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ASSESSMENT ON THE POTENTIAL USE OF SHOTCRETE LINED HIGH PRESSURE TUNNEL AT RASUWAGADHI HYDROELECTRIC PROJECT, NEPAL

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Hydropower Development
Submission date: June 2016
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Department of Geology
and Mineral Resources Engineering

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for
Ramesh Kumar KC

ASSESSMENT ON THE POTENTIAL USE OF SHOTCRETE LINED HIGH PRESSURE TUNNEL AT RASUWAGADHI HYDROELECTRIC PROJECT, NEPAL

Background

The Rasuwagadhi Hydroelectric Project is a run-of-river hydroelectric plant located at the upper reach of Trisilai River and is under construction. The project has an installed capacity of 111 MW. The project mainly consists of daily regulating reservoir, three underground settling basins, approximately 4.2 km long headrace tunnel, 160 m deep penstock shaft, surge-shaft, underground powerhouse, 0.56 km long tailrace tunnel.

This MSc-thesis is related to the analysis and evaluation of existing underground works of the project as well as to assess and re-design on the potential use of shotcrete lined pressure tunnel as a replacement of the vertical steel lined penstock shaft. Therefore, the MSc thesis should cover following scope of works:

- Literature review on the planning and design aspects of conventional and unlined/shotcrete lined waterway system
- Describe the existing features of the Rasuwagadhi project
- Evaluate and compare the engineering geological aspects of the lay out plans of the present design
- Purpose a new design alternative with shotcrete lined high pressure tunnel
- Asses on the hydraulic fracturing possibility using both analytical and numerical approaches
- Carry out stability analysis and support optimization for the newly proposed surge shaft
- Discussion on the long term stability of the high pressure tunnel and surge shaft

Relevant computer software packages

Candidate shall use *roc-science package, auto-CAD* and other relevant computer software for the master study.

Background information for the study

- Relevant information about the project such as reports, maps, information and data collected by the candidate.
- The information provided by the professor about rock engineering and hydropower.
- Scientific papers, reports and books related to the Himalayan geology and tunnelling.
- Scientific papers and books related to international tunnelling cases.
- Literatures in rock engineering, rock support principles, rock mechanics and tunnelling.

Mr. Chhatra Basnet will be the co-supervisor of this MSc thesis.

The project work is to start on January 11, 2016 and to be completed by June 10, 2016.

The Norwegian University of Science and Technology (NTNU)
Department of Geology and Mineral Resources Engineering

January 11, 2016

Krishna Panthi

Dr. Krishna K. Panthi
Associate Professor of geological engineering, main supervisor

FOREWORD

This master thesis titled **ASSESSMENT ON THE POTENTIAL USE OF SHOTCRETE LINED HIGH PRESSURE TUNNEL AT RASUWAGADHI HYDROELECTRIC PROJECT, NEPAL** is submitted to the Department of Geology and Mineral Resources Engineering for the requirement to the partial fulfillment of Master of Science in Hydropower Development Program (2014-2016) conducted by Department of Hydraulic and Environmental Engineering, Norwegian University of Science and Technology (NTNU), Trondheim, Norway.

The thesis work mainly focuses on the analysis of hydraulic fracturing in the shotcrete lined pressure tunnel, stability analysis of the pressure tunnel and stability analysis of surge shaft of Rasuwagadhi Hydroelectric Project in Rasuwagadhi, Nepal using two methods Norwegian rule of thumb and Numerical analysis. The thesis work has started in January 2016 and completed in June 2016. This thesis is purely an academic exercise carried out by the candidate and significant outside contributions have been highly acknowledged.

Ramesh Kumar K C

NTNU, Norway

June, 2016

ACKNOWLEDGEMENT

Today, the time has arrived to express my sincere gratitude to each and every person who helped me to the greatest achievement of finishing my Master Thesis. It has been an intense period and the duration of five Months have been one of the best period throughout my Master's degree. During this period, I got to learn so many things and improve my knowledge and expertise in my field of study. Though the thesis work entitles my name, the recognition goes to everybody who helped me directly and indirectly.

Firstly, I would like to express my deepest gratitude to my supervisor, Associate Professor Dr. Krishna Kanta Panthi. Without his supportive guidance, the completion of this task would be impossible. His continuous suggestions, discussion throughout the course of this thesis, motivation and kind behavior have really helped me to work with higher dedication and here come the fruitful result.

Secondly, I would like to thank my Co-supervisor Mr. Chhatra Bahadur Basnet for providing his valuable time and suggestions to me. He helped me with the sample collection and providing the necessary data and information required. He also helped me with my lab works and encourage me continuously to achieve my goal of this task. The motivation I got from him have been very much helpful. Also I am thankful to Mr. Gunnar Vistnes for his support during the lab work.

Similarly, I am grateful to the Department of Hydraulics and Environmental Engineering, Norwegian University of Science and Technology (NTNU) for providing me the privilege to pursue my Master Degree here in Norway. I am also thankful to all the department staffs, faculties, professors and students for creating friendly environment.

Additionally, I am thankful to my parents and all the family members, relatives and friends for the continuous love and support throughout my life. I would like to thank my wife Yashoda Karki, for her love, support and motivation and also creating a suitable environment for my studies. I cannot forget to mention my friends for their support and help in conducting the thesis work

Ramesh Kumar KC

ABSTRACT

In this thesis, Rasuwagahdi Hydroelectric Project located in central region of Nepal, has been taken as the case study. In this project, approximately 4.2 Km long headrace tunnel with 138m long vertical shaft has been designed and being constructed. Rock types along the headrace tunnel alignment are migmatitic gneisses, quartzite, schists, banded gneisses, and weak or shear zones. In this case study whole tunnel alignment is divided into headrace tunnel and shotcrete lined high pressure tunnel. Major part of this thesis is to proposed the pressure tunnel and analyze it against stability and hydraulic fracturing. And another part of the thesis is to design and analyze surge shaft. Shotcrete lined pressure tunnel is proposed from the construction audit two at chainage 1+850m. Surge shaft is proposed at chainage 2+064m. Along the proposed high pressure tunnel and surge shaft location quartzite is the dominant rock type.

In this thesis two different method have been used for the analysis of Hydraulic fracturing. Analytical evaluation has been done with Norwegian rule of thumb. All together 10 section have been selected and analysis has been performed. All sections have factor of safety more than two which suggest tunnel is safe from hydraulic fracturing. Numerical modelling of four selected section has been performed with the help of Phase^2 program. Results of the phase^2 also shows that pressure tunnel is safe against hydraulic fracturing. Stability analysis of the pressure tunnel and surge shaft has been carried out using Numerical modelling. Input parameter are estimated based on geological report of Rasuwagadhi Hydroelectric Project and lab testing of rock sample collected from powerhouse excavation, through Rocdata. Computer program Phase^2 has been used for numerical modelling with generalized Hoek and Brown failure criteria. Analysis of pressure tunnel is performed on 4 selected sections and support are estimated based on the analysis result.

It is concluded that the proposed shotcrete lined high pressure tunnel and surge shaft are stable and safe from hydraulic fracturing. Proposed pressure tunnel is good alternative of present headrace tunnel. It is also safe and economical.

