thickly to massively foliated, light grey, strong to very strong, fresh to slightly weathered banded gneiss. Three major joint sets along with other joints shall be encountered. The rock overburden within this stretch is between 15m to 225m.	F:134◦/32◦ J1:344◦/81◦ J2:278◦/57◦
(1+600)	The rock mass is thickly to massively foliated, light grey, strong to very strong, fresh to slightly weathered banded	F:051◦/27◦

To (1+900)	gneiss. Three major joint sets. Shear zones shall be encountered at some stretches. The tunnel alignment makes oblique to the major discontinuity(foliation). The rock overburden within this stretch is between 15m to 150m.	J1:272◦/62◦ J2:202◦/48◦
(1+900) To (2+900)	The rock mass are not exposed witin the area. The tunnel alignment makes less angle to the major discontinuity with the excavation driving towards dip. The rock overburden within this stretch is between 150m to 450m.	F:134◦/32◦ J1:344◦/81◦ J2:278◦/57◦
(2+900) To (3+400)	The rock mass is thickly to massively foliated, light grey, strong to very strong, fresh to slightly weathered banded gneiss with few partings of slightly weathered medium foliated schist. The tunnel alignment is perpendicular to the major discontinuity with the excavation driving towards dip. The rock overburden within this stretch is between 200m to 450m.	F:77◦/12◦ J2:182◦/72◦ J3:269◦/65◦
(3+400) To (3+700)	The rock mass are not exposed in the area. The tunnel alignment is almost perpendicular to the major discontinuity (foliation).The rock overburden within this stretch is between 200m to 250m.	F:075◦/15◦ J1:275◦/70◦ J2:180◦/65◦
(3+700) To (4+000)	The rock mass are not exposed in the area. The tunnel alignment makes oblique with the strike of the major discontinuity (foliation).The rock overburden within this stretch is between 240m to 300m.	F:134◦/32◦ J1:344◦/81◦ J2:278◦/57◦
(4+000) To (4+700)	The rock mass consist of thick to massive foliated gneiss and garnetiferous schist. The tunnel alignment makes oblique angle with the strike of the major discontinuity (foliation).The rock overburden within this stretch is between 200m to 300m.	F:109◦/33◦ J2:344◦/81◦ J3:278◦/57◦
(4+700) To (5+300)	The rock mass is thickly to massively foliated, light grey, strong to very strong, fresh to slightly weathered banded gneiss with three major joint sets. Foliation is major discontinuity. The rock overburden within this stretch is between 100m to 250m. The tunnel alignment makes very low angle with the strike of the major discontinuity (foliation).	F:115◦/21◦ J2:320◦/60◦ J3:240◦/57◦

4.6 The Study Section

The selected project is Super Madi Hydro-electric Project with installed capacity 44 MW to generate annual average energy of 242.65 GWh. This project has Headrace tunnel of length 5282.31 m inverted D shaped. This headrace tunnel is designed for low pressure flow. The above length of the headrace tunnel is excluding of settling basin, adits, inspection tunnel, flushing tunnels, kalbandi kholsi crossing, approach tunnel and tunnel stretch before vertical drop, surge shaft connecting tunnel. The excavation is in the section of 1+000m to 1+400m while doing this research.

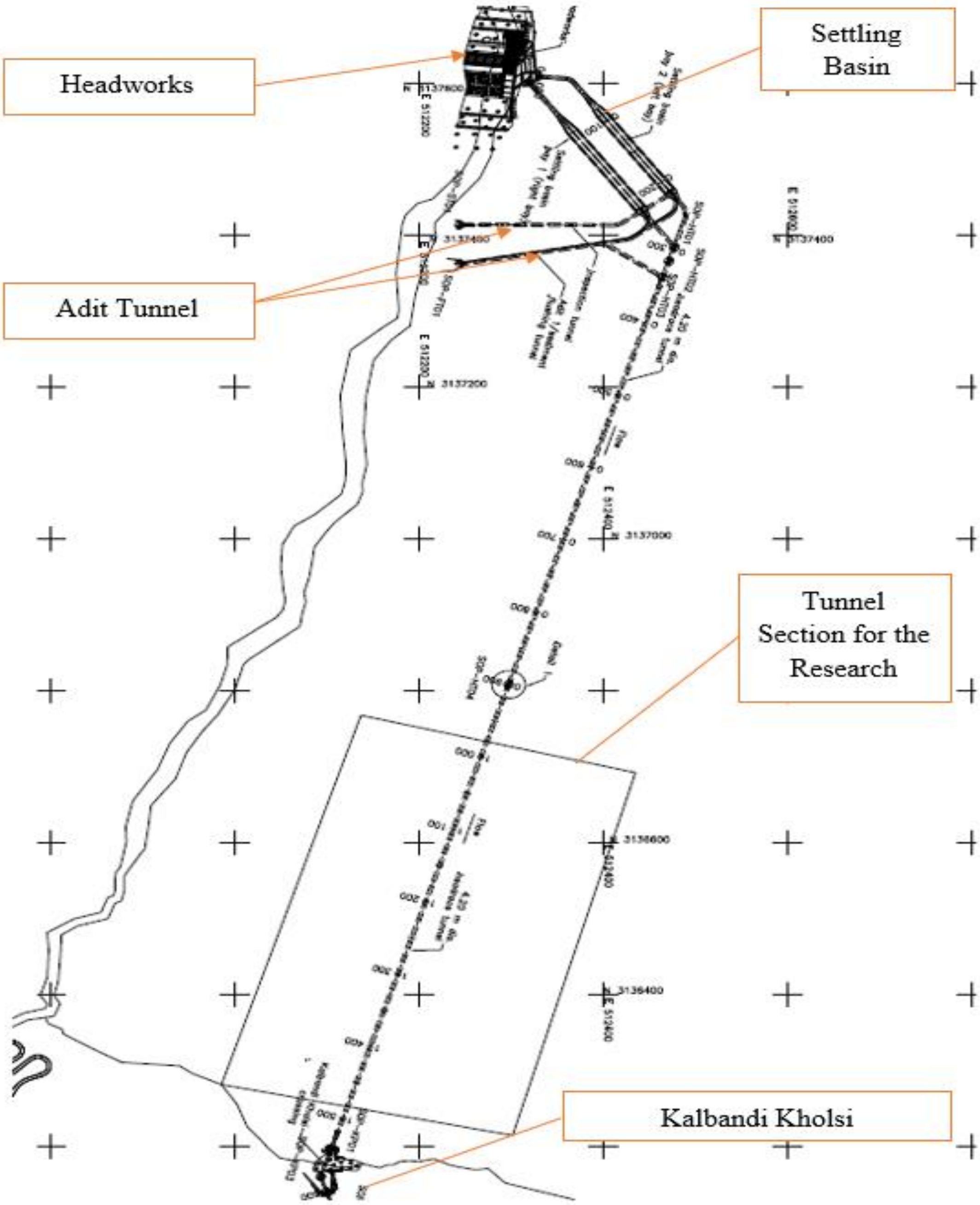


Figure 4-6 Selected Tunnel Section for the Study

The tunnel excavation size (Figure 4-6) without pay line is 4.2 m*4.2 m (B*H). The elevation of the tunnel section of this study varies from 1300.00 m to 1350.00m above mean sea level.

The tunnel is exposed at the bank of Kalbandi Kholsi which is also be used as adits during construction and shall be connected by penstock pipe at the final stage with river crossing structure.

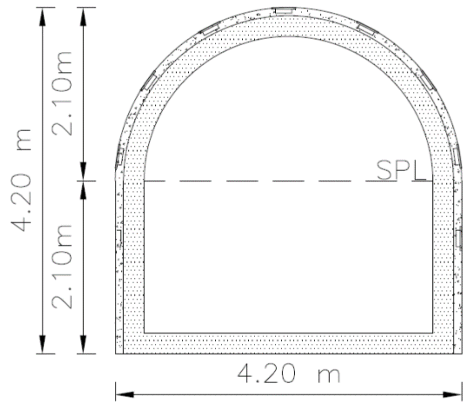


Figure 4-7 Tunnel Section

Overburden in the selected section for this study varies from 116.66m at Kalbandi Kholsi to 306.42 m at 500m upstream from the Kholsi.

4.6.1 Geology

The properties of the discontinuities along the study section was collected from the site office and were check to some section. The properties are listed in Table 4-3. Planes of weakness in rock are formed through failure in extension/tension, shear or in more complex failure modes that involve a combination of both. Failure surfaces formed in shear are usually smooth with some gouge material whereas failure surfaces formed in extension are rough and usually clean. Once formed, planes of weakness are more susceptible to weathering than the intact rock.

Table 4-3 Properties of Discontinuities along study section

Chainage	Discontinuities	Geological Description
1+000	F/J1 80˚/20˚	The Excavated tunnel face consists gray colored, medium grained, foliated, slight to moderately weathered, medium strong, banded Gneiss with quartz veins parallel to the foliation plane. The rough, planar, moderately weathered joints with fair RQD, have tight to few (1-3) mm aperture with clay fillings in some prominent joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.
	J2: 310˚/40˚	
	J3: 260˚/60˚	
1+050	F/J1: 90˚/15˚	The Excavated tunnel face consists gray colored, medium grained, foliated, slight to moderately weathered, medium strong, banded Gneiss with quartz veins parallel to the foliation plane. The rough, planar, moderately weathered joints with fair RQD, have tight to few (1-3) mm aperture with clay fillings in some prominent joints. Joints are closely to moderately spaced and have medium to high
	J2:10˚/60˚	
	J3:280˚/30˚	

		persistency. Surface water condition of the area is damp.
1+100	F/J1: 90˚/20˚ J2: 20˚/60˚ J3: 340˚/45˚	The Excavated tunnel face consists gray colored, medium grained, foliated, slight to moderately weathered, medium strong, banded Gneiss with quartz veiss parallel to the foliation plane. The rough, planar, moderately weathered joints with fair RQD, have tight to few (2-3) mm aperture with clay fillings in some prominent joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.
1+150	F/J1: 95˚/20˚ J2: 185˚/65˚ J3:220˚/80˚	The Excavated tunnel face consists gray colored, medium grained, foliated, highly weathered, weak, banded Gneiss with quartz veins parallel to the foliation plane. The individual beds are 20-40 cm thick. High amount of clay coating and filling can be seen on crown part and upper tunnel face. The rough, planar, moderately weathered joints with poor RQD, have tight to few (3 mm) aperture with clay fillings in some most of the joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is dry to damp.
1+200	F/J1: 85˚/20˚ J2: 300˚/50˚ J3: 180˚/75˚	The Excavated tunnel face consists gray colored, medium grained, foliated, highly weathered, weak, banded Gneiss with quartz veins parallel to the foliation plane. High amount of clay coating and filling can be seen on crown part and upper tunnel face. Below the spring line, clay filling upto 1m can see. The rough, planar, moderately weathered joints with poor RQD, have tight to few (2 mm) aperture with clay fillings in most of the joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.
1+250	F/J1: 100˚/20˚ J2: 170˚/60˚ J3: 240˚/85˚	The Excavated tunnel face consists gray colored, medium grained, foliated, highly weathered, weak, banded Gneiss with quartz veins parallel to the foliation plane. The individual beds are 20-40 cm thick. High amount of clay coating and filling can be seen on crown part and upper tunnel face. The rough, planar, moderately weathered joints with poor RQD, have tight to few (3 mm) aperture with clay fillings in most of the joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.
1+300	F/J1: 110˚/20˚	The Excavated tunnel face consists gray colored, medium grained, foliated, Low weathered, Medium

	J2: 280˚/40˚ J3: 95˚/80˚	to strong, banded Gneiss with quartz veins parallel to the foliation plane. The individual beds are 20-100 cm thick. The rough, planar, moderately weathered joints with good RQD, have tight to few (1 mm) aperture with clay coating in few joints. Joints are closely to moderately spaced and have medium persistency. Surface water condition of the area is dry to damp.
1+350	F/J1: 95˚/25˚ J2: 280˚/45˚ J3: 175˚/85˚	The Excavated tunnel face consists gray colored, medium grained, foliated, moderately weathered, Medium strong to strong, banded Gneiss with quartz veins parallel to the foliation plane. The individual beds are 20-80 cm thick. The rough, planar, moderately weathered joints with fair RQD, have tight to few (1-2 mm) aperture with clay coating in few joints. Joints are closely to moderately spaced and have medium persistency. Surface water condition of the area is dry to damp.
1+400	F/J1: 100˚/20˚ J2: 220˚/65˚ J3: 285˚/80˚	The Excavated tunnel face consists gray colored, medium grained, foliated, moderately weathered, Medium strong to strong, banded Gneiss with quartz veins parallel to the foliation plane. The individual beds are 10-60 cm thick. The rough, planar, moderately weathered joints with fair RQD, have tight to few (1-3 mm) aperture with clay coating and filling in few joints. Joints are closely to moderately spaced and have medium persistency. Surface water condition of the area is dry to damp.

4.6.2 Geotechnical Data

Overburden

Tunneling under high overburden stresses results in many tunnel instability problems due to the rock overstressing. Understanding and simulating the rock failure process is the major issue of a deep excavation to achieve an appropriate rock support system that provides possible cost-effective and stable construction. Overburden pressure is a geological term that denotes the pressure caused by the weight of the overlying layers of material at a specific depth under the earth's surface. Overburden pressure is also called lithostatic pressure, or vertical stress. Overburden stress (σ_v) is the pressure exerted on a formation at a given depth due to the total weight of the rocks and fluids above that depth.

The "roof" of the tunnel, or the top half of the tube, is the crown. The bottom half is the invert. The basic geometry of the tunnel is a continuous arch. Because tunnels must withstand tremendous pressure from all sides, the arch is an ideal shape. It is the stress that will include the pressure coming from the soil, water in pores and from the external load. When the load is applied to the soil, it transfers the load to water in the pores and soil grains. It increases with the increasing depth of soil. It is denoted using the term σ_v.

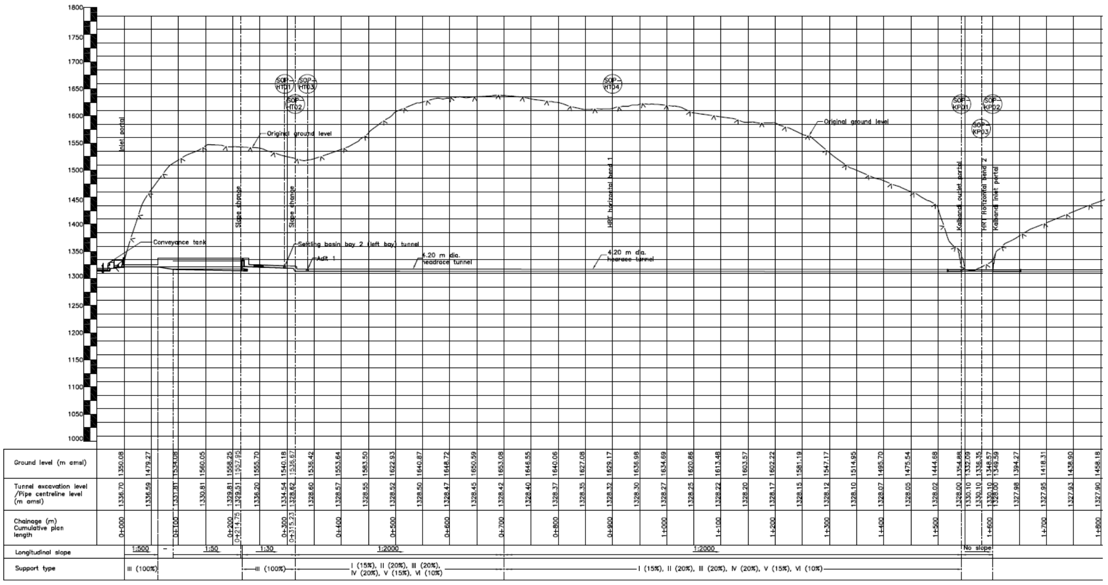


Figure 4-8 Ground Profile of Study Section

The overburden height of the ground above tunnel section is calculated using the profile shown in Figure 4-8 by subtracting tunnel section level from the ground surface and that was shown in Table 4-4.

Table 4-4 Overburden Calculation of Study Section

Chainage	Ground Level (masl)	Tunnel Excavation Level (masl)	Overburden (m)
1+000	1634.69	1328.27	306.42
1+050	1620.86	1328.25	292.61
1+100	1613.48	1328.22	285.26
1+150	1603.57	1328.20	275.37
1+200	1602.22	1328.17	274.05
1+250	1581.19	1328.15	253.04
1+300	1547.17	1328.12	219.05
1+350	1514.95	1328.10	186.85
1+400	1495.70	1328.07	167.63

4.6.3 Rock Mass Classification

<u>Calculation of Q-Value</u>

All the data were collected from the geological survey conducted by site office. Understanding to measure all the parameter required to calculate Q-value the site visit was conducted frequently. Table 4-5 shows that value along the study section which describe the rock mass class which was also noted on that table. This result shows the extremely variation along the study section from extremely poor rock to poor rock. The studied section was poor rock category that need much support and careful construction mechanism.

Table 4-5 Q-Value of Study Section

Chainage	RQD	JN	JR	JA	JW	SRF	Q value	Rock Class	Remarks
1+000	55	12	1.5	4	1	5	0.344	E	Very Poor
1+050	70	12	1.5	3	1	5	0.583	E	Very Poor
1+100	55	12	1.5	6	1	5	0.229	E	Very Poor
1+150	45	15	1.5	6	1	10	0.075	F	Extremely Poor
1+200	30	15	1.5	8	1	10	0.038	F	Extremely Poor
1+250	45	12	1.5	8	1	10	0.070	F	Extremely Poor
1+300	75	12	1.5	3	1	2.5	1.250	D	Poor
1+350	65	12	1.5	3	1	2.5	1.083	D	Poor
1+400	65	12	1.5	6	1	5	0.271	E	Very Poor

These Q-value is to be applied for the estimation of rock support. The rock mass quality (Q) is a very sensitive index and its value varies from 0.001 to 1000. Use of the Q-system is specifically recommended for tunnels and caverns with an arched roof.

<u>Calculation of RMR Value</u>

All the RMR value were calculated using the geological description provided by the site office. All the description was closely studied and the rating value was calculated from the RMR classification of rock masses (Bieniawski, 1989). These values describe the rock mass class which was also shown in Table 4-6. The rock class found in that section was varies from very poor rock to fair rock which is similar classification as Q-value. Very poor rock class of category V from section 1+150 to 1+250 m. these values are furthers used for the support estimation which was described later in this chapter.

Table 4-6 RMR Calculation of Study Section

Chainage	Rating											Rock Mass Class	Remark
	Strength of Intact Rock Material	RQD	Spacing of Discontinuities	Condition of Discontinuities					Ground Water Condition	Adjustment for Orientation	Total		
				Length, Persistence	Separation (aperture)	Roughness	Infilling	Weathered					
1+000	4	8	8	1	1	3	2	3	10	-5	35	IV	Poor
1+050	4	8	8	1	1	3	2	3	10	-5	35	IV	Poor
1+100	4	8	8	1	1	3	2	3	10	-5	35	IV	Poor
1+150	2	5	5	0	0	1	0	1	10	-5	19	V	Very poor
1+200	2	5	5	0	0	1	0	1	10	-5	19	V	Very poor
1+250	2	5	5	0	0	1	0	1	10	-5	19	V	Very poor
1+300	7	13	10	2	4	5	2	5	15	-5	58	III	Fair
1+350	7	13	10	2	4	5	2	5	15	-5	58	III	Fair
1+400	4	8	8	1	1	3	2	3	10	-5	35	IV	Poor

5 RESULT AND DISCUSSION

The term rock support is the term widely 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. Tunnels are generally grouped in four broad categories, depending on the material through which they pass: soft ground, consisting of soil and very weak rock; hard rock; soft rock. Three main types of primary support systems are presently used in rock.

The stretch of tunnel was excavated during the time of this study and has observed weak rock mass quality and challenges difficulty in stabilizing the tunnels with the designed rock support. This chapter includes tunnel stability analysis and numerical modelling of three typical sections in this stretch of tunnel.

5.1 Estimation of Rock Mass Properties

Rock mass properties such as Hoek–Brown constants, deformation modulus of rock masses and uniaxial compressive strength of rock mass were calculated in accordance with Q_c, Q_N, Q, RMR, RMi. Factors affecting the stress problems are rock mass properties such as jointing systems, strength properties, anisotropy, and elastic properties. Orientation of major principle stress relative to the direction of major joint sets and structural features, such as bedding and schistosity have a major influence on rock bursting and spalling. For gneiss, tunnel section having rich in mica are often characterized by stress relief, while the rock burst is confined to more quartz and feldspar rich sections (Panthi 2006).

The rock mass properties are presented in, Table 5-1 Table 5-2 and Table 5-3, UCS for the rock mass and deformation modulus of rock mass using various empirical analysis.

For SMHEP, there is no uni-axial compressive strength test data in the settling basin area. Panthi (2006) gives some good approximation of the UCS value of intact rock. These values are used for the guidance to estimate the UCS value of intact rock in Himalayan region. He found the UCS value for Banded rock is between 50 to 100 MPa and the value for schistose gneiss is 35 MPa.

Rock mass strength is defined as an ability to withstand stress and deformation. It is influenced by the foliation, schistosity, discontinuity and the orientation of structural features. Strength of rock mass and the intact rock have vast variation due to the nonhomogeneity, anisotropic and discontinuity of the rock mass (Bieniawaski and Van Heerden (1975)).

For SMHEP, banded gneiss is the metamorphic rock with few intercalations of mica schist. Therefore, for this type of schistose banded gneiss, value of M_i is taken as 28 for further analysis as an input parameter. The disturbance factor is taken as 0.5 to 0.8 for the poor blasting method. The Laboratory test for various mechanical properties like Poisson`s ration, bulk density and modulus of elasticity are not carried out. Therefore, those parameters have taken from the similar projects where laboratory test has been carried out. In this respect, UTKHEP and SMHEP have many geological similarities such as rock type (Banded gneiss) and geological location (Higher Himalaya). Panthi (2006) and Neupane (2010) suggested the various mechanical properties for the similar geological condition. The poisson's ratio is taken 0.2 (Neupane 2010) and bulk density 2.68 g/cm^3 (panthi, 2006) for the further analysis.

Excavation and geometry of the opening incorporate the stress distribution around the opening. These stresses are the resultant of the vertical stress caused by gravity, tectonic stresses, topographic and residual stresses. These vertical stresses are changed after the excavation. Vertical stress is induced because of the overlying strata. If we assumed the homogenous and isotropic rock mass the vertical stress due to the overlying strata, (gravitational stress) is calculated by using following relation.

$$\sigma_v = \gamma Z \qquad \text{Eq.5-1}$$

Where,

γ =specific weight of the overlying rock mass (MN/m^3)

Z=Depth of overburden (m)

Mathematical form of total horizontal stress is as follows:

$$\sigma h \frac{\upsilon}{1-\upsilon}\ \sigma_v + \sigma_{tec} \qquad \text{Eq. 5-2}$$

In SMHEP, there is not any test carried out to measure the tectonic stresses. this was neglected in this research. Sheorey (1994) suggested a relation to evaluate the value of `K` by considering the curvature of the crust and variation of elastic constants, density, and thermal expansion co-efficient through the crust to mantle. According to him:

$$K = 0.25 + 7E_h\,(0.001+1/Z) \qquad \text{Eq. 5-3}$$

Where E_h is the average deformation modulus of the upper part of the earth crust measured in a horizontal direction and Z (m) is the depth below the surface. For SMHEP, E_h is taken as 3.97 GPa (Gautam 2012) for banded gneiss.

Table 5-1 Rock Mass Properties

Rock Type/ Group	Chainage	H	GSI	γ	ν	m_i	σ_{cm}	E_{rm}	Disturbance Factor	m_b	s	a	σ_v	k	σ_h
		(m)	(value)	(MN/m^3)			(MPa)	(GPa)					(MPa)		(MPa)
Very Poor (G01)	1+000	306.42	34	0.026	0.20	28	2.835	3.872	0.8	0.5637	4.82E-05	0.5166	8.212	0.368	2.053
	1+050	292.61	39	0.026	0.20	28	3.325	4.200	0.8	0.7484	9.91E-05	0.5120	7.842	0.373	1.960
	1+100	285.26	31	0.026	0.20	28	2.601	3.676	0.8	0.4537	2.77E-05	0.5213	7.645	0.375	1.911
Extremely Poor (G02)	1+150	275.37	21	0.026	0.20	28	0.527	1.260	0.5	0.6411	2.55E-05	0.5418	7.380	0.379	1.845
	1+200	274.05	14	0.026	0.20	28	0.491	1.117	0.5	0.4763	1.11E-05	0.5634	7.345	0.379	1.836
	1+250	253.04	20	0.026	0.20	28	0.523	1.244	0.5	0.6236	2.36E-05	0.5434	6.781	0.388	1.695
Poor (G03)	1+300	219.05	46	0.026	0.20	28	9.303	11.972	0.8	1.1257	2.80E-04	0.5075	5.871	0.405	1.468
	1+350	186.85	45	0.026	0.20	28	8.963	11.574	0.8	1.0426	2.30E-04	0.5082	5.008	0.427	1.252
Very Poor (G01)	1+400	167.63	32	0.026	0.20	28	2.686	3.752	0.8	0.4961	3.48E-05	0.5192	4.492	0.444	1.123

Table 5-2 Calculation of UCS for Rock Mass

Rock Type/ Group	Chainage	H (m)	Q - Value	RMR	GSI	σ_{ci}	σ_{cm}							Remarks
						(MPa)	Hoek & Brown (1980b)	Yuhbir et al(1983)	Ramamurthy (1986)	Kalamaris & Bieniawski (1995)	Sheorey (1997)	Barton (2002)	Average	
Very Poor (G01)	1+000	306.42	0.344	35	30	75	2.0266	0.519436	2.34157	4.999	2.9081	4.2152	2.83492	
	1+050	292.61	0.583	35	30	75	2.0266	0.519436	2.34157	4.999	2.9081	7.1531	3.32457	
	1+100	285.26	0.229	35	30	75	2.0266	0.519436	2.34157	4.999	2.9081	2.8102	2.60074	
Extremely Poor (G02)	1+150	275.37	0.075	19	14	35	0.3888	0.071279	0.46550	1.198	0.6098	0.4292	0.52703	
	1+200	274.05	0.038	19	14	35	0.3888	0.071279	0.46550	1.198	0.6098	0.2146	0.49127	
	1+250	253.04	0.070	19	14	35	0.3888	0.071279	0.46550	1.198	0.6098	0.4024	0.52256	
Poor (G03)	1+300	219.05	1.250	58	53	75	7.2729	3.017679	7.98439	13.033	9.1842	15.3281	9.30339	
	1+350	186.85	1.083	58	53	75	7.2729	3.017679	7.98439	13.033	9.1842	13.2844	8.96277	
Very Poor (G01)	1+400	167.63	0.271	35	30	75	2.0266	0.519436	2.34157	4.999	2.9081	3.3211	2.68590	

Table 5-3 Deformation Modulus of Rock Mass

Chainage	H	GSI	Q	RMR	γ	ν	Em					Average Em
	(m)	(value)			(MN/m³)		Bieniawski (1978)	Serafim and Pereira (1983)	Grimstad and Barton (1993)	Read et al.	Hoek & Brown (1998)	
1+000	306.42	30	0.344	35	0.026	0.20	-	4.217	-	4.2875	2.418	3.641
1+050	292.61	30	0.583	35	0.026	0.20	-	4.217	-	4.2875	2.418	3.641
1+100	285.26	30	0.229	35	0.026	0.20	-	4.217	-	4.2875	2.418	3.641
1+150	275.37	14	0.075	19	0.026	0.20	-	1.679	-	0.6859	0.963	1.109
1+200	274.05	14	0.038	19	0.026	0.20	-	1.679	-	0.6859	0.963	1.109
1+250	253.04	14	0.070	19	0.026	0.20	-	1.679	-	0.6859	0.963	1.109
1+300	219.05	53	1.250	58	0.026	0.20	16	15.849	2.4227	19.5112	9.087	12.574
1+350	186.85	53	1.083	58	0.026	0.20	16	15.849	0.8690	19.5112	9.087	12.263
1+400	167.63	30	0.271	35	0.026	0.20	-	4.217	-	4.2875	2.418	3.641

5.2 Tunnel Stability Evaluation

Stability of the tunnel section was analyzed by block failure analysis and squeezing prediction. Poor rock quality and the presence of various discontinuities in the tunnel alignment cause the instability problem which was studied in this chapter.

5.2.1 Block Stability Analysis

The blocks are formed by the intersection of discontinuities. When a right combination of joints is present at a given location, a wedge is formed. During tunnel excavations, rock wedges could form due to the presence of joints. These Blocks were falling due to the influence of gravity and other forces, roof and wall wedges may fail either by falling, sliding or rotating out of their sockets. The analysis of these Block/wedges are very important for the stability of excavation. Most existing algorithms for underground wedge stability analysis assume that stresses are sufficiently low and can therefore be ignored. If the in-situ stress is low than we have to neglect the wedge but for the deep excavation it needs importantly to analyze the wedge analysis. In this research work, wedge analysis was done for each section and analyze whether they are stable by applying the minimum support as designed.

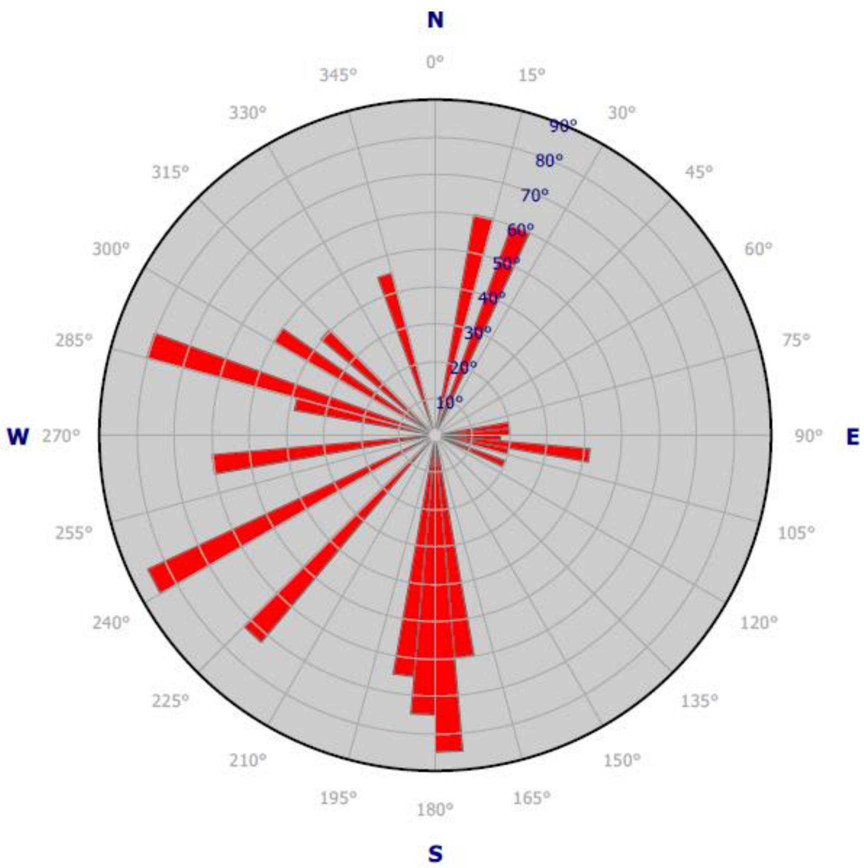


Figure 5-1 Rosette Diagram Showing Discontinuities (Dip/Plunge)

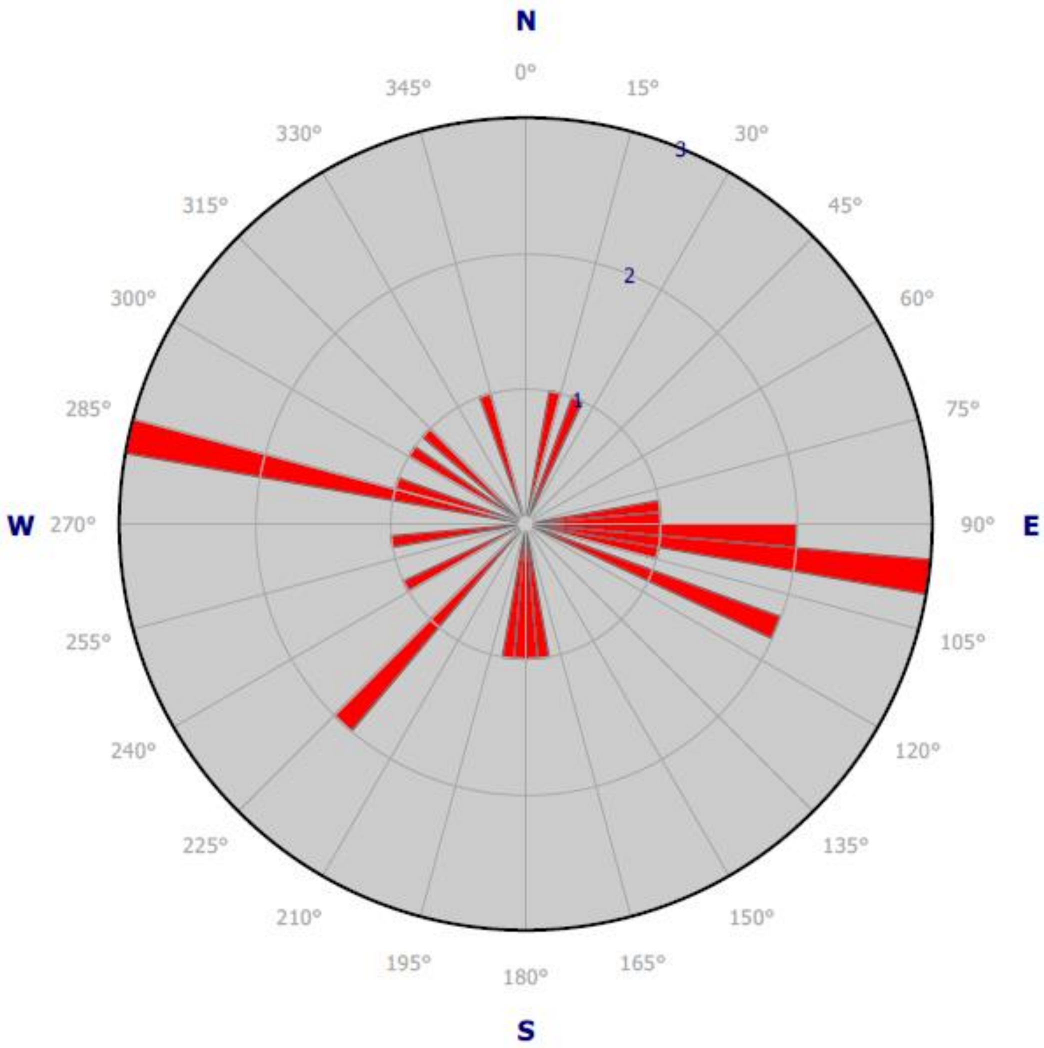


Figure 5-2 Rosette Diagram Showing Joints (Dip/Trend)

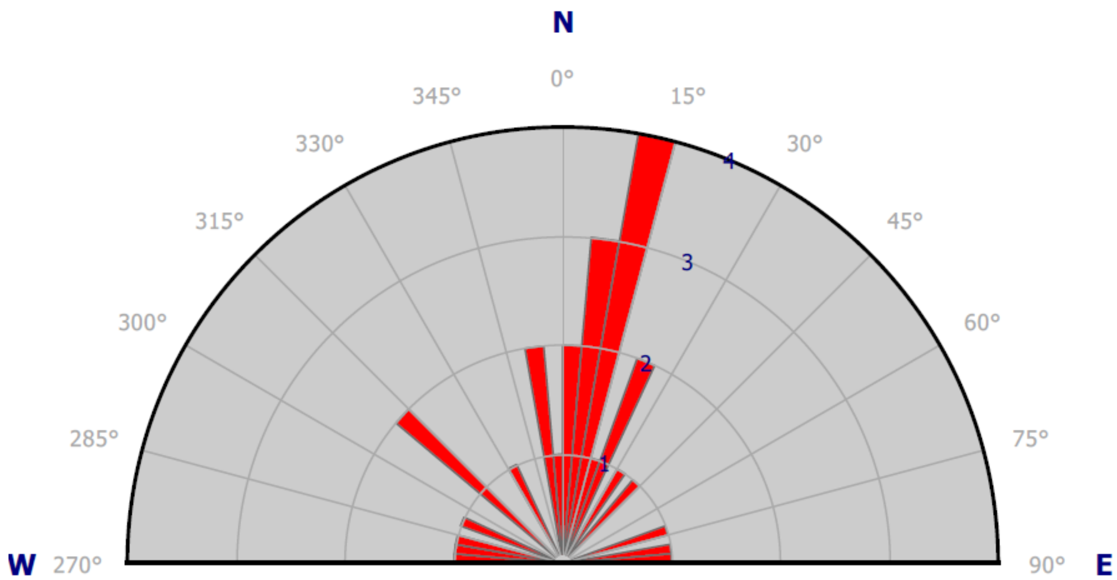


Figure 5-3 Rosette Diagram Showing Strike

The above diagram present clear view of joints presented in the study section of Headrace tunnel. Dip direction (Figure 5-1) dip amount (Figure 5-2) and strike (Figure 5-3) of important joint of site condition its self a clear analysis tool for the block failure analysis.

For the analysis of wedge stability, UNWEDGE computer program was used. This is very popular software for the stability analysis of wedge formed during tunnel excavation which is developed by rockscience. UNWEDGE is a 3D stability analysis and visualization program for underground excavations in rock containing intersecting structural discontinuities. Safety factors are calculated for potentially unstable wedges and support requirements can be modeled using various types of pattern and spot bolting and shotcrete as designed after finite element method in previous topic.

<u>Input Parameter</u>

Table 5-4 Input Parameter for Wedge Analysis

Tunnel Axis Orientation		Shear Strength	
Trend:	17.46^{o}	Model Used:	Barton-Bandis
Plunge:	Zero	JRC:	10
Unit Weight		JCS	78 MPa
Rock	0.027 MPa	Φ_r	30
Water:	0.00981 MPa	Assume water pressure and waviness are zero.	

In this program wedges are subjected to gravity loading only, stress field are not taken into consideration that may affect in the result and lead to lower the factor of safety. This is the limitation of this software. It is assumed that displacement of wedges is takes place at the discontinuities and wedges move as rigid bodies with no internal deformation. All the section was analyzed for the wedge failure but only three section were discussed here and rest of the section were taken on appendix. These three sections were selected as different rock mass class categorized by the Q-value.

<u>**Chainage 1+000m**</u>

At all the sections three joint set are considered, Figure 5-4 shows the joint orientation and stereonet at this chainage with the tunnel axis. This is the input parameter of discontinuities for the wedge analysis.