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LIST OF ABBREVIATION

ESR	Excavation Support Ratio
FOS	Factor of Safety
HEP	Hydroelectric Project
GSI	Geological Strength Index
ISRM	International Society of Rock Mechanics
MPa	Mega Pascal
GPa	Giga Pascal
RMR	Rock Mass Rating
RQD	Rock Quality Designation
UCS	Uniaxial Compressive Strength
SRF	Stress Reduction Factor
MCT	Main Central Thrust
MBT	Main Boundary Thrust


CHINA
Project Location
NEPAL
INDIA
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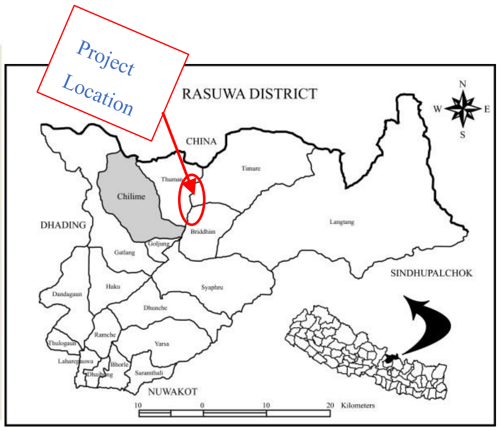
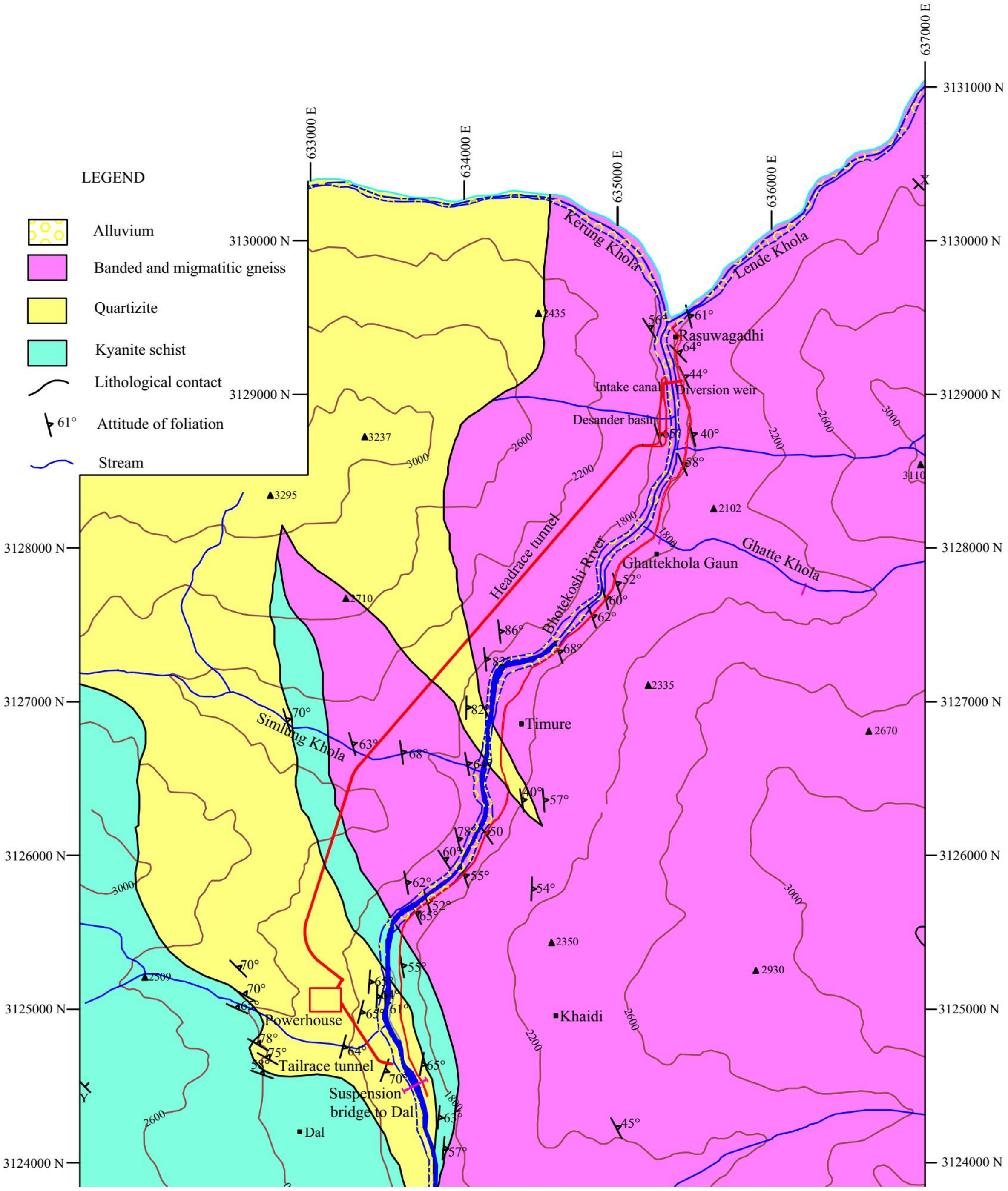

Project Location
RASUWA DISTRICT
CHINA
Chilime
DHADING
SINDHUPALCHOK
NUWAKOT
Kilometers


LEGEND
Alluvium
Banded and migmatitic gneiss
Quartzite
Kyanite schist
Lithological contact
61° Attitude of foliation
Stream
633000 E
634000 E
635000 E
636000 E
637000 E
3131000 N
3130000 N
3129000 N
3128000 N
3127000 N
3126000 N
3125000 N
3124000 N
Kerung Khola
Lende Khola
Rasuwagadhi
Intake canal
Diversion weir
Desander basin
Ghattekhola Gaun
Ghatte Khola
Headrace tunnel
Bhotekoshi River
Simling Khola
Timure
Khaidi
Powerhouse
Tailrace tunnel
Suspension bridge to Dal
Dal

CHAPTER 1

1 Introduction

Energy production of any country reflects economic growth of the country. Hydropower is one of the major source of energy. Combination of steep topography and perennial rivers originating from the high snow covered mountains leads to huge potential of hydropower in Nepal. Nepal has a potential of 83,000 MW of which 43,000 MW is economically viable (Shrestha, 1966). Other studies shows different estimation upto 200,000 MW. Different estimates of capacities are due to the assumption made on different factor and different estimates methodology.

Tunneling through Young Himalayas with very fragile geology of Nepal is a challenging job because of high overburden, vegetation cover and highly varying geology with the presence of shear zone, thrust zone and faults. Difficult mountainous terrain is not favorable for surface structure to convey water from intake to power plant. Water conveyance through tunnel is the favorable solution for this kind of rough terrain. The majority of the tunnel carried out in this region have suffered severe stability problem (Panthi, 2006).

From few years back tunneling activities for hydropower are increased subsequently in Nepal. Development of medium scaled hydropowers are on progress. Some of the plants are under construction like Rasuwagadhi Hydroelectric Project, Upper Tamakoshi Hydroelectric Project and some of them are in operation. Large scale hydropower plants are yet to be developed. Most of the hydropower tunnel constructed till now and under construction are designed and constructed in a conventional approach which is time consuming and expensive. Most of the countries in the world have already gone with the modern design technique and are implementing successfully. From 1950's/60's onwards Norway is constructing unlined high pressure tunnel which is the efficient solution to conventional approach (Edvardsson and Broch, 2002). In this thesis one of the under construction project of Nepal Rasuwagadhi Hydro Electric Project (RHP) will be proposed and analyzed with shotcrete lined pressure tunnel and surge shaft instead of conventional design vertical steel lined shaft.

1.1 Objectives and scope of the work

- Assessments on the engineering geological aspects of the layout plan of present design.
- Purpose a new alternative design with shotcrete lined high pressure tunnel and surge shaft
- Assessment on the stability and support estimation of the alternative design.

- Assessment on the hydraulic fracturing possibility using both analytical and numerical approaches.
- Assessment on the stability and support optimization for the proposed surge shaft.
- Discussion on the long term stability of the high pressure tunnel and surge shaft.

1.2 Methodology of the study

Rasuwagadhi Hydroelectric Project was chosen as a case which is under construction. To meet the objective, the following research methodology is applied in this study.

- Literature review

  Review of relevant information, scientific publication on stability analysis and hydraulic fracturing.
- Data Collection and Lab investigation

  Data consist of geological reports, photographs, project layout plan and profile and other related reports. Sample has been collected from powerhouse site of Rasuwagadhi HEP and tested in IGB Laboratory at NTNU. Some data have been assumed based on the literature available.
- Proposal and analysis of Alternative design

  Based on the layout plan and profile and geological report of the project, shotcrete lined high pressure tunnel has been proposed on the same alignment. Stability and hydraulic fracturing analysis have been done by Norwegian rule of thumb and numerical method. Support estimation is done based on both Q-system and numerical modelling.
- Stability analysis and support estimation of surge shaft

  Based on the hydrological, hydraulic and geological data, surge shaft has been designed. Numerical modelling with is done for stability analysis. Support is calculated from both Q-system and numerical modelling and results have been compared.