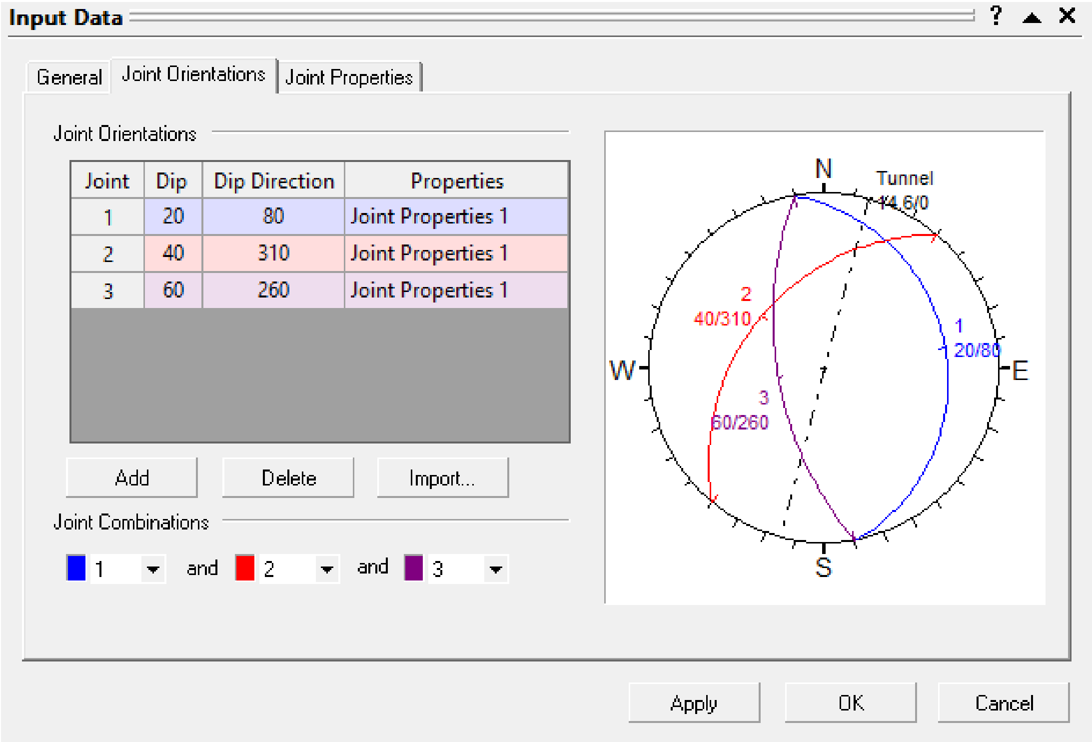


Figure 5-4 Joint Orientation and Stereonet at chainage 1+000m

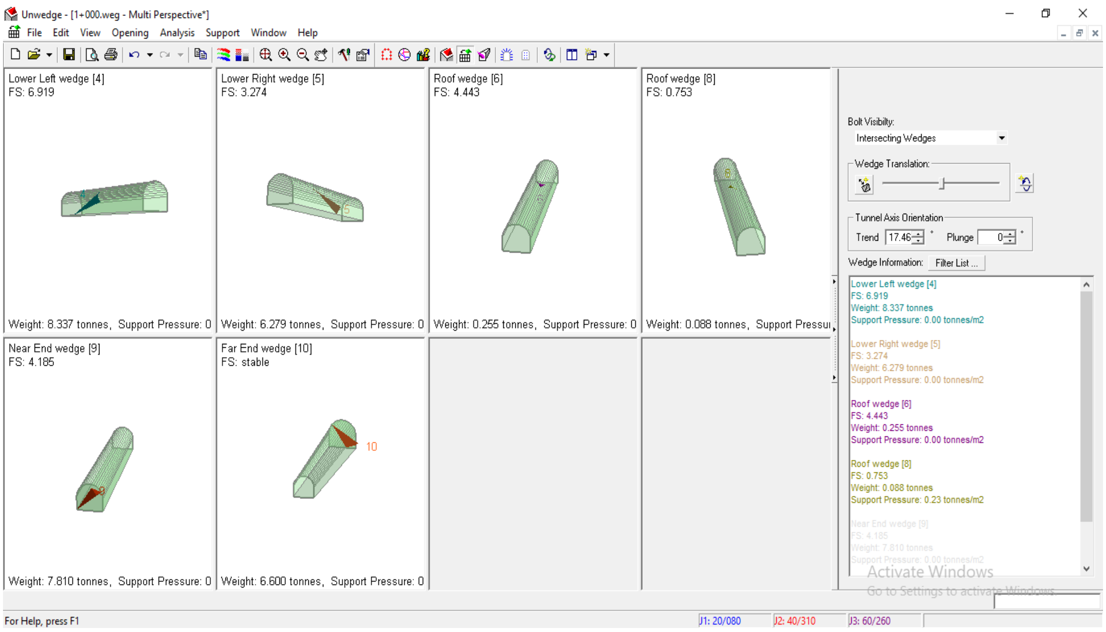


Figure 5-5 Multi perspective view of wedge failure without Support at chainage 1+000m

The joints all have the same Mohr-Coulomb strength parameters: a cohesion value of 0.1 MPa and a friction angle of 30˚. The total of six wedges formed around the perimeter of the tunnel. Wedge #4 is in the lower left wall, #3 is on the lower right sidewall, while #6 and #8 are in the roof. Wedge #9 and #10 are on the near and far end. All the wedges are shown with the factor of safety before Figure 5-5 and after Figure 5-6 support installed. Roof wedge #8 have very low factor of safety i.e. 0.753

which is further increase to stable condition after installing minimum support designed earlier.

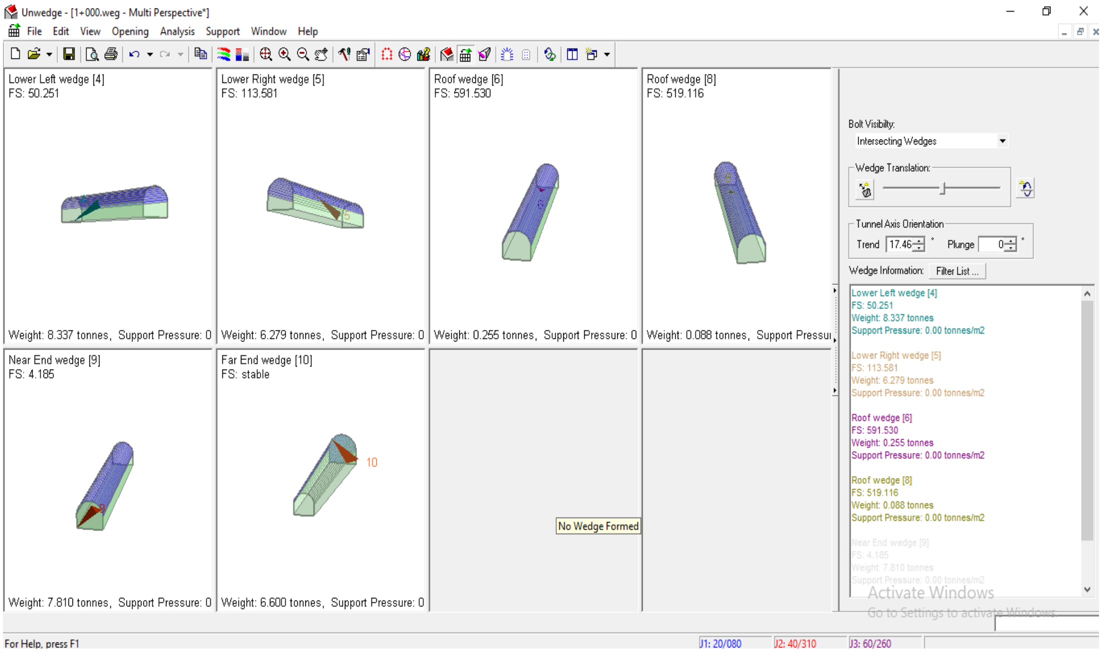


Figure 5-6 Multi perspective view of wedge failure with Support at chainage 1+000m

Chainage 1+150m

Three joint set are considered, Figure 5-7 shows the joint orientation and stereonet at

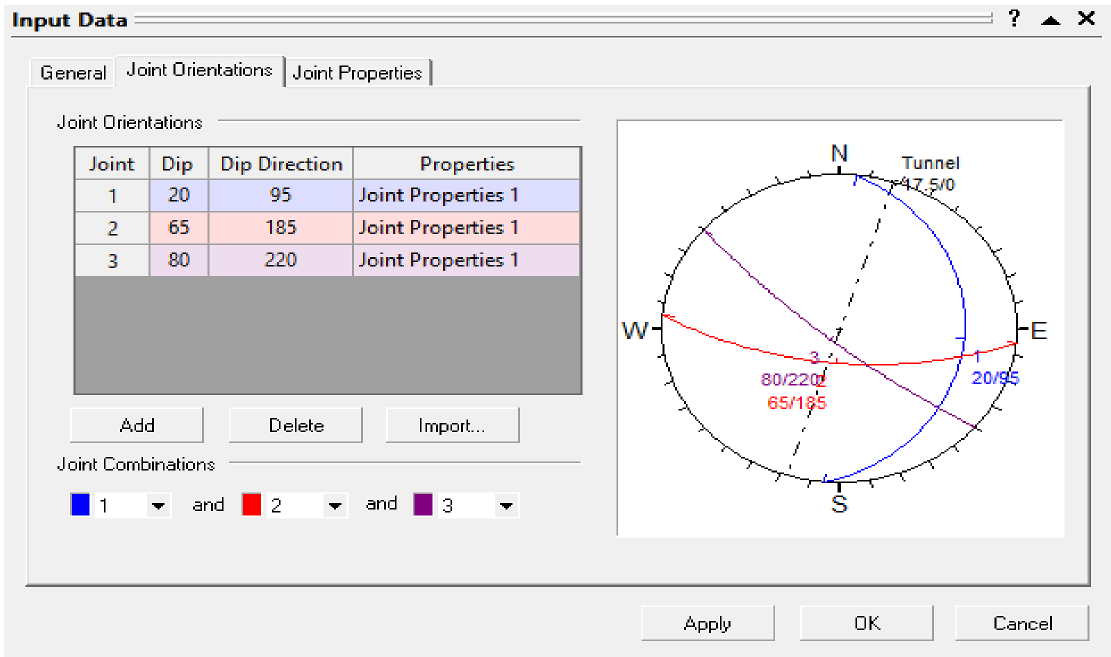


Figure 5-7 Joint Orientation and Stereonet at chainage 1+150m

this chainage with the tunnel axis. This is the input parameter of discontinuities for the wedge analysis. The joints set with dip amount of 20, 65 and 80 were found in this section with dip direction 95, 185, 220 respectively.

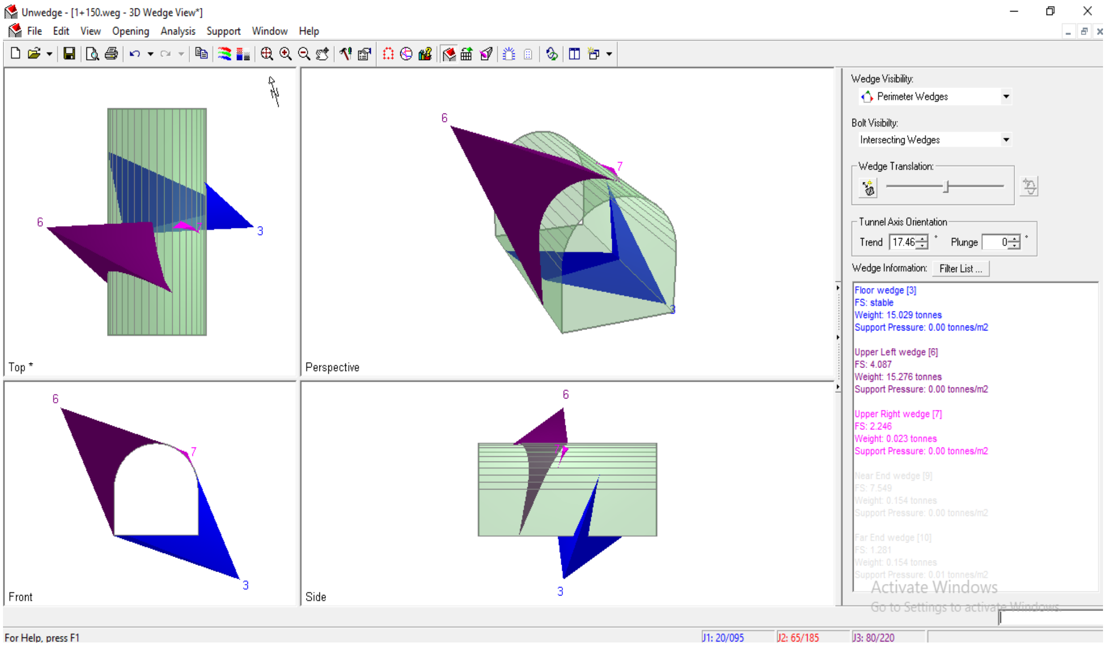


Figure 5-8 Multi perspective view of wedge failure without Support at chainage 1+150m

The joints all have the same Mohr-Coulomb strength parameters: a cohesion value of 0.1 MPa and a friction angle of 30˚. The total of five wedges formed around the perimeter of the tunnel. Wedge #3 is in the floor, #6 is on the upper left sidewall, while #7 in the upper right wall. Wedges #9 and #10 are on the near and far end. All the wedges are shown with the factor of safety before Figure 5-8 and after Figure 5-9 support installed.

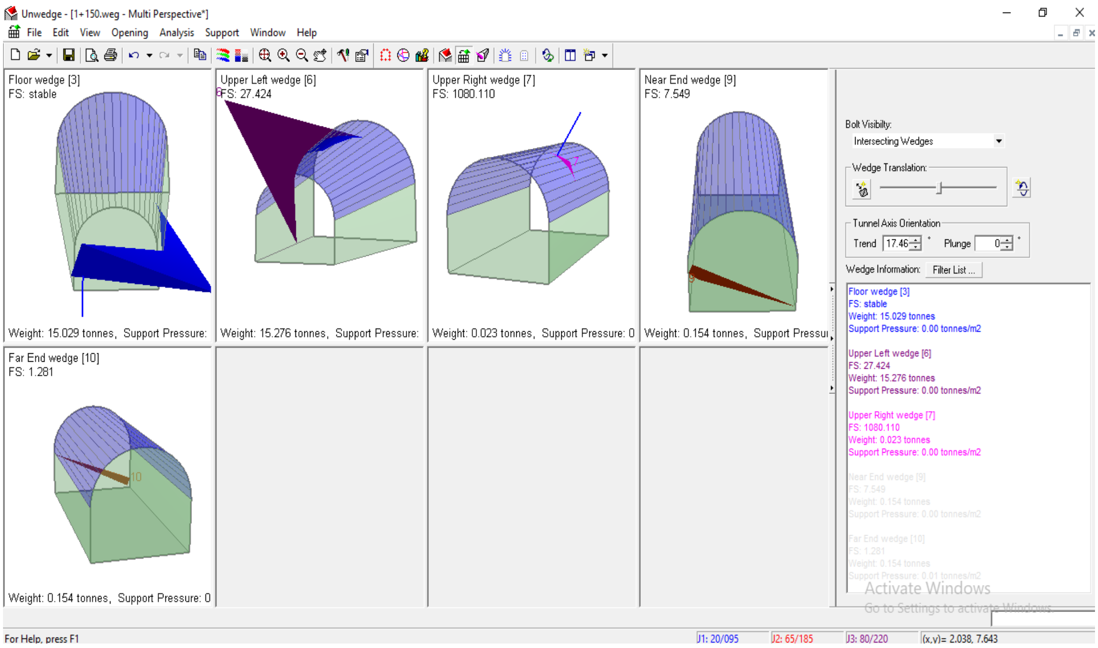


Figure 5-9 Multi perspective view of wedge failure with Support at chainage 1+150m

Far end wedge #10 have very low factor of safety i.e. 1.281 which is further increase to stable condition after installing minimum support designed earlier. Wedge #3 will not fail since it is in the floor so it will also not be discussed further.

Chainage 1+350m

Three joint set are considered, Figure 5-10 shows the joint orientation and stereonet at this chainage with the tunnel axis. This is the input parameter of discontinuities for the wedge analysis. The joints set with dip amount of 25, 45 and 85 were found in this section with dip direction 95, 280, 175 respectively.

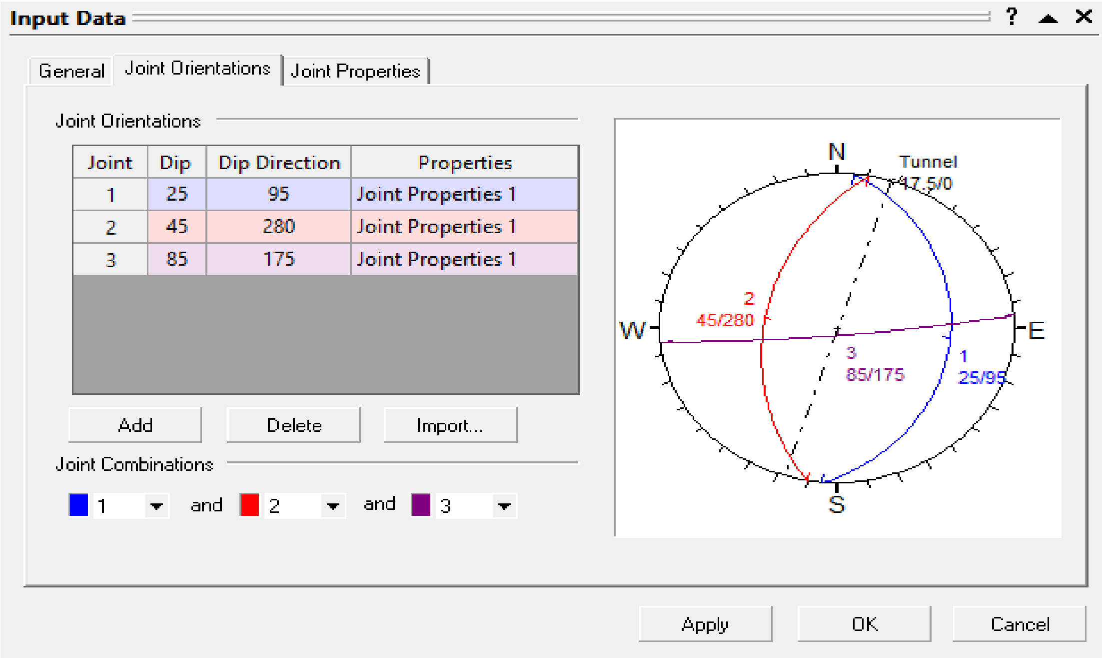


Figure 5-10 Joint Orientation and Stereonet at chainage 1+350m

The joints all have the same Mohr-Coulomb strength parameters: a cohesion value of 0.1 MPa and a friction angle of 30˚. The total of seven wedges formed around the perimeter of the tunnel.

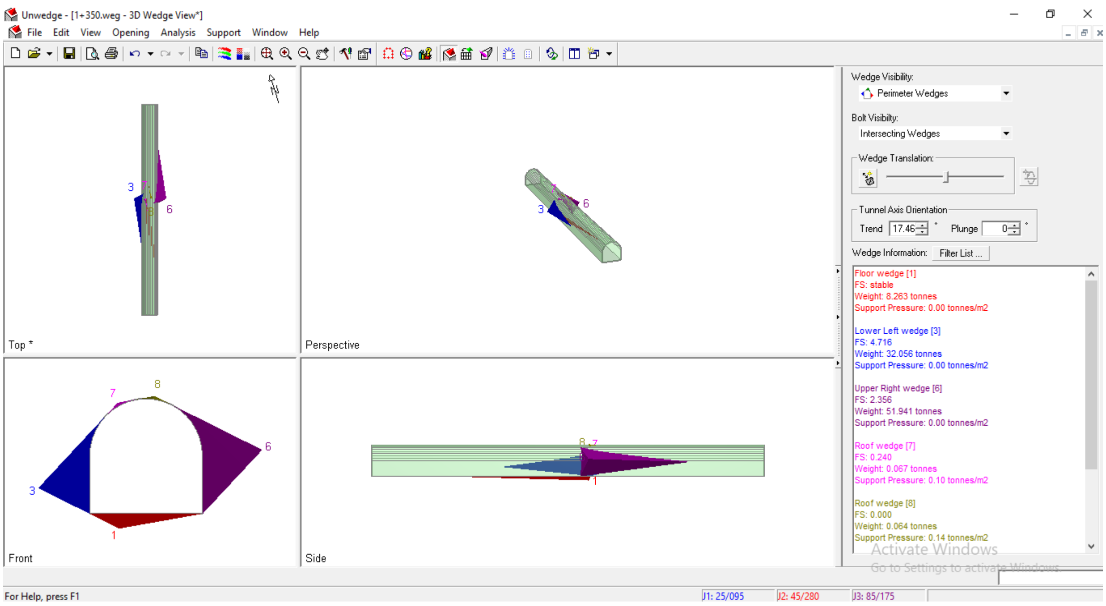


Figure 5-11 Multi perspective view of wedge failure without Support at chainage 1+350m

Wedge #1 is in the floor, #3 is on the lower left sidewall, while #5 in the upper right wall. Wedges #7 and #8 are on the roof. Wedges #9 and #10 are on the near and far end. All the wedges are shown with the factor of safety before Figure 5-11 and after Figure 5-12 support installed. Roof wedge #7 and #8 have very low factor of safety i.e. 0.240 and 0.00 respectively, which is further increase to stable condition after installing minimum support designed earlier. Wedge #1 will not fail since it is in the floor so it will also not be discussed further.

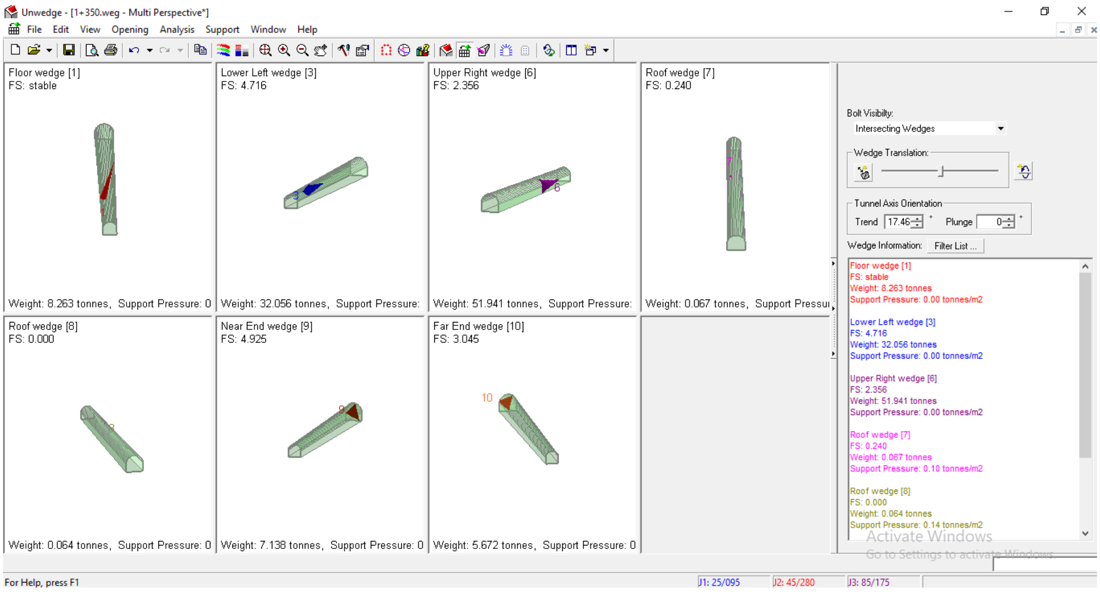


Figure 5-12 Multi perspective view of wedge failure with Support at chainage 1+350m

5.2.2 Squeezing Prediction

Squeezing ground conditions refer to large convergence of excavations occurring during excavation and that may continue over time. These conditions are encountered in tunneling drives in poor quality or weak rock but also in structurally defined rock masses. In this portion the tunnel section from 1+000m to 1+400m was analyzed for the squeezing and that was study for safe support system which are minimum as designed earlier.

Squeezing prediction in tunnel alignment were analyzed by Empirical, Semi-Analytical and Numerical methods. In Empirical methods Singh et al. (1992) and Goel (1994) approaches were used. For the analysis by Semi Analytical Method, Hoek and Morinas (2000) was used which found quite more acceptable and is recommended by different scientists. Finite Element Method (FEM) using RS2 computer software was used for the Numerical analysis.

5.2.2.1 Empirical Approach

Singh et al. (1992) approach

Singh et al. (1992) has given a demarcation line to differentiate squeezing condition from non-squeezing condition. This approach was developed by collecting data on rock mass quality Q (Barton et al., 1974) and overburden depth H explained in table 4-32. The equation of the line is:

$$H = 350\ Q^{1/3}\ (m) \qquad \textit{Eq. 5-4}$$

With the rock mass uniaxial compressive strength σ_{cm} estimated as σ_{cm}

$$\sigma_{cm} = 0.75\ \gamma\ Q^{1/3}\ (MPa) \qquad \textit{Eq. 5-5}$$

Where,

H = Overburden Depth

Q = Rock mass quality

γ = Rock mass unit weight.

Considering the above equations, the different data needed for this approach were obtained from my study area and tabulated as below Table 5-5.

Table 5-5 Tunnel overburden and Q value for Singh et al. (1992) approach.

Chainage	Overburden (m)	Q value	Equation of Line (H)	Remarks
1+000	306.42	0.344	245.18	Squeezing
1+050	292.61	0.583	292.44	Squeezing
1+100	285.26	0.229	214.18	Squeezing
1+150	275.37	0.075	147.60	Squeezing
1+200	274.05	0.038	117.15	Squeezing
1+250	253.04	0.070	144.46	Squeezing
1+300	219.05	1.250	377.03	Non Squeezing
1+350	186.85	1.083	359.46	Non Squeezing
1+400	167.63	0.271	226.45	Non Squeezing

The graph obtained Figure 5-13 shows about the ground condition prediction based on rock mass quality Q. From the graph we can conclude that most of the area (about 67%)

falls under the squeezing zone and the remaining (33%) falls under the non- squeezing zone. The section from 1+000m to 1+250m where high overburden pressure lies falls under squeezing condition and rest of the section with lesser overburden pressure is safe for squeezing problem.

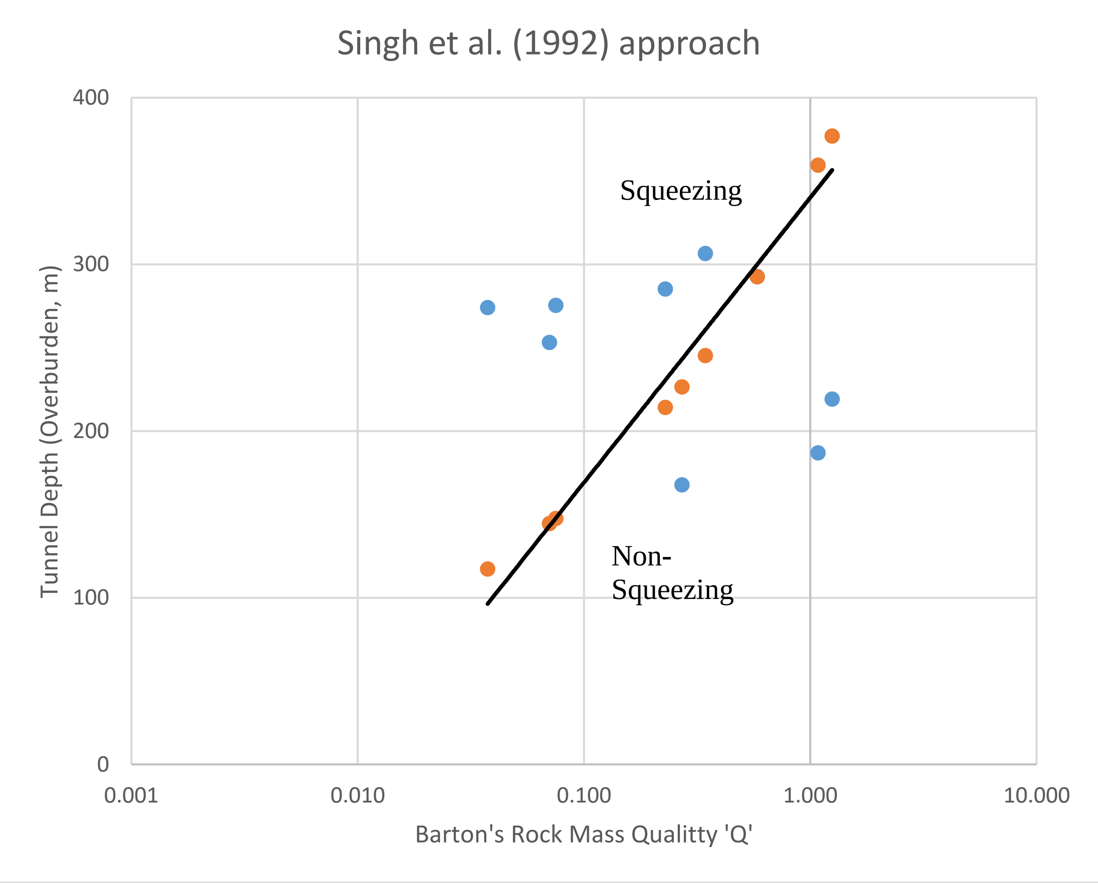


Figure 5-13 Ground condition prediction based on rock mass quality 'Q'.

Goel (1994) approach

Goel (1994) developed an empirical approach based on the rock mass number N, defined as Q with SRF = 1. N was used to avoid the problems and uncertainties in obtaining the correct rating of parameter SRF in Q method. Considering the overburden depth H, the tunnel span or diameter B, and the rock mass number N from our tunnel sections, we have plotted the available data on log-log diagram between N and $HB^{0.1}$. All the section is studied for the squeezing problems and found five section fall for squeezing and rest lies for non-squeezing condition. As shown in the Table 5-6, distinguishes the squeezing and non-squeezing cases. The equation of this line is

$$H= (275N^{0.33})\ B^{-0..1} \qquad \textit{Eq. 5-6}$$

Considering the above equations, the different data needed for this approach were obtained from my study area and tabulated as below Table 5-6.

Table 5-6 Different parameters for Goel (1994) approach.

Chainage	Overburden (m)	Rock Mass Number 'N'	Tunnel Diameter 'B'	Equation of Line 'H'	$HB^{0.1}$	Remarks
1+000	306.42	1.72	4.200	284.86	328.82	Squeezing
1+050	292.61	2.92	4.200	339.17	391.51	Non-Squeezing
1+100	285.26	1.15	4.200	249.18	287.64	Squeezing
1+150	275.37	0.75	4.200	216.66	250.09	Squeezing
1+200	274.05	0.38	4.200	172.36	198.96	Squeezing
1+250	253.04	0.70	4.200	212.09	244.82	Squeezing
1+300	219.05	3.13	4.200	346.98	400.53	Non-Squeezing
1+350	186.85	2.71	4.200	330.98	382.05	Non-Squeezing
1+400	167.63	1.35	4.200	263.31	303.94	Non-Squeezing

The Figure 5-14 represent the ground condition at the study section for different value of 'N', the most of the section fall under minor squeezing and other has non-squeezing condition.

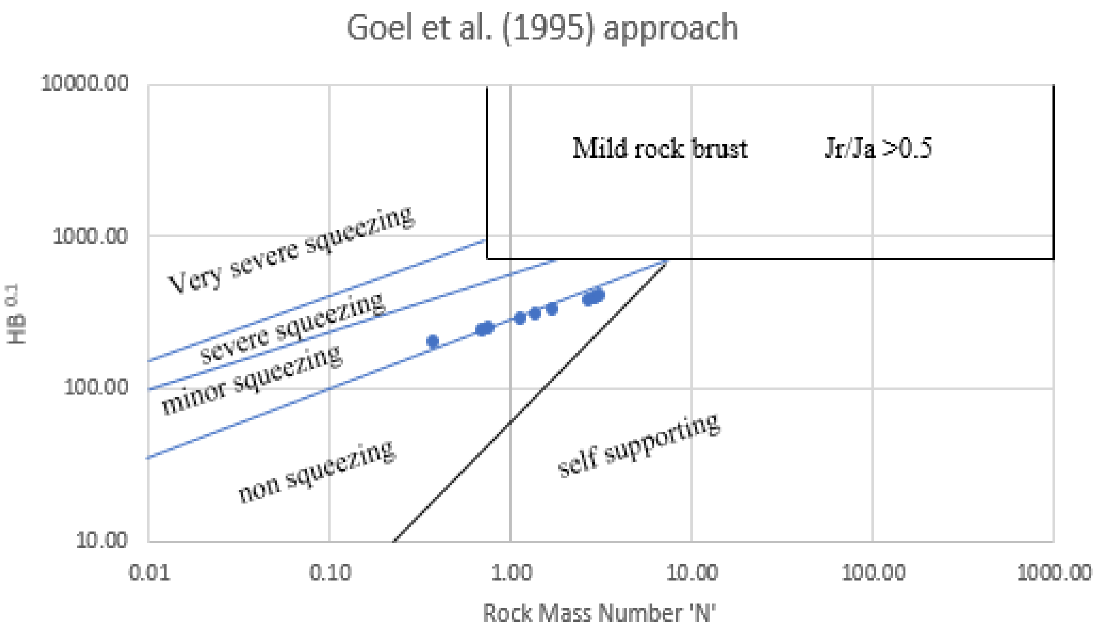


Figure 5-14 Ground condition prediction using rock mass number 'N'.

5.2.2.2 Semi Empirical Method

Jethwa et al. approach (1984)

The degree of squeezing in this approach is described using coefficient Nc which is equal to the ratio of rock mass uniaxial compressive strength (UCS) to insitu stress. Based on this value, type of behavior of tunnel can be estimated. Jethwa et al. (1984) define the degree of squeezing based on following relation:

$$Nc = \frac{\sigma cm}{Po} \qquad \textit{Eq. 5-7}$$

Where, σ_{cm} = rock mass uniaxial compressive strength Po = in-situ stress γ = Unit weight of rock mass H = tunnel depth below surface.

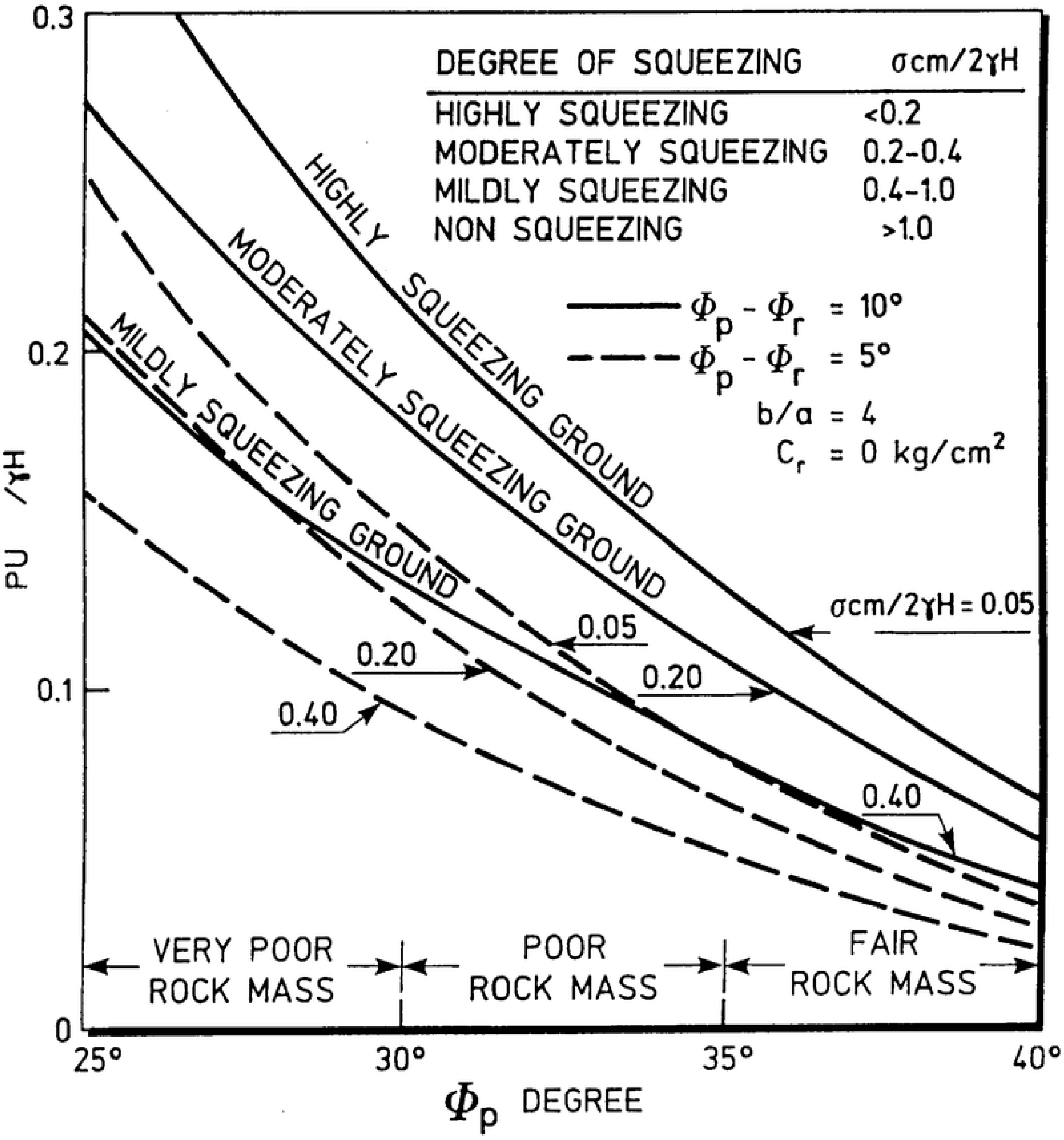


Figure 5-15 Chart for Degree of Squeezing Analysis (Jethwa et al.)

The calculation is done for the prediction of squeezing at each section of this research on the basic of Jethwa et al. approach. For the various rock class i.e. very poor rock, poor rock and fair rock with degree of squeezing Figure 5-15 the prediction of squeezing was done for the nine section of this study Table 5-7.

Table 5-7 Squeezing prediction using Jethwa et al.

Chainage	Overburden (m)	RMR	Q	σcm	Po= γH	Nc = σcm/2Po	Remarks
1+000	306.42	35	0.344	16.66	8.27	1.01	Non-Squeezing
1+050	292.61	35	0.583	16.66	7.90	1.05	Non-Squeezing
1+100	285.26	35	0.229	16.66	7.70	1.08	Non-Squeezing
1+150	275.37	19	0.075	8.55	7.43	0.58	Mildly-Squeezing
1+200	274.05	19	0.038	8.55	7.40	0.58	Mildly-Squeezing
1+250	253.04	19	0.070	8.55	6.83	0.63	Mildly-Squeezing
1+300	219.05	58	1.250	31.01	5.91	2.62	Non-Squeezing
1+350	186.85	58	1.083	29.88	5.04	2.96	Non-Squeezing
1+400	167.63	35	0.271	16.66	4.53	1.84	Non-Squeezing

5.2.2.3 Semi Analytical Method

Hoek and Marinos (2000)

A semi-analytical approach given Hoek and Marinos (2000) have been used for estimation of the deformation caused by squeezing and estimation of support pressure required in the squeezing tunnel. Hoek and Marinos showed that a plot **Error! Reference source not found.** of tunnel strain (ξ) against the ratio cm/po could be used effectively to assess tunneling problems under squeezing condition. Hoek and Brown's criteria for estimating the strength and deformation characteristics of rock masses assume that rock mass behaves isotropically. However, if the rock mass is heavily fractured, the continuity of the bedding surfaces will have been disrupted and the rock may behave as an isotropic mass. Thus, this criterion can be adapted to weak heterogeneous rock masses too.

This curve can be obtained by using the following equation:

$$\varepsilon = \frac{\delta i}{do} = [0.002\text{-}0.0025\frac{pi}{po}]\ \frac{\sigma cm}{po}^{(2.4\frac{pi}{po}-2)} \qquad \textit{Eq. 5-8}$$

$$\sigma_{cm} = (0.0034mi^{0.8})\ \sigma_{ci}\ \{1.029+0.025e^{(-0.1mi)}\}^{GSI} \qquad \textit{Eq. 5-9}$$

In the equation given above, value of support pressure (pi) will be zero for an unsupported condition. The value of 'pi' should be increased until the strain reaches an acceptable range. The rock mass compressive strength (cm) is estimated by the following equation:

In case semi analytical method of squeezing analysis, Hoek and Marinos (2000) approach was used with correlation with ground condition. This method shown quite acceptable result and is more recommended by different scientists in rock engineering. In this method, tunnel strain (Ɛ) was calculated by using different parameters such as Tunnel overburden depth, vertical stress, intact strength, material constant, GSI, rock mass strength and support pressure. Support pressure Pi was calculated by using the supports used in the tunnel such as steel ribs, shotcrete and rock bolts. From the calculated value of Pi i.e. Pimax tunnel strain was calculated.

The various physical parameters are determined in the empirical method. The empirically derived relationship between rock mass parameters and supports are then utilized to predict the support types and quantities and possibly the excavation procedure. Empirical method is generally applied during two circumstances when there might be limited geological information but relatively unlimited time and during construction when there is ample geological information but time is critical.

Among the above method of predicting squeezing for our tunnel section, the semi analytical method is most appropriate for the study in which we can use the installed support pressure. The describe the strain in percentage at every section that lies in the < 1 %. That indicate the minor support problem not the squeezing problem. So, the designed support is safe for the installation. It generally happens in weak rock such as shale, phylite, and slates or at weakness zone. In case of SMHEP, rock mass seems (from surface) strong and brittle, there is less chance of squeezing.

Table 5-8 Tunnel strain by using Pi

Chainage	Tunnel Depth (m)	γ (KN/m²)	Po (Mpa)	σci	mi	GSI	σcm	σcm/Po	Pi	Pi/Po	ξ (%)	Remarks
1+000	306.42	27	5.133	75.00	28	30	2.835	0.55	3.739	0.729	0.021%	
1+050	292.61	27	4.901	75.00	28	30	3.325	0.68	3.574	0.729	0.020%	
1+100	285.26	27	4.778	75.00	28	30	2.601	0.54	3.574	0.748	0.015%	
1+150	275.37	27	4.612	35.00	28	14	0.527	0.11	3.376	0.732	0.029%	
1+200	274.05	27	4.590	35.00	28	14	0.491	0.11	3.351	0.730	0.030%	
1+250	253.04	27	4.238	35.00	28	14	0.523	0.12	3.307	0.780	0.006%	
1+300	219.05	27	3.669	75.00	28	53	9.303	2.54	2.389	0.651	0.025%	
1+350	186.85	27	3.130	75.00	28	53	8.963	2.86	2.091	0.668	0.022%	
1+400	167.63	27	2.808	75.00	28	30	2.686	0.96	2.091	0.745	0.014%	

5.2.3 Support Estimation

The support system was estimated considering empirical, analytical and finite element Modelling which was further more analyzed for the ground characterization i.e. block stability and squeezing. Rock mass characterization using Q value and RMR value was used for the rock support estimation under empirical method. The applied support pressure compared with the critical support pressure is used for the estimating rock support analytically. Al last all the support are applied to the finite element model to optimize the support individually.

5.2.3.1 Empirical Methods

Statistical analysis of underground observations is the empirical technique for evaluating the stability of underground structures. Engineering rock mass classification is the best empirical approach for assessing the underground opening (Goodman, 1980; Hoek and Brown, 1980). Q-system and RMR (Geo-mechanics) system, are the commonly used type of Empirical methods to evaluate the rock support requirement and support design. Some empirical approaches, which are commonly used and universally accepted, are described here in detail.

Support Estimation by Q- Chart

Based on the Q value the permanent support is estimated using Q-chart. In addition to the rock mass quality (the Q-value) two other factors are decisive for the support design in underground openings and caverns. These factors are the safety requirements and the dimensions, i.e., the span or height of the underground opening. Generally, there will be an increasing need for support with increasing span and increasing wall height. Safety requirements will depend on the use (purpose) of the excavation. To express safety requirements, a factor called ESR (Excavation Support Ratio) is 1.6 used. The estimated support from the Q value are listed in Table 5-9.

The support chart gives an average of the empirical data from examined cases. For a given combination of Q-value and Equivalent dimension, a given type of support has been used and the support chart has been divided into areas according to type of support. The support chart indicates what type of support is used in terms of the centre to centre spacing for rock bolts and the thickness of sprayed concrete. It also indicates the energy absorption of the fibre reinforced sprayed concrete, as well as the bolt length and design of reinforced ribs of sprayed concrete. All the estimated support was further analyzed using finite element method from RS^2 software from rock science.

Table 5-9 Rock Support Estimation Using Q-Value

Chainage	Q-Value	Class Number	Rock Mass Description	Final-Support
1+000	0.344	V	Very Poor	Systematic Bolt of φ-20mm, length 2.5m at 1.3m c/c, Fibre Reinforced shotcrete of 70 mm, (Sfr + B)
1+050	0.583	IV	Very Poor	Systematic Bolt of φ-20mm, length 2.0m at 1.5m c/c, Fibre Reinforced shotcrete of 40mm. (Sfr + B)
1+100	0.229	V	Very Poor	Systematic Bolt of φ-20mm, length 2.5m at 1.5m c/c, Fibre Reinforced shotcrete of 50 mm, (Sfr + B)
1+150	0.075	VI	Extremely Poor	Systematic Bolt of φ-20mm, length 2.5 m at 1.2m c/c, Fibre Reinforced shotcrete of 100 mm. (Sfr + B)
1+200	0.038	VI	Extremely Poor	Systematic Bolt of φ-20mm, length 2.5 m at 1.2m c/c, Fibre Reinforced shotcrete of 150 mm. (Sfr + B)
1+250	0.070	VI	Extremely Poor	Systematic Bolt of φ-20mm, length 2.5 m at 1.2m c/c, Fibre Reinforced shotcrete of 100 mm. (Sfr + B)
1+300	1.250	IV	Poor	Systematic Bolt of φ-20mm, length 2m at 2m c/c, unreinforced shotcrete of 50mm. (Sfr + B)
1+350	1.083	IV	Poor	Systematic Bolt of φ-20mm, length 2m at 2m c/c, unreinforced shotcrete of 50mm. (Sfr + B)
1+400	0.271	V	Very Poor	Systematic Bolt of φ-20mm, length 2.5m at 1.5m c/c, Fibre Reinforced shotcrete of 50 mm, (Sfr + B)

The thickness of the sprayed concrete increases towards decreasing Q-value and increasing span, and lines are drawn in the support chart indicating thicknesses. For positions between these lines the thicknesses will have an intermediate value. If deformation occurs, for instance caused by high stresses, reinforced concrete should be used in all categories. The length of the bolts depends on the span or wall height of the underground opening and to some degree on the rock mass quality.

Sometimes alternative methods of support are given. At high Q-values in the support chart, sprayed concrete may or may not be used. The mean bolt spacing in such cases will be dependent upon whether or not sprayed concrete is used. Due to this, the support chart is divided into two areas. The area defined as “Bolt spacing in fibre reinforced sprayed concrete” refers to bolting in combination with sprayed concrete.

Support Estimation using RMR Value

Rock support was also estimated from RMR value using support table design by Bieniawski, 1989. These supports are listed in Table 5-10.

Table 5-10 Rock Support Estimation Using RMR Method

Chainage	Rock Support-RMR Method			
	RMR	**Rock Class**	**Rock Type**	**RMR Support**
1+000	35	IV	Poor	Systematic 20mm diameter bolts 2m long, spaced 1–1.5 m in crown and wall with wire mesh, shotcrete with a thickness range of between 100mm and 150m in crown and 100mm in the sides of tunnel, and Light to medium ribs spaced 1.5 m steel set where required
1+050	35	IV	Poor	
1+100	35	IV	Poor	
1+150	19	V	Very poor	Systematic 20mm diameter bolts 2.5m long, spaced 1–1.5 m in crown and walls with wire mesh; bolt invert, shotcrete with a thickness range of between 150mm and 200m in crown and 150mm in the sides of tunnel and 50mm on face, and Medium to heavy ribs spaced 0.75 m with steel lagging and fore poling if required; close invert
1+200	19	V	Very poor	
1+250	19	V	Very poor	

1+300	58	III	Fair	Systematic 2m long systematic bolts of 20mm diameter and fully grouted, spacing range between bolts of 1.5–2m in crown and walls with wire mesh in crown, shotcrete with a thickness range of between 50mm and 100m in crown and 30mm in the sides of tunnel, and no steel set required
1+350	58	III	Fair	
1+400	35	IV	Poor	Systematic 20mm diameter bolts 2m long, spaced 1–1.5 m in crown and wall with wire mesh, shotcrete with a thickness range of between 100mm and 150m in crown and 100mm in the sides of tunnel, and Light to medium ribs spaced 1.5 m steel set where required

RMR method is generally used for horseshoe shaped tunnel of width 10m under the vertical stress 25 MPa, Calculated supports were compared with the support from various other methods.

5.2.3.2 Analytical Method

Now the model is validated through the comparison of total displacement computed and result shown by the model. For this first we have calculated the total inward displacement here.

For this first we have to calculate hydrostatic stresses P_o and uniform internal support pressure P_i. These pressures are subjected to the support applied at the section. Generally, we have applied the shotcrete, systematic bolting and steel ribs.

Estimated support can be optimized using analytical equation such that there is no failure condition. If the Internal support pressure is greater than the critical pressure than no failure. With the help of this condition, we can optimize the support and these models is said to be model set III.

Table 5-11 Calculation of Hydrostatic & Internal Support Pressure

Chainage	p_o	Bolt Spacing	Shotcrete Thickness	Steel Rib Type	p_i			Total
					Bolt	Shotcrete	Steel Rib	p_i
1+000	8.212	1.600	100	ISMB 150	0.0719	1.7887	3.755	5.6156
1+050	7.842	1.400	100	ISMB 150	0.0939	1.7887	2.888	4.7710

1+100	7.645	1.400	100	ISMB 150	0.0939	1.7887	3.414	5.2962
1+150	7.380	1.400	200	ISMB 150	0.0939	3.4304	3.129	6.6534
1+200	7.345	1.200	200	ISMB 150	0.1278	3.4304	2.888	6.4466
1+250	6.781	1.400	200	ISMB 150	0.0939	3.4304	2.347	5.8711
1+300	5.871	2.000	100	ISMB 150	0.0460	1.7887	1.173	3.0081
1+350	5.008	2.000	100	ISMB 150	0.0460	1.7887	1.173	3.0081
1+400	4.492	2.000	100	ISMB 150	0.0460	1.7887	1.173	3.0081

Table 5-12 Checking Failure Condition for Support from Analytical Approach

Chainage	p_o	p_i	p_{cr}	Remarks
1+000	5.133	5.6156	5.4295	No Failure
1+050	4.901	4.7710	4.7189	No Failure
1+100	4.778	5.2962	5.0578	No Failure
1+150	4.612	6.6534	6.3087	No Failure
1+200	4.590	6.4466	6.3003	No Failure
1+250	4.238	5.8711	5.7323	No Failure
1+300	3.669	3.0081	1.3991	No Failure
1+350	3.130	3.0081	1.8950	No Failure
1+400	2.808	3.0081	2.0295	No Failure

Table 5-13 Estimated Support from Analytical Approach

Chainage	Description
1+000	Systematic Bolting of 20mm-φ & 2m length with 1.6m c/c spacing. Fibre reinforced sprayed concrete 100 mm + Steel Ribs (ISMB 150) at 0.5m c/c.
1+050	Systematic Bolting of 20mm-φ & 2m length with 1.4m c/c spacing. Fibre reinforced sprayed concrete 100 mm + Steel Ribs (ISMB 150) at 0.65m c/c.
1+100	Systematic Bolting of 20mm-φ & 2m length with 1.4m c/c spacing. Fibre reinforced sprayed concrete 100 mm + Steel Ribs (ISMB 150) at 0.55m c/c.
1+150	Systematic Bolting of 20mm-φ & 2m length with 1.4m c/c spacing. Fibre reinforced sprayed concrete 200 mm + Steel Ribs (ISMB 150) at 0.6m c/c.

1+200	Systematic Bolting of 20mm-φ & 2m length with 1.2m c/c spacing. Fibre reinforced sprayed concrete 200 mm + Steel Ribs (ISMB 150) at 0.65m c/c.
1+250	Systematic Bolting of 20mm-φ & 2m length with 1.4m c/c spacing. Fibre reinforced sprayed concrete 200 mm + Steel Ribs (ISMB 150) at 0.8m c/c.
1+300	Systematic Bolting of 20mm-φ & 2m length with 2m c/c spacing. Fibre reinforced sprayed concrete 100 mm + Steel Ribs (ISMB 150) at 0.8m c/c.
1+350	Systematic Bolting of 20mm-φ & 2m length with 2m c/c spacing. Fibre reinforced sprayed concrete 100 mm + Steel Ribs (ISMB 150) at 0.8m c/c
1+400	Systematic Bolting of 20mm-φ & 2m length with 2m c/c spacing. Fibre reinforced sprayed concrete 100 mm + Steel Ribs (ISMB 150) at 0.8m c/c

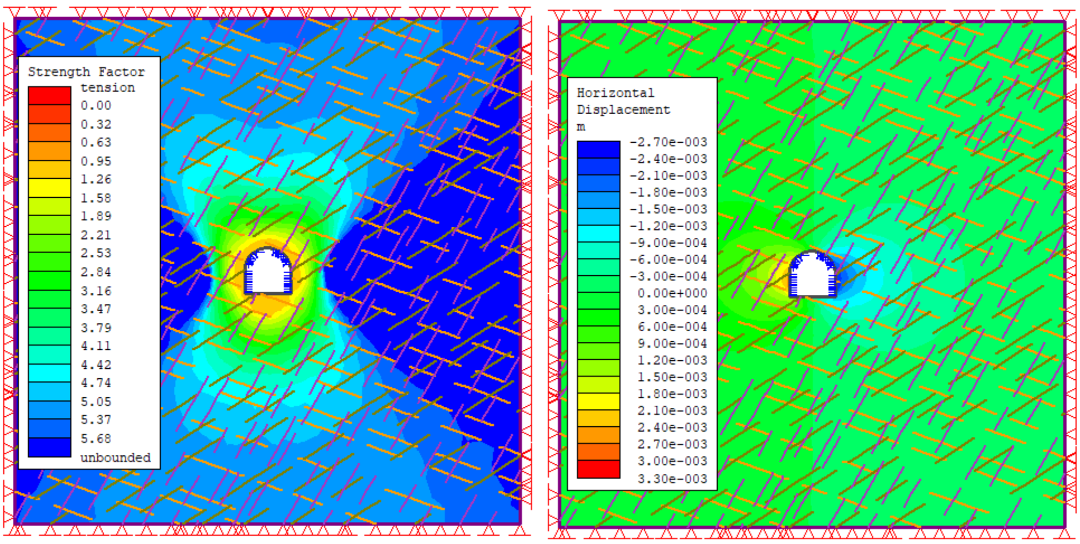
5.2.3.3 Numerical Modelling

Modeling of rock mass is a very difficult job due to the presence of discontinuities, anisotropic, heterogeneous, and nonelastic nature of rock mass, using empirical and numerical methods. Complex nature and different formation make the rock masses a difficult material for empirical and numerical modeling. During initial stages of excavation projects, the detailed data are not available about strength properties, deformation modulus, in situ stresses, and hydrological of rock masses. To handle the no availability of the detailed project data, the empirical methods like rock mass classification systems are considered to be used for solving engineering problems. empirical methods used defined input parameters in designing of any underground structures, recommendation of support systems, and determination of input parameters for numerical modeling. Empirical methods classified the rock mass quantitatively into different classes having similar characteristics for easily understanding and construction of underground engineering structures. Despite its wide applications, the empirical methods do not evaluate the performance of support systems, stress redistribution, and deformation around the tunnel. Therefore, it is very important to consider these parameters in designing of optimum underground structure and support systems. .is deficiency of empirical method is solved by numerical methods.

Numerical modeling is gaining more attention in the field of civil and rock engineering for prediction of rock mass response to various excavation activities. The numerical methods are convenient, less costly, and less time consuming for the analysis of redistribution stresses and their effects on the behavior of rock mass and designing of structures within the rock mass environment. Numerical methods give the exact mathematical solution for the problem based on the engineering judgment and input parameters like physical and strength parameters of rock masses In this study, the rock mass along the tunnel axis was assessed using rock mass rating (RMR) and tunneling quality index (Q-system). The support system was recommended by these two classification systems. the rock mass behavior with the interaction of two different support systems was analyzed based on stresses, total deformation, and plastic yield thickness around the tunnel using finite element method- (FEM-) based Phase^2 software for selection of an appropriate support system for tunnel, which is of great importance for the practicing engineers in the field.

<u>Chainage 1+000m</u>

The selected chainage consists gray colored, medium grained, foliated, slight to moderately weathered, medium strong, banded Gneiss with quartz veins parallel to the foliation plane. The rough, planar, moderately weathered joints with fair RQD, have tight to few (1-3) mm aperture with clay fillings in some prominent joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.



(a) (b)

Figure 5-16 Phase Model to showing (a) Strength factor (b) Total displacement at Chainage 1+000m

In elastic analysis, the material type is considered as elastic that means rock mass behaves elastically. The major concern of this analysis is to find the strength factor around tunnel periphery that was shown in the Figure 5-16 (a). The strength factor is less than one around the tunnel in both cases with and without support. If the strength factor is less than one in elastic analysis, there will be failure of the material and for more additional information plastic analysis would be necessary (Phase^2 tutorial no.1). Strength factor is less than one for all tunnel sections considered. Hence Plastic analysis has been done in each chainage.

The total displacement of the tunnel is 2.71mm. This is about 0.06% of the tunnel span. The extend of the plastic zone (Rp) is about 3.684m as shown in tables (Annex A). The ration of distance from tunnel face to tunnel radius (X/Rt) is 1.19. And plastic zone to tunnel radius (Rp /Rt) is 1.754. By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal to 0.74. Therefore, the closure equals 2.005mm. This is about 74% of the total closure 2.71mm. 74% of total deformation will already take place before support is installed. Internal pressure factor of 0.04 yields the tunnel wall displacement computed above for the point of support installation.

In Plastic analysis, uniform distributed load is added to the tunnel in the initial stage. The factor is taken such that it will gradually reduce the magnitude of the pressure. As a result, tunnel deformation will increase as the pressure is lowered to zero. At this stage the internal pressure is removed, simulating the reduction of support due to the advance of tunnel face.

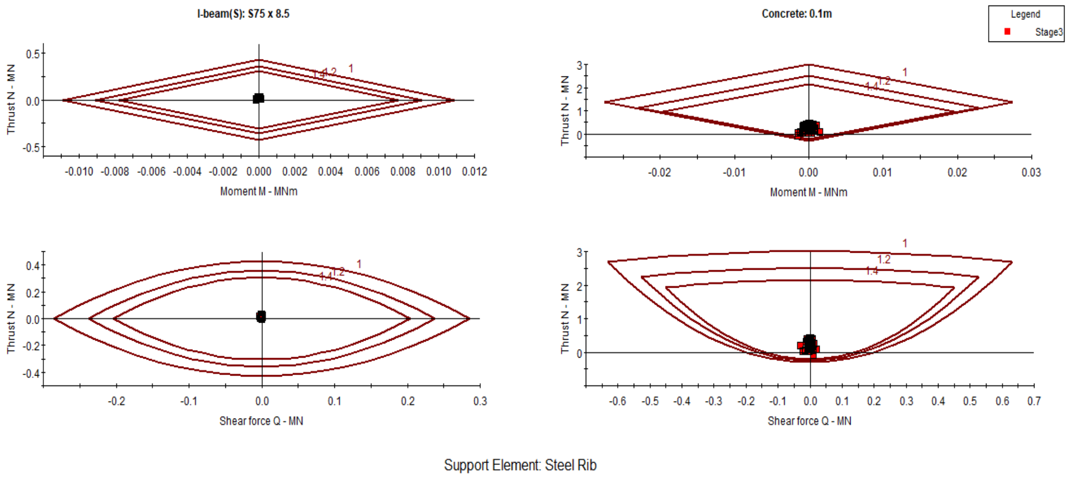


Figure 5-17 Support Capacity Plot at Chainage 1+000

Support is installed as shown in Table 5-14 the support capacity diagram, which are presented as Thrust Vs Shear Force and Thrust vs Moment for support system as suggested in Figure 5-17.

The supports are safe for the installation as it comes under the all three envelops, so the support for the similar geological condition with the similar Q-value, Steel Sets and shotcrete are preferred.

Table 5-14 Revised Support at Chainage 1+000

Steel sets			Tensile Strength	400	MPa
Type	ISSB 75		Weight	8.5	Kg/m
Sectional Depth	0.0762	m	Moment of Inertia	$1*10^6$	
Area	0.00107	m2	Shotcrete		
Young's Modulus	200000	MPa	Unconfined Compressive Strength	30	Mpa
Poisson Ratio	0.25		Young's Modulus	30000	Mpa
Spacing	0.5	m	Thickness of Shotcrete	100	mm
Compressive Strength	400	MPa	Poisson's Ratio	0.25	

Chainage 1+050m

The selected tunnel section consists gray colored, medium grained, foliated, slight to moderately weathered, medium strong, banded Gneiss with quartz veins parallel to the foliation plane. The rough, planar, moderately weathered joints with fair RQD, have tight to few (1-3) mm aperture with clay fillings in some prominent joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.

A uniform distributed load to the tunnel is in the initial stage. The factor is taken such that it will gradually reduce the magnitude of the pressure. As a result, tunnel deformation will increase as the pressure is lowered to zero. At this stage the internal pressure is removed, simulating the reduction of support due to the advance of tunnel face.

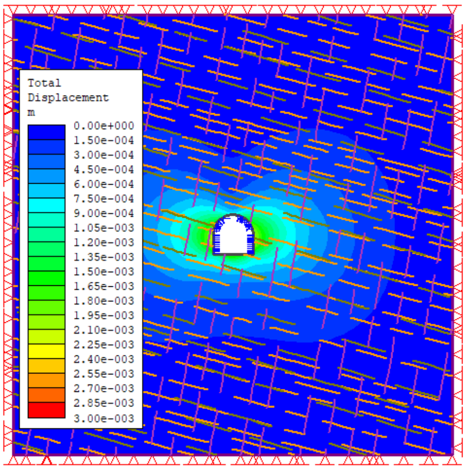


Figure 5-18 Phase Model to show Total Displacement at Chainage 1+050m

The total displacement of the tunnel is 2.26mm. This is about 0.054% of the tunnel span. The extend of the plastic zone (Rp) is about 3.058m as shown in tables (Annex A). The ration of distance from tunnel face to tunnel radius (X/Rt) is 1.19. And plastic zone to tunnel radius (Rp /Rt) is 1.456. By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal to 0.78. Therefore, the closure equals 1.767mm. This is about 78% of the total closure 2.26mm. 78% of total deformation will already take place before support is installed. Internal pressure factor of 0.1 yields the tunnel wall displacement computed above for the point of support installation.

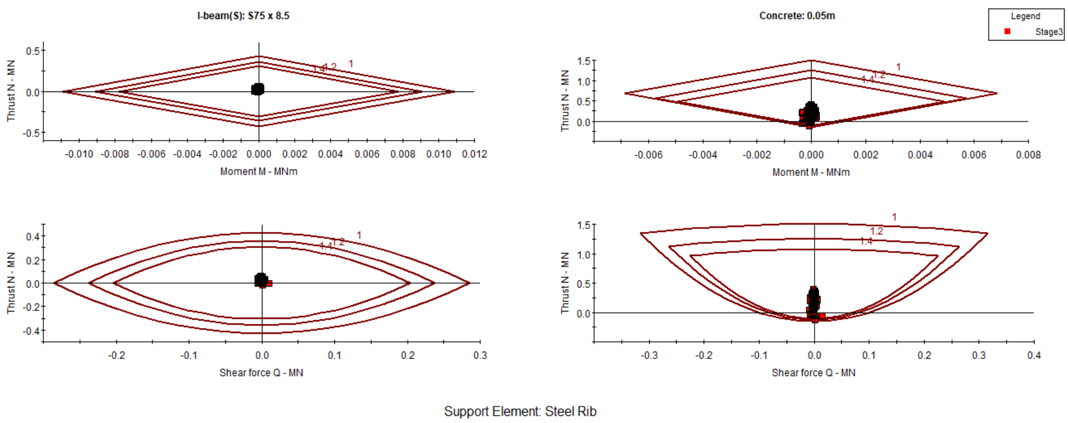


Figure 5-19 Support Capacity Plot at Chainage 1+050m

The support needs to redesign because of some elements was lies outside the envelop. The thickness of shotcrete was increases from 50mm to 75mm which comes under all the three envelop i.e. safe as shown in Figure 5-20

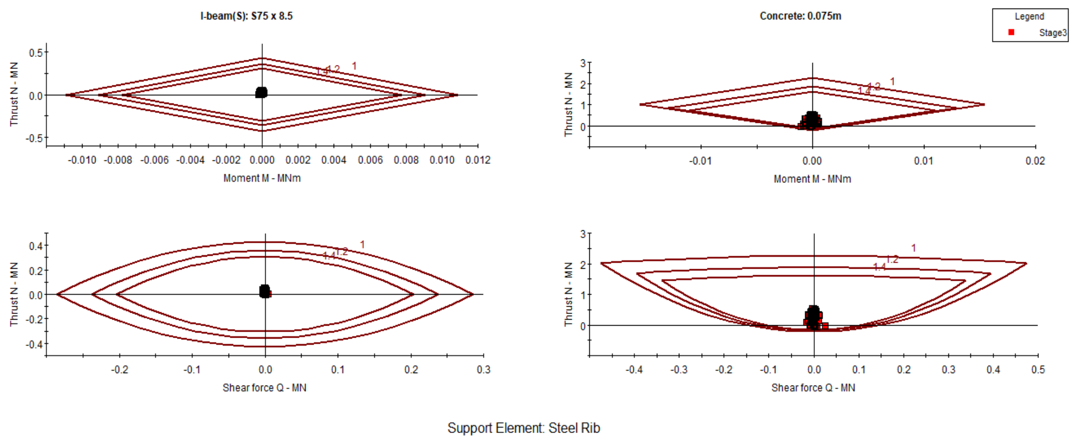


Figure 5-20 Support Capacity Plot at Chainage 1+050m after redesigned

Support is installed as shown in Table 5-15 the support capacity diagram, which are presented as Thrust Vs Shear Force and Thrust vs Moment for support system as suggested in Figure 5-20.

Table 5-15 Revised Support at Chainage 1+050

Steel sets			Tensile Strength	400	MPa
Type	ISSB 75		Weight	8.5	Kg/m
Sectional Depth	0.0762	m	Moment of Inertia	$1*10^6$	
Area	0.00107	m2	Shotcrete		
Young's Modulus	200000	Mpa	Unconfined Compressive Strength	30	Mpa
Poisson Ratio	0.25		Young's Modulus	30000	Mpa
Spacing	0.85	m	Thickness of Shotcrete	75	mm
Compressive Strength	400	Mpa	Poisson's Ratio	0.25	

<u>Chainage 1+100m</u>

The selected tunnel section consists gray colored, medium grained, foliated, slight to moderately weathered, medium strong, banded Gneiss with quartz veins parallel to the foliation plane. The rough, planar, moderately weathered joints with fair RQD, have tight to few (2-3) mm aperture with clay fillings in some prominent joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.

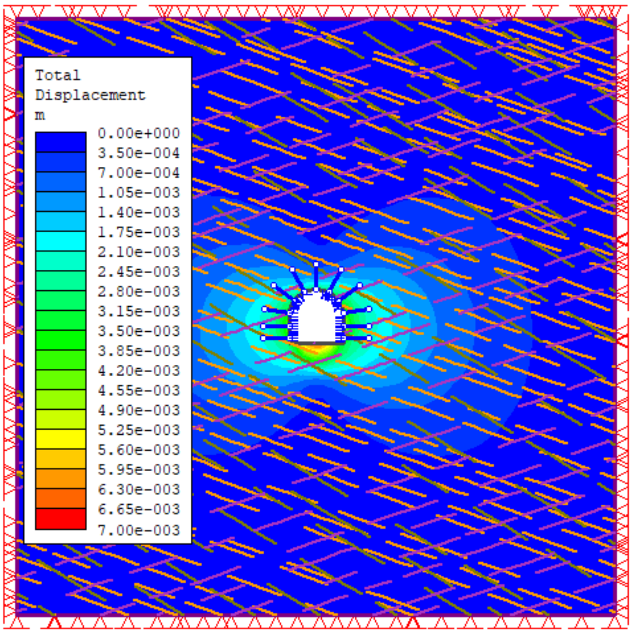


Figure 5-21 Phase Model Showing Total Displacement at Chainage 1+100m

The total displacement of the tunnel is 2.59mm. This is about 0.062% of the tunnel span. The extend of the plastic zone (Rp) is about 3.656m as shown in tables (Annex A). The ration of distance from tunnel face to tunnel radius (X/Rt) is 1.19. And plastic zone to tunnel radius (Rp /Rt) is 1.741. By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal to 0.73. Therefore, the closure equals 1.898mm. This is about 73% of the total closure 2.59mm. 73% of total deformation will already take place before support is installed. Internal pressure factor of 0.08 yields the tunnel wall displacement computed above for the point of support installation.

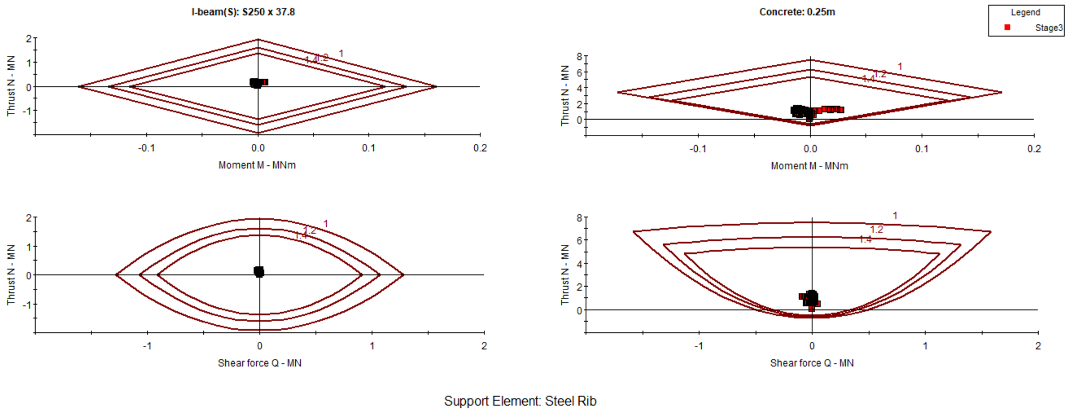


Figure 5-22 Support Capacity Plot at Chainage 1+100m

In Plastic analysis, uniform distributed load is added to the tunnel in the initial stage. The factor is taken such that it will gradually reduce the magnitude of the pressure. As a result, tunnel deformation will increase as the pressure is lowered to zero. At this stage the internal pressure is removed, simulating the reduction of support due to the advance of tunnel face.

Support system is redesigned and installed as shown in Table 5-16 the support capacity diagram, which are presented as Thrust Vs Shear Force and Thrust vs Moment for support system as suggested as in Figure 5-22.

Table 5-16 Revised Support at Chainage 1+100m

Steel sets			Rock Bolt		
Type	ISSB 250		Type	End Anchored	
Sectional Depth	0.203	m	Length	2	m
Area	0.0048	m2	Diameter	20	mm
Young's Modulus	200000	Mpa	Bolt Modulus	200000	Mpa
Poisson Ratio	0.25		Spacing	1.5	m
Spacing	0.5	m	Shotcrete		
Compressive Strength	400	Mpa	Unconfined Compressive Strength	30	Mpa
Tensile Strength	400	Mpa	Young's Modulus	30000	Mpa
Weight	37.8	kg/m	Thickness of Shotcrete	250	mm
Moment of Inertia	5.12*10-5		Poisson's Ratio	0.25	

Chainage 1+150m

The selected tunnel face consists gray colored, medium grained, foliated, highly weathered, weak, banded Gneiss with quartz veins parallel to the foliation plane. The individual beds are 20-40 cm thick. High amount of clay coating and filling can be seen on crown part and upper tunnel face. The rough, planar, moderately weathered joints with poor RQD, have tight to few (3 mm) aperture with clay fillings in some most of the joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is dry to damp.

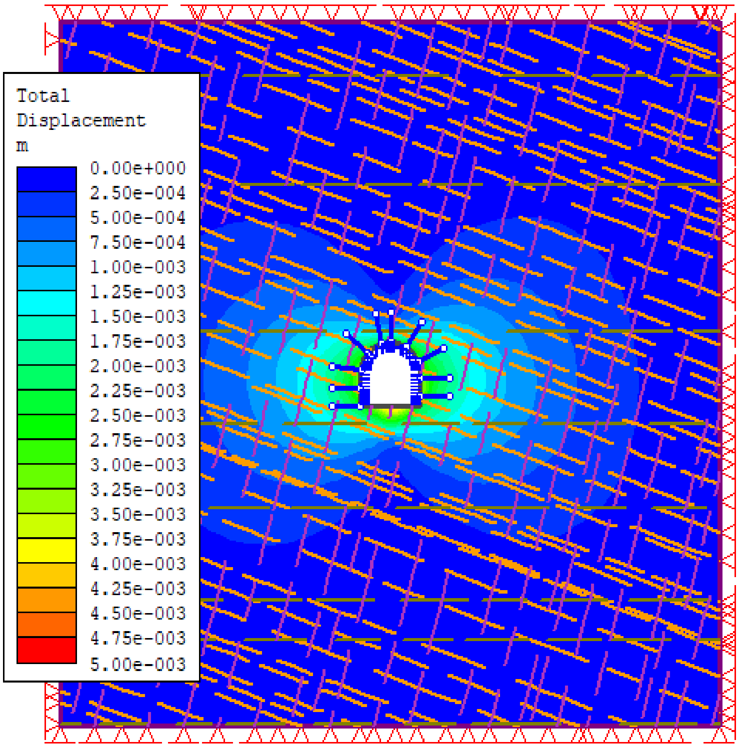


Figure 5-23 Phase Model Showing Total Displacement at Chainage 1+150m

The total displacement of the tunnel is 4.37mm. This is about 0.104% of the tunnel span. The extend of the plastic zone (Rp) is about 4.849m as shown in tables (Annex A). The ration of distance from tunnel face to tunnel radius (X/Rt) is 1.19. And plastic zone to tunnel radius (Rp /Rt) is 2.309. By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal to 0.64. Therefore, the closure equals 2.802mm. This is about 64% of the total closure 4.37m. 64% of total deformation will already take place before support is installed. Internal pressure factor of 0.02 yields the tunnel wall displacement computed above for the point of support installation.

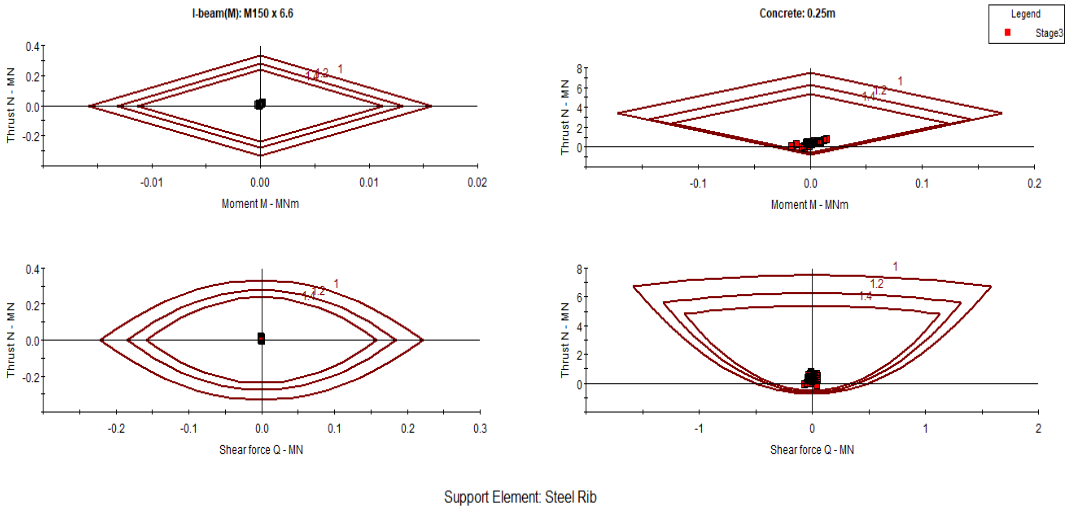


Figure 5-24 Support Capacity Plot at Chainage 1+150m

Support system is installed as shown in Table 5-17 the support capacity diagram, which are presented as Thrust Vs Shear Force and Thrust vs Moment for support system as suggested as in Figure 5-24.

Table 5-17 Revised Support at Chainage 1+150m

Steel sets			Rock Bolt		
Type	ISMB 150		Type	End Anchored	
Sectional Depth	0.152	m	Length	2	m
Area	0.000832	m2	Diameter	20	mm
Young's Modulus	200000	Mpa	Bolt Modulus	200000	Mpa
Poisson Ratio	0.25		Spacing	1.4	m
Spacing	0.5	m	Shotcrete		
Compressive Strength	400	Mpa	Unconfined Compressive Strength	30	Mpa
Tensile Strength	400	Mpa	Young's Modulus	30000	Mpa
Weight	6.6	kg/m	Thickness of Shotcrete	250	mm
Moment of Inertia	3.01E-06		Poisson's Ratio	0.25	

Chainage 1+200m

The selected tunnel face consists gray colored, medium grained, foliated, highly weathered, weak, banded Gneiss with quartz veins parallel to the foliation plane. High amount of clay coating and filling can be seen on crown part and upper tunnel face.

Below the spring line, clay filling upto 1m can see. The rough, planar, moderately weathered joints with poor RQD, have tight to few (2 mm) aperture with clay fillings in most of the joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.

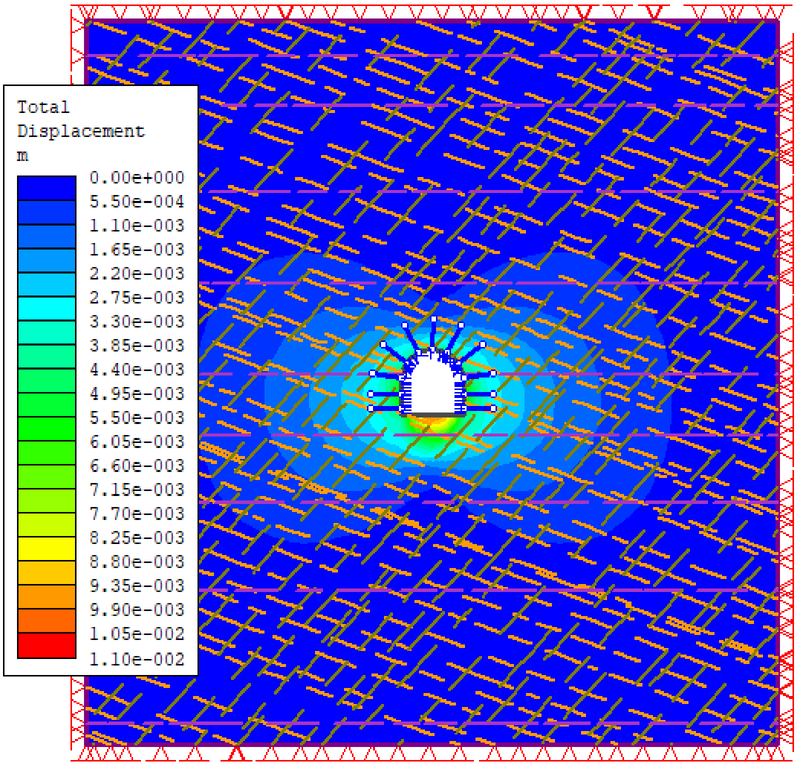


Figure 5-25 Phase Model showing Total Displacement at Chainage 1+200m

The total displacement of the tunnel is 12.71mm. This is about 0.303% of the tunnel span. The extend of the plastic zone (Rp) is about 9.125m as shown in tables (Annex A). The ration of distance from tunnel face to tunnel radius (X/Rt) is 1.19. And plastic zone to tunnel radius (Rp /Rt) is 4.345. By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal to 0.44. Therefore, the closure equals 5.596mm. This is about 44% of the total closure 12.71m. 44% of total deformation will already take place before support is installed. Internal pressure factor of 0.08 yields the tunnel wall displacement computed above for the point of support installation.

In Plastic analysis, uniform distributed load is added to the tunnel in the initial stage. The factor is taken such that it will gradually reduce the magnitude of the pressure. As a result, tunnel deformation will increase as the pressure is lowered to zero. At this stage the internal pressure is removed, simulating the reduction of support due to the advance of tunnel face.

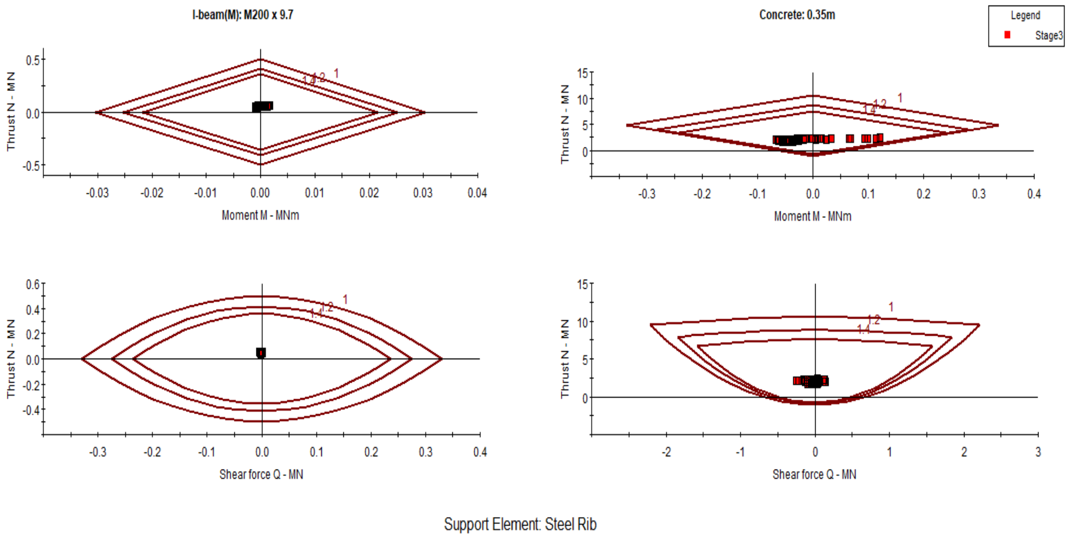


Figure 5-26 Support Capacity Plot at Chainage 1+200m

Support system is designed and installed as shown in Table 5-18 the support capacity diagram, which are presented as Thrust Vs Shear Force and Thrust vs Moment for support system as suggested as in Figure 5-26.