1.3 Thesis outline

The analysis and findings of this thesis are presented in altogether 8 chapters. Chapter 1 covers the introduction of the thesis with its objectives. Similarly, chapter 2 covers the general information of the rock mass which is the key of rock mechanics. Chapter 3 explains about general description of the location, features and geology of the case. Findings of the lab investigation and rock mass properties of the project area are detailed in chapter 4. Likewise, chapter 5 covers design part of shotcrete lined pressure tunnel and surge shaft and chapter 6 covers analysis of the shotcrete lined pressure tunnel with stability assessment and support estimation. Additionally, chapter 7 covers the arrangement of surge tank in alternative design with its assessment. Last but not the list chapter 8 covers the conclusion of the thesis and recommendation for the further study.

1.4 Limitation of the study

One of the main problem faced during the study is input parameter estimation. The main source of input data is information gathered from reports of Rasuwagadhi HEP. Many datas were lacking in the report. So, the information was not sufficient to estimate all the required parameters. Hence many literatures such as books, journals, thesis reports and discussions with supervisor and co-supervisor have been used to estimate the remaining parameters that were not found from the project documents. The parameters estimated from literatures or similar reference project may not represent the reality of study case. In addition to input parameter estimation, the difficulty is also with availability of time for the analysis and verification. It would be far better to have at least one field visit to the project site before or during study period in order to see the geological condition and parameter and to test the rock stresses. But because of the time and money constraints, it was not possible to go Nepal from Norway.

CHAPTER 2

2 Engineering properties of rocks and rock masses

Rock is naturally composed of one or many minerals. Some of the rocks are composed with only one mineral where most of rocks are heterogeneous mixture of naturally occurring minerals. Different kind of the minerals have different properties. Each mineral has a unique combination of diagnostic physical features color, surface luster, hardness, density and cleavage (Goodman, 1993). Some of the minerals are very hard like quartz while other are very weak like talc. Most of the properties of rocks are determined by the properties of minerals. In general rock forming minerals have density of 2.6 to 2.8 times that of water.

Rocks may vary enormously in mechanical and physical properties also. To discuss mechanical and physical properties, it is important to distinguish between rock and rock mass. The term "Rock Mass" denotes a large volume of deformed rock, in which yield of the intact material and discontinuities both must occur for overall failure to take places. The difficulty in testing directly a large extent of rock mass makes it difficult to characterize them. The proportion and configuration of discontinuities determines the strength of composite material, there is a pronounced size effect, such that large volumes appear weaker than small volumes (Cundall et al., 2008).

2.1 Rock mass properties

Progress of any underground work and behavior of underground opening is almost dependent on the properties of the intact rock. Inhomogeneity and anisotropy is a distinctive feature of most of the rock types (Nilsen and Thidemann, 1993).

2.1.1 Physical properties

The physical properties of the rock masses are mainly govern by its density, porosity and wave velocity.

Density and porosity

Most of the rocks have density in the range of 2.5-3.2 g/cm3. Young sedimentary rocks are in the lower range and gabbro and amphibolite are in higher range.

Table 2-1: Typical density and porosity (Panthi, 2015)

	Dry (t/m3)	Effective porosity	
Rock type	mean	Typical values	St. dev
Branded gneiss	2.68	0.87	0.10
Micagneiss	2.73	0.76	0.10
Limestone, dolomite and marble	2.82	0.50	0.10
Phyllite	2.78	0.45	0.05
Trondemite	2.70	0.84	0.05
Granite and quartzite	2.65	0.22	0.05
Gabbro and amphybolite	3.15		
Metasandstone	2.65	0.81	0.20
Young sandstone	2.50	30%	

Majority of rocks have very less porosity, often have a porosity of less than 1%. Young sedimentary rocks are more porous having porosity greater than 30%. Limestones reacts with water easily so the porosity of such rocks increases when contact with water in pressure tunnel.

Wave Velocity

Wave velocity test is a non-destructive test. It is measured on core bit which is later be used for Uniaxial Compressive Strength test. Results of the test depend on the degree of saturation of the specimen as shown in Figure 2-1.

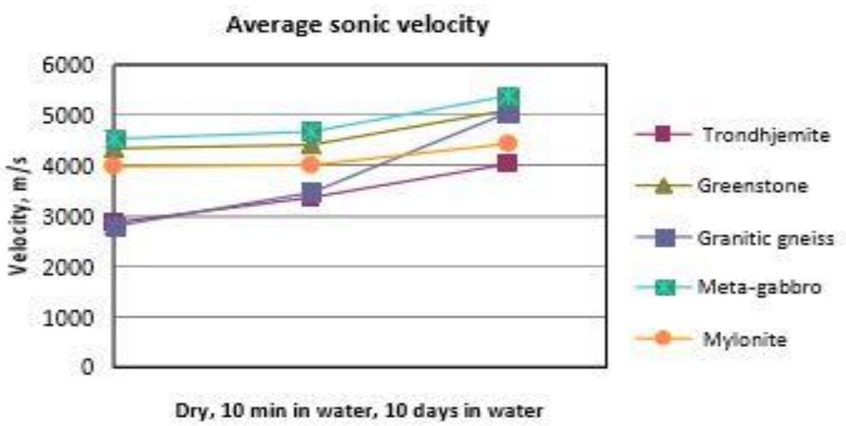


Figure 2-1: Wave velocity as a function of water content (Panthi, 2015)

From the test results it can be said that sonic velocity depends upon the water content in the sample. Sonic velocity increase with water content. It is low in dry state and high in saturated state.

2.1.2 Mechanical Properties

Mechanical properties of the rock masses are mainly govern by its strength.

2.1.2.1 Strength of rock

Ability of rock to sustain high stress and deformation is defined as strength of rock. It is difficult to test directly entire rock mass because it is expensive, time consuming rather than intact rock mass. Smaller specimen is expected to have higher strength than large specimen (Bieniawski and Heerden, 1975).

2.1.2.2 Strength testing methods

The most commonly used method for testing strength of rock are field identification, uniaxial compression, point load and triaxial compression.

Field identification

Strength of the rocks are assumed approximately on the field with very simple tools like geological hammer. Soft rocks are scratched with knife and hard rocks are hit with hammer and with the intensity of scratches in soft rock or intensity of fracture in hard rock strength of rock is approximated. Estimation of rock strength in field is can be done according to ISRM (Barton, 1995), which is in the Appendix D1.

Uniaxial compressive strength

Intact rock specimen of cylindrical shape as in Figure 2-2 prepared from the core drill, has been used to measure uniaxial compressive strength. Generally, the specimen of height to diameter ratio 2.5 are used. Modulus of elasticity is also determined through measurement of axial and lateral strength using electronic strain gauge. The uniaxial compressive strength of the intact rock is used in classification of rock mass and a basic parameter for rock mass strength. Result depends on the nature and composition of rock and the water content in test specimen.

Figure 2-2: Cylindrical shape specimen for Uniaxial Compression Test

The uniaxial compressive strengths of rock having same geological name also varies widely due to its porosity, degree of weathering, and increasing degree of microfissuring (Brady and Brown, 2013). Standard test procedure and interpretation of result for determining the uniaxial compressive strength is given by ISRM (Bieniawski and Bernede, 1979).

Point load test

Point load test gives the tensile strength indirectly by loading the rock specimen between two conical or pointed shaped platens. Test specimen can be cores or irregular lumps of rock. In diametrical point load test strength of the specimen decreases while increase in core diameter. In axial point load test specimen length and specimen diameter both influence the result of the test. Shape and size effect are more severe in lump test than testing specimen of regular geometry (Broch and Franklin, 1972). According to Broach and Franklin, (1972) point load strength index can be calculated as

$$I_s = \frac{P}{D^2} \tag{2.1}$$

Where D is diameter of specimen.

I_s could be correlated with the uniaxial compressive strength of the rock.