Table 5-18 Revised Support at Chainage 1+200m

Steel sets			Rock Bolt		
Type	ISMB 200		Type	End Anchored	
Sectional Depth	0.0203	m	Length	2	m
Area	0.00124	m2	Diameter	20	mm
Young's Modulus	200000	Mpa	Bolt Modulus	200000	Mpa
Poisson Ratio	0.25		Spacing	1	m
Spacing	0.5	m	Shotcrete		
Compressive Strength	400	Mpa	Unconfined Compressive Strength	30	Mpa
Tensile Strength	400	Mpa	Young's Modulus	30000	Mpa
Weight	9.7	kg/m	Thickness of Shotcrete	350	mm
Moment of Inertia	7.70E-06		Poisson's Ratio	0.25	

Chainage 1+250m

The Excavated tunnel face consists gray colored, medium grained, foliated, highly weathered, weak, banded Gneiss with quartz veins parallel to the foliation plane. The individual beds are 20-40 cm thick. High amount of clay coating and filling can be seen on crown part and upper tunnel face.

The rough, planar, moderately weathered joints with poor RQD, have tight to few (3 mm) aperture with clay fillings in most of the joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.

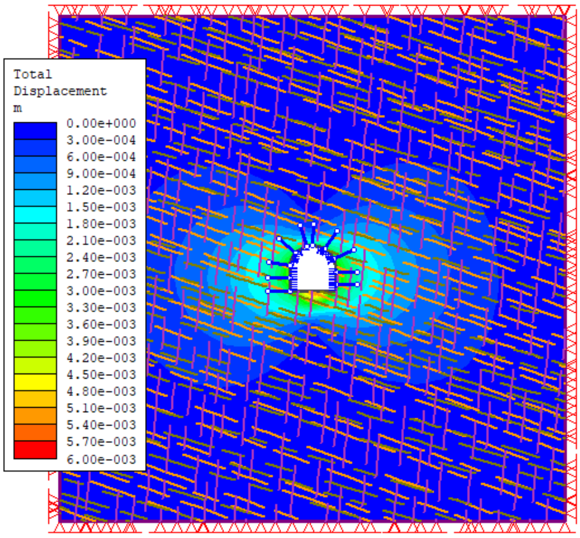


Figure 5-27 Phase Model showing Total Displacement at Chainage 1+250m

The total displacement of the tunnel is 6.04mm. This is about 0.144% of the tunnel span. The extend of the plastic zone (Rp) is about 5.941m as shown in tables (Annex A). The ration of distance from tunnel face to tunnel radius (X/Rt) is 1.19. And plastic zone to tunnel radius (Rp /Rt) is 2.829. By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal to 0.58. Therefore, the closure equals 3.5mm. This is about 58% of the total closure 6.06mm. 58% of total deformation will already take place before support is installed. Internal pressure factor of 0.04 yields the tunnel wall displacement computed above for the point of support installation.

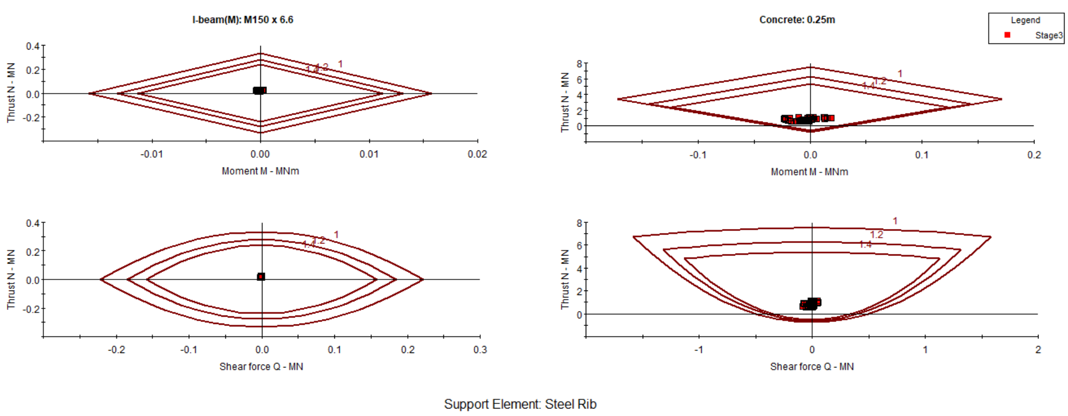


Figure 5-28 Support Capacity Plot at Chainage 1+250m for Model Set-III

Support system is redesigned and installed as shown in Table 5-19 the support capacity diagram, which are presented as Thrust Vs Shear Force and Thrust vs Moment for support system as suggested as in Figure 5-28.

Table 5-19 Revised Support at chainage 1+250m

Steel sets			Rock Bolt		
Type	ISMB 150		Type	End Anchored	
Sectional Depth	0.152	m	Length	2	m
Area	0.000832	m2	Diameter	20	mm
Young's Modulus	200000	Mpa	Bolt Modulus	200000	Mpa
Poisson Ratio	0.25		Spacing	1.4	m
Spacing	0.5	m	Shotcrete		
Compressive Strength	400	Mpa	Unconfined Compressive Strength	30	Mpa
Tensile Strength	400	Mpa	Young's Modulus	30000	Mpa
Weight	6.6	kg/m	Thickness of Shotcrete	250	mm
Moment of Inertia	3.01E-06		Poisson's Ratio	0.25	

Chainage 1+300m

The Excavated tunnel face consists gray colored, medium grained, foliated, Low weathered, Medium to strong, banded Gneiss with quartz veins parallel to the foliation plane. The individual beds are 20-100 cm thick. The rough, planar, moderately weathered joints with good RQD, have tight to few (1 mm) aperture with clay coating in few joints. Joints are closely to moderately spaced and have medium persistency. Surface water condition of the area is dry to damp.

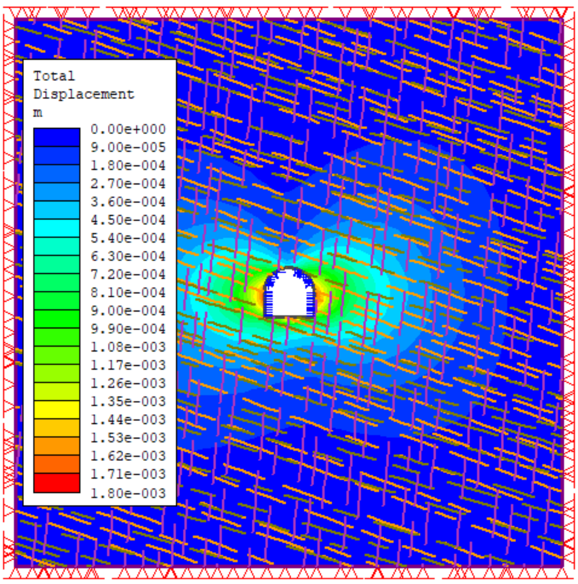


Figure 5-29 Phase Model showing Total Displacement at Chainage 1+300m

The total displacement of the tunnel is 1.77mm. This is about 0.042% of the tunnel span. The extend of the plastic zone (Rp) is about 3.21m as shown in tables (Annex A). The ration of distance from tunnel face to tunnel radius (X/Rt) is 1.19. And plastic zone to tunnel radius (Rp /Rt) is 1.529. By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal to 0.76. Therefore, the closure equals 1.349mm. This is about 76% of the total closure 1.77mm. 76% of total deformation will already take place before support is installed. Internal pressure factor of 0.1 yields the tunnel wall displacement computed above for the point of support installation.

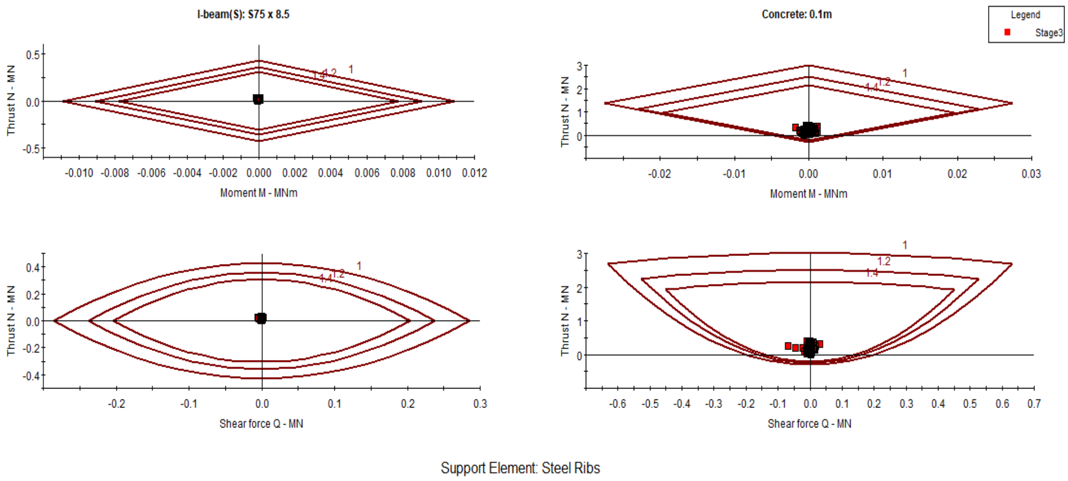


Figure 5-30 Support Capacity Plot at Chainage 1+300m

Support system is redesigned and installed as shown in Table 5-20 the support capacity diagram, which are presented as Thrust Vs Shear Force and Thrust vs Moment for support system as suggested as in Figure 5-30.

Table 5-20 Revised Support at Chainage 1+300m

Steel sets			Tensile Strength	400	MPa
Type	ISSB 75		Weight	8.5	Kg/m
Sectional Depth	0.0762	m	Moment of Inertia	1.21E06	
Area	0.00107	m2	Shotcrete		
Young's Modulus	200000	Mpa	Unconfined Compressive Strength	30	Mpa
Poisson Ratio	0.25		Young's Modulus	30000	Mpa
Spacing	0.5	m	Thickness of Shotcrete	100	mm
Compressive Strength	400	Mpa	Poisson's Ratio	0.25	

Chainage 1+350m

The Excavated tunnel face consists gray colored, medium grained, foliated, moderately weathered, Medium strong to strong, banded Gneiss with quartz veins parallel to the foliation plane. The individual beds are 20-80 cm thick. The rough, planar, moderately weathered joints with fair RQD, have tight to few (1-2 mm) aperture with clay coating in few joints. Joints are closely to moderately spaced and have medium persistency. Surface water condition of the area is dry to damp.

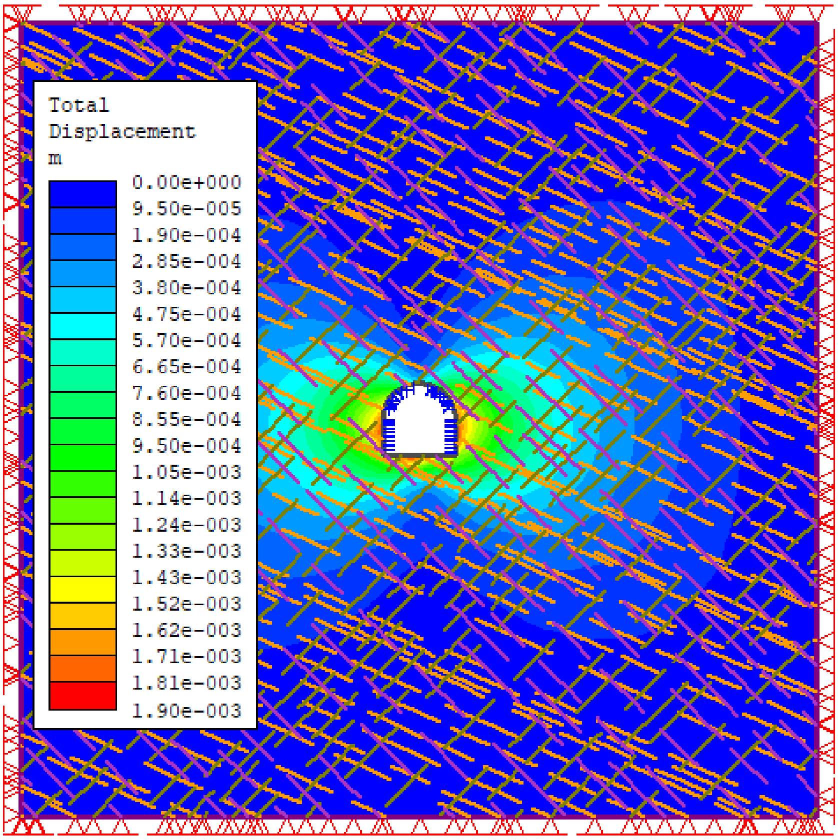


Figure 5-31 Phase Model showing Total Displacement at Chainage 1+350m

The total displacement of the tunnel is 1.88mm. This is about 0.045% of the tunnel span. The extend of the plastic zone (Rp) is about 3.577m as shown in tables (Annex A). The ration of distance from tunnel face to tunnel radius (X/Rt) is 1.19. And plastic zone to tunnel radius (Rp /Rt) is 1.703. By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal to 0.82. Therefore, the closure equals 1.547mm. This is about 82% of the total closure 1.88mm. 82% of total deformation will already take place before support is installed. Internal pressure factor of 0.02 yields the tunnel wall displacement computed above for the point of support installation.

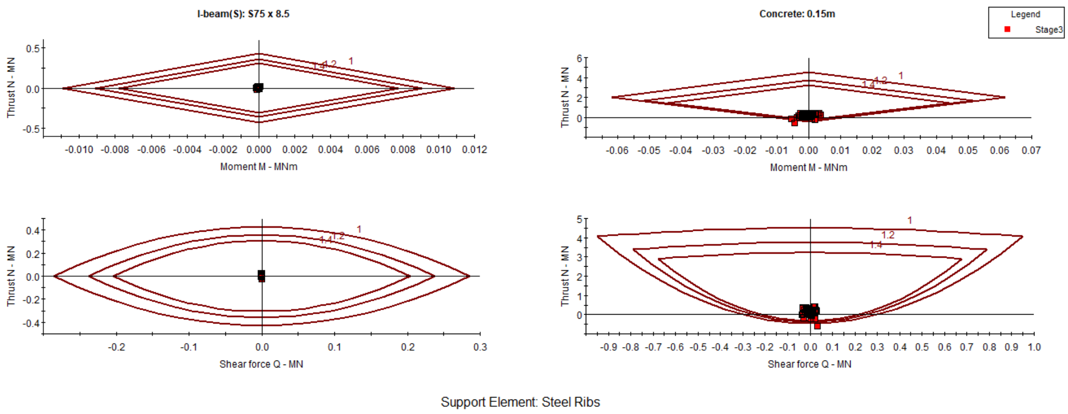


Figure 5-32 Support Capacity Plot at Chainage 1+350m

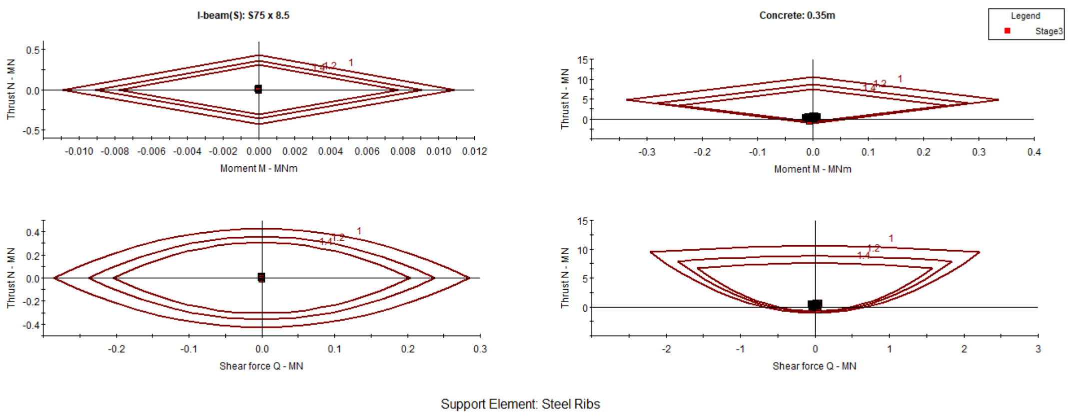


Figure 5-33 Support Capacity Plot at Chainage 1+350m after redesign

Support system is redesigned and installed as shown in Table 5-21 the support capacity diagram, which are presented as Thrust Vs Shear Force and Thrust vs Moment for support system as suggested as in Figure 5-33.

Table 5-21 Revised Support at chainage 1+350m

Steel sets			Tensile Strength	400	MPa
Type	ISSB 75		Weight	8.5	Kg/m
Sectional Depth	0.0762	m	Moment of Inertia	1.21E-06	
Area	0.00107	m2	Shotcrete		
Young's Modulus	200000	Mpa	Unconfined Compressive Strength	30	Mpa
Poisson Ratio	0.25		Young's Modulus	30000	Mpa
Spacing	0.5	m	Thickness of Shotcrete	250	mm
Compressive Strength	400	Mpa	Poisson's Ratio	0.25	

Chainage 1+400m

The Excavated tunnel face consists gray colored, medium grained, foliated, moderately weathered, Medium strong to strong, banded Gneiss with quartz veins parallel to the foliation plane. The individual beds are 10-60 cm thick. The rough, planar, moderately weathered joints with fair RQD, have tight to few (1-3 mm) aperture with clay coating and filling in few joints. Joints are closely to moderately spaced and have medium persistency. Surface water condition of the area is dry to damp.

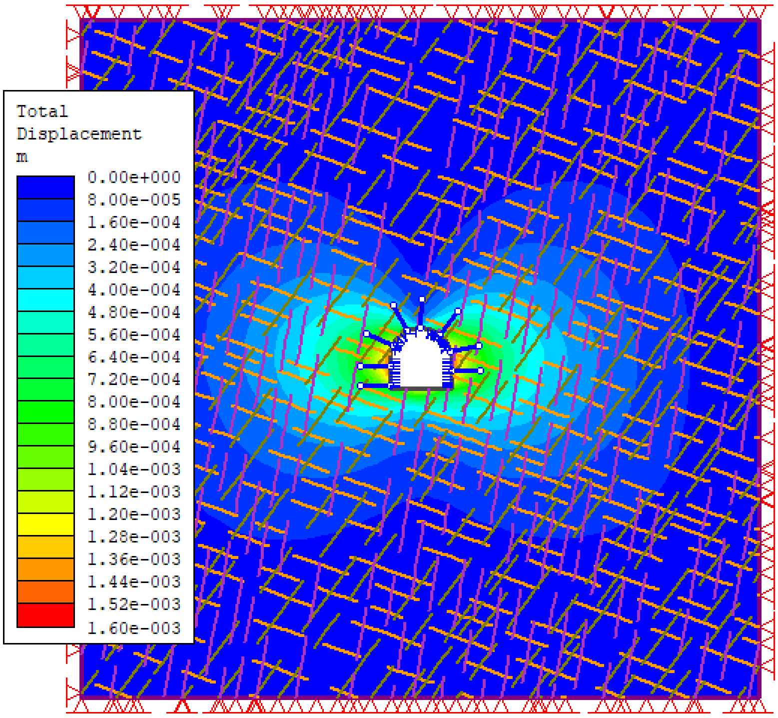


Figure 5-34 Phase Model showing Total Displacement at chainage 1+400m

In Plastic analysis, uniform distributed load is added to the tunnel in the initial stage. The factor is taken such that it will gradually reduce the magnitude of the pressure. As a result, tunnel deformation will increase as the pressure is lowered to zero. At this stage the internal pressure is removed, simulating the reduction of support due to the advance of tunnel face.

The total displacement of the tunnel is 1.58mm. This is about 0.038% of the tunnel span. The extend of the plastic zone (Rp) is about 3.567m as shown in tables (Annex A). The ration of distance from tunnel face to tunnel radius (X/Rt) is 1.19. And plastic zone to tunnel radius (Rp /Rt) is 1.699. By using Vlachopoulos and Diederichs method, the above values are plotted gives ratio of closure to maximum closure equal to 0.73. Therefore, the closure equals 1.15mm. This is about 73% of the total closure 1.58mm.

73% of total deformation will already take place before support is installed. Internal pressure factor of 0.1 yields the tunnel wall displacement computed above for the point of support installation.

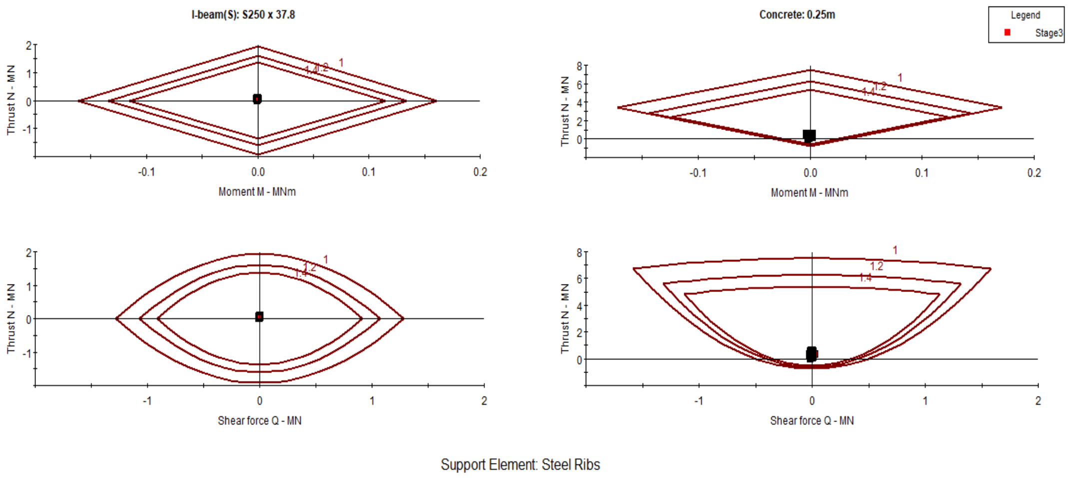


Figure 5-35 Support Capacity Plot at Chainage 1+400m

Support system installed as shown in Table 5-22 the support capacity diagram, which are presented as Thrust Vs Shear Force and Thrust vs Moment for support system as suggested as in Figure 5-35.

Table 5-22 Revised Support at chainage 1+400m

Steel sets			Rock Bolt		
Type	ISSB 250		Type	End Anchored	
Sectional Depth	0.203	m	Length	2	m
Area	0.0048	m2	Diameter	20	mm
Young's Modulus	200000	Mpa	Bolt Modulus	200000	Mpa
Poisson Ratio	0.25		Spacing	1.5	m
Spacing	0.5	m	Shotcrete		
Compressive Strength	400	Mpa	Unconfined Compressive Strength	30	Mpa
Tensile Strength	400	Mpa	Young's Modulus	30000	Mpa
Weight	37.8	kg/m	Thickness of Shotcrete	250	mm
Moment of Inertia	5.12*10-5		Poisson's Ratio	0.25	

5.2.3.4 Support Adopted by the Project

The site is under-construction and they have designed the support with the preliminary data for the construction purpose. We will analysis these supports using numerical modelling and comparing with the above designed support. The site office categories the various support system for different Q value. The support adopted by the construction company at site are shown in Figure 5-36 and Figure 5-37. The all four type of support are used in our study section.

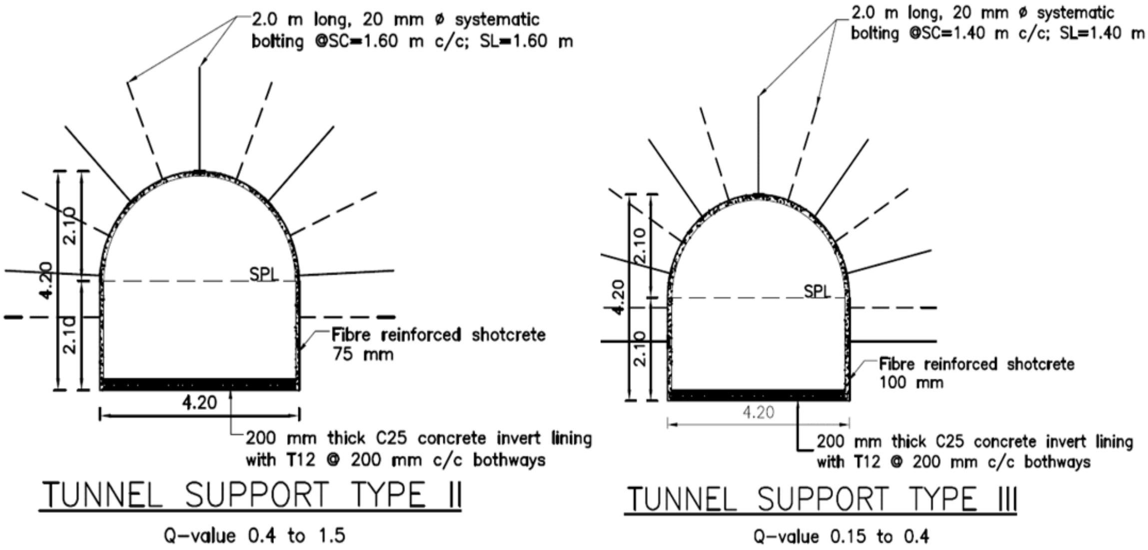


Figure 5-36 Rock Support Type II & Type III

Support type II and Type III have fibre reinforce shotcrete of 75mm and 100 mm respectively and same size of bolt used with center to center spacing 1.6m and 1.4 m respectively. These two types of support are applied for the rock class E.

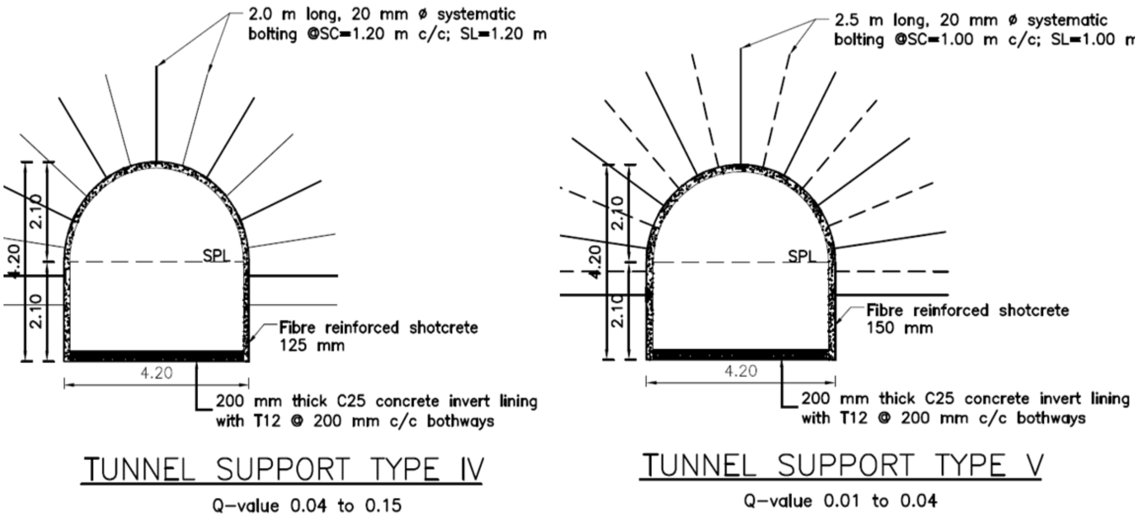


Figure 5-37 Rock Support Type IV & V

Similarly, for rock class F they used a support type IV and Type V. In this type of support fibre reinforced shotcrete of 125mm -150mm thickness was applied and of same size bolt with spacing 1m – 1.2m.

5.3 Discussion

Different methods i.e. empirical approaches, analytical and numerical were used - either in combine or in individual form - to analyze the support for the headrace tunnel of Super-Madi Hydroelectric Project. All the methods have some uniqueness, such as RMR value gives the idea for stand-up time whereas Q-system does not. Since Q-system consider the more parameters (Including strength reduction factor) and have better support chart for estimation.

As an empirical approach, Rock mass classification system is used to predict the quality of rock mass either in the form of Q-Parameter or RMR parameter. Selection or rock bolt and lining type are also depending upon their market availability in the vicinity area. For more accurate and optimum solution to estimate the rock support system, numerical analysis technique was suggested which is discussed in this chapter. For the analysis through numerical approach, initial values of rock support for the simulation were needed. In this regard, suggested values of rock supports from the Q-system would be better choice and was used. Various combination of lining (shotcrete, Concrete, RCC) and Bolts are analyzed through numerical analysis and have eventually estimated the optimum values of support combinations which provides the required degree of safety and have relatively least cost. In numerical analysis, this requirement was achieved by reducing the maximum number of yielded elements. The estimated rock support was then reduced using critical pressure criteria in analytical method. The support pressure greater than the critical pressure indicates no failure condition. Therefore, support estimated from empirical methods were reduced such that the support pressure was higher than that critical pressure developed during excavation.

Analysis shows that rock support estimated from analytical method at chainage 1+100m, 1+200m and 1+350m tends to fail using numerical analysis that needs to redesign for the required factor of safety. Joint properties play crucial role in the rock mass stability during excavation. The depth geological survey of the area enhances the tunnel design, construction and stability. Which also help in the estimating required rock support and help in optimization. It was strongly recommended to perform numerical analysis, which may give more reliable result than any other methods.

The steel ribs were recommended to apply as a major rock support for stability with better factor of safety. All the section are numerically analyzed and suitable support are listed in Table 5-23 Revised Rock Support with Comparing with Project SupportTable 5-23 with the comparison of support taken by the project.

Table 5-23 Revised Rock Support with Comparing with Project Support

Chainage	Support Type	Support Designed by Project	This Study Reviewed Support
1+000	III	Systematic bolting of 2 m long, 20 mm diameter at 1.4m c/c spacing and Fibre reinforced sprayed concrete of 100 mm thickness.	Fibre Reinforce Shotcrete of 100 mm thickness and Steel Sets ISSB75*8.5 at 0.5 c/c.
1+050	II	Systematic bolting of 2 m long, 20 mm diameter at 1.6m c/c spacing and Fibre reinforced sprayed concrete of 75 mm thickness.	Fibre Reinforce Shotcrete of 50 mm thickness and Steel Sets ISSB75*8.5 at 0.85 c/c.
1+100	III	Systematic bolting of 2 m long, 20 mm diameter at 1.4m c/c spacing and Fibre reinforced sprayed concrete of 100 mm thickness.	Systematic Bolting of 2m length -20mm dia. @ 1.5m c/c. Fibre Reinforce Shotcrete of 250 mm thickness and Steel Sets ISSB250*37.8 at 0.5 c/c.
1+150	IV	Systematic bolting of 2 m long, 20 mm diameter at 1.2m c/c spacing and Fibre reinforced sprayed concrete of 125 mm thickness.	Systematic Bolting of 2m length -20mm dia. @ 1.4m c/c. Fibre Reinforce Shotcrete of 250 mm thickness and Steel Sets ISMB150*6.6 at 0.5
1+200	V	Systematic bolting of 2.5 m long, 20 mm diameter at 1.0m c/c spacing and Fibre	Systematic Bolting of 2m length -20mm dia. @ 1.0m c/c. Fibre Reinforce Shotcrete of 350 mm

		reinforced sprayed concrete of 150 mm thickness.	thickness and Steel Sets ISMB200*9.7 at 0.5
1+250	IV	Systematic bolting of 2 m long, 20 mm diameter at 1.2m c/c spacing and Fibre reinforced sprayed concrete of 125 mm thickness.	Systematic Bolting of 2m length -20mm dia. @ 1.4m c/c. Fibre Reinforce Shotcrete of 250 mm thickness and Steel Sets ISMB150*6.6 at 0.5
1+300	II	Systematic bolting of 2 m long, 20 mm diameter at 1.6m c/c spacing and Fibre reinforced sprayed concrete of 75 mm thickness.	Fibre Reinforce Shotcrete of 100 mm thickness and Steel Sets ISSB75*8.5 at 0.5 c/c.
1+350	II	Systematic bolting of 2 m long, 20 mm diameter at 1.6m c/c spacing and Fibre reinforced sprayed concrete of 75 mm thickness.	Fibre Reinforce Shotcrete of 150 mm thickness and Steel Sets ISSB75*8.5 at 0.5 c/c.
1+400	III	Systematic bolting of 2 m long, 20 mm diameter at 1.4m c/c spacing and Fibre reinforced sprayed concrete of 100 mm thickness.	Systematic Bolting of 2m length -20mm dia. @ 1.5m c/c. Fibre Reinforce Shotcrete of 250 mm thickness and Steel Sets ISSB250*37.8 at 0.5

The estimated support using various empirical as well as analytical method were tested in Phase^2 model. The presence of various joint in the rock mass tends to block failure during exaction. The installed support must stabilize such blocks. All the section is tested for the block failure before and after estimated minimum support applied. All the sections were safe against block failure with factor of safety is always greater than desired value (1.5). some section has critical factor of safety for the applied minimum support. So, it can be increase while excavation at the site. Such section is 1+100m, 1+200m, and 1+350m, these sections has lower factor of safety for the roof wedge.

The most common problem in the tunneling is squeezing. Hoek and Marinos (2000) methods were applied for the prediction of squeezing at the study section. This method was widely used for estimation of the deformation caused by squeezing and estimation of support pressure required in the squeezing tunnel. The strain in percentage at every section that lies in the < 1 %. That indicate the minor support problem not the squeezing problem. So, the alignment of 400m was safe against the squeezing problem.

Therefore, the research shows that the different method of estimating rock support may provide different support system. By the analysis using finite element approach the supports are optimized for the economic project completion. The many section has overestimated support from the chart approach which were reduced in the analytical as well as finite element methods. These supports can withstand the block stability and squeezing problems at the weakest part of the tunnel. The Total displacements and stress related problems are very less in all sections. It may be due to limitation to account the effect of discontinuities. Therefore, the deformation may be higher in actual case than the computed values. Lack of consideration of all the minor joint sets that prevails in the rock mass may be one of major cause of error in the analysis. Effect of ground water was not considered as it may create problem during excavation. Therefore, it is suggested to make drain holes to pass out the possible water.

Wedge failure analysis shows that the alignment of tunnel was good for block stability. All the detach blocks have factor of safety more than one except roof wedge at some section, which means the tunnel is stable without support system as well. The squeezing problem was not actually identified along the headrace tunnel. The empirical method suggests same type of support if they fall under same category but geological condition has to be verified. Therefore, the rock support suggested by empirical relation only is not adequate for apply. Numerical analysis gives better result on that.

6 CONCLUSION AND RECOMMENDATION

6.1 Conclusions

In this study, rock mass classification is carried out in conjunction with numerical analysis to study the support requirement of the headrace tunnel of Super-Madi Hydroelectric Project, located in the Himalayan Region of Nepal. These simulated models were developed based on the following assumptions: Supports were installed instantly after excavation, Elastoplastic behavioral model using generalized Hoek–Brown criterion is used to simulate the models and Tunnel model is 2D considering plane strain problem.

The study explored the design of a support system for an underground Inverted D-shaped tunnel. Overall this study found that a comprehensive design of an adequate tunnel support system cannot be accomplished using only one approach. The required support selected should be the minimum supports results of the different solutions. Through rigorous designs, underground geotechnical engineering instability problems causing tunnel failure can be minimized. The stability analysis of models developed for each geotechnical unit in RS^2 was carried out after installment of support estimation from rock mass classification (Q & RMR system), Analytical Method (using critical pressure criteria) and support used by the project. From all the result and discussion of that, this research concludes as following:

- The empirical method gives very first estimate for the support analysis. The Q and RMR method also help in rock mass classification which was helpful in further design.
- The empirical method suggests same type of support if they fall under same category but geological condition has to be verified.
- Some input parameter for the numerical analysis were calculated form empirical relations.
- The support capacity plot was very useful for the analysis of rock support estimation. That can be used for the optimizing supports.
- Therefore, rock mass classification approach only is not adequate to design and estimation of tunnel support. Numerical analysis was very helpful to estimate

the tunnel support in such geological region where rock masses are very poor with high rock cover.

- These revised estimated supports are further analyzed for the wedge failure using UNWEDGE programming. This must helpful for the verification of these support can stand for possible wedge failure.
- Block failure analysis shows that the alignment of the Headrace Tunnel is good. Most of the detach wedges have factor of safety more than one, some have lesser than that but all the wedge is safe after applying revised support, which means the headrace tunnel section is stable from the block failure.
- The tunnel section is analyzed for the squeezing problem using various empirical, semi empirical and finite element modelling. No any serious squeezing problem was found.
- Effect of ground water is not considered as it may create problem during excavation. Therefore, it is suggested to make drain holes to pass out the possible water.

6.2 Recommendations

- The effect of water has not been considered in the analysis of the thesis. The result can be improved by considering the water effect in the analysis.
- The seismic effect in the tunnel is not considered in this research. The result can be improved by applying the seismic effect.