According to (Broch and Franklin, 1972) for diameter 50mm

$$\sigma_c = 24 * I_s \tag{2.2}$$

For other diameter, (Bieniawski and Heerden, 1975) suggested a relationship

$$\sigma_c = (14 + 0.175\mathrm{D})I_s \tag{2.3}$$

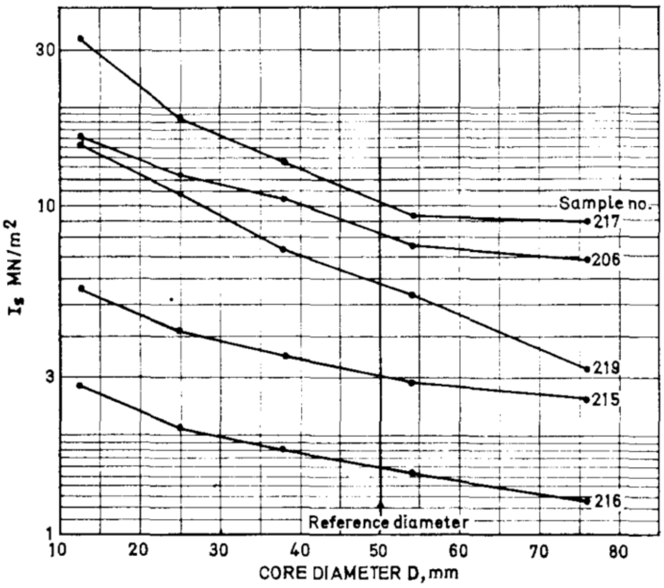


Figure 2-3: size effect in the diametrical point test (Broch and Franklin, 1972)

Figure 2-3 shows the effect of size in which strength value is decreasing when the sample size is increasing. Strength changes more rapidly in smaller diameter but for larger diameter effect of size is much less.

Triaxial compression

This test is carried out on cylindrical specimen prepared in the same manner as those used for uniaxial compression test. The specimen is placed inside a pressure vessel and a fluid pressure is applied on its surface. Axial stress is applied to the specimen through ramp. When the specimen is initially loaded it compresses, but a point is reached soon before the peak of the axial strain- axial stress curve, at which the specimen begin to increase its volume as a result of internal fracture. After the peak strength is reached the net volumetric strain of the specimen became dilation. The amount of dilation decrease with increase in pressure (Brady and Brown, 2013).

2.1.3 Factor affecting rock mass strength

Strength of rock is often influenced by discontinuities, foliation or schistocity planes and the orientation of these features relatively to the direction in which the strength is assessed. An intact rock specimen is usually strong and homogeneous with few discontinuities, and much stronger than the rock mass.

Discontinuities

Discontinuities in rock are named as cracks, fractures, bedding plane, foliation plane, weakness zone etc. Most discontinuities are outcome of movement in the rock mass in geological past. According to Barton, (1978) discontinuity is a general term for any mechanical discontinuity in the rock mass having zero or low tensile strength. According to Panthi, (2006) ten parameter is to be considered for describing any discontinuities characteristics in rock which is illustrated in Figure 2-4

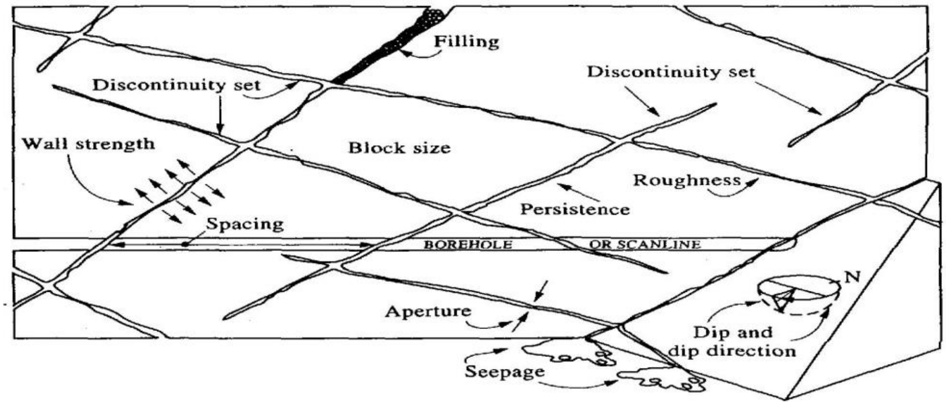


Figure 2-4: Discontinuity characteristics in the rock mass (Panthi, 2006).

Discontinuity roughness may vary from slickenside to very rough. Infilling materials on the discontinuity surfaces may either be gouge material formed as a result of shear movement or material transported by groundwater through open joints in the rock mass. Size of the individual rock block is determined from spacing of discontinuities. Closely spaced discontinuities lead to reduction of the interlocking effect, increase in rock mass permeability and seepage characteristics, which again lead to decrease in cohesion that may result complete rock mass failure due to reveling ground conditions (Panthi, 2006).

When aerial photographs are taken, number of lines can be observed making a parallel pattern. These pattern are similar with joints but are on a larger distance. These lines normally represent weakness zones and faults in the rock. Weakness zones and faults are major discontinuities in the rock mass. They appear as a trenches on the surface and extend down to the bed rock. According to Nilsen and Thidemann, (1993) weakness zones are classified into two main categories:

- Layer of weak rock in a series of sedimentary or metamorphic rocks
- A zone of crushed or altered rock formed by faulting or other tectonic movement.

Weakness zones and faults are weakest part of the rock mass which should be handled very carefully before and during tunneling.

Size effect

Due to the fact that actual rock mass are discontinuous in most case, test conducted on small specimen in the laboratory generally do not give strength and deformation data of the rock which would directly applicable to the rock mass from which specimen is taken. Smaller the specimen fewer the discontinuity is present and hence stronger the specimen (Bieniawski and Heerden, 1975).

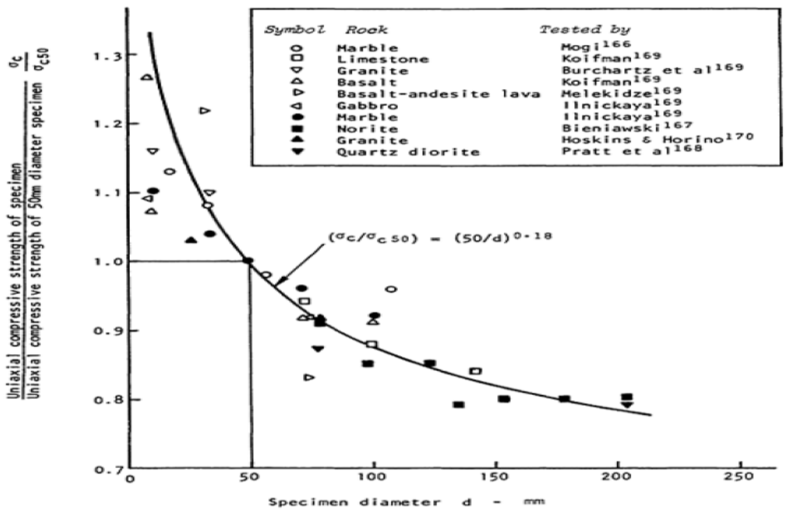


Figure 2-5: Influence of the sample size on the strength of the intact rock (Panthi, 2006)

Figure 2-5 shows the influence of sample size on strength. The reduction in strength is due to the greater opportunity for failure through and around grains, the 'building blocks' of the intact rock, as more and more of these grains are included in the test sample. Eventually, when a sufficiently large number of grains are included in the sample, the strength reaches a constant value (Hoek and Brown, 1997).

Schistocity effect

Compared to common construction materials, anisotropy is also distinctive common feature of many rock types. The term anisotropy is used when the properties of the rock are different in different direction. The degree of anisotropy is normally governed by the content of flaky minerals and mica content (Nilsen and Thidemann, 1993).

The rocks of the Himalaya are highly directional concerning strength and deformability. In many occasions, thin bands of very weak and highly sheared rocks such as slate, phyllite and schists are intercalated within the bands of relatively strong and brittle rocks such as gneiss, quartzite and dolomite. Being weaker in their mechanical characteristics and highly schistose, these weak rocks lack sufficient bonding / friction and have reduced self-supporting capability, and as a result severe stability problems have been faced during tunneling (Panthi, 2006).

2.1.4 Rock mass strength estimation

The rock mass strength is difficult to estimate directly in the field or by laboratory testing. Some authors who have suggested empirical formulae for the estimation of rock mass strength which is presented in the Table 2-2 below. Specimen of 50mm is compulsory for these empirical relationship which are presented in the table below.

Table 2-2: Empirical formula for estimation of rock mass strength (Source:-Panthi, 2006)

Proposed by	Empirical relationship
Bieniawaski (1993)	$\sigma_{cm} = \sigma_{ci} \times exp\left(\frac{RMR - 100}{18.75}\right)$
Hoek et al. (2002)	$\sigma_{cm} = \sigma_{ci} \times \frac{\left(m_b + 4s - a(m_b - 8s)\right)(m_b/4 + s)^{a-1}}{2(1+a)(2+a)}$
Barton (2002)	$\sigma_{cm} = 5\gamma \times Q_c^{1/3} = 5\gamma \left[\frac{\sigma_{ci}}{100} \times Q\right]^{1/3} = 5\gamma \left[\frac{\sigma_{ci}}{100} \times 10^{\frac{RMR-50}{15}}\right]^{1/3}$
Panthi (2006)	$\sigma_{cm} = \frac{\sigma_{ci}{}^{1.5}}{60}$

Where, σ_{cm} is the unconfined compressive strength of rock mass in MPa. σ_{ci} is the uniaxial compressive strength of intact rock in MPa. RMR is the Bieniawaski's rock mass rating and the detail is discussed in section 2.4 and a is the material constant related to Hoek-Brown failure criteria .GSI is the geological strength index, γ is the rock density in t/m^3. Q_c is the normalized rock mass quality rating and Q is the rock mass quality rating. The detail of Q-system is discussed in section 2.4.

$$RMR=15xlogQ+50 \tag{2.4}$$

$$GSI=RMR-5 \tag{2.5}$$

$$\sigma_{cm} = \frac{\sigma_{ci}{}^{1.5}}{60} \tag{2.6}$$

However, in case of availability of Q-value; RMR and GSI value can be calculated using the equations 2.4 and 2.5 proposed by (Barton, 1995)and (Hoek and Diederichs, 2006) respectively. Equation 2.6 defines the best representing power curve established by Panthi (2006)

2.2 Rock stress

The engineering mechanical problem created by underground opening is the prediction of displacement field generated in the opening and surrounding rock by any excavation. The rock in which excavation occurs is stressed by gravitational, tectonic and other forces. Excavating any underground opening is mechanically equivalent to the application of set of distributed forces over the surface generated by excavation. Formation of the opening also induces a set of displacement at the excavation surface. From a knowledge of induced surface forces and displacements it is possible to determine the stresses (Brady and Brown, 2013).

Tunnels passing through areas of high rock cover (overburden) may be subject to instabilities related to induce rock stresses. In relatively unjointed and massive strata, if the rock mass strength is less than the induced stresses the instability may be mainly associated with rock spalling or rock bursting. On the other hand, if the rock mass is weak, schistose, sheared, deformed and thinly foliated/bedded squeezing is the most likely scenario (Panthi, 2012b).

2.2.1 Origin of rock stresses

According to (Nilsen and Thidemann, 1993) virgin rock stress have following components

- Gravitational stresses
- Topographic stresses
- Tectonic stresses
- Residual stresses

Gravitational stresses

Rock stresses originated from the effect of gravity is termed as gravitational stresses. Gravity induces both vertical and horizontal stresses at depth (z) which can be calculated as:

$$\sigma_z = \Upsilon * h \qquad (2.7)$$

$$\sigma_x = \sigma_y = \frac{\nu}{1-\nu} \Upsilon * h \qquad (2.8)$$

Where, σz is vertical stress in MPa and σx and σy are horizontal stresses in MPa, γ is the specific weight in MN/m3, h is the depth in meters and ν is the Poisson's ratio. The horizontal stress induced by gravity alone is only a part of total horizontal stress.

Topographic stresses

When the surface is not horizontal the topography will affect the rock stresses situation. In high valley sides where the hydropower are often located, the stresses situation are totally dominated by topographic effects. In such cases the major principle stress (σ_1) near the surface will be more or less parallel to the slope of the valley, and the minor principle stress (σ_3) will be approximately perpendicular to the slope of the valley (Nilsen and Thidemann, 1993).

Tectonic stresses

Due to the action of plate tectonics, tectonic stress as well as faulting and folding also occur. The horizontal stress induced by tectonic stress is much higher than horizontal stress induced by gravity alone. This is particularly the case of the shallow and moderate depth. Tectonic stresses vary according to the extent of tectonic movement, its movement direction and degree of schistosity and shearing. Orientation of the tectonic stress in the central part of the Himalaya is very close to North-South. Thus, tunnels oriented North-South will have least effect of the tectonic stress across its section. Under such circumstances, the total in-plane horizontal stress in a tunnel at high depth can be well low, resulting to high degree of stress anisotropy (Shrestha, 2014).

2.2.2 Redistribution of rock stresses along the tunnel

After the excavation in the rock mass, stresses which are previously existed in rock mass are distributed and new stresses are induced in the rock along the periphery of the opening.

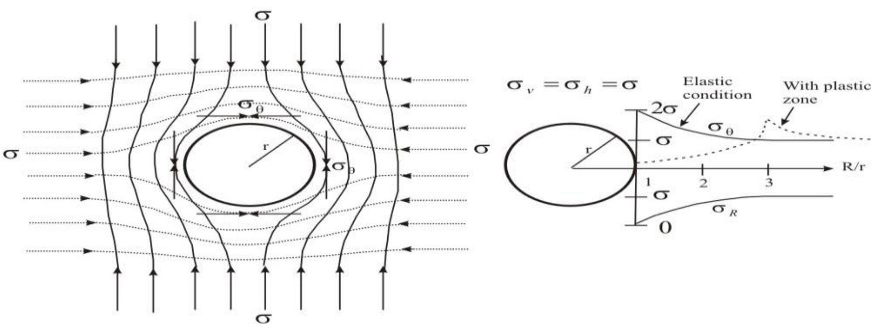


Figure 2-6: Stress trajectories in rock mass surrounding a circular opening (left) and tangential and radial stress distribution in elastic and non-elastic conditions (right) (Panthi, 2006)

As shown in Figure 2-6 if the radius of the opening is r, the tangential stresses ($\sigma\theta$) and the radial stress (σR) at the periphery of a circular opening in fully isostatic stress condition and for elastic rock material will be twice and zero times the isostatic stress respectively. Stresses become normalized as the ratio between radial distance (R) and opening radius (r) increases. (Panthi, 2006). The magnitudes of $\sigma\theta$ and σR are:

$$\sigma\theta = \sigma(1 + \frac{r^2}{R}) \qquad (2.9)$$

$$\sigma R = \sigma(1 - \frac{r^2}{R}) \qquad (2.10)$$

Due to highly anisotropic stress condition the tangential stress will vary around the periphery of a circular opening. For anisotropic condition Kirsch's equation are used for the evaluation of tangential stresses. According to Kirsch the tangential stress will reach its maximum value ($\sigma\theta_{max}$) when the σ_1 direction is a tangent to a contour, and its minimum value ($\sigma\theta_{min}$) when the σ_3 direction is a tangent to a contour with its values:

$$\sigma\theta_{max} = 3\sigma_1 - \sigma_3 \qquad (2.11)$$

$$\sigma\theta_{min} = 3\sigma_3 - \sigma_1 \qquad (2.12)$$

In case of non-symmetric geometry and sharp corner in particular, will affect the magnitude of the tangential stress (Nilsen and Thidemann, 1993)

2.3 Stability problem of tunnel

In a tunnel contour, tangential stress are concentrated normally in two diametrically opposite area. Normally, when the stress problem occur, it is confined to the area of maximum tangential stress. There are mainly two form of stability problem by induced stresses which are:

- Rock burst / spalling
- squeezing

2.3.1 Rock burst

If the compressive tangential stress ($\sigma\theta_{max}$) exceeds the rock mass strength (σ_{cm}) in hard and brittle rock, fracture parallel to the tunnel contour with loud noise is commonly preferred as rock burst. At moderate stress levels the fracturing will result in a loosening of thin slabs, often referred to as rock spalling. When the rock stresses are very high, rock burst may be a major threat to safety if right kind of support is not installed at right moment. Rock burst activity is most intensive at the working face immediately after excavation (Nilsen and Thidemann, 1993).