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ANNEX A: DETAILS OF CALCULATIONS

Calculation of Closure and Tunnel Relaxation Stage using Vlachopoulos and Diederichs (2009)

Chainage	Distance from Tunel Face	Tunel Radius	Plastic Zone Radius	Dis. From Tunnel face/Tunnel radius	Plastic zone radius/Tunnel Radius	Closure/Max. Closer	Max Displacement	So, Closure	Tunnel Relaxation Stage
1+000	2.5	2.100	3.684	1.190	1.754	0.740	0.00271012	0.002005	Stage 7
1+050	2.5	2.100	3.058	1.190	1.456	0.780	0.00226542	0.001767	Stage 5
1+100	2.5	2.100	3.656	1.190	1.741	0.730	0.00259994	0.001898	Stage 6
1+150	2.5	2.100	4.849	1.190	2.309	0.640	0.00437814	0.002802	Stage 8
1+200	2.5	2.100	9.125	1.190	4.345	0.440	0.01271910	0.005596	Stage 6
1+250	2.5	2.100	5.941	1.190	2.829	0.580	0.00604210	0.003504	Stage 7
1+300	2.5	2.100	3.210	1.190	1.529	0.760	0.00177558	0.001349	Stage 5
1+350	2.5	2.100	3.577	1.190	1.703	0.820	0.00188657	0.001547	Stage 8
1+400	2.5	2.100	3.567	1.190	1.699	0.730	0.00158035	0.001154	Stage 5

Calculation of Field Stress for Numerical Modeling

Chainage	Overburden (m)	Q-Value	GSI	σ_h	σ_v	σ_z	Remarks
1+000	306.42	0.344	34	2.053	8.212	8.212	
1+050	292.61	0.583	39	1.960	7.842	7.842	
1+100	285.26	0.229	31	1.911	7.645	7.645	
1+150	275.37	0.075	21	1.845	7.380	7.380	
1+200	274.05	0.038	14	1.836	7.345	7.345	
1+250	253.04	0.070	20	1.695	6.781	6.781	
1+300	219.05	1.250	46	1.468	5.871	5.871	
1+350	186.85	1.083	45	1.252	5.008	5.008	
1+400	167.63	0.271	32	1.123	4.492	4.492	

ANNEX B: PHASE MODELLING & RESULT

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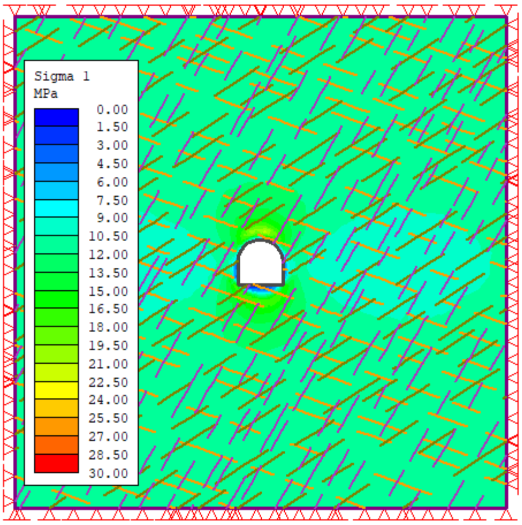

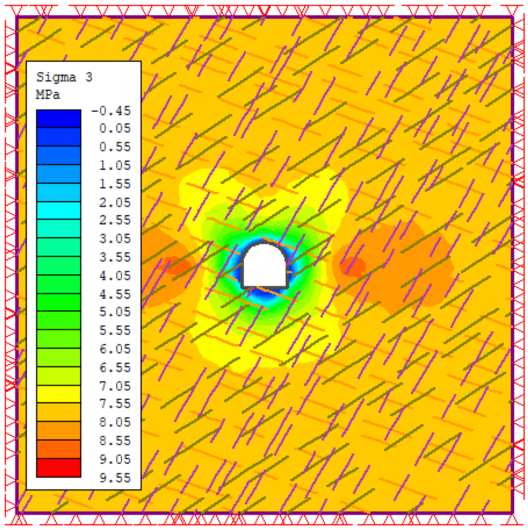

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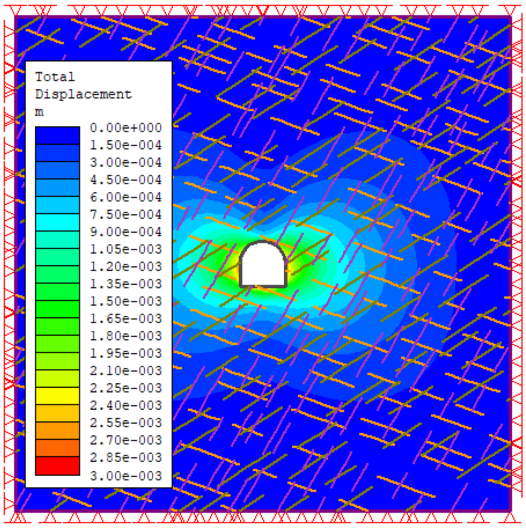

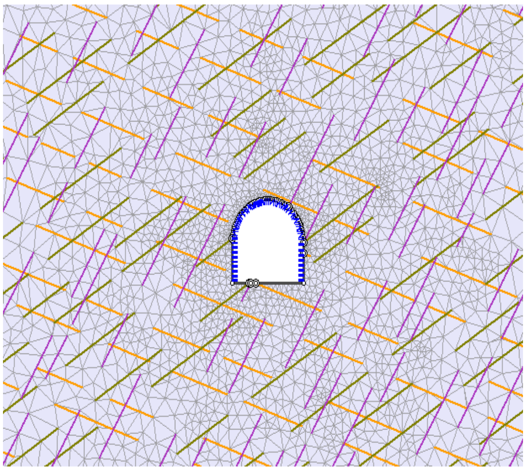

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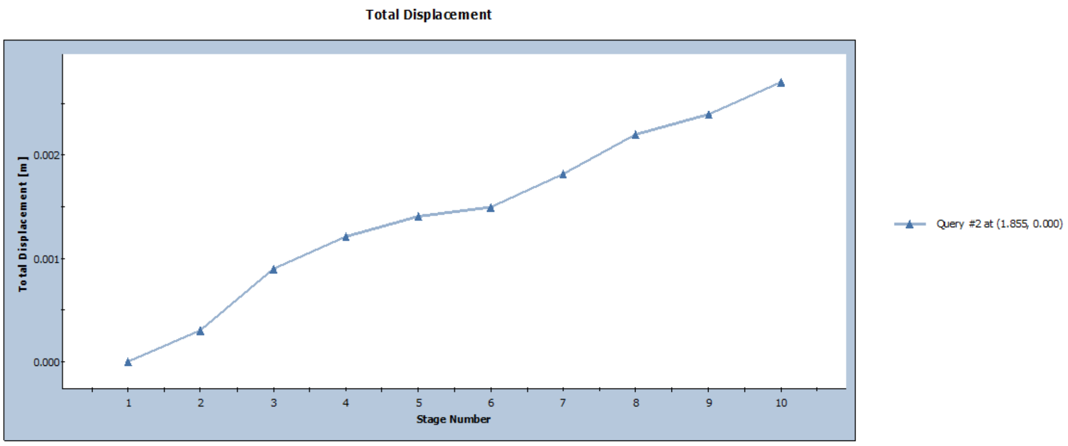

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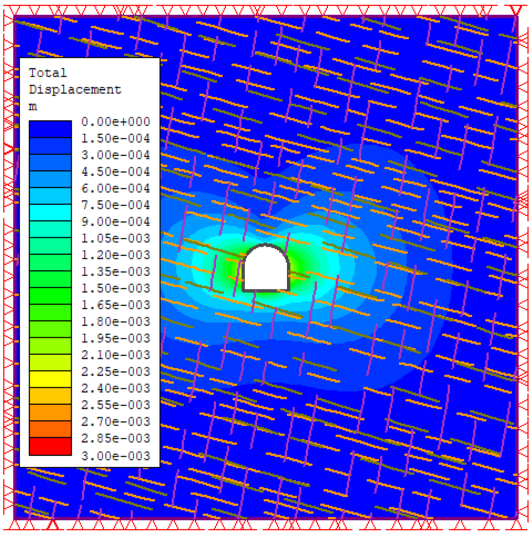

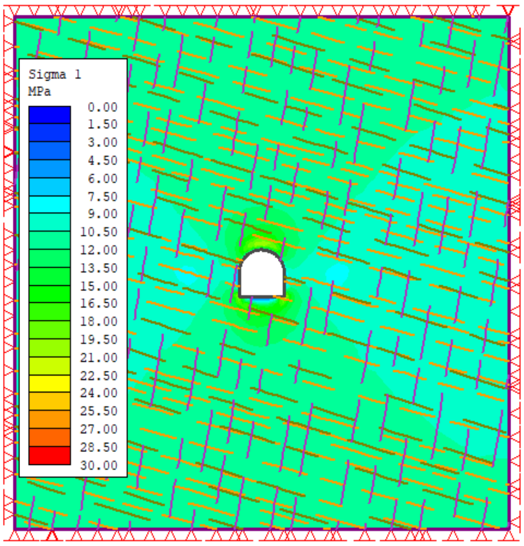

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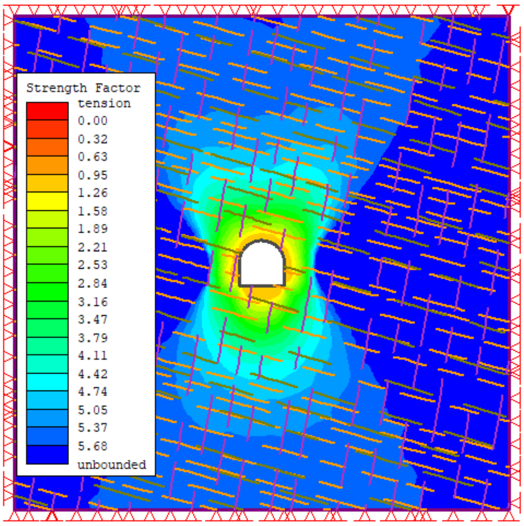

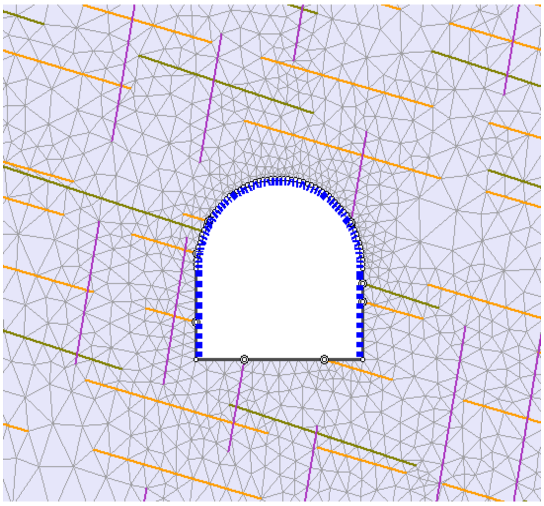

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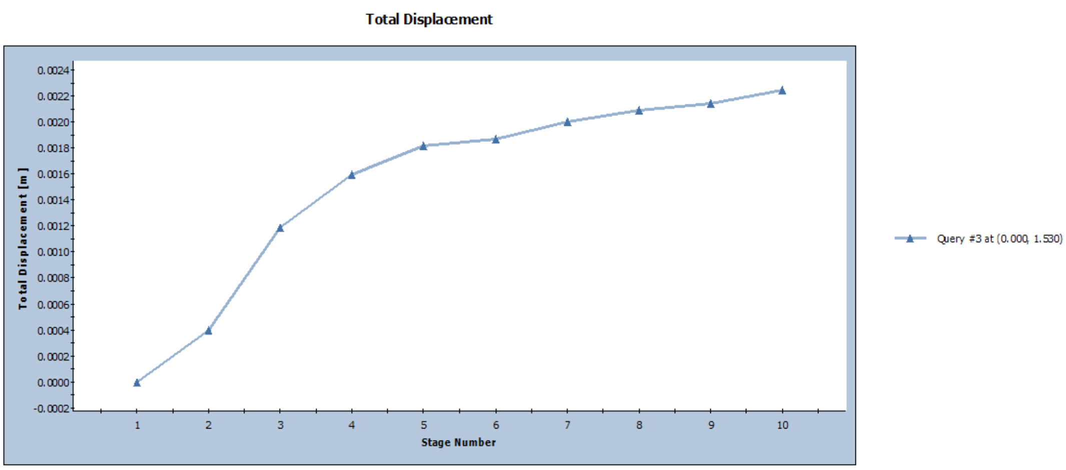

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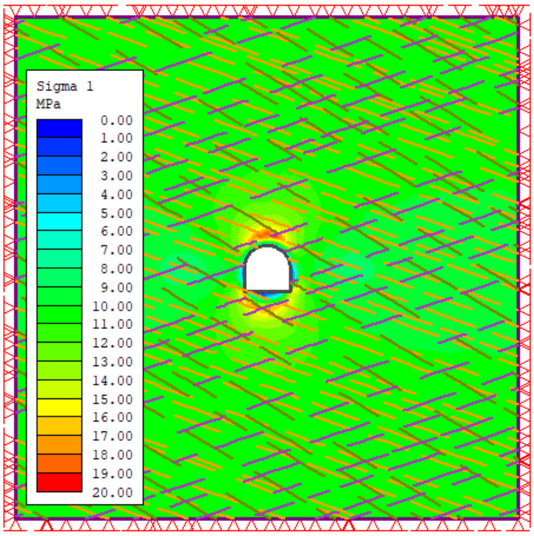

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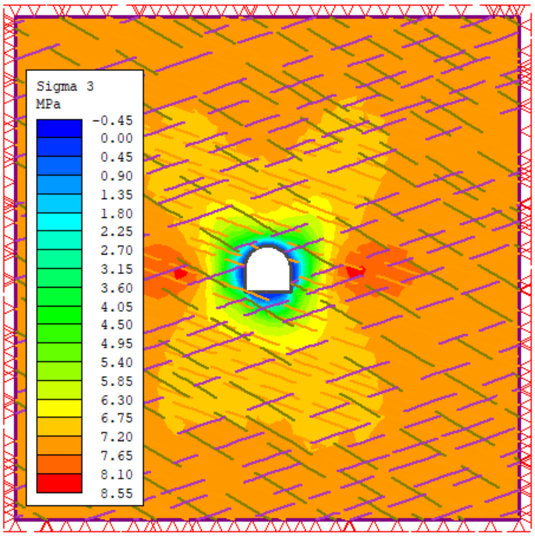

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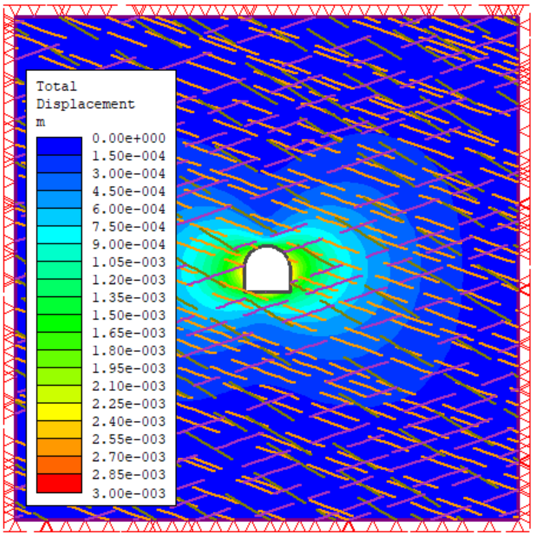

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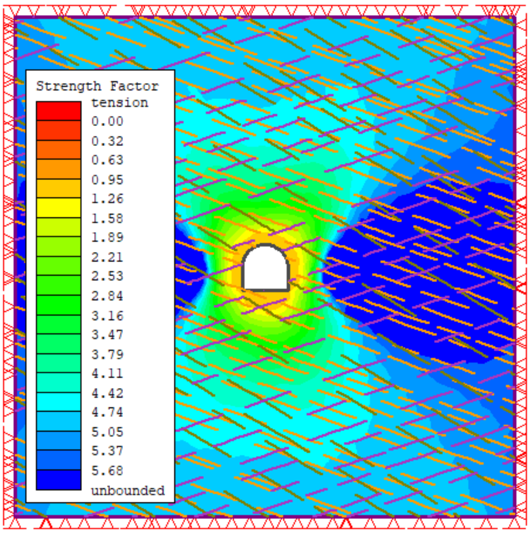

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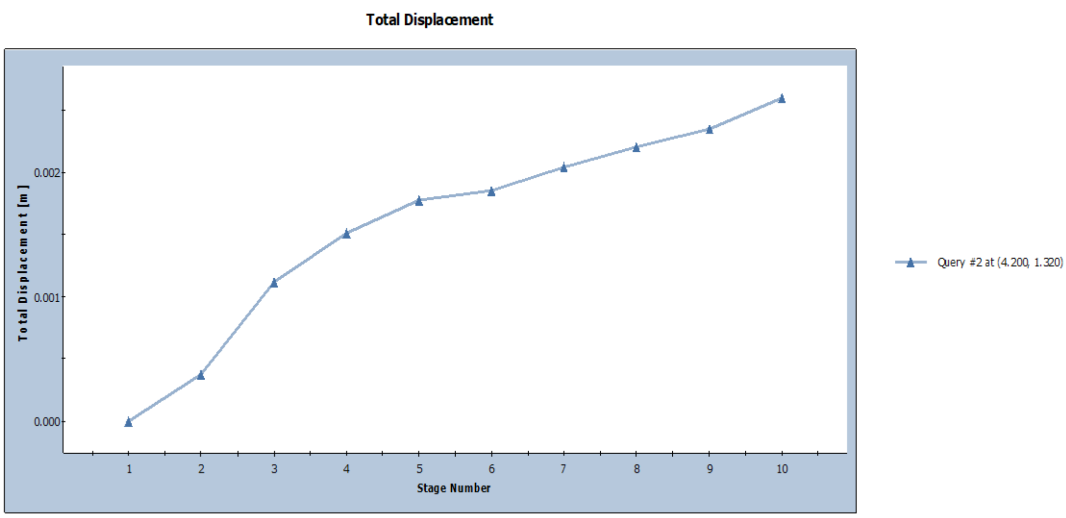

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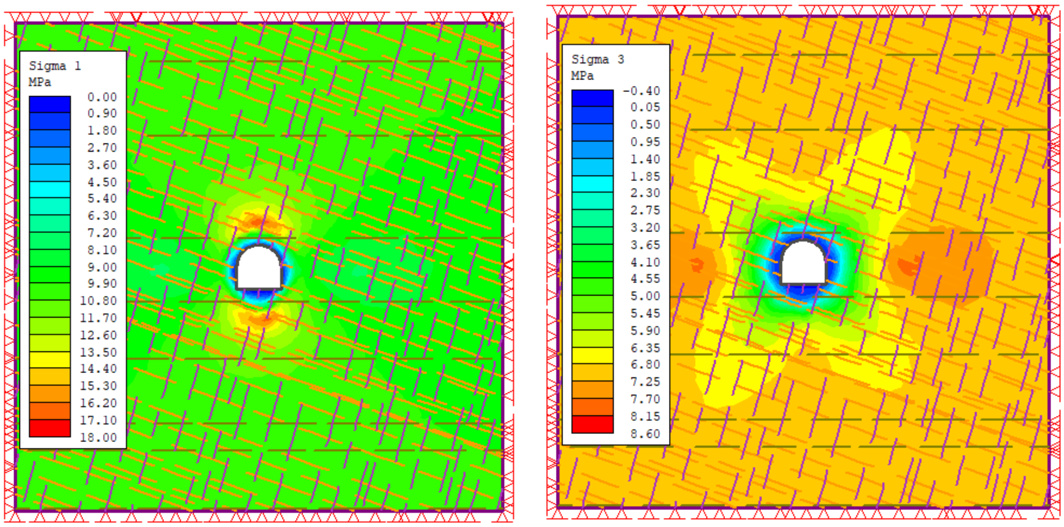

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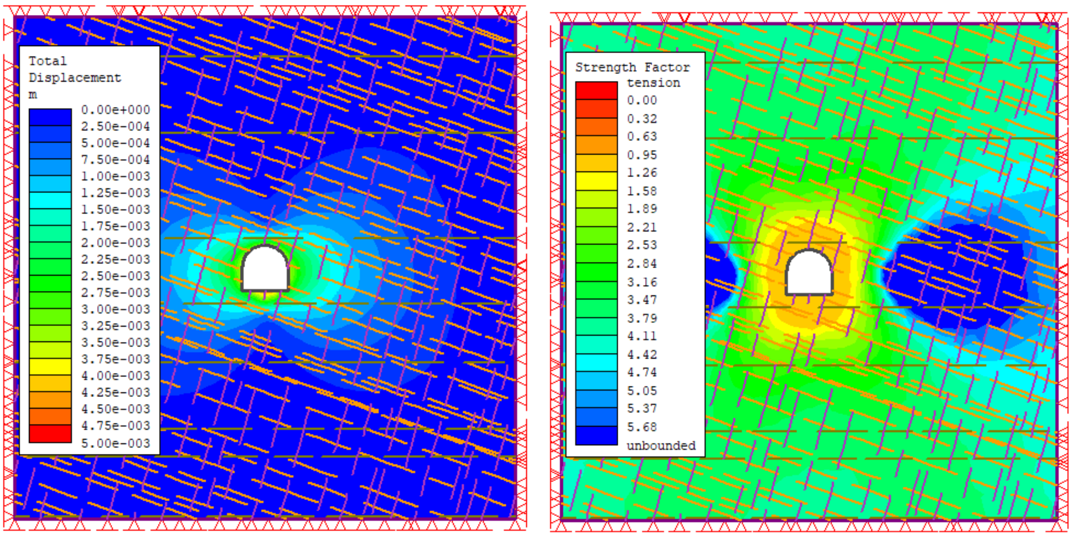

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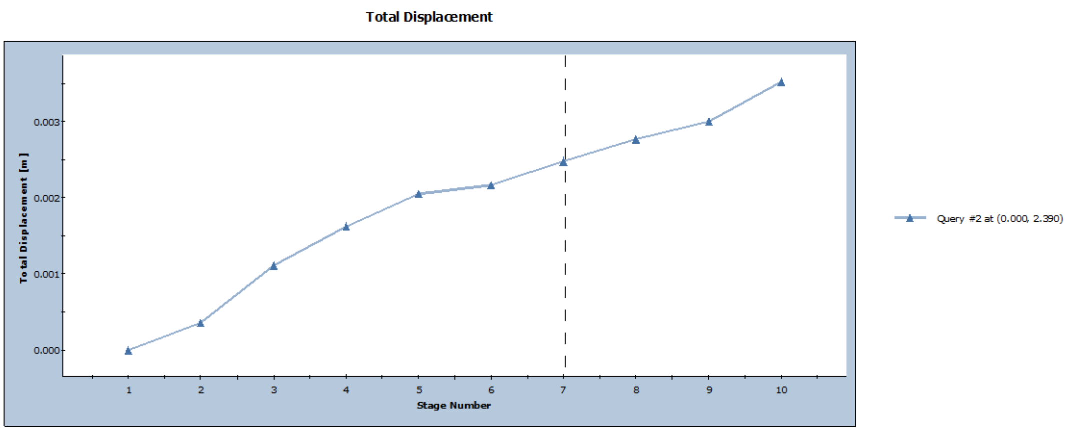

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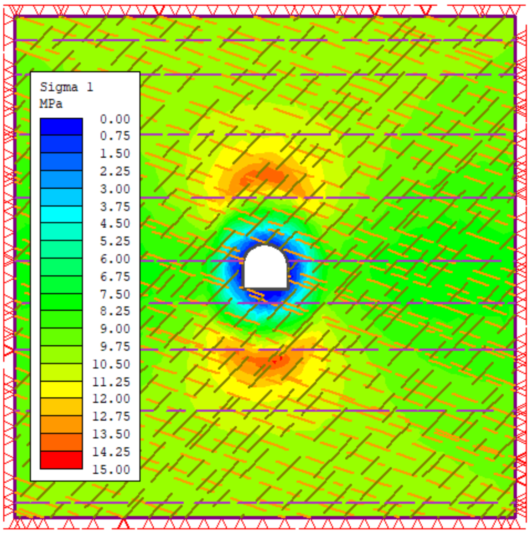

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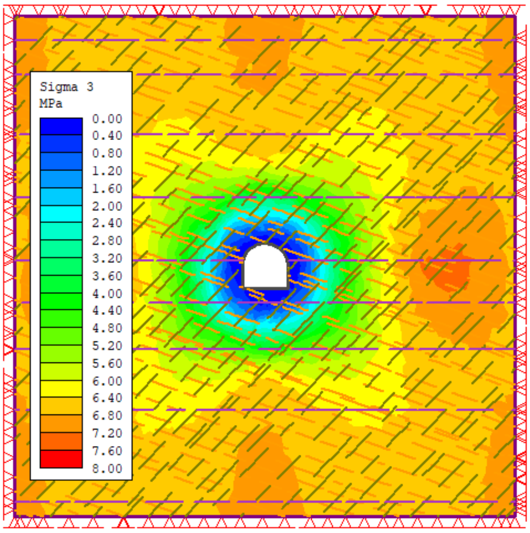

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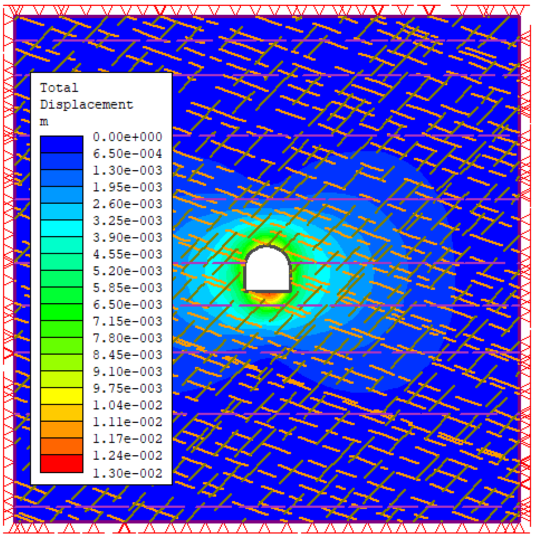

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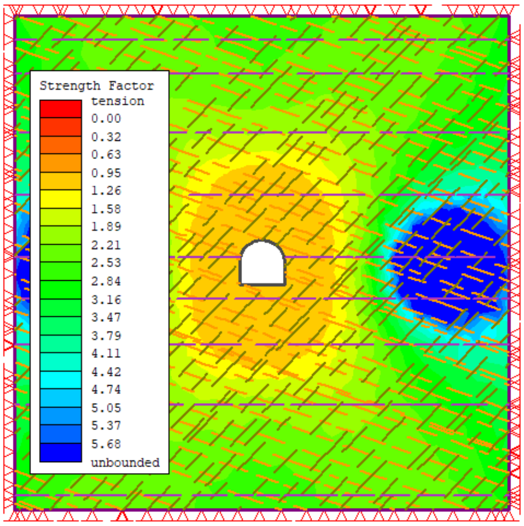

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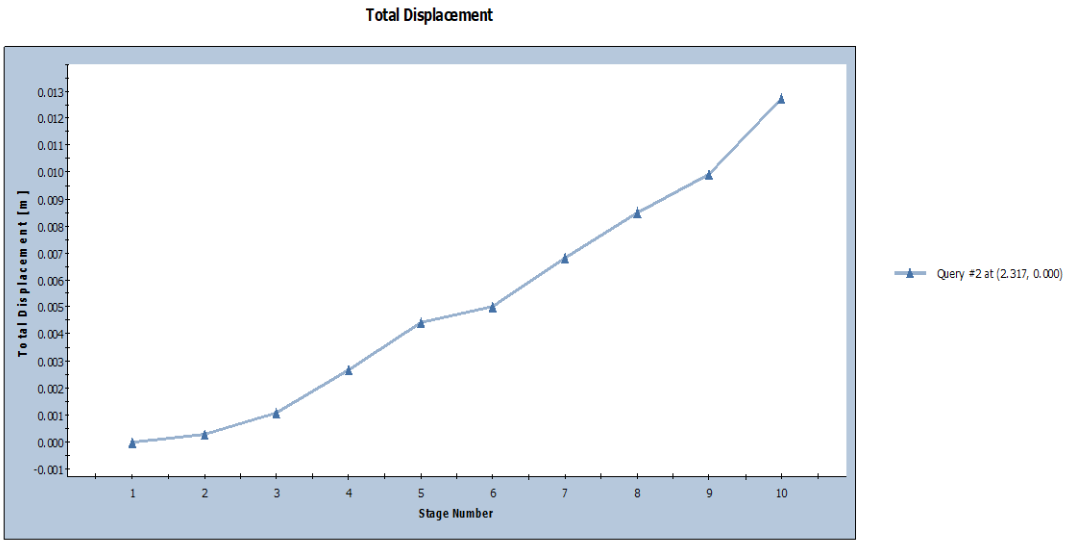

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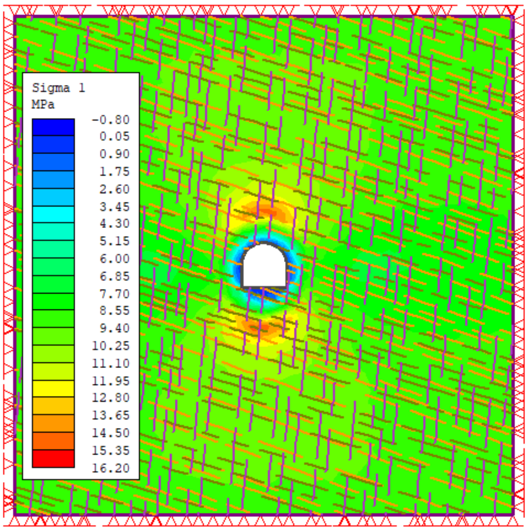

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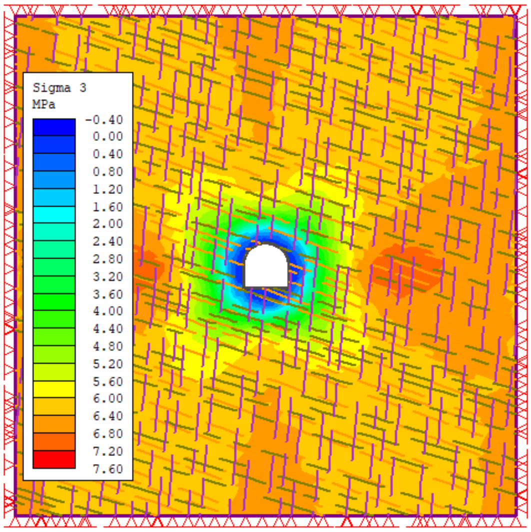

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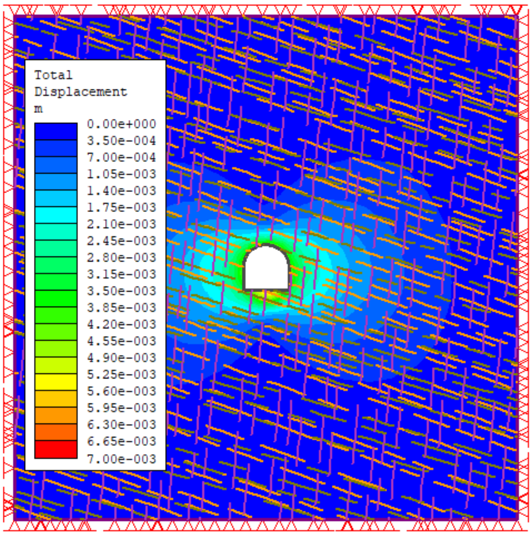

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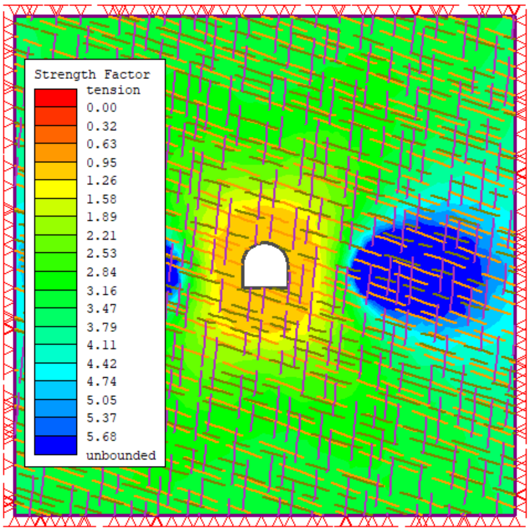

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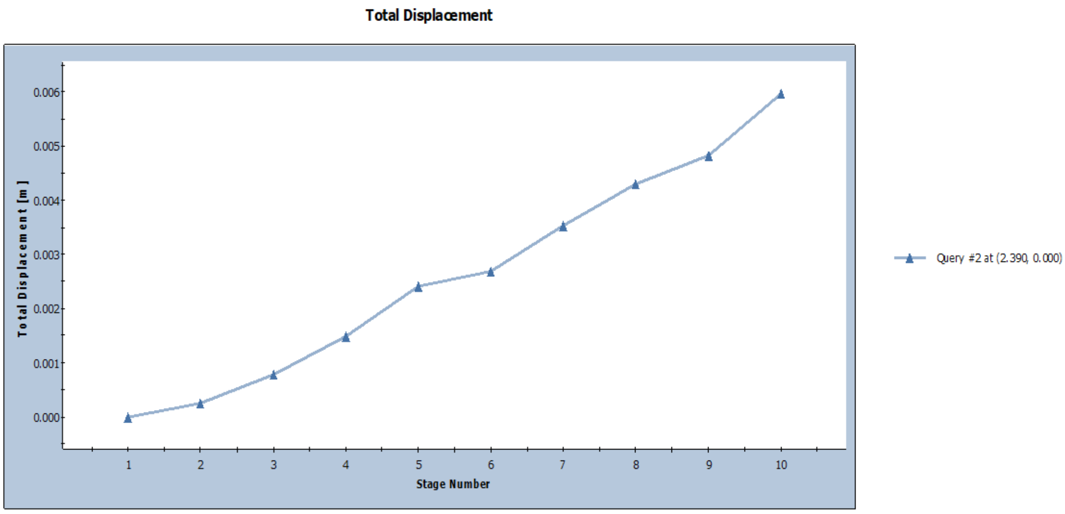

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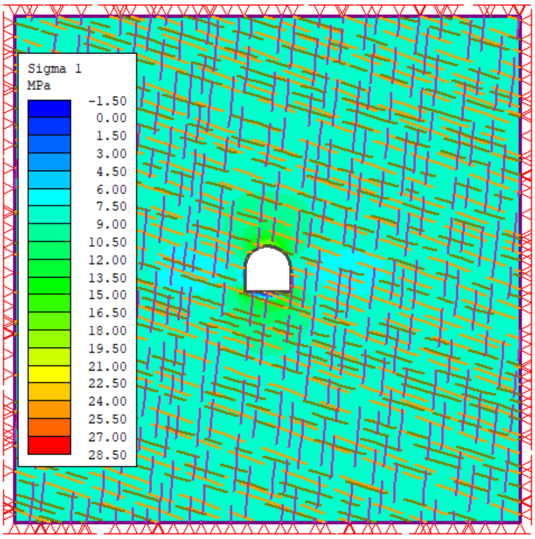

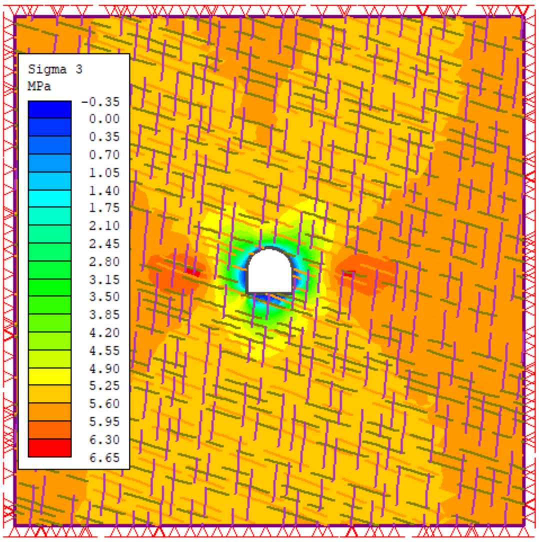

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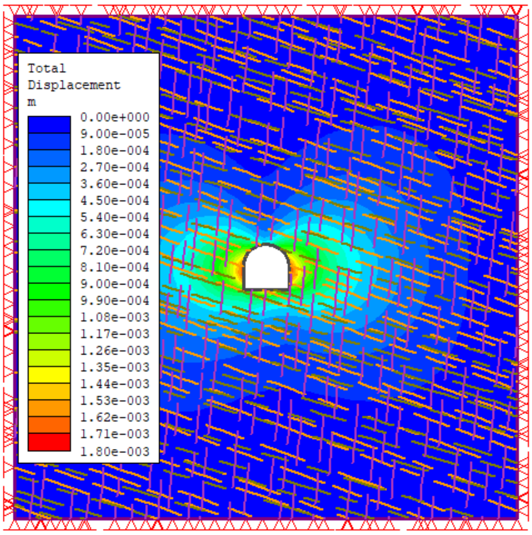

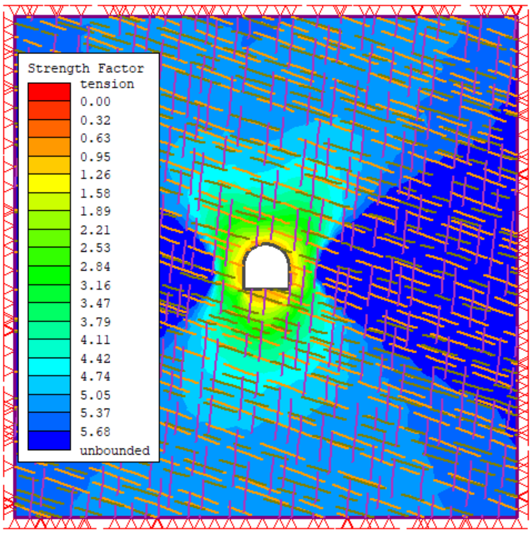

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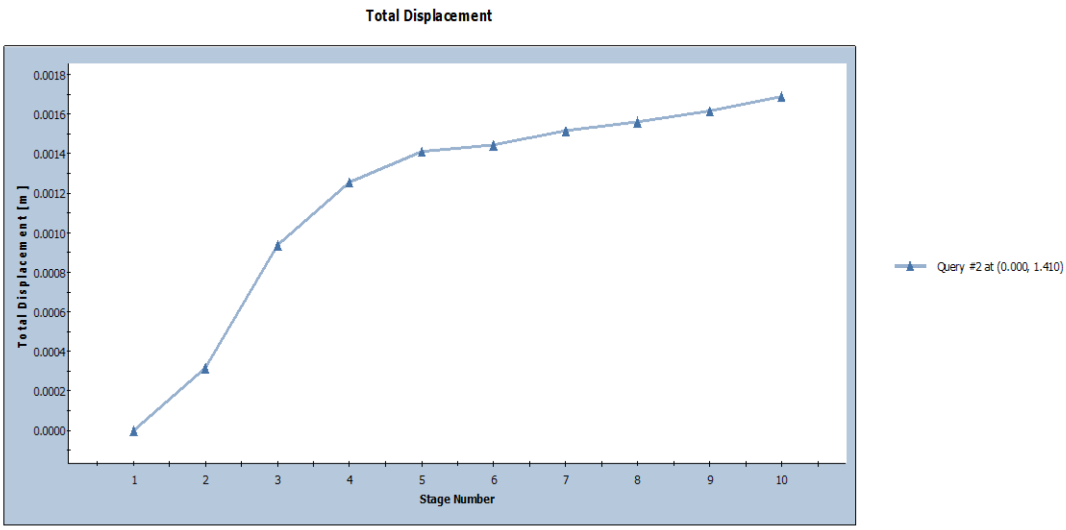

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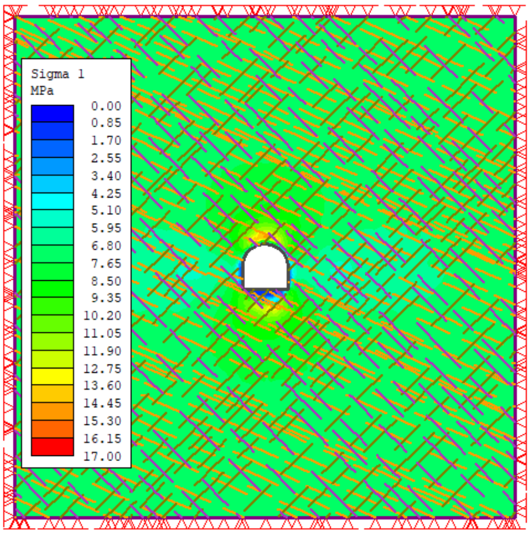

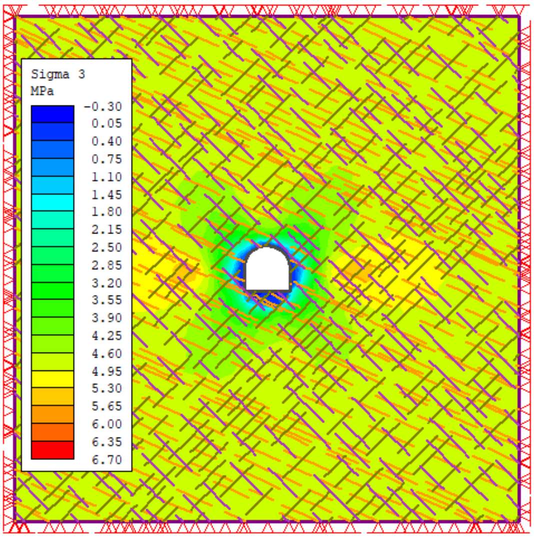

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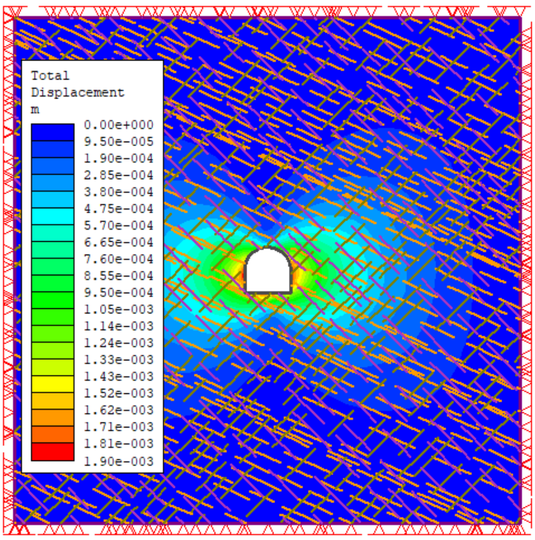

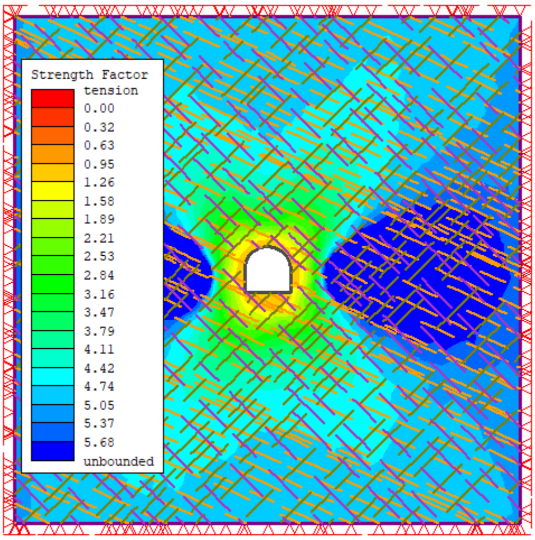

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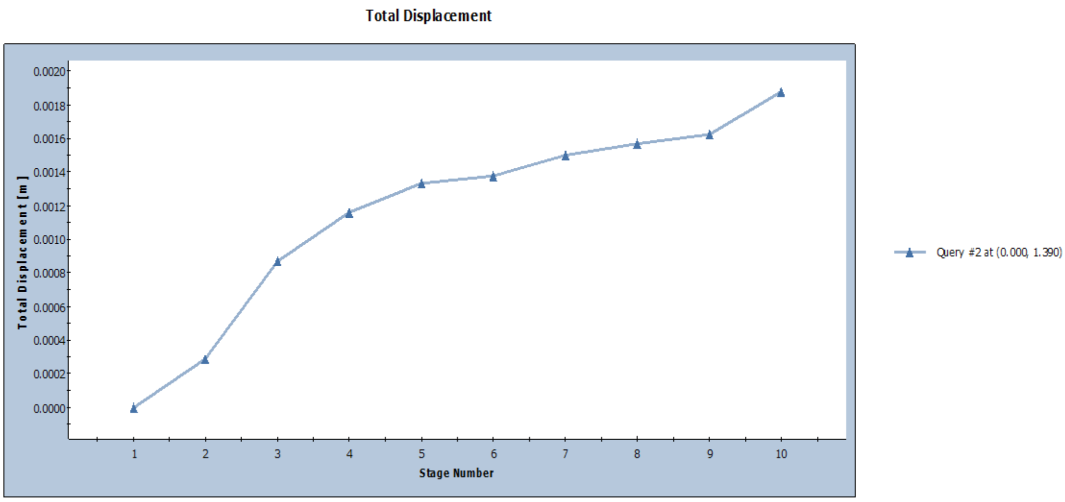