According to the Norwegian rule of thumb, rock spalling/ rock burst is likely to occur when overburden above the rock exceed 500m. The extent of this type of failure is likely to be severe, even if the tunnel runs parallel to the valley side with a slope angle exceeding 25˚. The rock burst depth-impact reaches its maximum where the maximum tangential stress is concentrated and depends upon the magnitude of the maximum tangential stress, the brittleness characteristics (mineral composition) of the rock and the rock mass strength (Panthi, 2012a).

2.3.2 Squeezing

In soft rock when the strength is less than induced tangential stresses along the tunnel periphery, gradual formation of micro cracks along the schistocity or foliation plane will takes place. As a result, a viscous-plastic zone of micro-fractured rock mass is formed deep into the walls, as shown in Figure 2-7. The induced maximum tangential stresses are moved beyond the plastic zone.

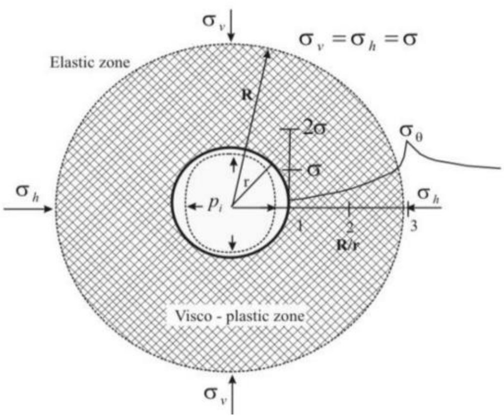


Figure 2-7: An illustration of squeezing in a circular tunnel. In the figure, r is the tunnel radius, R is the radius of visco-plastic zone and pi is the support pressure (Panthi, 2006).

As a result, a time dependent inward movement of rock material will take place and supports in the opening will experience gradual buildup of pressure which is known as squeezing of rock (Panthi, 2006).

2.4 Rock mass classification

Rock mass classification system are empirical tools which are used to classify rock engineering properties and obtain a general rating of rock mass properties. Rock mass classification are used for:

1. Quantitative classification of rock mass quality
2. Estimating rock support based on quality description
3. Use quality rating for predicting (estimating) properties of rock mass
4. Use quality rating in some failure criterion

There are more than one system that are widely use around the world for rock mass quality rating as well as estimating rock support, some of them are presented in Table 2-3.

Table 2-3: Rock mass classification system

Name of Class.	Author & version	Applications	Form and type	Remarks
Rock Load Theory	Terzhagi, 1946	Tunnels with steel support	Descriptive Behavior Functional	Use for modern tunneling
NATM	Rabcewicz, 1964/65 and 1975	Tunneling in incompetent ground	Descriptive Behavior	Tunneling concept Used in squeezing ground conditions
GSI	Hoek, 1994 and Hoek et al 1998	Tunnels, mines and failure criterion	Descriptive Behavior Numerical	Widely used for failure criterion
RMR	Bieniawski, 1974 and 1989	Tunnels, mines, foundations etc.	Descriptive Numerical Functional	Widely used but lacks good support system
Q-System	Barton et at, 1974, Grimstad & Barton, 1993	Tunnels and large caverns	Descriptive Numerical Functional	Popular and widely accepted

In 1946 Terzaghi proposed a simple classification system. It represents a category of classification systems with a limited field of application. It is restricted to steel arch support in tunnel only.

Geological Strength Index (GSI)

Both RMR and Q system are heavily dependent on RQD. But in most of the soft rock, RQD value is zero. The GSI system is based upon an assessment of the lithology, structure and condition of discontinuity surface of the rock mass and it is estimated from visual examination of rock mass exposed in outcrops, in the surface excavations such as road cuts and in tunnel faces and borehole cores. The GSI, by combining the two fundamental parameters of the geological process, the blockiness of the mass and the conditions of discontinuities, respects the main geological constraints that govern a formation and is thus a geologically sound index that is simple to apply in the field. The index is used in conjunction with appropriate values for the unconfined compressive strength of the intact rock (σ_{ci}) and the petrographic constant mi, to calculate the mechanical properties of a rock mass, in particular the compressive strength of the rock mass (σ_{cm}) and its deformation modulus (E) (Marinos et al., 2005).

Rock Mass Rating System (RMR)

Rock mass rating system was developed by Z.T. Bieniawski during 1992-93. Bieniawski suggested that the classification for jointed rock mass should:

1. Identify the most significant parameters influencing the characteristics of a rock mass.
2. Divide a particular rock mass formation into groups of similar behavior.
3. Derive quantitative data required for the real engineering problems.
4. To provide a common basis for communication between engineers and geologists.

In order to fulfill these requirements, Bieniawski (1988)suggested RMR should incorporated the following parameter.

1. Uniaxial compressive strength of rock material.
2. Rock quality designation (RQD).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities

These parameter are evaluated by field measurement and rating is allocated to each range of values for each parameter from the table in Appendix D4 and overall rating is calculated by adding the rating of each of the parameter. Rating adjustment is done for joint orientation as given in the table on Appendix D4. Final value of rating after the adjustment is used to define the rock mass quality and its class.

Q-method

Based on the analysis and evaluation of a large number of cases of histories of underground excavation stability Barton et al. (1974) of the Norwegian Geotechnical Institute proposed Q-system of rock mass classification to determine the quality of rock mass for tunneling. Six parameter are used to determine the rock mass quality in the following ways:

$$Q = \frac{RQD}{J_n} * \frac{J_r}{J_a} * \frac{J_w}{SRF} \qquad (2.13)$$

Where,

RQD is Rock quality designation

J_n is the joint set number

J_r is the joint roughness number

J_a is the joint alteration number

J_w is the joint water reduction factor

SRF is the stress reduction factor

The rock mass description and rating for each of the six parameter is given in the Appendix D5.

The first quotient (RQD/J_n) represents the block size of the rock mass. The second quotient (J_r/J_a) represented the inter-block shear strength which is also the representation of the roughness and frictional characteristics of the joint walls. The third quotient (J_w/SRF) represents the active stress for the discontinuities around the tunnel opening.

The range of possible Q-values (approx. 0.001 to 1000) encompass the wide range of rock mass qualities from heavy squeezing-ground right up to sound unjointed rock. In the original version of Q-system, Barton et al (1974) defined rock mass quality in nine different classes ranging from ‘exceptionally poor’ to ‘exceptionally good’ and the Q-value was correlated with actually applied rock support measures in the tunnels. Grimstad and Barton (1993) modified the Q-system, particularly its support chart and inclusion of squeezing conditions on the SRF rating. The most recent version of the support chart is in appendix D5.

Chapter 3

3 Description of Case

In this thesis, the case study has been carried out for ‘Rasuwagadhi Hydroelectric Project’. This Project is owned by Rasuwagadhi Hydropower Company Limited (RGHPCL). The company is promoted by Chilime Hydropower Company Ltd.

Rasuwagadhi Hydroelectric Project is planned in Rasuwa district , Bagmati Zone of the Central Development Region in Nepal. The project is accessible from Kathmandu -Trisuli - Somdang road up to Syabrubensi (about 130 Km) and then Syabrubesi to headwork site at Rasuwagadhi (about 16 km) northwest of Kathmandu. About 5 km long internal access road will be required to access the headworks, powerhouse, surge tank, tailrace. The proposed project lies between 20°14’ 05” N, 85°21’ 22” E to 28° 16’ 39”N, 82°23’03”E (RHEP, 2011)

Figure 3-1: Location of Rasuwagadhi Hydroelectric Project of Nepal.(Panthi, 2006)

This project is basically a run-of-river type scheme having the capacity of 111 MW with the design discharge of 80 m3/sec and available gross head of 168m. The source river is Bhote Koshi (Trishuli) which flows down from Tibet, China entering Nepal at Rasuwagadhi reaches down to Trishuli in Rasuwa District. The headworks site is located about four hundred meters downstream from the confluence of Kerung khola and Lende khola which are the Boundary Rivers of Nepal and China (Tibet). The Project area lies in the Higher Himalayan succession. The migmatitic gneisses and banded gneisses occupy the intake area whereas quartzite comprise the headrace tunnel and powerhouse area. There is also about 200 m wide zone of grey kyanite schist between the quartzite and banded gneisses.