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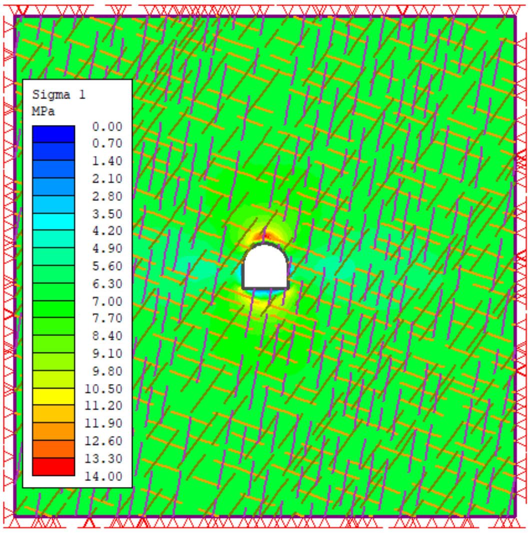
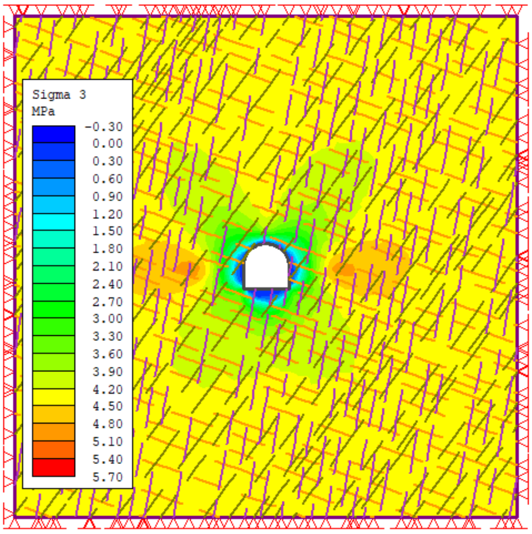


(a)



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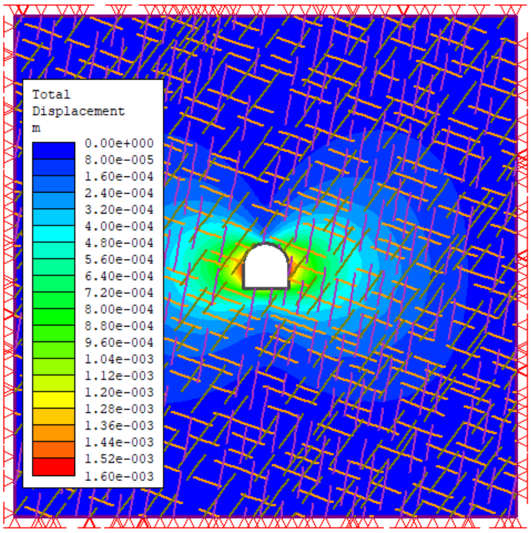

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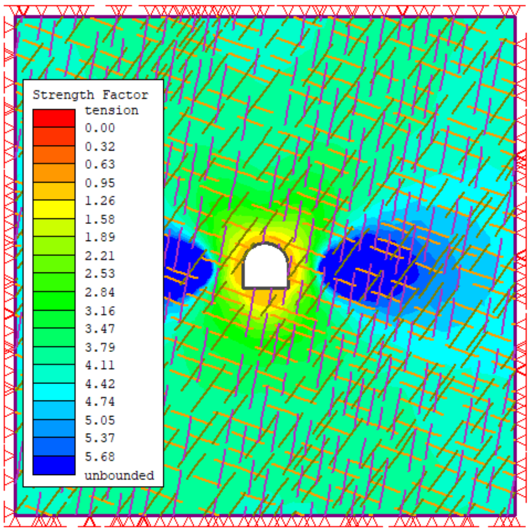

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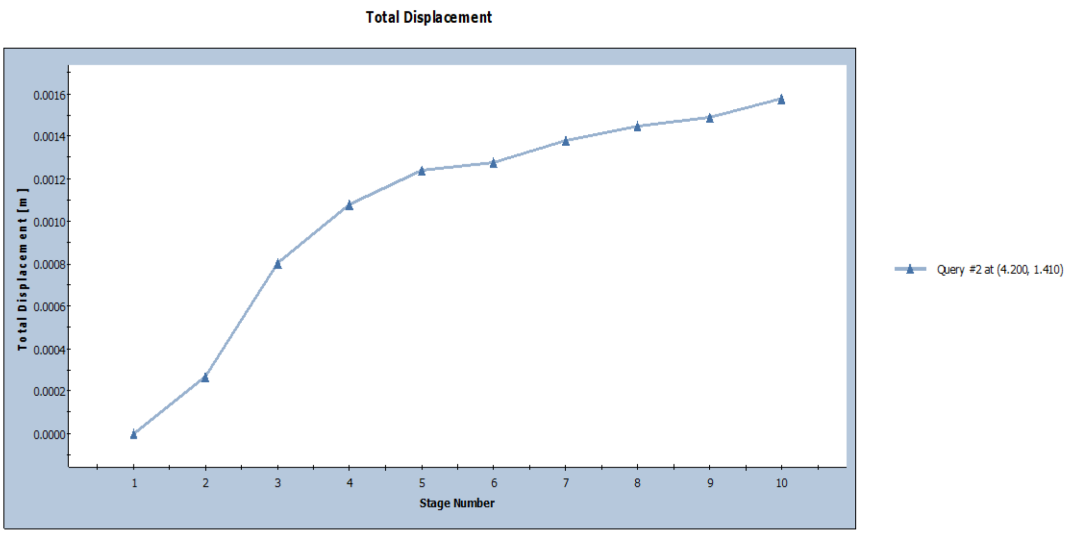
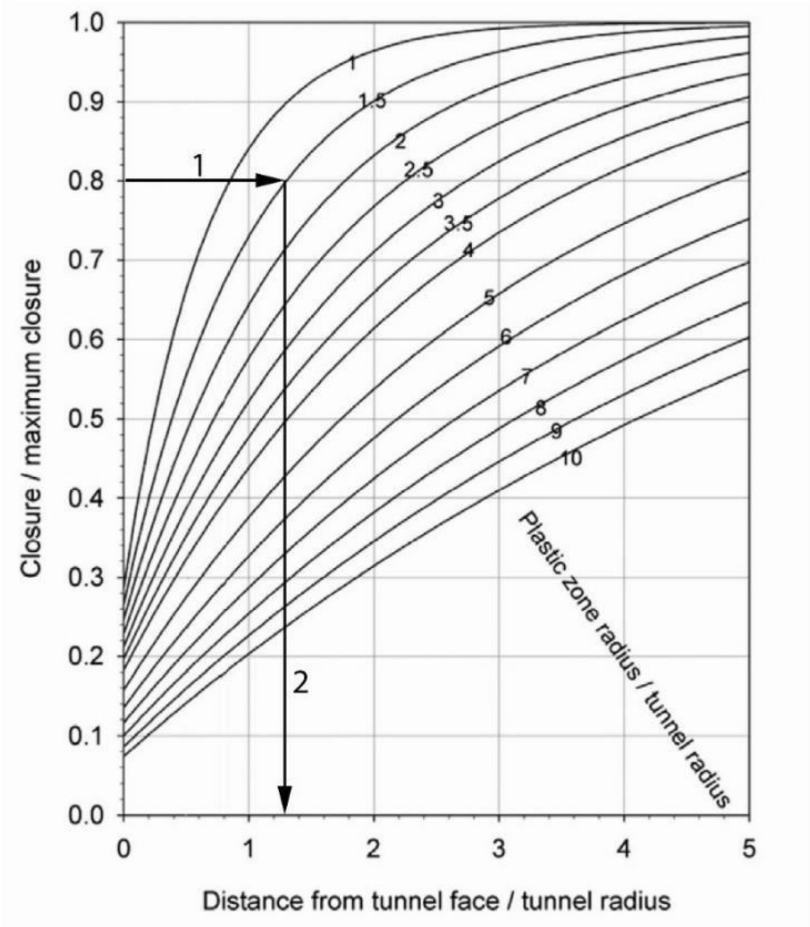
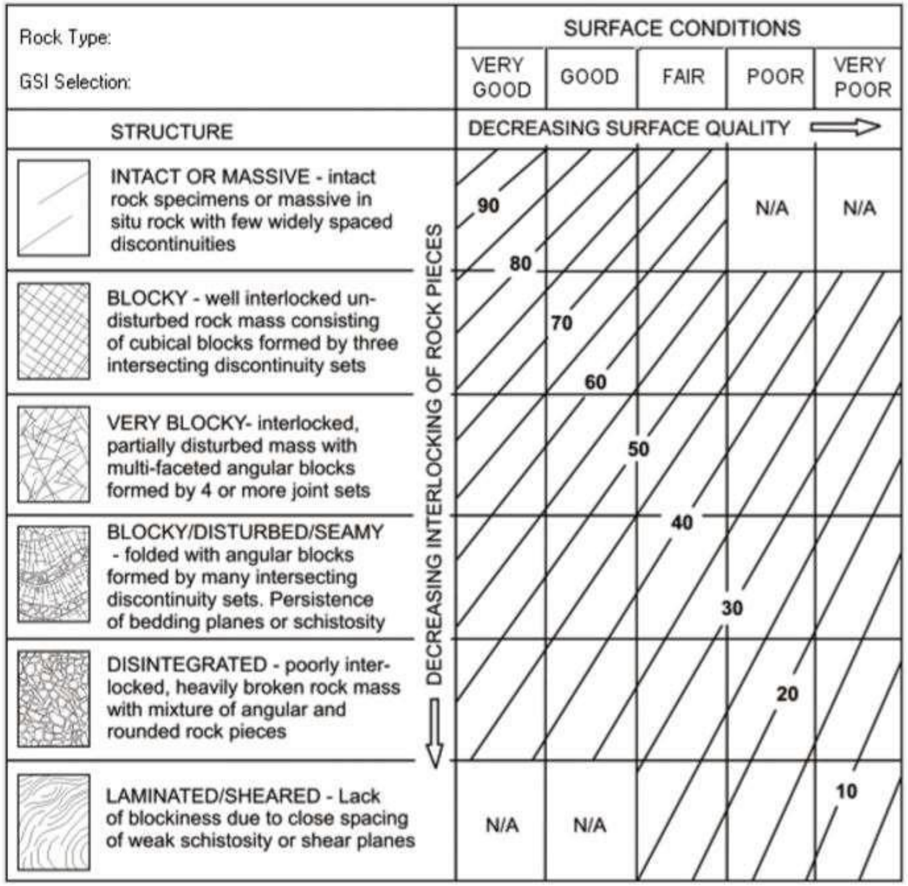


ANNEX C: STANDARD TABLES, CHARTS & FUGURES



Vlachopoulos and Diederichs (2009) Diagram



General chart for GSI estimates from the geological observations

Rock Type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very Fine
SEDIMENTARY	Clastic		Conglomerates (22) Breccias (10)	Sandstones 19	Siltstones 9 Greywackes (18)	Claystones 4
	Non-Clastic	Organic				Chalk 7
		Carbonates		Sparitic Limestone 8	Micritic Limestone 20	
		Evaporites		Gypstone 16	Anhydrite 13	
METAMORPHIC	Non-Foliated		Marble 9	Hornfels (19)	Quartzites (24)	
	Slightly foliated		Migmatite (30)	Amphobolites 31		
	Foliated*		Gneiss 33	Schist (10)	Phyllites (10)	Slate 9
IGNEOUS	Plutonic	Light	Granite 33 Granodiorite (30)	Diorite (28)		
		Dark	Gabbro 27 Norite 22	Dolerite (19)		
	Volcanic	Lava		Rhyolite (16) Andesite 19	Dacite (17) Basalt (17)	Obsidian (19)
		Pyroclastic	Agglomerates (20)	Breccia (18)	Tuff (15)	

Values of constant mi given by intact rock (Hoek & Brown, 1997)

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

Guidelines for estimating disturbance factor D

Rock Quality Designation (RQD) Values

1.0	*Rock Quality Designation*	*RQD*	Notes
a.	Very Poor (Completely weathered rock)	0 - 25	1. Where ROD is reported or measured as <10 (including 0) a nominal value of 10 is used to evaluate Q. 2. RQD intervals of 5 i.e. 100, 95 etc. are sufficiently accurate
b.	Poor (weathered rocks)	25 - 50	
c.	Fair (Moderately weathered rocks)	50 - 75	
d.	Good (*Hard Rock*)	75 - 90	
e.	Excellent (*Fresh rocks*)	90 - 100	

Joint Set Number Values

2.0	*Joint Set Number*	*Jn*	Notes
a.	Massive, no or few Joints	0.5 - 1.0	1. For Intersections use (3.0*Jn). 2. For Portal use (2.0*Jn)
b.	One Joint Set	2	
c.	One Joint Set Plus Random	3	
d.	Two Joint Set	4	
e.	Two Joint Set Plus Random	6	
f.	Three Joint Sets	9	
g.	Three Joint Set Plus Random	12	
h.	Four or More Joint Set, Random, Heavily Jointed, 'Sugar Cube' etc.	15	
i.	Crushed Rock, Earthlike	20	

Joint Roughness Values

3.0	*Joint Roughness Number*	*Jr*	*Notes*
a.	Rock Wall Contact		1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3m. 2. Jr = 0.5 can be used for planar, slikensided joints having lineations, proveded that the lineactions are oriented for minimum strength
b.	Rock Wall Contact before 10 cm shear		
i)	Discontinuous Joint	4	
ii)	Rough and Irregular, Undulating	3	
iii)	Smooth Undulating	2	
iv)	Slickensided Undulating	1.5	
v)	Rough or Irregular, Planar	1.5	
vi)	Smooth, Planar	1	
vii)	Slickensided Planar	0.5	
c.	No Rock Wall Contact When Sheared		
i)	Zones containing Clay mineral thick enough to prevent rock wall contact	1.0 (nominal)	
ii)	Sandy, Gravely or Crushed zone thick enough to prevent rock wall contact	1.0 (nominal)	

Joint Alteration Values

4.0	Joint Alteration Number	Ja	Φ_r degree
a.	Rock Wall Contact		
i)	Tightly healed, hard, non-softening, impermeable filling	0.75	
ii)	Unaltered joint walls, surface staining only	1.0	25-35
iii)	Slightly altered joint walls, non softening mineral coatings, sandy particles, clasy-free disintegrated rock etc.	2.0	35-30
iv)	Silty- or Sandy-Clay coatings, small clay-fraction (non-softening)	3.0	20-25
v)	Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc. and small quantities of swelling clays (Discontinuous coating, 1-3 mm or less	4.0	8-16
b.	Rock Wall Contact before 10 cm Shear		
i)	Sandy particles, clay-free, disintegrating rock etc.	4	25-00
ii)	Strongly over-consolidated, non-softening clay mineral filling (Continuous<5mm thick	6	16-24
iii)	Medium or low over-consolidation, softening clay mineral filling(Continuous<5mm thick)	0	12-10
iv)	Swelling clay filling, i.e. montmorillonite,(Continuous<5mm thick). Value of Ja depend on percent of swelling clay size particles and access to water.	8.0 - 12.0	6-12
c.	No rock Wall Contact when Sheared		
i)	Zones or bands of disintegrated or crushed	6	
ii)	Rock and Clay (see b-ii,iii,iv) for Clay	8	
iii)	Conditions	8.0-12.0	
iv)	Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5	
v)	Thick Continuous Zones or Band of Clay	10.0-13.0	
vi)	See b-ii,iii,iv for Clay Conditions	6.0-24.0	

Joint Water Reduction Values

5.0	Joint Water Reduction	Jw	Approx. Water pressure (kgf/cm2)
i)	Dry Excavation or Minor Inflow i.e. <5 l/m locally	1	<1.0
ii)	Medium inflow or Pressure, occasional outwash of joint fillings	0.66	1.0-2.5
iii)	Large inflow or high pressure in compelent rock with unfillied joints	0.5	2.5-10.0
iv)	large inflow or high pressure	0.333	2.5-10.0
v)	Exceptionally high infow or pressure at blasing, decaying with time.	0.2-0.1	>10
vi)	Exceptionally high infow or pressure	0.1-0.05	>10

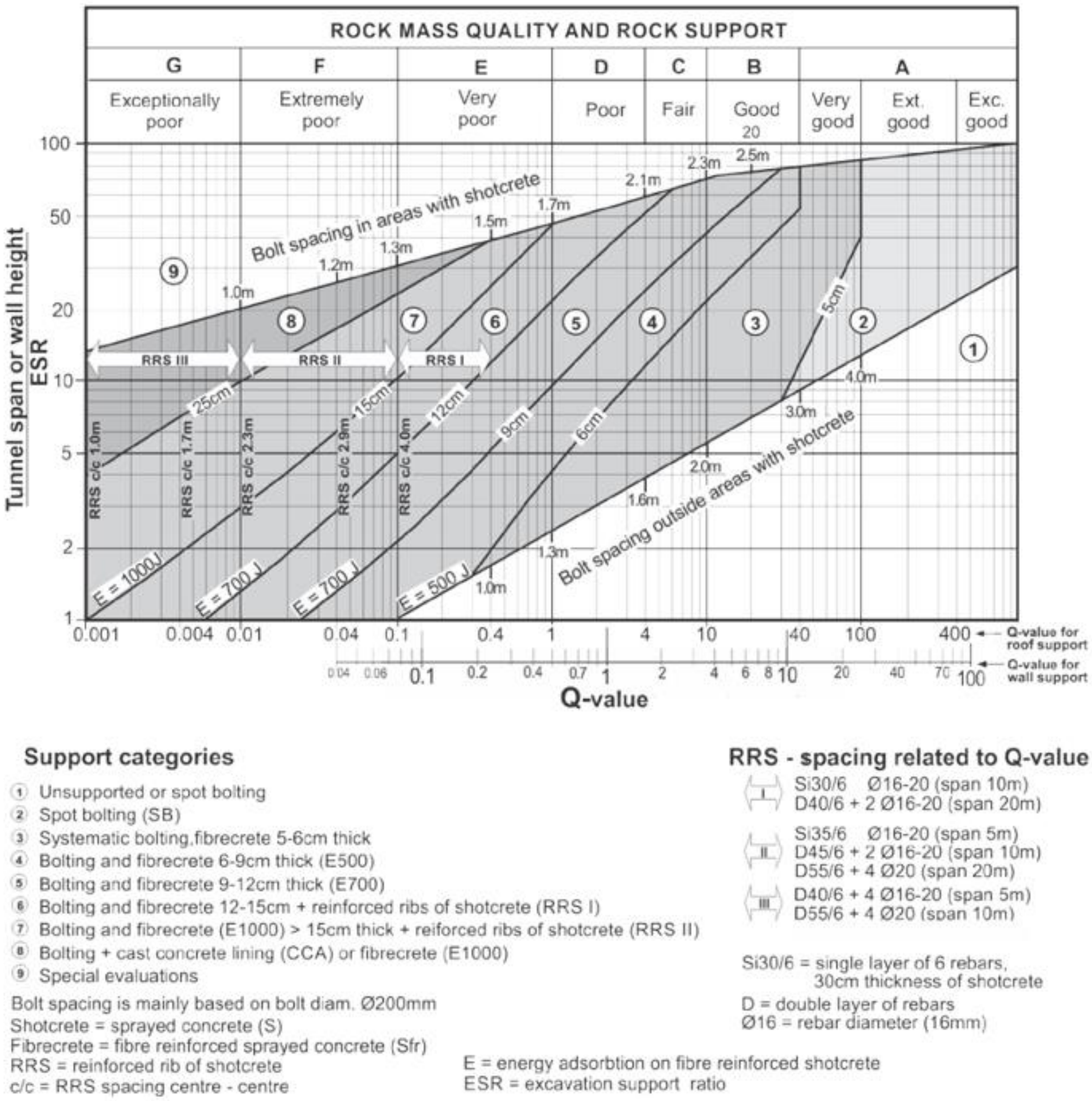
Stress Reduction Factor Values

6.0	***Stress Reduction Factor***	***SRF***	***Notes***	
a.	Weakness zones intersecting excavation, while may cause loosening of rock mass when tunnel is excavated			
i)	Multiple Occurances of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)	10	1. Reduce these values of SRF by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation	
ii)	Single Weakness zones containing clay, or chemically distegrated rock (excavation depth <50 m)	5		
iii)	Single Weakness zones containing clay, or chemically distegrated rock (excavation depth >50 m)	2.5	2. For strongly anisotropic virgin stress field (if measured): when $5 < \sigma_1/\sigma_3 < 10$, reduce σ_c to $0.08\sigma_c$ and σ_t to $0.08\sigma_t$. When $\sigma_1/\sigma_3 > 10$ reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$. Where σ_c- unconfined compressive strength, σ_t tensile strength(point load) and σ_1 and σ_3 are the major and minor principal stresses	
iv)	Multiple Shear zone in competent rock (Clay free) loose surrounding rock (any Depth)	7.5		
v)	Single Shear Zone in Competent rock (Clay free). Depth of excavation <50m.	5		
vi)	Single Shear Zone in Competent rock (Clay free). Depth of excavation >50m.	2.5		
vii)	Loose open joints, heavily jointed or Sugar cube (any depth)	5		
b.	Competent rock, rock stress problem	σ_c/σ_1	$\sigma_t.\sigma_1$	SRF
i)	Low Stress, near surface	>200	>13	2.5
ii)	Medium Stress	200-10	13-0.66	1
iii)	high Stress, very tight Structure (Usually favorable to stability, may be unfavorable to wall stability)	10-5	0.66-0.33	0.5-2
iv)	Mild rockbrust (massive rock)	5-2.5	0.33-0.16	5-10
v)	Heavy rockbrust (massive rock)	<2.5	<0.16	10-20
c.	Squeezing rock, plasitc flow of incompetent rock under influence of high rock pressure			
i)	Mild squeezing rock pressure			5-10
ii)	Heavy squeezing rock pressure			10-20
d.	Swelling rock, chemical swelling activity depending on pressence of water			
i)	Mild swelling rock pressure			5-10
ii)	heavy swelling rock pressure			10-15

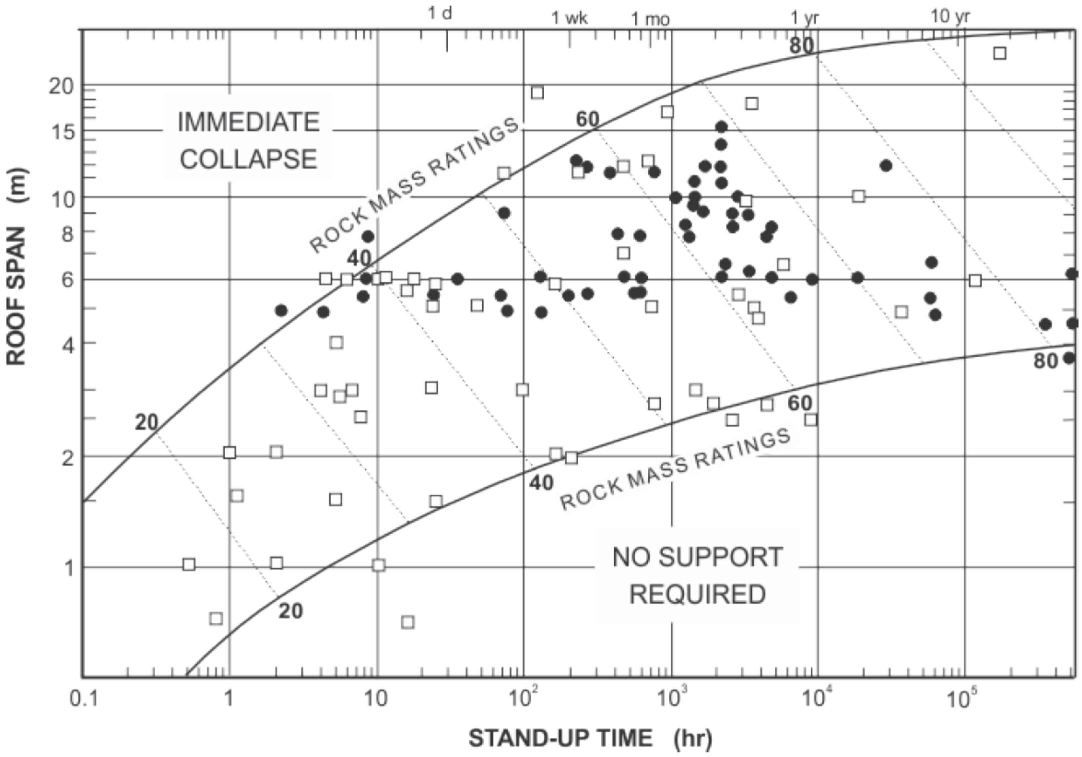
Ratings of the excavation support ratio (ESR) (Barton et. al., 1974).

SN	Type or use of Underground Opening	ESR
1.0	Temporary mine opening	3.5
2.0	Vertical Shaft, rectangular and Circular Respectively	2.0-2.5
3.0	Water Tunnels, Permanent mine openings, Adits, Drifts	1.6
4.0	Storage Caverns, road tunnels with little traffic, access tunnels etc.	1.3
5.0	Power stations, road and railways tunnels with heavy traffic, civil defence shelters etc.	1.0
6.0	Nuclear power plants, railroad stations, sport arenas etc.	0.8

The Q system chart for rock support estimate, developed by the Norwegian Geotechnical Institute (NGI), (based on www.ngi.no, 2014).



RMR classification of rock masses. (Contour lines indicate limits of applicability) (Bieniawski, 1989)



Strength of Intact Rock Material & Ranking

1.0 Strength of Intact Rock Material (Sources: Bieniawski, 1979)

SN	*Qualitative Description*	*Compressive Strength (MPa)*	*Point Load Strength (MPa)*	*Rating*
1	*Extremely Strong**	*> 250*	*8*	*15*
2	*Very Strong*	*100-250*	*4-8*	*12*
3	*Strong*	*50-100*	*2-4*	*7*
4	*Medium Strong*	*25-50*	*1-2*	*4*
5	*Weak*	*5-25*	*Use of UCS is preferred*	*2*
6	*Very Weak*	*1-5*	*-do-*	*1*
7	*Extremely Weak*	*< 1*	*-do-*	*0*

At compressive strength of rock material less than 1.0 MPa, many rock materials would be regarded as soil.
**Term redefined according to ISO 14689*

Rock Quality Designation & Ranking

2.0 Rock Quality Designation (Bieniawski, 1979)

SN	*Qualitative Description*	*RQD (%)*	*Rating*
1	*Excellent*	*90-100*	*20*
2	*Good*	*75-90*	*17*
3	*Fair*	*50-75*	*13*
4	*Poor*	*25-50*	*8*
5	*Very Poor*	*<25*	*3*

Spacing of Discontinuities & Ranking

3.0 Spacing of Discontinuities (Bieniawski, 1979)			
SN	***Qualitative Description***	***Spacing (m)***	***Rating***
1	*Very Wide*	*>2*	*20*
2	*Wide*	*0.6-2*	*15*
3	*Moderate*	*0.2-0.6*	*10*
4	*Close*	*0.06-0.2*	*8*
5	*Very Close*	*<0.06*	*5*
If more than one discontinuity set is present and the spacing of discontinuities of each set varies, consider the unfavorably oriented set with lowest rating. ISO 14689 uses the term "extremely close" for joint spacing less than 0.02m.			

Condition of Discontinuities & Ranking

4.0 Condition of Discontinuities (Bieniawski, 1979)

SN	***Description***	***Joint Separation (mm)***	***Rating***
1	*Very rough and unweathered, wall rock tight and discontinuous, no separation*	*0*	*30*
2	*Rough and Slightly weathered, wall rock surface separation <1mm*	*<1*	*25*
3	*Slightly rough and moderately to highly weathered, wall rock surface separation <1m*	*<1*	*20*
4	*Slickenside wall rock surface or 1-5 mm thick gouge or 1-5 mm wide continuous discontinuity*	*1-5*	*10*
5	*5mm thick soft gouge, 5mm wide continuous discontinuity*	*>5*	*0*

Ground Water Condition & Ranking

6.0 Groundwater Condition (Bieniawski, 1979)

1	*Inflow per 10 m tunnel length (L/min)*	*None*	*<10*	10-25	25-125	>125
2	*Ratio of Joint water pressure to major principal Stress*	*0*	*0-0.1*	0.1-0.2	0.2-0.5	>0.5
3	*General Description*	*Completely dry*	*Damp*	Wet	Dripping	Flowing
4	*Rating*	*15*	*10*	7	4	0

Assessment of Joint Orientation Effect on Tunnels

8.0 Assessment of Joint Orientation effect on Tunnels (Source: Bieniawski, 1984)						
Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Irrespective of Strike
Drive with dip		Drive against dip				
Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°
Very Favorable	Favorable	Fair	Unfavorable	Fair	Very unfavorable	Fair

The RMR System: Guideline for Classification of Discontinuity Condition

5.0 The RMR System: Guidelines for Classification of Discontinuity Conditions (Source: Bieniawski, 1993)

SN	Parameter	Rating				
1	Discontinuity length (Persistence/Continuity)	<1 m	1-3 m	3-10 m	10-20 m	>20m
	Rating	6	4	2	1	0
2	Separation (Aperture)	None	<0.1 mm	0.1 - 1.0 mm	1-5 mm	>5mm
	Rating	6	5	4	1	0
3	Roughness of Discontinuity surface	Very Rough	Rough	Slightly rough	Smooth	Slickensided
	Rating	6	5	3	1	0
4	Infilling (Gouge)	Hard Filling			Soft filling	
		None	<5 mm	>5 mm	<5 mm	>5 mm
	Rating	6	4	2	2	0
5	Weathering discontinuity surface	Unweathered	Slightly Weathered	Moderately Weathered	Highly Weathered	Decomposed
	Rating	6	5	3	1	0

**Some conditions are mutually exclusive. For example, if infilling is present, it is irrelevant what the roughness may be, since its effect will be overshadowed by the influence of the gouge. In such cases use Table 3-11 directly*

Adjustment for Joint Orientation

9.0 Adjustment for Joint Orientation (Source: Bieniawski, 1979)

Joint Orientation assessment for	Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Tunnels	0	-2	-5	-10	-12
Raft Foundation	0	-2	-7	-15	-25
Slopes*	0	-5	-25	-50	-60

**It is recommended to use slope mass rating*

Design Parameter & Engineering Properties of Rock

10.0 Design Parameter & Engineering Properties of Rock (Bieniawski, 1993)

SN	Parameter/Properties of Rock Mass	RMR (rock class)				
		100-81	80-61	60-41	40-21	<20
		(I)	(II)	(III)	(IV)	(V)
1	Classification of rock mass	Very Good	Good	Fair	Poor	Very Poor
2	Average stand-up time	20 years for 15m span	1 year for 10 m span	1 week for 5m span	10 hours for 2.5m span	30 minutes for 1m span
3	Cohesion of rock mass (MPa)*	>0.4	0.3-0.4	0.2-0.3	0.1-0.2	<0.1
4	Angle of Internal friction of rock mass	>45˚	35 -45˚	25-35˚	15-25˚	<15˚
5	Allowable bearing pressure (T/m2)	600-440	440-280	280-135	135-45	45-30
6	Safe cut Slope (˚) (Waltham, 2002)	>70	65	55	45	<40

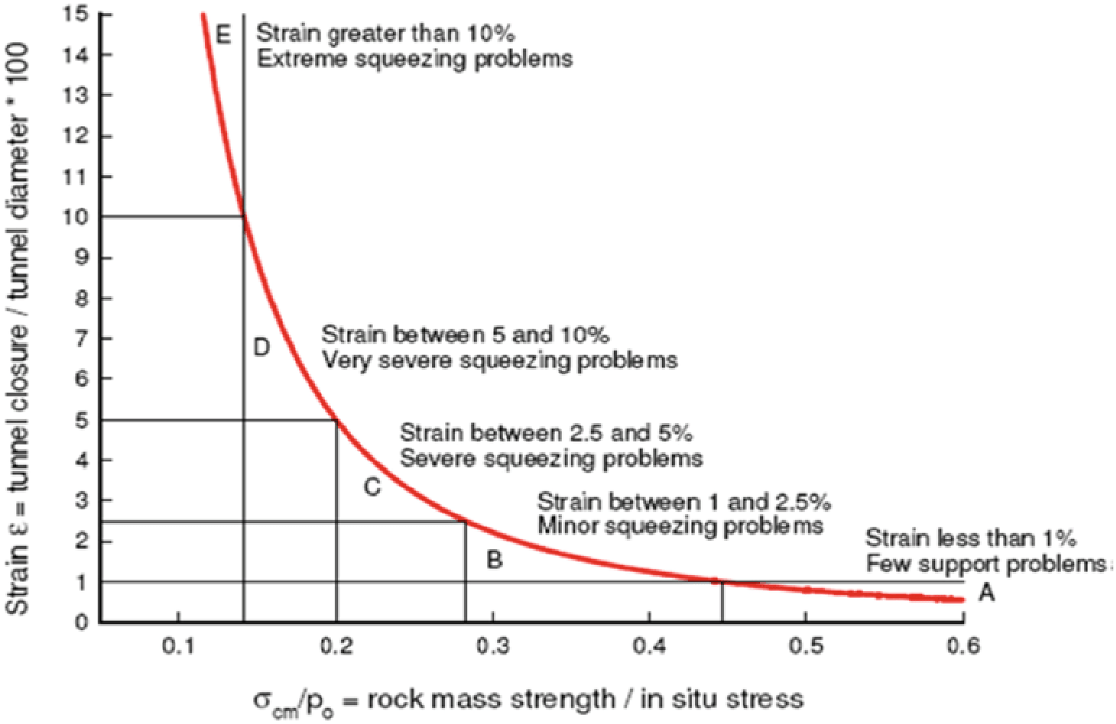
During earthquake loading, the above values of allowable bearing pressure may be increased by 50% in view of rheological behavior of rock masses.
**These values are applicable to slopes only in saturated and weathered rock mass.*

RMR classification guide for excavation and support in rock tunnels (Bieniawski, 1989).

Rock mass class	Excavation	Supports: *Rock bolts (20 mm diameter, fully grouted)*	Supports: *Conventional shotcrete*	Supports: *Steel sets*
Very good rock RMR = 81–100	Full face; 3 m 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 3 m long, spaced 2.5 m, with occasional wire mesh	50 mm in crown where required	None
Fair rock RMR = 41–60	Heading and bench; 1.5–3 m advance in heading; commence support after each blast; complete support 10 m from face	Systematic bolts 4 m long, spaced 1.5–2 m in crown and walls with wire mesh in crown	50–100 mm in crown and 30 mm in sides	None
Poor rock RMR = 21–40	Top heading and bench; 1.0–1.5 m advance in top heading; install support concurrently with excavation 10 m from face	Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and wall with wire mesh	100–150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
Very poor rock RMR <20	Multiple drifts; 0.5–1.5 m advance in top heading; install support concurrently with excavation; shotcrete as soon as possible after blasting	Systematic bolts 5–6 m long, spaced 1–1.5 m in crown and walls with wire mesh; bolt invert	150–200 mm in crown, 150 mm in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required; close invert

Shape: Horseshoe; width: 10 m; vertical stress <25 MPa; construction: drilling and blasting.
Source: Bieniawski, 1984.

Classification of squeezing behavior (as per Hoek and Marinos 2000).



ANNEX D: PROJECT SITE PHOTOGRAPHS

<u>Measuring Joint Properties at the Tunnel Face</u>

<u>Drilling Pattern in face of Tunnel</u>

Portal of The Headrace Tunnel

Shotcreting and finishing the excavated face

Support Installed in Headrace Tunnel

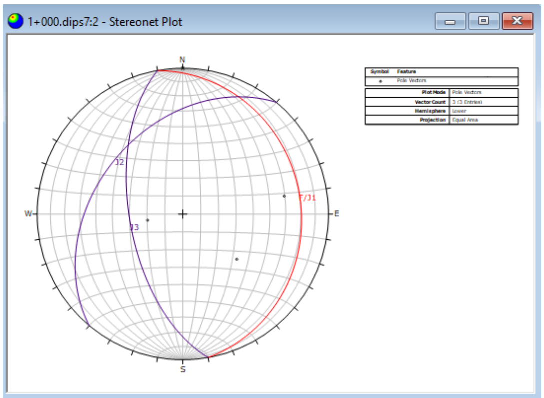
Sample of Rock Bolt used in the Site

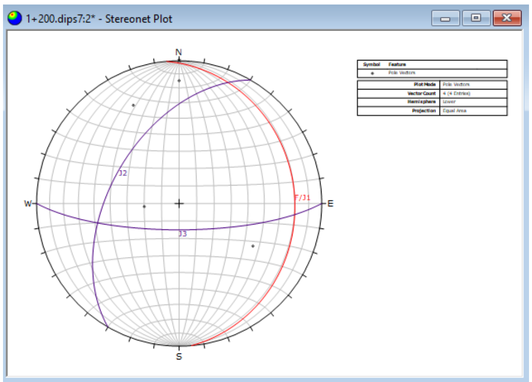
Prepared for the Collection of Geological Data

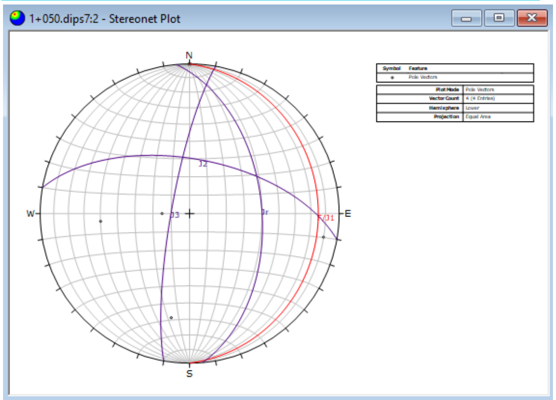
I-Beam Used as the Rock Support Sample from Site

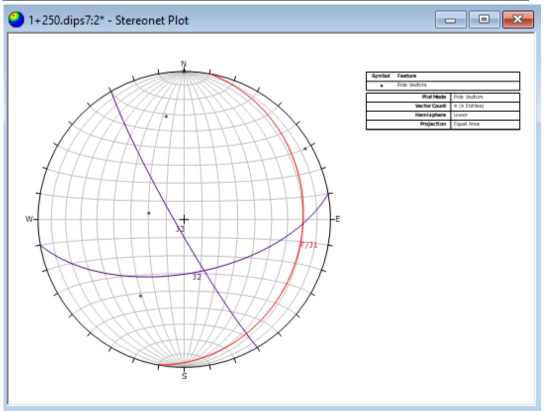
ANNEX E: STEREONET & UNWEDGE ANALYSIS

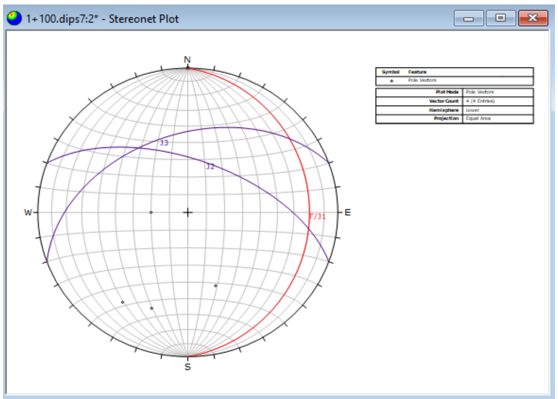
<u>Stereonet Plot at each section using Dip 7.0 software.</u>

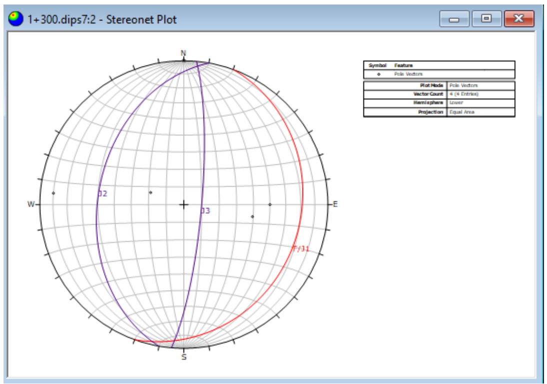


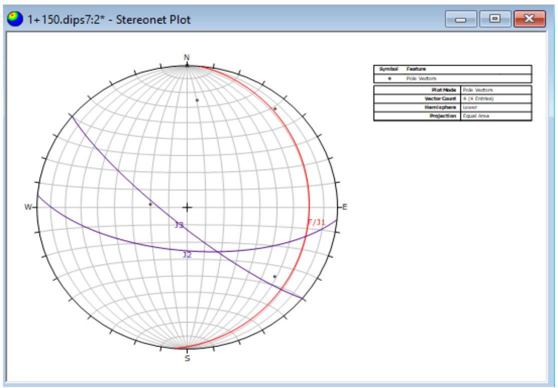


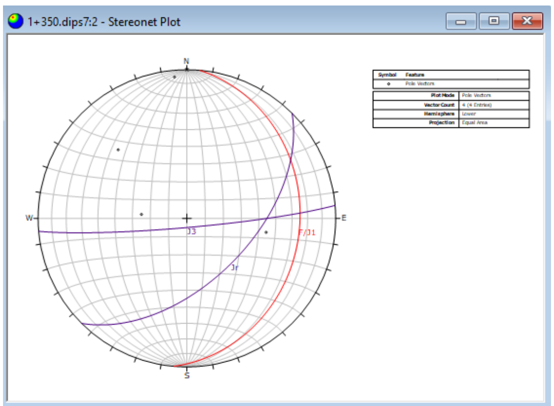






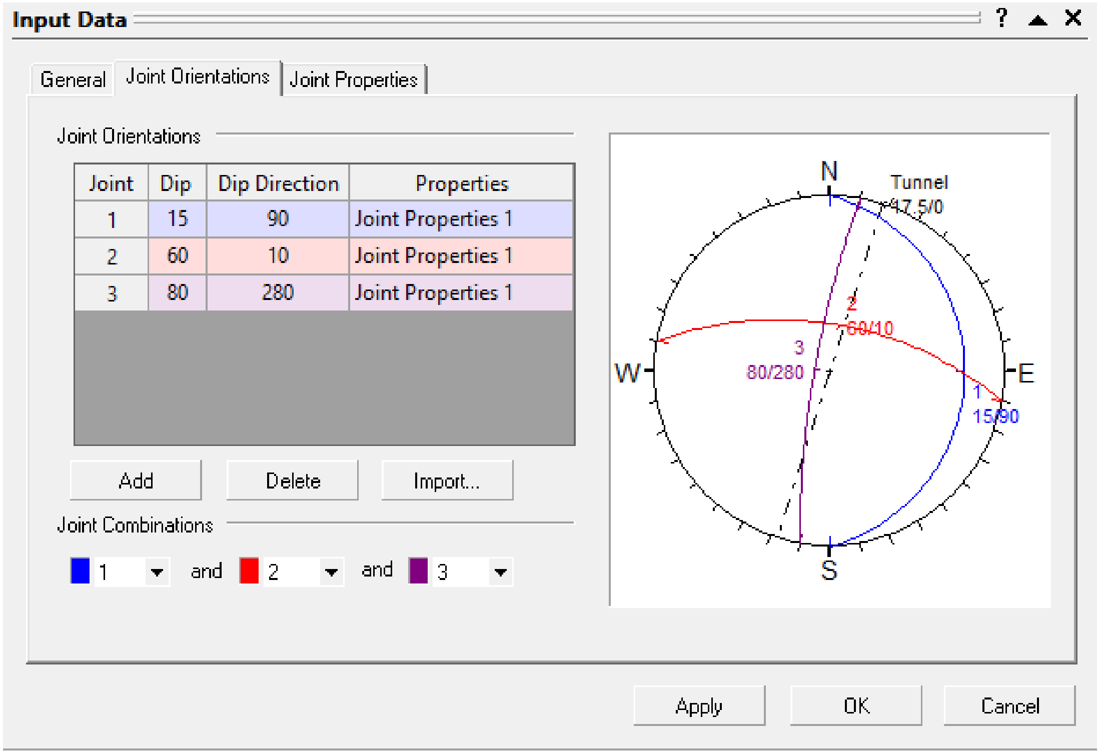


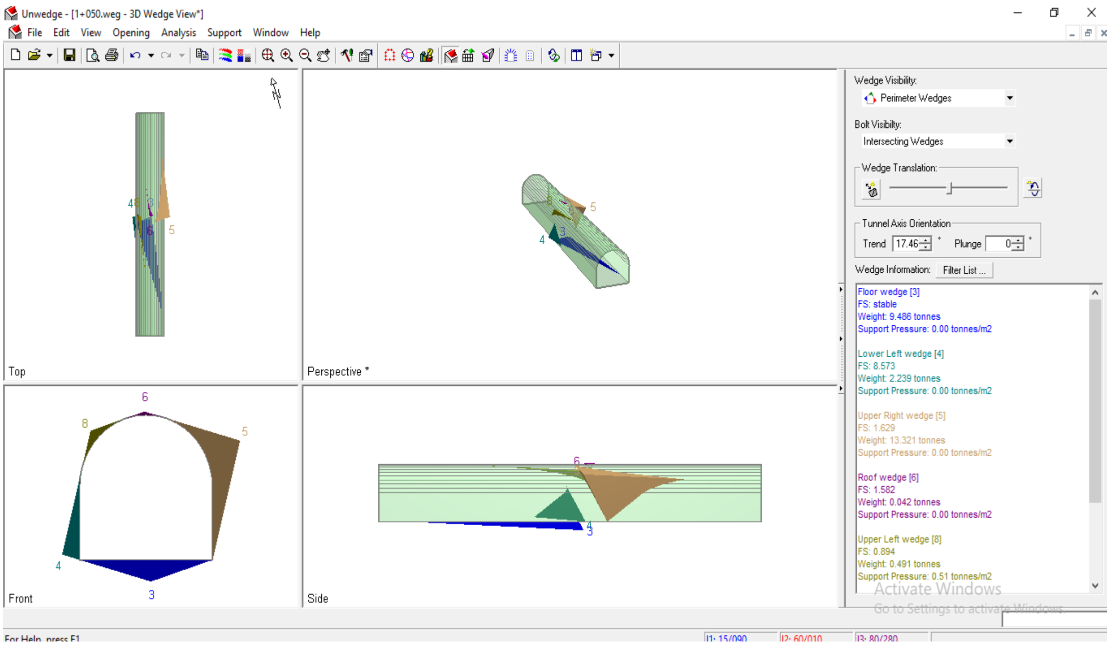


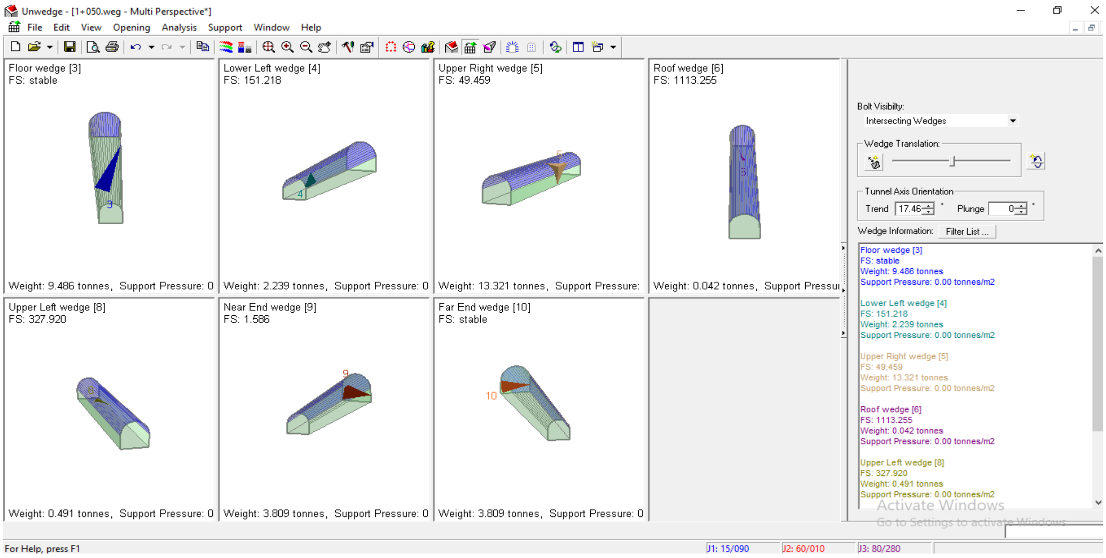


Input Data and Unwedge Analysis pictures at each section using UNWEDGE Software.

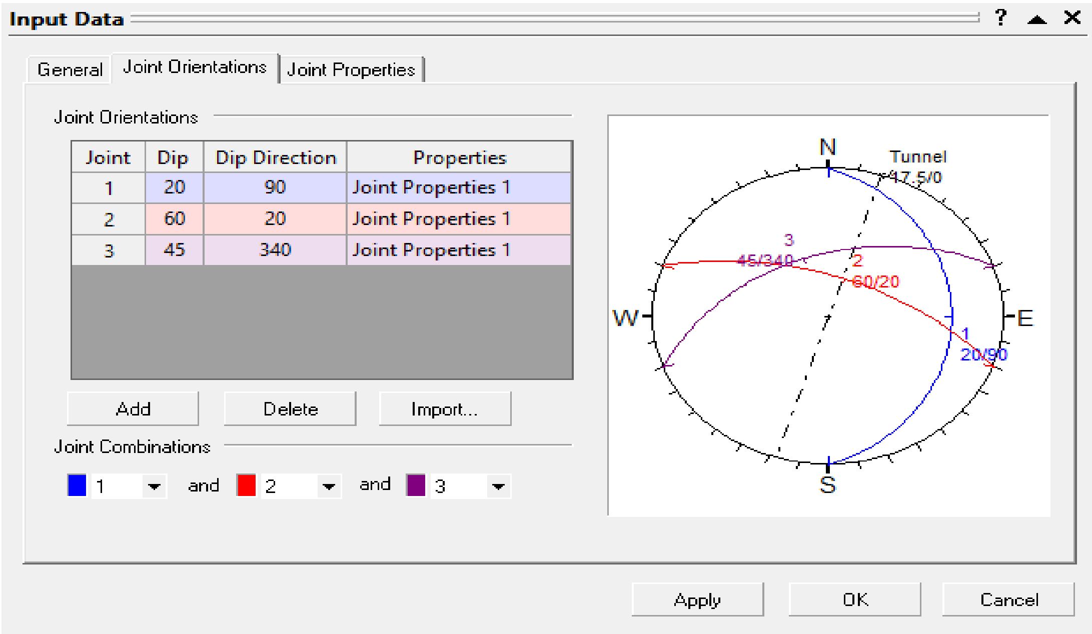
Chainage 1+050m

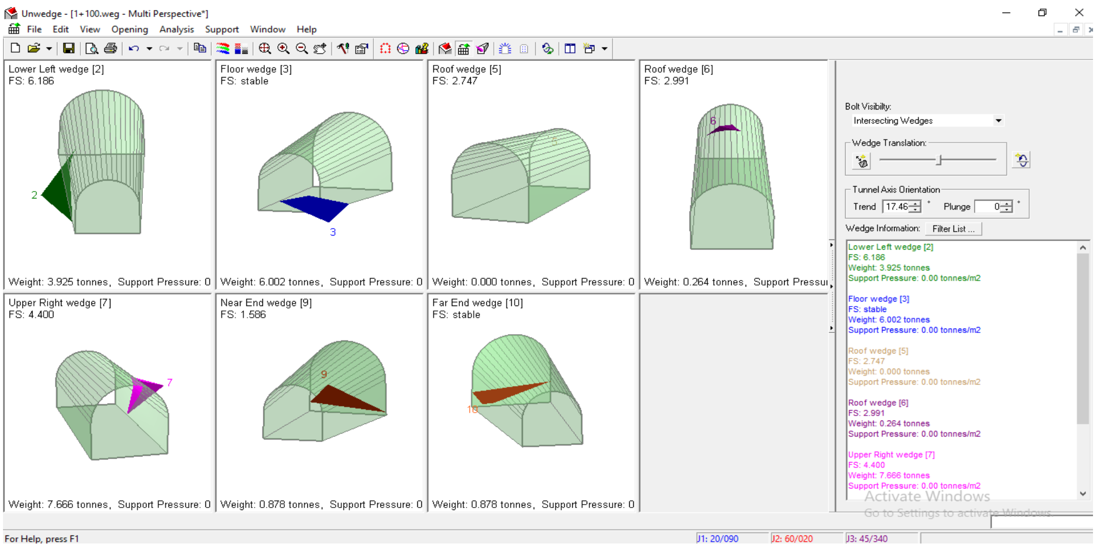


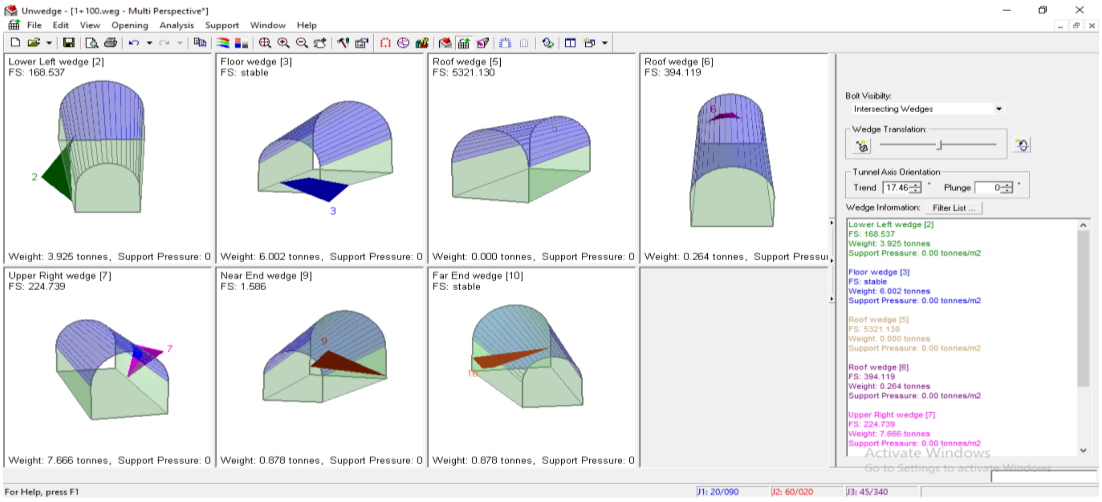




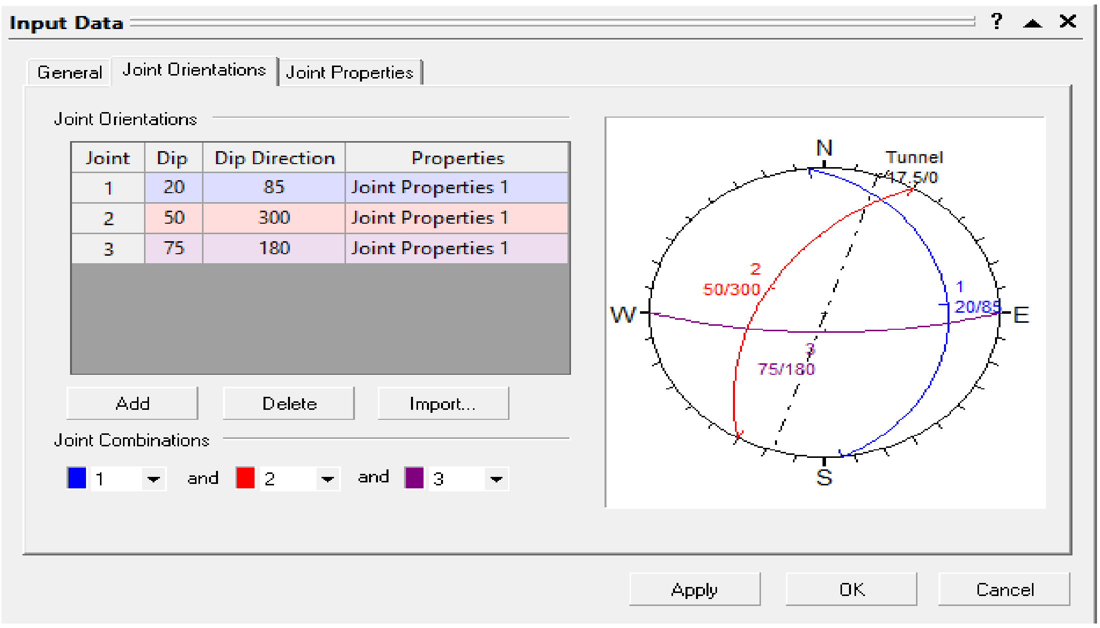
Chainage 1+100m

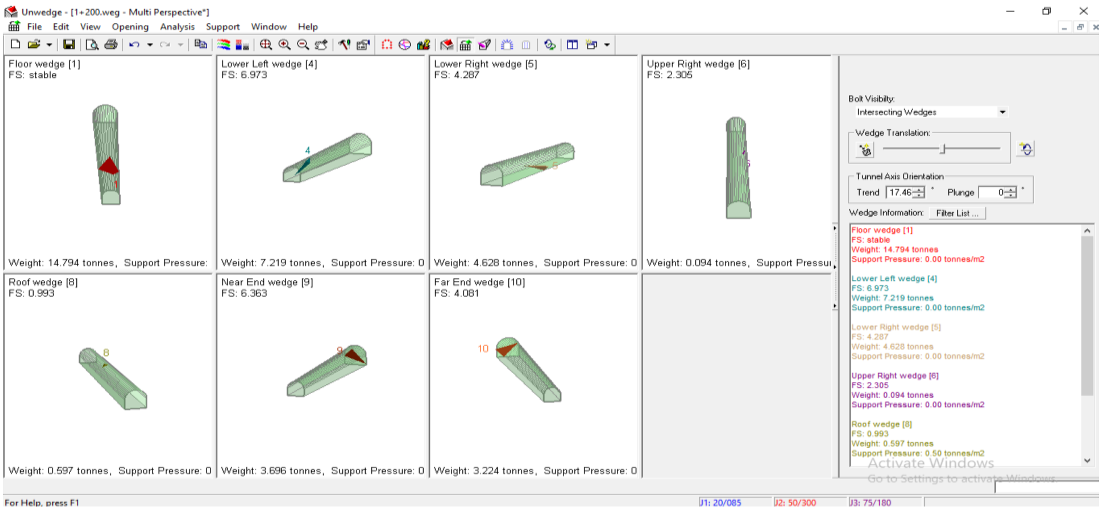


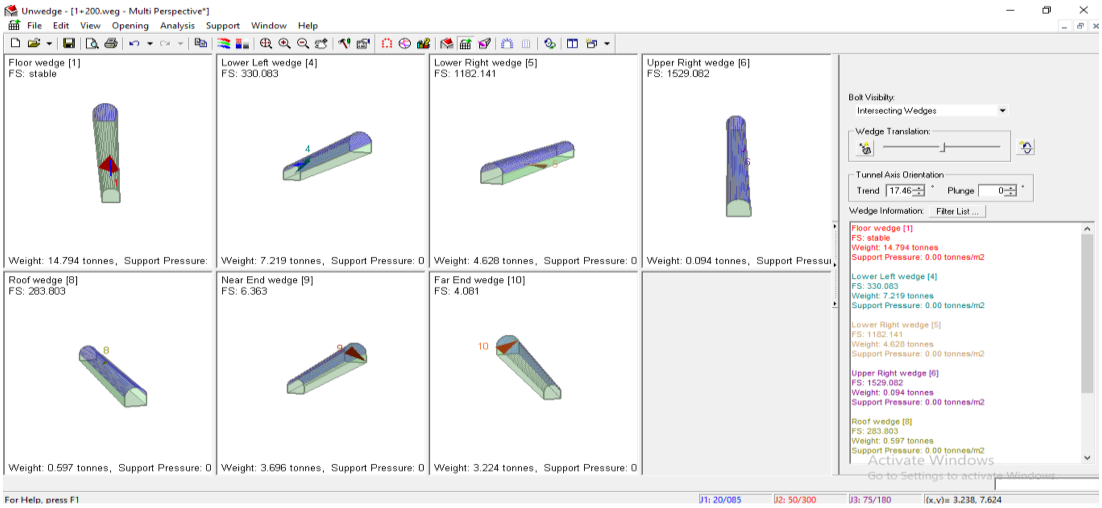




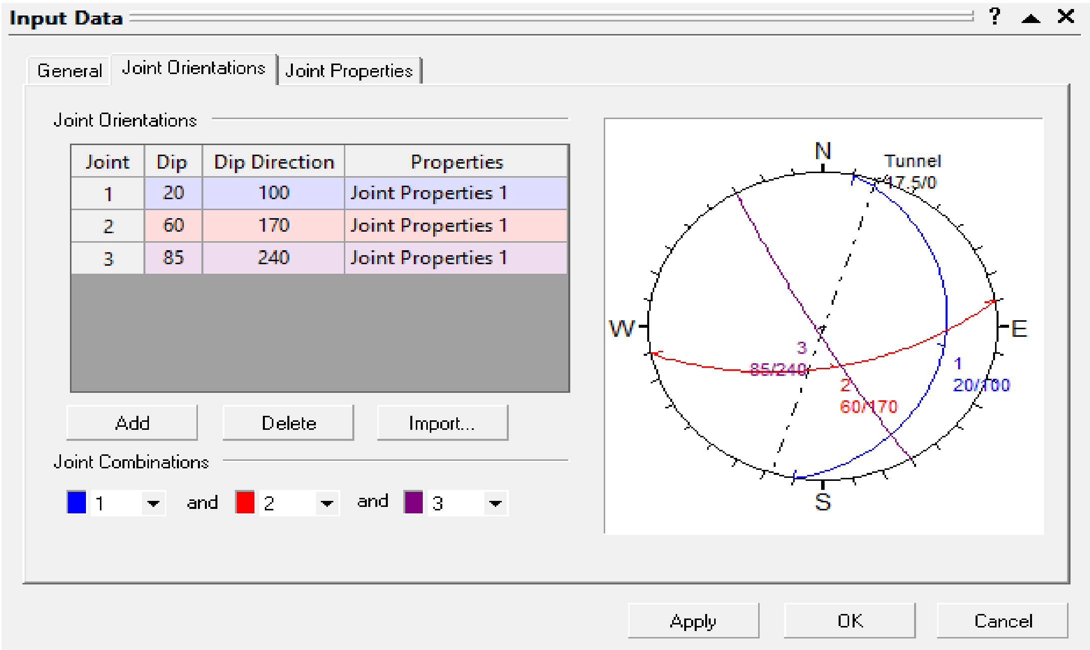
Chainage 1+200m

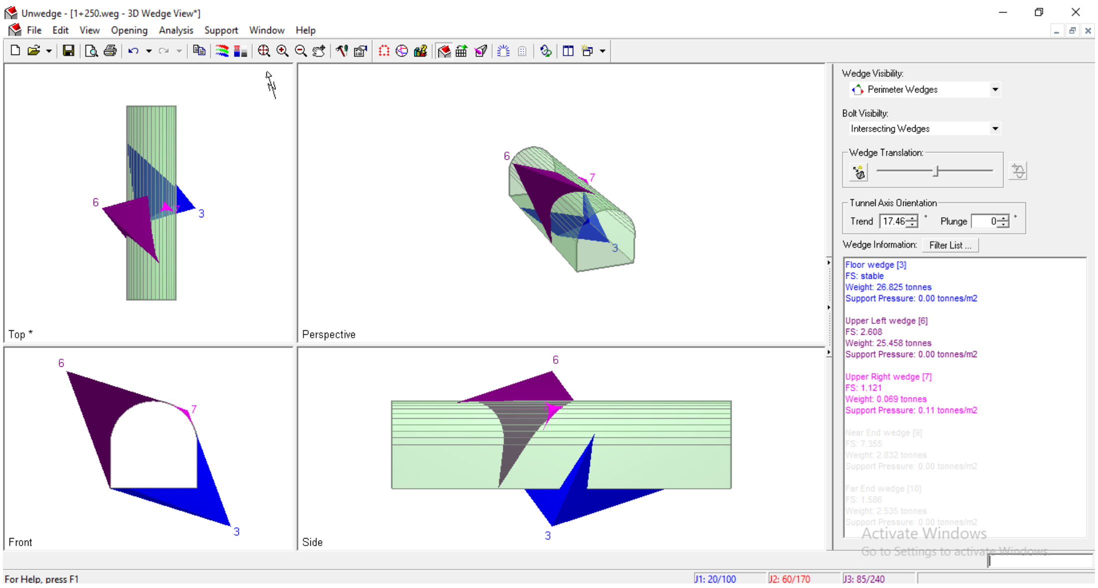


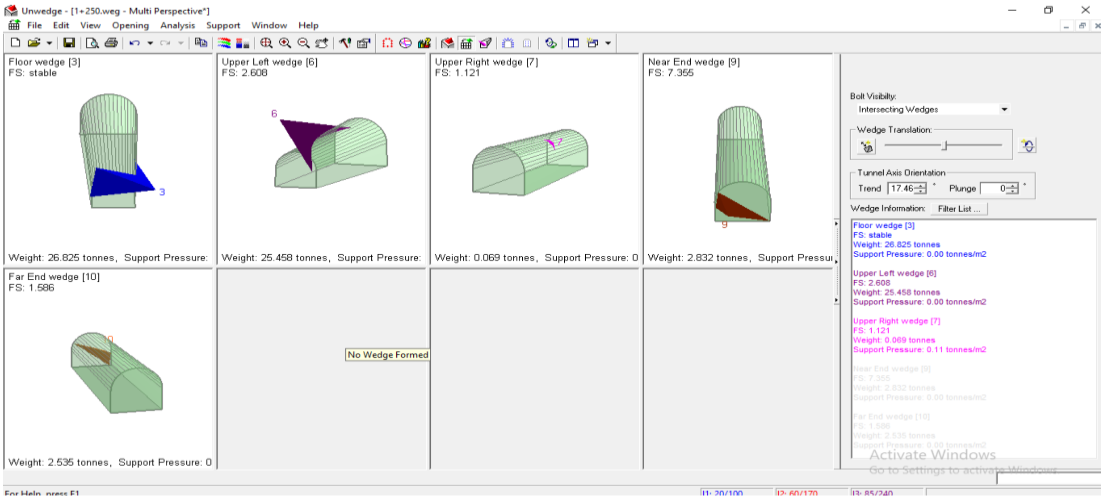




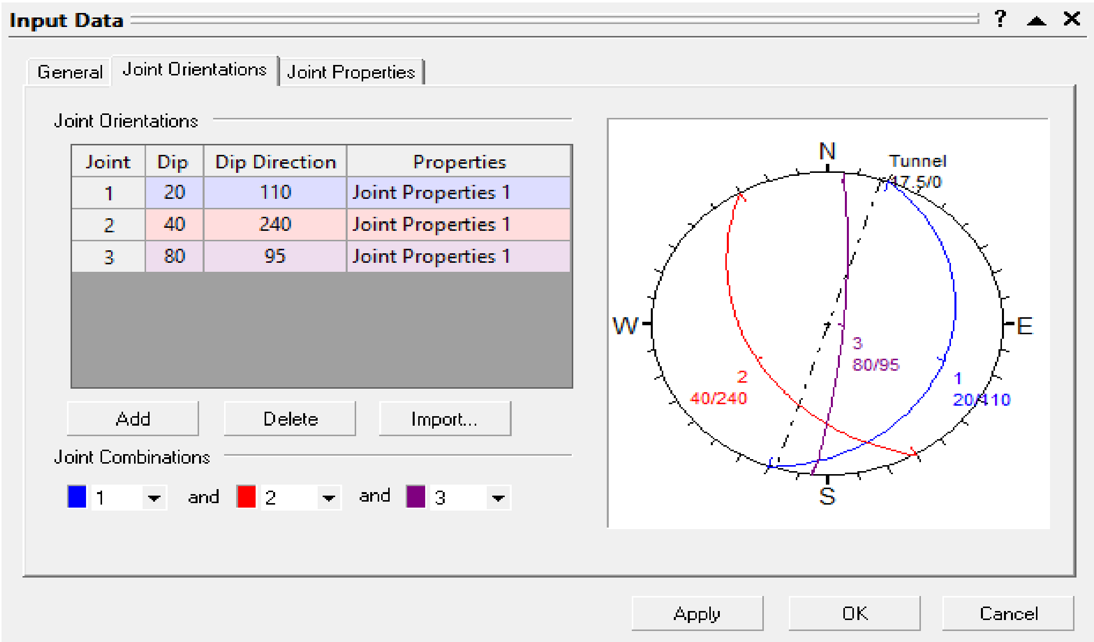
Chainage 1+250m

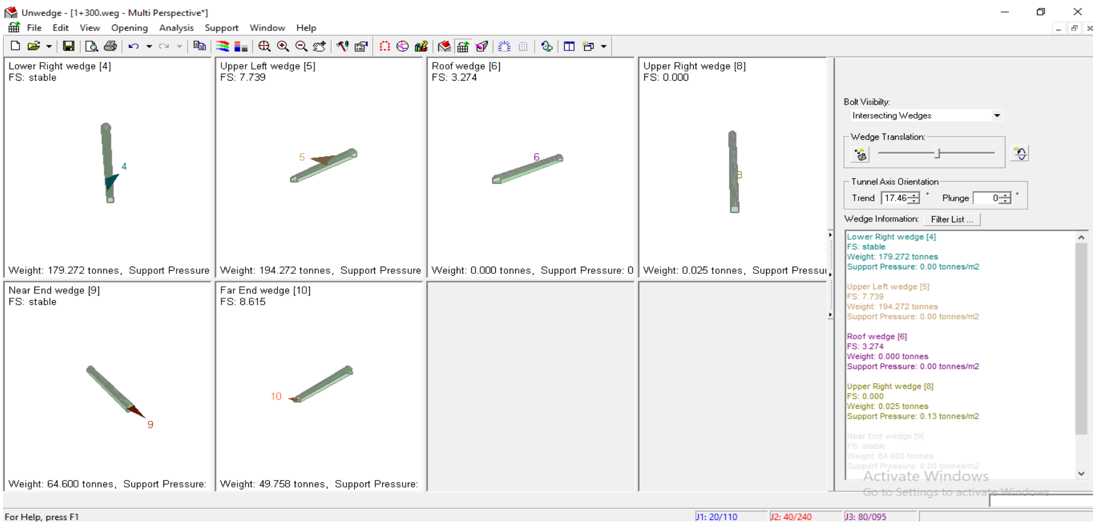


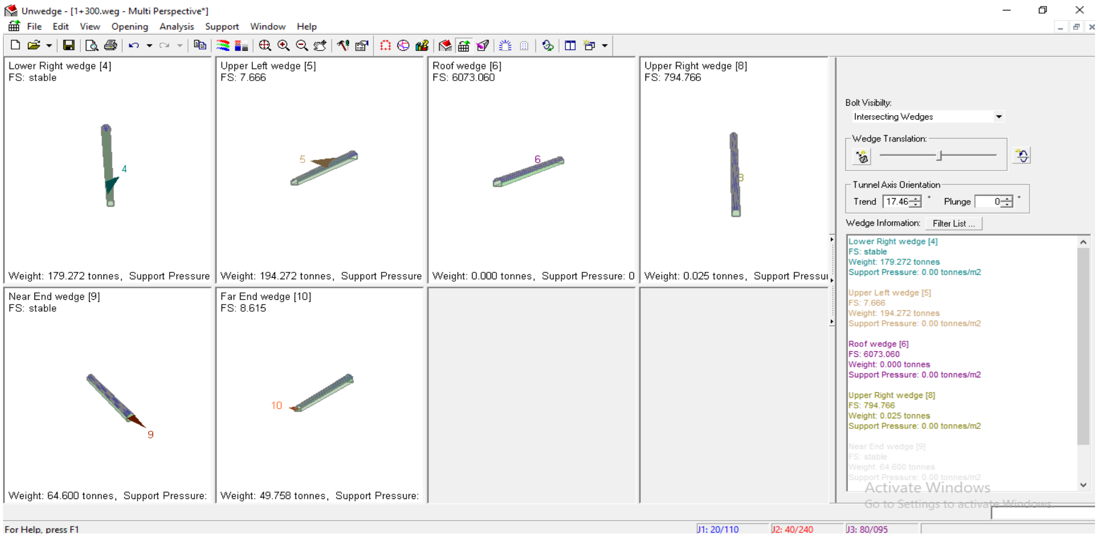




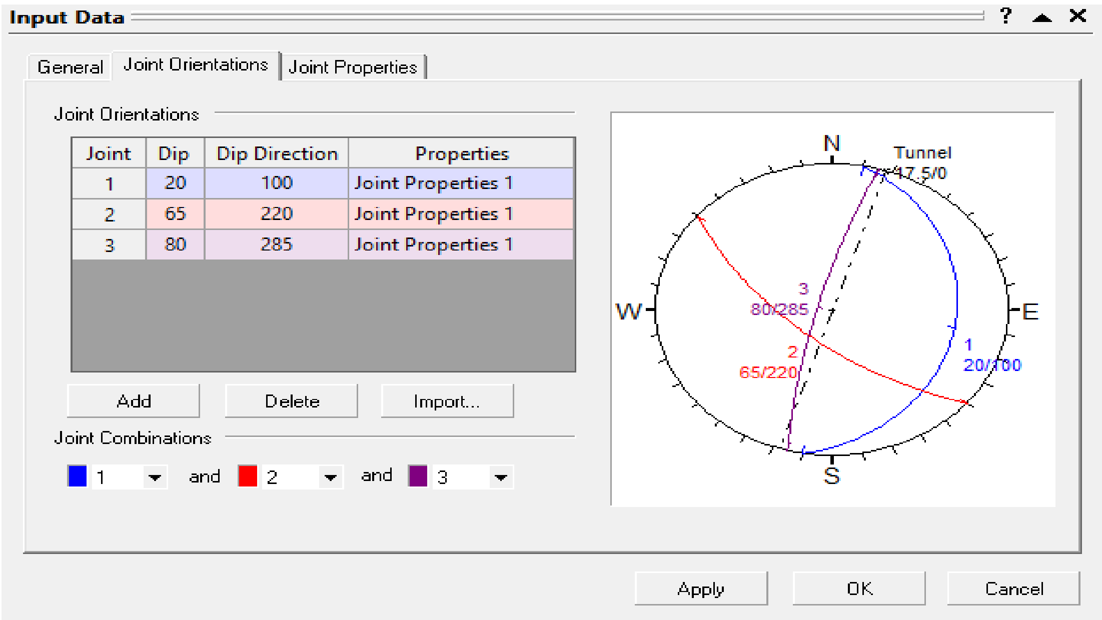
Chainage 1+300m

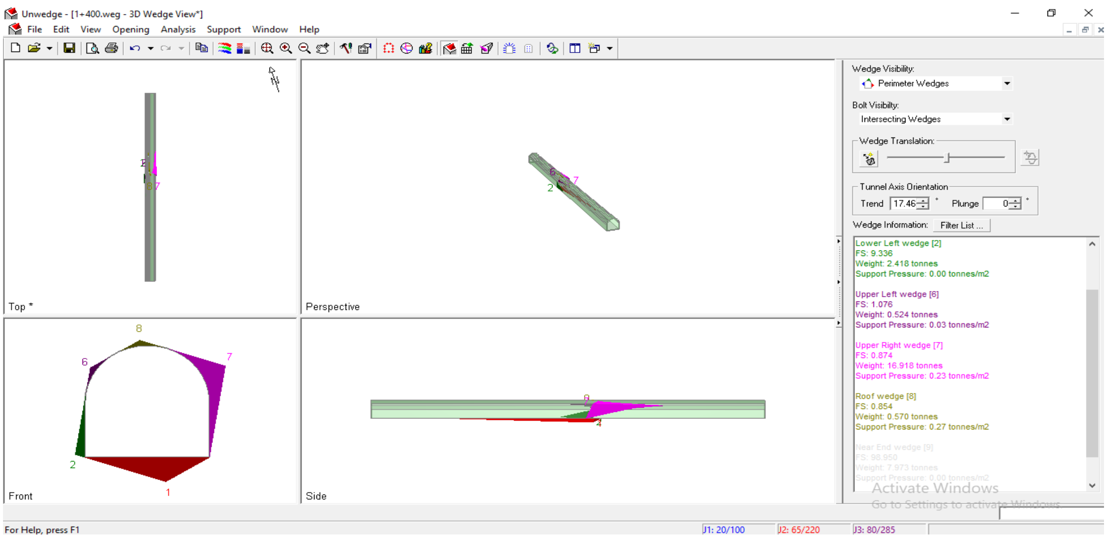


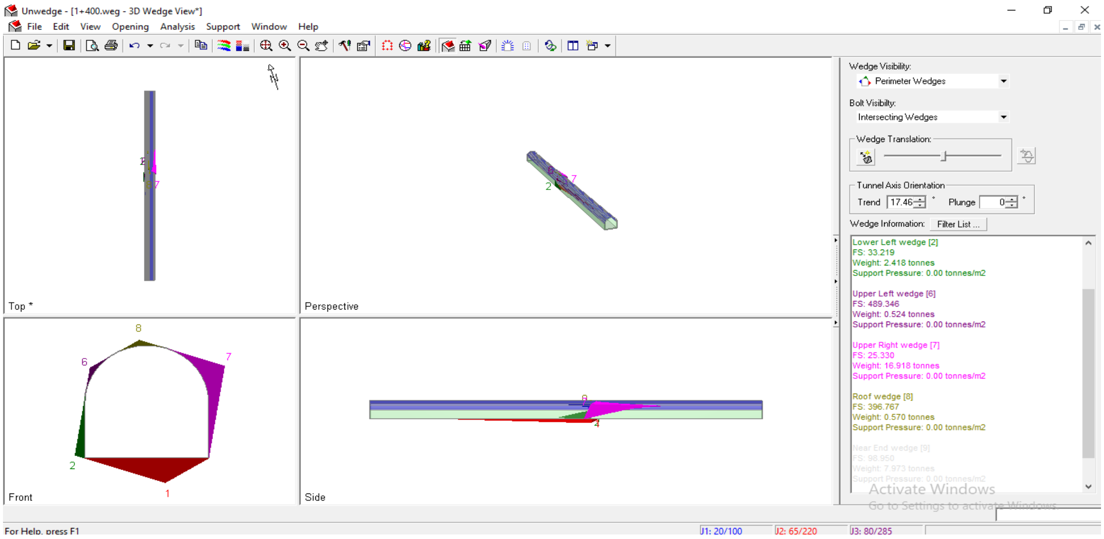




Chainage 1+400m

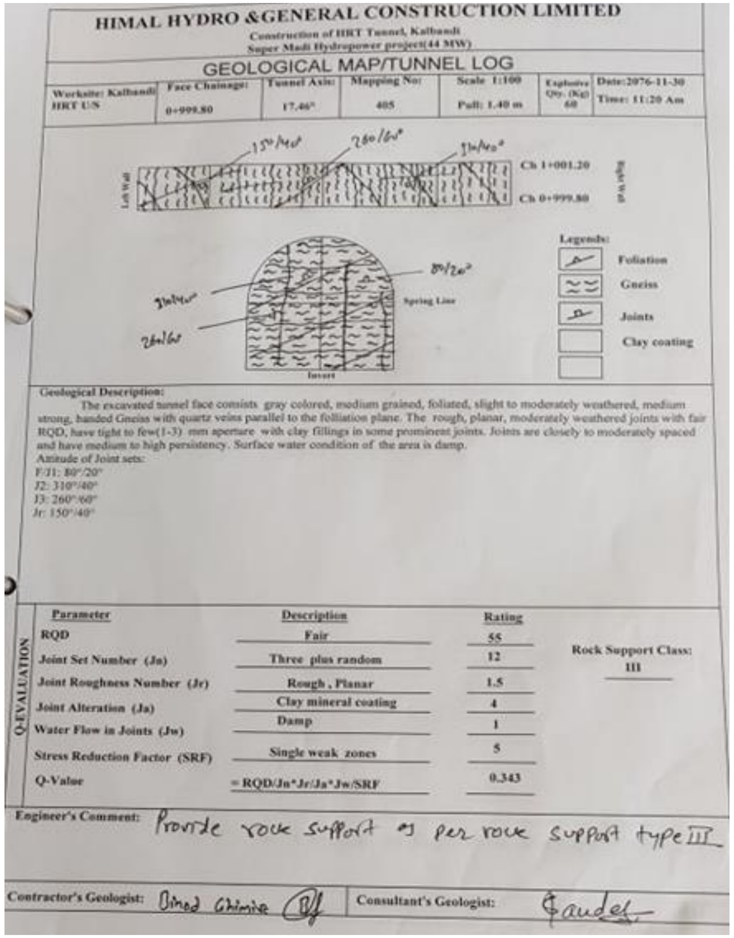





Unwedge - [1+400.weg - 3D Wedge View*]
File Edit View Opening Analysis Support Window Help
Top *
Perspective
Front
Side
Wedge Visibility:
Perimeter Wedges
Bolt Visibility:
Intersecting Wedges
Wedge Translation:
Tunnel Axis Orientation
Trend 17.46
Plunge 0
Wedge Information:
Filter List ...
Lower Left wedge [2]
FS: 33.219
Weight: 2.418 tonnes
Support Pressure: 0.00 tonnes/m2
Upper Left wedge [6]
FS: 489.346
Weight: 0.524 tonnes
Support Pressure: 0.00 tonnes/m2
Upper Right wedge [7]
FS: 25.330
Weight: 16.918 tonnes
Support Pressure: 0.00 tonnes/m2
Roof wedge [8]
FS: 396.767
Weight: 0.570 tonnes
Support Pressure: 0.00 tonnes/m2
For Help, press F1
J1: 20/100
J2: 65/220
J3: 80/285