The average annual precipitation over the entire Bhotekoshi basin is estimated to be about 1000 mm. The long term mean monthly flow in the project area reveals a maximum of 216.40 m3/s in August and minimum 20.60 m3/s in March. The flow duration curve was constructed from the generated mean monthly flow series for the duration of 1994-2003 at the weir site. It is noted that the design flow equivalent to 40% dependable is about 80.00 m3/s.(RHEP, 2011)

3.1 Project layout features

The project is a run-of-river (ROR) type. The gross head and design discharge of the project are estimated at 168m and 80.0 m3/ s respectively. The general project layout is shown Appendix A1. Briefly the project comprises:

- 3 no of side off take intake with dimension W 4.0m x H 4.3m.
- Underground 3 bay settling basin cavern with dimension L 125m x W 15m x H 10m.
- Headrace Tunnel of about 4200m long with Horseshoe, W 6.3m x H 6.3m (unlined section) (Present design).
- Headrace Tunnel of about 1895m long with Horseshoe, W 6.3m x H 6.3m (unlined section) (Alternative design).
- Pressure Tunnel of about 2364.2m long with Horseshoe, W 8.5m x H 8.5m (unlined section) (Alternative design).
- Surge shaft with height 53.75 m and internal diameter16 m, (Present design).
- Surge shaft with height 43.5m and internal diameter16 m, (Alternative design).
- Underground Powerhouse with dimension L 76.3m x W 15.0m x H 39.0m

3.2 Geological Aspects of Case

3.2.1 Regional geology

Broadly, Nepal is divided into five lithologic zones, from north to south which are Tibetan Tethys zone, Higher Himalayan zone, Lesser Himalayan zone, Siwalik zone and Terai zone. (Upreti, 1999).

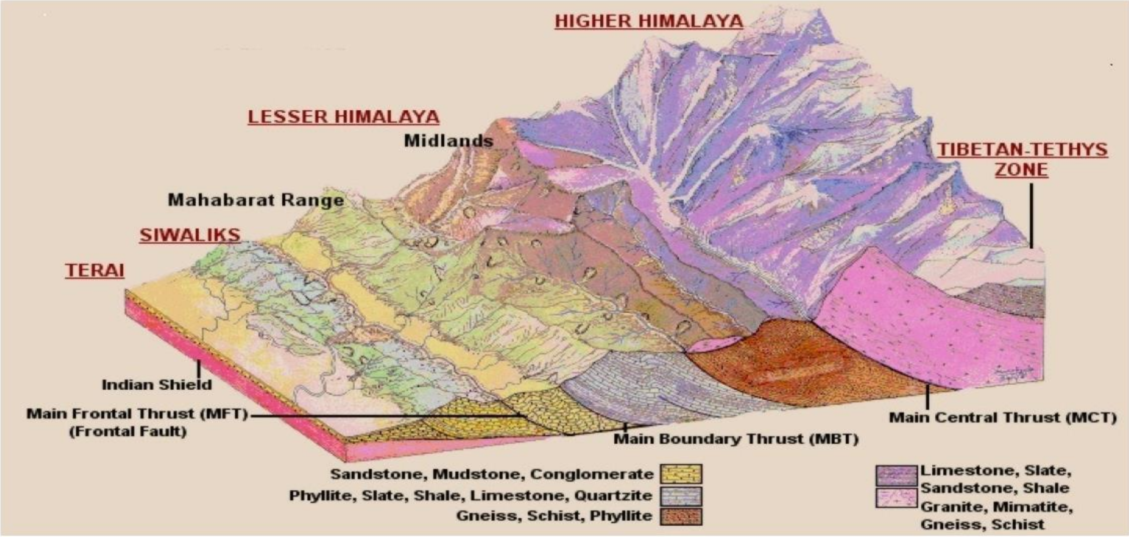


Figure 3-2: Block diagram of the Nepal from North to South giving different litho-tectonic units (Panthi, 2006)

These tectonic zones are characterized by special lithology, geological features and history and consist of different types of rock as shown in Figure 3-2.

The Tibetan Tethys zone

The northernmost tectonic zone of the Himalaya occupies a wide belt consisting of sedimentary rocks known as the Tethyan Sedimentary Series which exposes only occasionally within the territory of Nepal. The Tibetan-Tethys Zone begins at the top of the Higher Himalayan Zone and extends to the north in Tibet. Most of the high peaks including Everest Himalayan peak are composed of the rocks of this zone. This zone is composed of sedimentary rocks such as shale, sandstone, meta-sandstone, phyllite and some crystalline limestone.

The Higher Himalayan Zone

The Higher Himalayan zone include the rocks lying north of the MCT and below the Tibetan-Tethys Zone. This zone consists of an approximately 5-10km thick succession of crystalline rocks. The zone mainly consists of a high-grade metamorphic sequence of various kinds of gneisses. Biotitesillimanite gneiss, garnet-biotite gneiss and quartzite are the dominant rocks in the higher Himalayan zone. According to the geological division of Nepal, the Project area lies in the Higher Himalaya which is occupied by high grade metamorphic rocks.

The Lesser Himalayan Zone

The Lesser Himalayan Zone is bordered in the south by the Main Boundary Thrust (MBT) and in the north by the Main Central Thrust (MCT).According to (Stöcklin, 1980) some of these rocks are practically unmetamorphosed, but most are converted by low grade metamorphism to slates, phyllites, quartzites and finely crystalline limestones. This zone can be distinguish into narrow outer (southern) and a broad inner or Midland sedimentary belt. Tectonic deformation is intense in the outer belt, where tight folds with steep and often disturb the stratigraphic sequences. Undisturbed sequences are more readily found in inner belt. The MBT itself is a fault zone that has brought older Lesser Himalayan rocks over the Siwalik.

Siwalik Zone

The Siwalik is bounded in the north by the Main Boundary Thrust (MBT) and in the south by the Main Frontal Thrust (MFT). It consists basically fluvial deposits of the Neogene age. Lower Siwalik consists of finely laminated sandstone, siltstone and mudstone. Likewise, the middle Siwalik is comprised of medium to coarse grained salt and pepper type sandstones and the Upper Siwalik is consist of conglomerates and boulder beds. The dun valleys within the Siwalik consist of Quaternary fluvial sediments.

The Terai Zone

The Terai Zone represents the northern edge of the Indo-Gangetic alluvial plain and is the southernmost tectonic division of Nepal. Though physio graphically this zone does not belong to the main part of the Himalayas, it is a foreland basin and owes its origin to the rise of the Himalayas, it is thus genetically related. To the north, this zone is often delineated by an active fault, the main Frontal Thrust (MFT). The Siwalik rocks are found to rest over the sediments of the Terai in many places along this thrust. Terai is covered by Pleistocene to recent alluvium with average thickness about 1500 m. The basement topography of Terai below the alluvium is not uniform

3.2.2 Geology and engineering geology of the project area

According to the geological report of Rasuwagadhi HEP (RHEP, 2011), Project area comprises of three rock groups namely quartzite, banded and migmatitic gneisses alternating with few band of kyanite schist. Migmatitic and banded gneiss is present in intake area where as quartzite comprises the major part of the Project area. Most of the tunnel alignment and location of power house are located in quartzite. Small band (about 200m) of kyanite schist lie in-between quartzite and banded gneiss. Kyanite, garnet, and biotite are main metamorphic index minerals of the area. These rocks of amphibolite facies show relatively moderate deformation. None of the samples show high deformation and high visual strain. Based on the surface rock mass classification, the head race tunnel runs through about 15% in poor to very poor rock and about 85% in fair to good rock.