ANNEX F: PROJECT RELATED DATA & DOCUMENTS

Facemap showing Geological Condition at every Chainage



HIMAL HYDRO &GENERAL CONSTRUCTION LIMITED
Construction of HRT Tunnel, Kalbandi
Super Madi Hydropower project(44 MW)

GEOLOGICAL MAP/TUNNEL LOG

Worksite: Kalbandi HRT U/S	Face Chainage: 0+999.80	Tunnel Axis: 17.46°	Mapping No: 405	Scale 1:100 Pull: 1.40 m	Explosive Qty. (Kg) 60	Date:2076-11-30 Time: 11:20 Am

Ch 1+001.20
Ch 0+999.80
Left Wall
Right Wall
Spring Line
Invert

Legends: Foliation, Gneiss, Joints, Clay coating

Geological Description:
The excavated tunnel face consists gray colored, medium grained, foliated, slight to moderately weathered, medium strong, banded Gneiss with quartz veins parallel to the folliation plane. The rough, planar, moderately weathered joints with fair RQD, have tight to few(1-3) mm aperture with clay fillings in some prominent joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.
Attitude of Joint sets:
F/J1: 80°/20°
J2: 310°/40°
J3: 260°/60°
Jr: 150°/40°

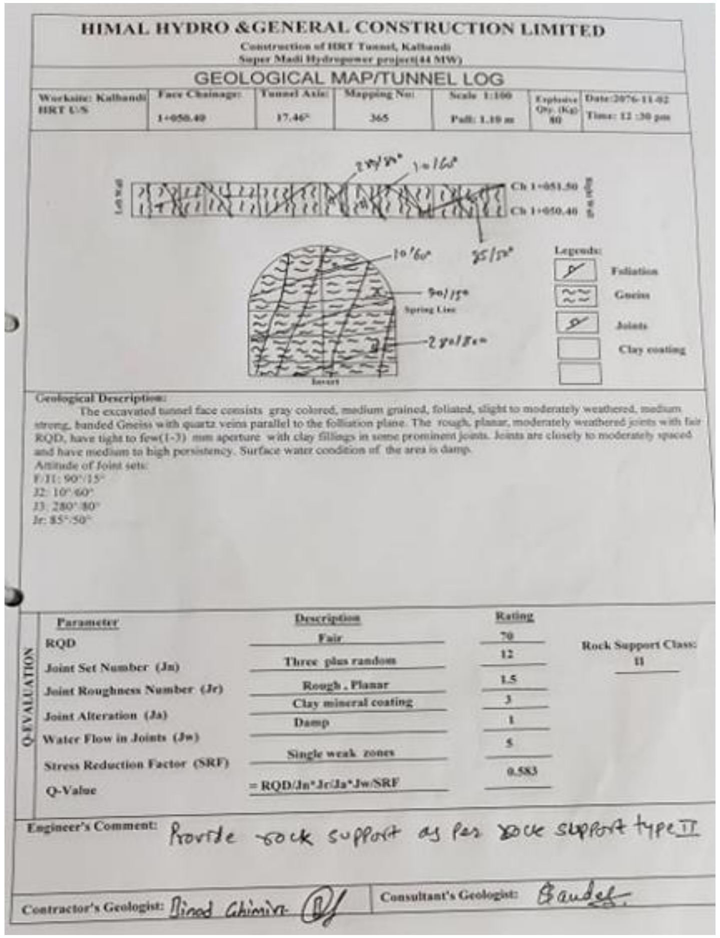
Q-EVALUATION

Parameter	Description	Rating
RQD	Fair	55
Joint Set Number (Jn)	Three plus random	12
Joint Roughness Number (Jr)	Rough , Planar	1.5
Joint Alteration (Ja)	Clay mineral coating	4
Water Flow in Joints (Jw)	Damp	1
Stress Reduction Factor (SRF)	Single weak zones	5
Q-Value	= RQD/Jn*Jr/Ja*Jw/SRF	0.343

Rock Support Class: III

Engineer's Comment: Provide rock support as per rock support type III

Contractor's Geologist: Binod Ghimire
Consultant's Geologist: Gaudel



HIMAL HYDRO &GENERAL CONSTRUCTION LIMITED
Construction of HRT Tunnel, Kalbandi
Super Madi Hydropower project(44 MW)

GEOLOGICAL MAP/TUNNEL LOG

Worksite: Kalbandi HRT U/S	Face Chainage: 1+050.40	Tunnel Axis: 17.46°	Mapping No: 365	Scale 1:100 Pull: 1.10 m	Explosive Qty. (Kg) 80	Date:2076-11-02 Time: 12 :30 pm

Ch 1+051.50
Ch 1+050.40
Left Wall
Right Wall
Spring Line
Invert

Legends: Foliation, Gneiss, Joints, Clay coating

Geological Description:
The excavated tunnel face consists gray colored, medium grained, foliated, slight to moderately weathered, medium strong, banded Gneiss with quartz veins parallel to the folliation plane. The rough, planar, moderately weathered joints with fair RQD, have tight to few(1-3) mm aperture with clay fillings in some prominent joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.
Attitude of Joint sets:
F/J1: 90°/15°
J2: 10°/60°
J3: 280°/80°
Jr: 85°/50°

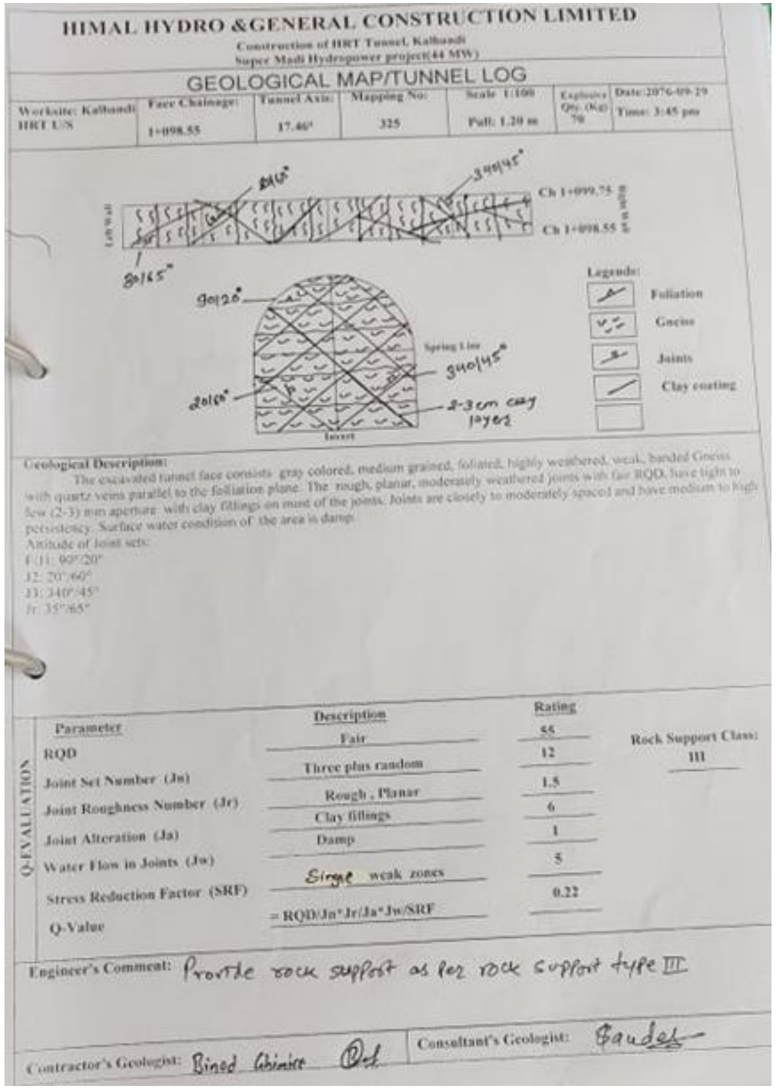
Q-EVALUATION

Parameter	Description	Rating
RQD	Fair	70
Joint Set Number (Jn)	Three plus random	12
Joint Roughness Number (Jr)	Rough , Planar	1.5
Joint Alteration (Ja)	Clay mineral coating	3
Water Flow in Joints (Jw)	Damp	1
Stress Reduction Factor (SRF)	Single weak zones	5
Q-Value	= RQD/Jn*Jr/Ja*Jw/SRF	0.583

Rock Support Class: II

Engineer's Comment: Provide rock support as per rock support type II

Contractor's Geologist: Binod Ghimire
Consultant's Geologist: Gaudel



HIMAL HYDRO &GENERAL CONSTRUCTION LIMITED

Construction of HRT Tunnel, Kalbandi

Super Madi Hydropower project(44 MW)

GEOLOGICAL MAP/TUNNEL LOG

Worksite: Kalbandi HRT U/S	Face Chainage: 1+098.55	Tunnel Axis: 17.46°	Mapping No: 325	Scale 1:100 Pull: 1.20 m	Explosive Qty. (Kg) 70	Date:2076-09-29 Time: 3:45 pm

Geological Description:

The excavated tunnel face consists gray colored, medium grained, foliated, highly weathered, weak, banded Gneiss with quartz veins parallel to the foliation plane. The rough, planar, moderately weathered joints with fair RQD, have tight to few (2-3) mm aperture with clay fillings on most of the joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.

Attitude of Joint sets:

F/J1: 90°/20°

J2: 20°/60°

J3: 340°/45°

Jr: 35°/65°

Q-EVALUATION

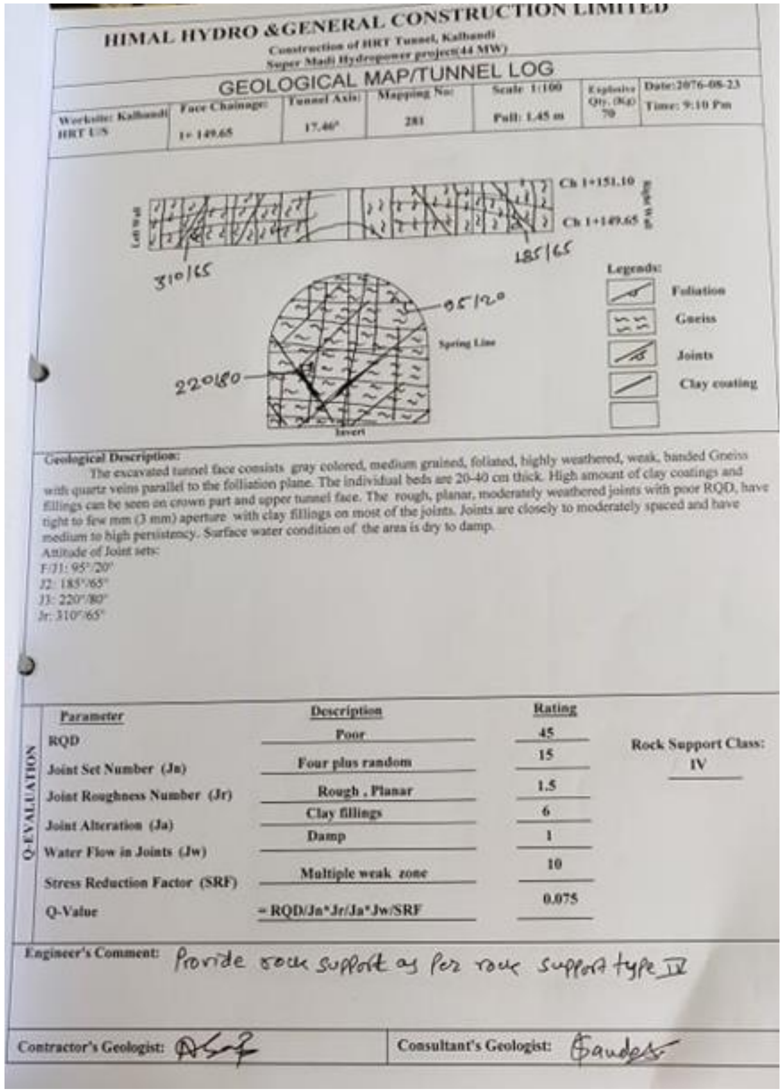
Parameter	Description	Rating
RQD	Fair	55
Joint Set Number (Jn)	Three plus random	12
Joint Roughness Number (Jr)	Rough , Planar	1.5
Joint Alteration (Ja)	Clay fillings	6
Water Flow in Joints (Jw)	Damp	1
Stress Reduction Factor (SRF)	Single weak zones	5
Q-Value	= RQD/Jn*Jr/Ja*Jw/SRF	0.22

Rock Support Class: III

Engineer's Comment: Provide rock support as per rock support type III

Contractor's Geologist: Binod Ghimire

Consultant's Geologist:



HIMAL HYDRO &GENERAL CONSTRUCTION LIMITED

Construction of HRT Tunnel, Kalbandi

Super Madi Hydropower project(44 MW)

GEOLOGICAL MAP/TUNNEL LOG

Worksite: Kalbandi HRT U/S	Face Chainage: 1+ 149.65	Tunnel Axis: 17.46°	Mapping No: 281	Scale 1:100 Pull: 1.45 m	Explosive Qty. (Kg) 70	Date:2076-08-23 Time: 9:10 Pm

Geological Description:

The excavated tunnel face consists gray colored, medium grained, foliated, highly weathered, weak, banded Gneiss with quartz veins parallel to the folliation plane. The individual beds are 20-40 cm thick. High amount of clay coatings and fillings can be seen on crown part and upper tunnel face. The rough, planar, moderately weathered joints with poor RQD, have tight to few mm (3 mm) aperture with clay fillings on most of the joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is dry to damp.

Attitude of Joint sets:

F/J1: 95°/20°

J2: 185°/65°

J3: 220°/80°

Jr: 310°/65°

Q-EVALUATION

Parameter	Description	Rating
RQD	Poor	45
Joint Set Number (Jn)	Four plus random	15
Joint Roughness Number (Jr)	Rough , Planar	1.5
Joint Alteration (Ja)	Clay fillings	6
Water Flow in Joints (Jw)	Damp	1
Stress Reduction Factor (SRF)	Multiple weak zone	10
Q-Value	= RQD/Jn*Jr/Ja*Jw/SRF	0.075

Rock Support Class: IV

Engineer's Comment: Provide rock support as per rock support type IV

Contractor's Geologist:

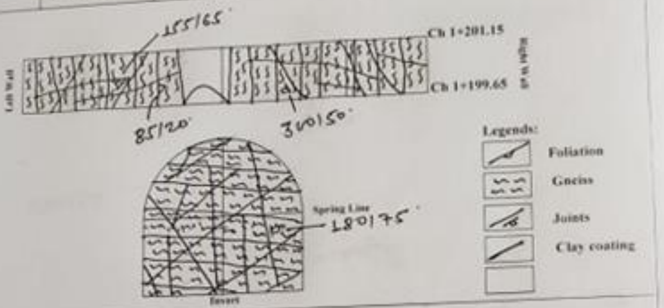
Consultant's Geologist:

HIMAL HYDRO &GENERAL CONSTRUCTION LIMITED

Construction of HRT Tunnel, Kalbandi
Super Madi Hydropower project(44 MW)

GEOLOGICAL MAP/TUNNEL LOG

Worksite: Kalbandi HRT U/S	Face Chainage: 1+ 199.65	Tunnel Axis: 17.46°	Mapping No: 246	Scale 1:100 Pull: 1.5 m	Explosive Qty. (Kg) 50	Date:2076-07-26 Time: 11:30 Pm



Geological Description:
The excavated tunnel face consists gray colored, medium grained, foliated, highly weathered, weak, banded Gneiss with quartz veins parallel to the folliation plane. High amount of clay coatings and fillings can be seen on crown part and upper tunnel face. Below the spring line, clay fillings upto 1 m can be seen. The rough, planar, moderately weathered joints with poor RQD, have tight to few mm (2 mm) aperture with clay fillings on most of the joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.
Attitude of Joint sets:
J1: 85°/20°
: 300°/50°
: 180°/75°
: 155°/65°

Parameter	Description	Rating
RQD	Poor	30
Joint Set Number (Jn)	Heavily jointed	15
Joint Roughness Number (Jr)	Rough , Planar	1.5
Joint Alteration (Ja)	Clay fillings	8
Water Flow in Joints (Jw)	Damp	1
Stress Reduction Factor (SRF)	Multiple weak zones	10
Q-Value	= RQD/Jn*Jr/Ja*Jw/SRF	0.0375

Rock Support Class: V

Engineer's Comment: Provide rock support as per rock support type V

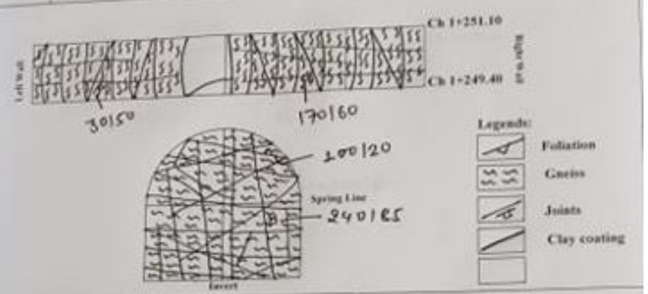
Contractor's Geologist: [signature] | Consultant's Geologist: [signature]

HIMAL HYDRO &GENERAL CONSTRUCTION LIMITED

Construction of HRT Tunnel, Kalbandi
Super Madi Hydropower project(44 MW)

GEOLOGICAL MAP/TUNNEL LOG

Worksite: Kalbandi HRT U/S	Face Chainage: 1+ 249.40	Tunnel Axis: 17.46°	Mapping No: 215	Scale 1:100 Pull: 1.7 m	Explosive Qty. (Kg) 62.6	Date:2076-06-11 Time: 2:30 Pm



Geological Description:
The excavated tunnel face consists gray colored, medium grained, foliated, highly weathered, weak, banded Gneiss with quartz veins parallel to the folliation plane. The individual beds are 20-40 cm thick. High amount of clay coatings and fillings can be seen on crown part and upper tunnel face. The rough, planar, moderately weathered joints with poor RQD, have tight to few mm (3 mm) aperture with clay fillings on most of the joints. Joints are closely to moderately spaced and have medium to high persistency. Surface water condition of the area is damp.
Attitude of Joint sets:
J1: 100°/20°
: 170°/60°
: 240°/85°
: 30°/50°

Parameter	Description	Rating
RQD	Poor	45
Joint Set Number (Jn)	Three plus random	12
Joint Roughness Number (Jr)	Rough , Planar	1.5
Joint Alteration (Ja)	Clay fillings	8
Water Flow in Joints (Jw)	Damp	1
Stress Reduction Factor (SRF)	Multiple weak zones	10
Q-Value	= RQD/Jn*Jr/Ja*Jw/SRF	0.07

Rock Support Class: IV

Engineer's Comment: Provide rock support as per rock support type IV

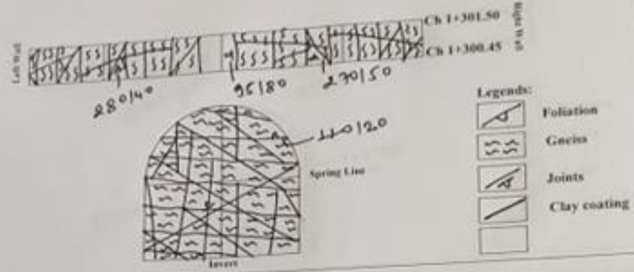
Contractor's Geologist: [signature] | Consultant's Geologist: [signature]

HIMAL HYDRO &GENERAL CONSTRUCTION LIMITED

Construction of Adit Tunnel,Bagalethar
Super Madi Hydropower project(44 MW)

GEOLOGICAL MAP/TUNNEL LOG

Worksite: Kalbandi HRT U/S	Face Chainage: 1+300.45	Tunnel Axis: 17.46°	Mapping No: 178	Scale 1:100 Pull: 1.05 m	Explosive Qty. (Kg) 99.4	Date:2076-05-14 Time: 4:30 Pm



Geological Description:

The excavated tunnel face consists gray colored, median grained, foliated, low weathered, medium strong to strong, banded Gneiss with quartz veins parallel to the folliation plane. The individual beds are 20-100 cm thick. The rough, planar, moderately weathered joints with good RQD, have tight to few mm(1 mm) aperture with clay coating in few joints. Joints are closely to moderately spaced and have medium persistency. Surface water condition of the area is dry to damp.

Attitude of Joint sets:
F/J1: 110°/20°
J2: 280°/40°
J3: 95°/80°
Jr: 270°/50°

Q-EVALUATION

Parameter	Description	Rating
RQD	Good	75
Joint Set Number (Jn)	Three plus random	12
Joint Roughness Number (Jr)	Rough , Planar	1.5
Joint Alteration (Ja)	Clay coatings	3
Water Flow in Joints (Jw)	Damp	1
Stress Reduction Factor (SRF)	Low stress	2.5
Q-Value	= RQD/Jn*Jr/Ja*Jw/SRF	1.25

Rock Support Class: II

Engineer's Comment: Provide rock support as per rock support type II

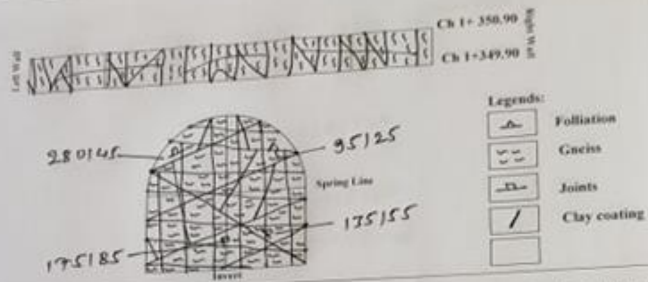
Contractor's Geologist: [signature] Consultant's Geologist: [signature]

HIMAL HYDRO &GENERAL CONSTRUCTION LIMITED

Construction of Adit Tunnel,Bagalethar
Super Madi Hydropower project(44 MW)

GEOLOGICAL MAP/TUNNEL LOG

Worksite: Kalbandi HRT U/S	Face Chainage: 1+349.90	Tunnel Axis: 17.46°	Mapping No: 135	Scale 1:100 Pull: 1 m	Explosive Qty. (Kg) 91	Date:2076-04-17 Time: 1:30 Pm



Geological Description:

The excavated tunnel face consists gray colored, medium grained, foliated, moderately weathered, medium strong to strong, banded Gneiss with quartz veins parallel to the folliation plane. The individual beds are 20-80 cm thick. The rough, planar, moderately weathered joints with fair RQD , have tight to few mm(1-2 mm) aperture with clay coating in few joints. Joints are closely to moderately spaced and have medium persistency. Surface water condition of the area is dry to damp.

Attitude of Joint sets:
F/J1: 95°/25°
J2: 280°/45°
J3: 175°/85°
Jr: 135°/55°

Q-EVALUATION

Parameter	Description	Rating
RQD	Fair	65
Joint Set Number (Jn)	Three plus random	12
Joint Roughness Number (Jr)	Rough , Planar	1.5
Joint Alteration (Ja)	Clay coatings	3
Water Flow in Joints (Jw)	Damp	1
Stress Reduction Factor (SRF)	Low stress	2.5
Q-Value	= RQD/Jn*Jr/Ja*Jw/SRF	1.08

Rock Support Class: II

Engineer's Comment: Provide rock support as per rock support type II

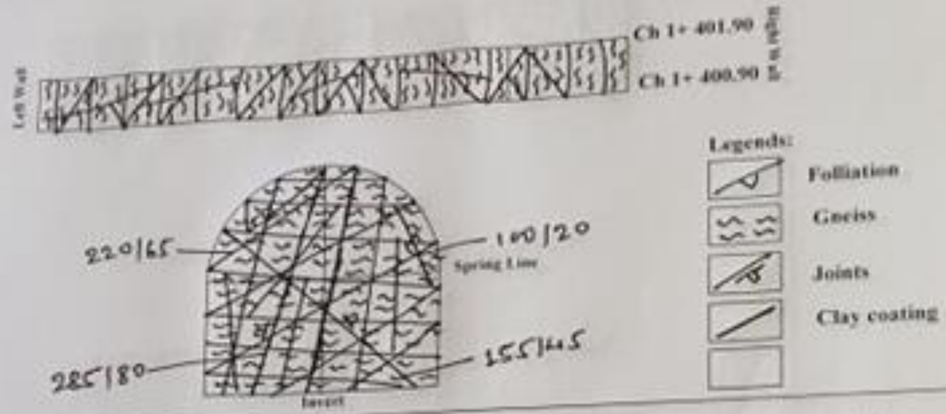
Contractor's Geologist: [signature] Consultant's Geologist: [signature]

HIMAL HYDRO &GENERAL CONSTRUCTION LIMITED

Construction of Adit Tunnel,Bagaletbar
Super Madi Hydropower project(44 MW)

GEOLOGICAL MAP/TUNNEL LOG

Worksite: Kalbandi HRT U/S	Face Chainage: 1+ 400.90	Tunnel Axis: 17.46°	Mapping No: 97	Scale 1:100 Pull: 1 m	Explosive Qty. (Kg) 98.6	Date:2076-3-20 Time: 12:45 Pm



Geological Description:

The tunnel face consists gray colored, medium grained, foliated, moderately weathered, medium strong to strong, banded Gneiss with quartz veins parallel to the folliation plane. The individual beds are 10-60 cm thick. The rough, planar, moderately weathered joints with fair RQD , have tight to few mm(1-3mm) aperture with clay coatings and fillings in few joints. Joints are closely to moderately spaced and have medium persistency. Surface water condition of the area is dry to damp.

Attitude of Joint sets:

1: 100°/20°
220°/65°
285°/80°
155°/45°

Parameter	Description	Rating
RQD	Fair	65
Joint Set Number (Jn)	Three plus random	12
Joint Roughness Number (Jr)	Rough , Planar	1.5
Joint Alteration (Ja)	Clay fillings	6
Water Flow in Joints (Jw)	Damp	1
Stress Reduction Factor (SRF)	Single weak zone	5
Q Value	= RQD/Jn*Jr/Ja*Jw/SRF	0.27

Rock Support Class: III

Geologist's Comment: Provide rock support as per rock support type III

Contractor's Geologist: [signature]

Consultant's Geologist: Gautam

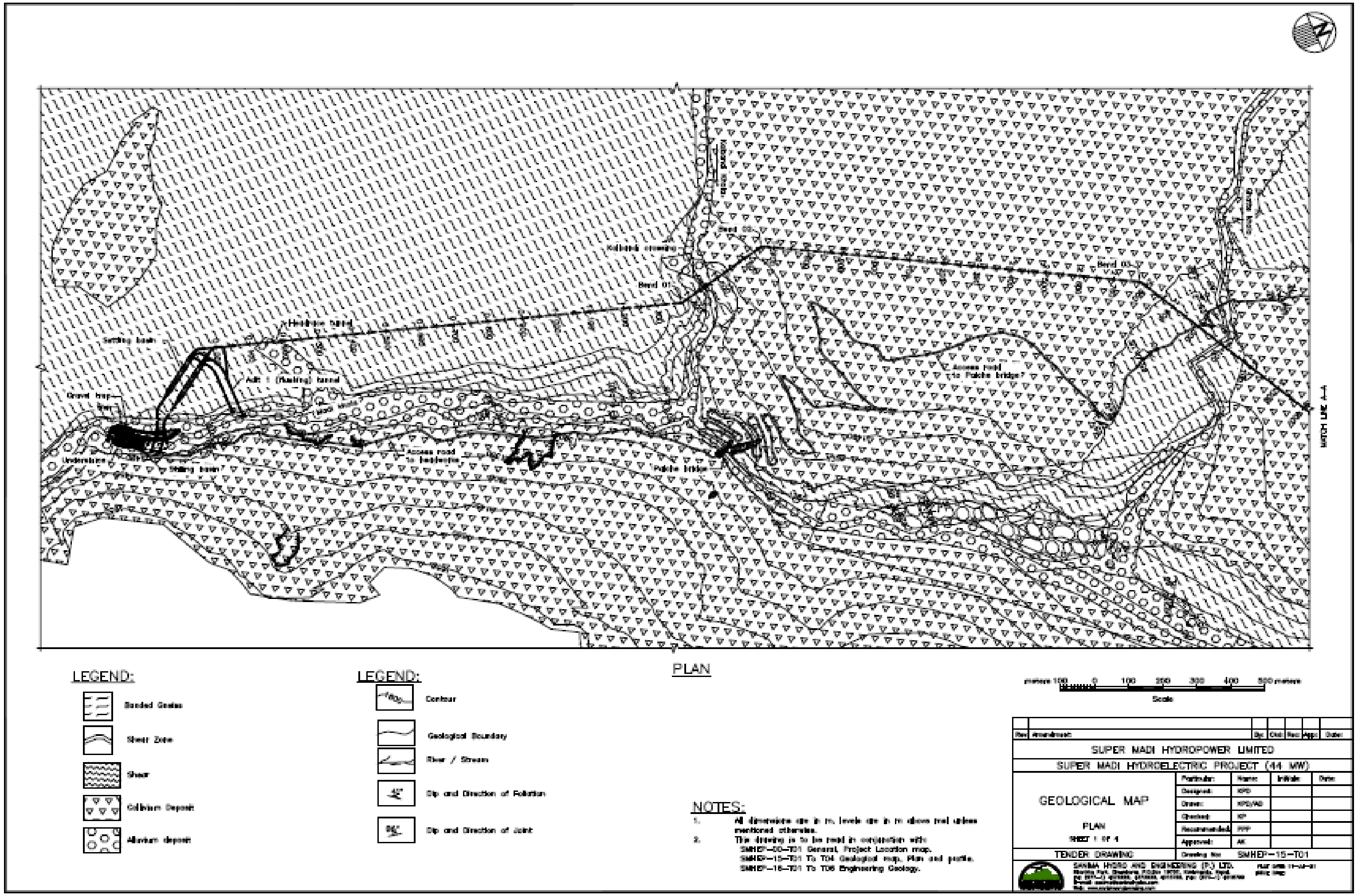
Salient Features of Project

GENERAL	
Name of the Project	Super Madi Hydroelectric Project
Developer	Super Madi Hydropower Ltd.
Name of the River	Madi River
Type of Scheme	Run-of-River
Project Location	Madi Rural Municipality
District	Kaski
Province	Province-04
Latitude	28°19’ 02” N to 28°21’ 39” N
Longitude	84° 04’ 45” E to 84° 08’ 34” E
Nearest Town	Pokhara
Gross Head	295.76 m
Rated Net Head	278.59 m
Normal Operating Headwater Level	1344.00m amsl
Tailrace water level	1048.24 m amsl
WEIR	
Weir Crest Level	1344.00 m amsl
Type of Weir	Concrete ogee shaped weir
Length of overflow weir	44.00 m
Lowest River Bed Level at Weir Axis	1335.00 m amsl
100 Years Flood	1508 m^3/s
INTAKE	
Catchment Area at Intake site	282.843 km^2
Design Flow	18.00 m^3/s
Riparian Release	1.52 m^3/s
Intake Type	Side intake
Intake orifice number/size	4 Nos./ 3.6 m wide x 2.0 m high
UNDERSLUICE	
No., Dimension (length x breadth)	2 nos., 4.0 x 4.0 m
Under-sluice Invert level at gate	EL 1336.00 m amsl
GRAVEL TRAP	
Length (effective)	20.0 m

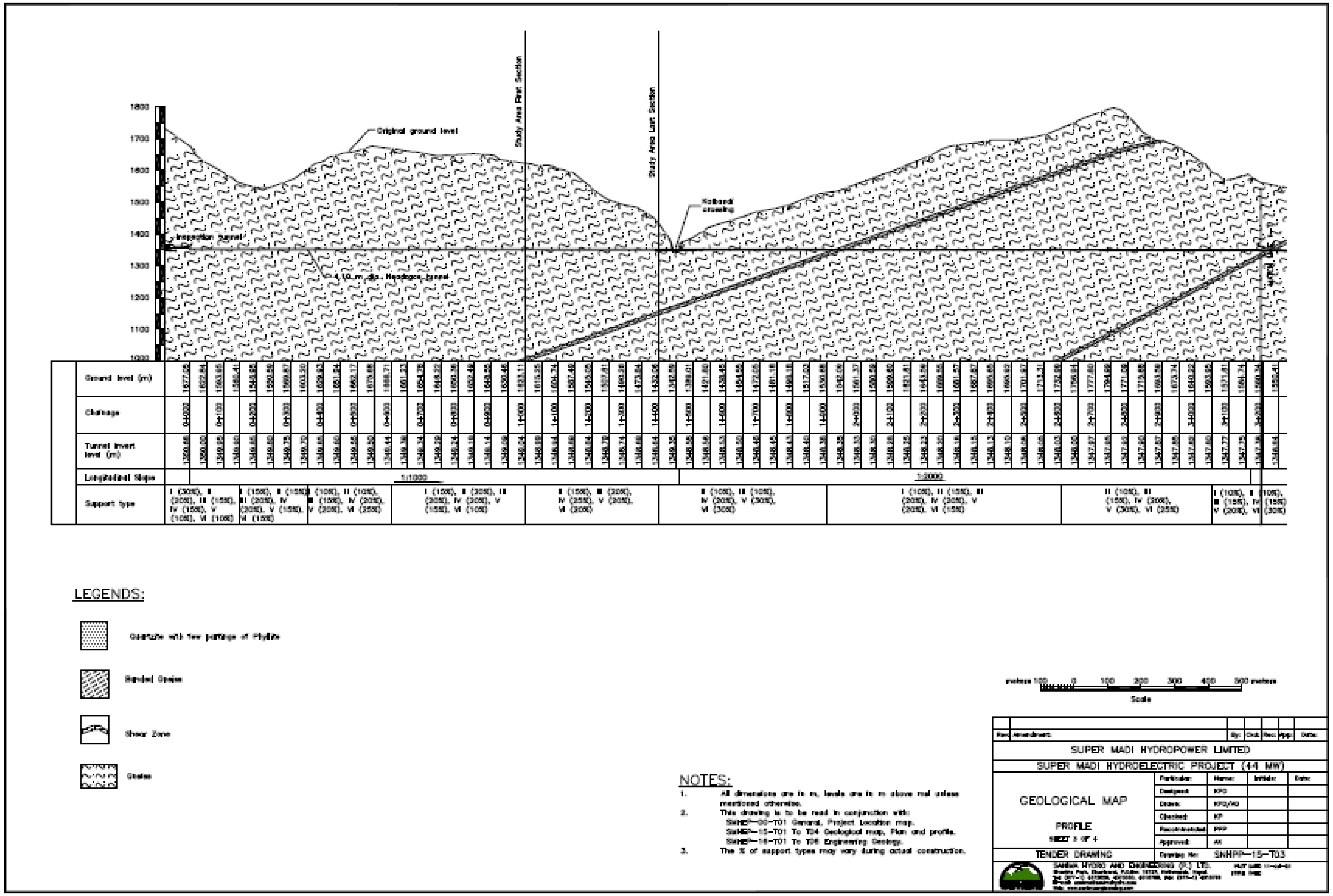
Width Height Particle size to settle	8.0 m 9 m 5 mm (at 85% efficiency)
<u>UNDERGROUND SETTLING BASIN</u> Type Settling Criteria Number of Chambers Inlet Transition Length (m) Settling Basin Size (uniform section) Flushing System Size of Flushing Channel	 Double Chamber, Intermittent Flushing Type 90%, ≥ 0.20 mm 2 28.0 m 124 m long x 12.30 m wide and 13.05 m high each Intermittent 1.00 m x 1.5 m (B x H), slope 1:50
<u>Headrace Tunnel (HRT)</u> Type of Flow Shape of Tunnel Length (excluding settling basin and Kalbandi Kholsi crossing) Excavation Size Support Type Tunnel inlet portal invert level before settling basin (finish) Length of Kalbandi Level crossing Diameter of Pipe in Crossing	 Low Pressure Tunnel Inverted U 5270.95 m 4.2 m x 4.2 m (B x H) Concrete/ Shotcrete/Rockbolt 1337 m amsl 55.50 m 2.7 m
<u>Adit-I (to HRT)</u> Type Total length of Adit (excluding flushing tunnel) Excavation Size Adit portal invert level (excavation)	 Inverted “U” Type, Shotcrete lined 237.16 m 4.2 m x 4.2 m (B x H) 1324.02 m amsl
<u>Surge Shaft</u> Type Diameter - upper section Height - upper section Diameter (excavation) – bottom restricted section	 Cylindrical 8.0 m 45.60 m 4.0 m

Height – bottom restricted section	31.68 m
Total Height	77.28 m
Invert Level	EL 1287.72 m amsl
Penstock Pipe	
Type	Exposed steel lined
Length (from HRT outlet portal)	562 m (center), 90 m (left branch), 66.7 m (right branch)
Diameter/Thickness	2.70 m dia. / 14-40 mm thick, 2.2 m dia 32 mm thick, 1.6 m dia 28 mm thick of E-350 Grade Steel or Equivalent
Adit-II (Bagalethar adit)	
Type	Inverted "U" Type, Shotcrete lined
Total length of Adit	254.36 m
Excavation Size	4.2 m x 4.2 m (B x H)
Adit portal invert level (excavation)	1300 m amsl
Powerhouse	
PH dimension (L X B X H)	46.00 x 16.0 m x 28.5 m
Turbine Axis Level	EI 1045.65 m amsl
PH Access Floor Level	EL 1052.90 m amsl
Tailrace	
Type	Concrete box culvert
Size	3.2 m x 3.0 m (B x H)
Length of Tailrace	195 m
Slope	1 in 500
Invert Level of Tailrace outlet	1045.43 m amsl
Turbine	
Type	Vertical Axis Francis Turbine
Number of Unit	3 (Three)
Rated Capacity per Unit	15.28 MW
Discharge per Unit	6.0 m^3/s
Turbine Axis Elevation	1045.65 m amsl
Tail water level	1048.24 m amsl
Turbine efficiency	93.5 %

Generator	
Type of Generators	Three phase, Synchronous, Brushless
Number of Unit	3 (Three)
Rated Output	17260 kVA
Generation Voltage	11.0 kV
Frequency	50 Hz
Power Factor	0.85
Generator efficiency	97 %
Transformers	
Type	Three Phase, Oil immersed
Number of Units	3
Frequency	50 Hz
Transformer Efficiency	99.0%
Rated capacity per unit	17,500 kVA
Primary (LV Side)	11 kV
Secondary (HV Side)	132 kV
Transmission Line	
Transmission Voltage (kV)	132 kV
Length (km)	10 km Upto Upper Madi Powerhouse (under study)
Type of Circuit	Single Circuit
Line Conductor	ACSR (Bear)
Proposed Interconnection Point	132 kV NEA Substation at Lekhnath via Upper Madi Transmission line(Proposed)
Power and Energy	
Installed Capacity	44 MW
Deemed / Contract Energy	242.65 GWh
Wet Saleable Energy	204.893 GWh
Dry Saleable Energy	37.756 GWh



Geological Map along Headrace Tunnel



Geological Cross-Section along Headrace Tunnel