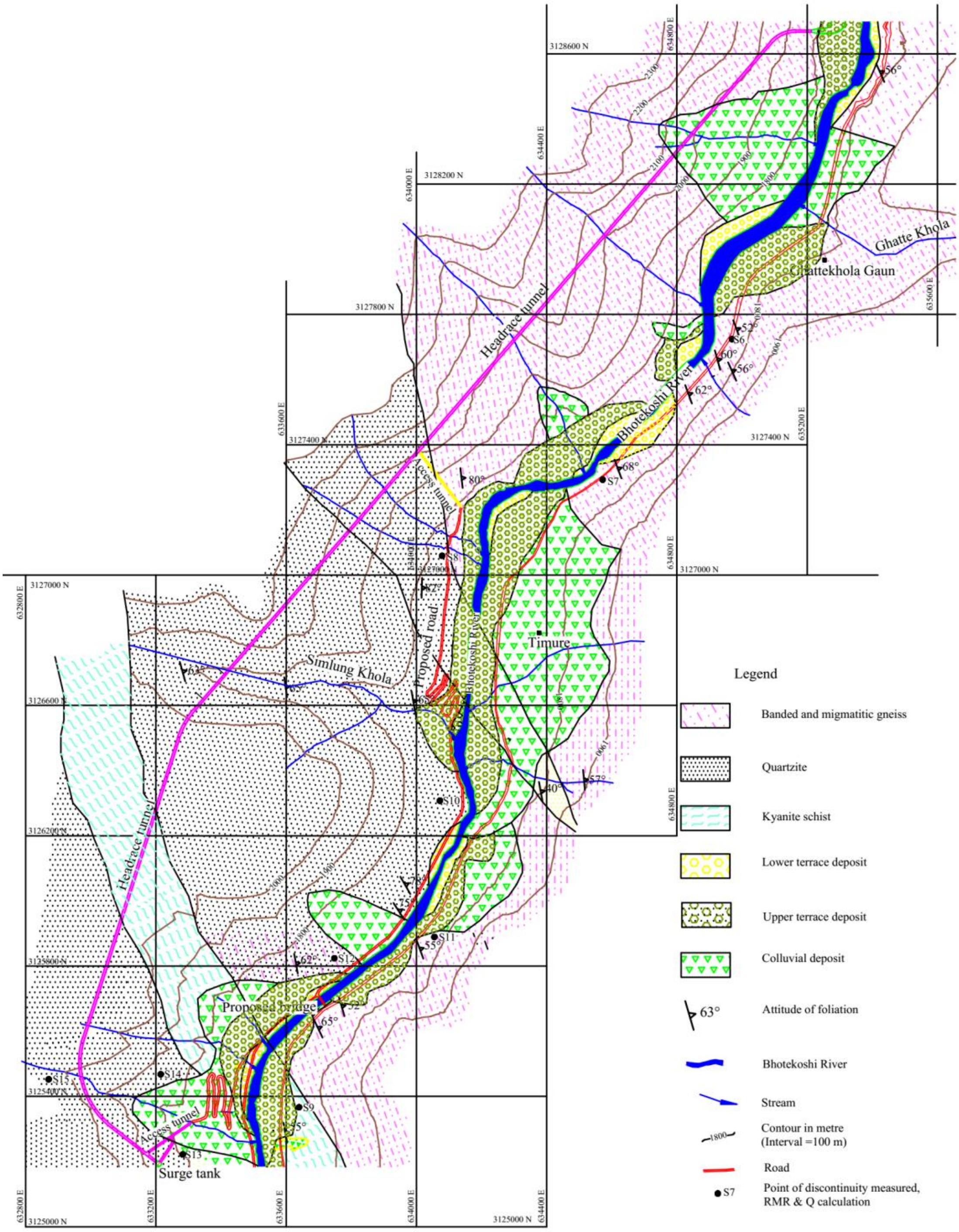


Figure 3-3: Geological map of project area (RHEP, 2011)

The well-known thrust zone namely the main central thrust (MCT) is located 8 Km south from the project area as shown in Figure 3-3.

Headworks Area

Bhotekoshi River at weir site is a symmetrical ''U' shaped valley more or less equal hill slope on the both bank which is favorable for weir site. The diversion weir is located at a straight course of the Bhote Koshi River about 500 m downstream from the confluence of the Lende

Khola and Kerung Khola. The intake canal is located on the right bank of the Bhote Koshi River. Three jointsets are present in the headworks area. Joints area generally tight to 1 mm open, rough surface to smooth surface with unaltered to slightly weathered wall. Main joint set around the headworks area is foliation plane. Hillslope of the headworks site is 65̊ to 70̊.

Dominant rock of the headrace tunnel inlet portal area is migmatitic gneisses .They are grey, medium-grained, slightly weathered, massive to blocky, and very strong. The RMR values in the vicinity of intake range from 65 and 72 (Good Rock, Class II).

Desander

The underground desander with three separate chambers is designed inside rock hill on the right bank of Bhotekoshi River at 400 m downstream from confluence of Lende and Kerung Khola. The axis of desander cavern is 242°. The length of each basin is 125 m with equal width 15 m. with 30 m side cover. The hill slope of the site ranges from 65° to 70°. The rock mass is grey, medium-grained, slightly weathered, massive to blocky and very strong. The RQD of rock mass on the portal area ranges from 61% to 92%. Similarly, the RMR and Q-value are 62 and 11.33 (Class II) respectively. The major joint around the desander area is foliation which oblique to the desander axis.

Tunnel

The present headrace tunnel alignment and alternative alignment (Appendix A2) passes through the migmatitic gneisses, quartzite, schists, banded gneisses, and weak or shear zones. The tunnel alignment will be oriented obliquely to the foliation plane. Hence, the tunnel driving conditions are fair. The tunnel passes through the rough hill on the right bank of the Bhotekoshi River. The rock dips upstream throughout the tunnel length. On the basis of surface rock mass classification, about 15% of the head race tunnel length runs through poor to extremely poor rock and 85% through fair to good rock. It is important to note that extremely poor conditions are encountered only at the minor shear zones of less than 10m in length at different chainage. Whereas, fair to poor conditions are confined to the schist. The orientation (strike) of foliation in the headrace tunnel varies from 318° to 352° with dips ranging between 50° and 70° due NE. Generally there are other two to three prominent joint sets of attitude 040°- 060°/75°-86° SE and 100°-135°/40°-55° SW.

A horse shoe shaped 6.3m x 6.3m size headrace tunnel is proposed with 4203m long in the present design. In alternative design horse shoe shaped 6.3m x 6.3m size head race tunnel upto 1850m is same as present design. After the chainage 1850m straight headrace tunnel is replaced by inclined pressure tunnel with 6.35% slope with horse shoe shaped 8.5m x 8.5m pressure tunnel. Summary of the tunnel description along alignment is presented in Table 3-1.

Table 3-1: Summary of rock description of Tunnel alignment.

Chainage	Rock Type	RQD	RMR	Ground Water	Description
0+000 - 1+367	grey to light grey, medium-grained, massive, very strong migmatitic gneisses with dykes of leucogranite	50 - 75	54 - 66	Dry to damp on the surface	Fair to Good Rock (Class, Class II &III)
1+367 - 2+194	light grey, pale yellow to white, fine-grained, massive to blocky, very strong quartzite with schist partings and thin bands	75 - 90	65 - 70	Dry to damp on the surface	Good rock (Class II)
2+194 - 2+556	Grey to light grey, medium-grained banded gneiss with schist partings and thin bands.	50 -75	42 - 63	Damp	Fair to Good Rock (Class, Class II &III)
2+556 - 2+687	Grey to green-grey, medium grained garnet-kyanite schist.	25 - 75	50 - 60	Damp	Fair Rock,(Class III)
2+687 - 4+203	pale yellow to grey, fine- to medium-grained, blocky quartzite with schist partings	50 - 75	47 - 63	Damp	Fair to Good Rock, (Class III -II)

Powerhouse

Geologically the powerhouse site falls on the quartzite. The main rock type of the powerhouse area is thick to very thick-banded, coarse-grained, light grey to grey and white, massive quartzite with schist partings. There are a number of quartz veins intersecting the rocks. Four discontinuity sets are present in the area.

By surface investigation, there is no structural disturbance as fault, folds, and shear zone. The unadjusted RMR values in the vicinity of powerhouse range from 70 to 71 whereas the adjusted RMR values lie between 58 and 59 (Fair Rock, Class III) The quality of rock on powerhouse area is fair to good (class II-III). There are planar failures on foliation plane and toe cutting by the Bhotekoshi River is active in the area.

Surge Shaft

The purposed surge shaft in an alternative design is underground type having 16 m in diameter and 43.5m high and is connected to the Pressure tunnel. The surge shaft is located after the pressure tunnel cross the small kholsi, where there can be a possibility of water leakage. It is located at chainage 2+064m.

The proposed surge shaft lies on the quartzite zone. The quartzite is strong, dark grey to white, medium- to coarse-grained and slightly weathered having three to four sets of joints. The joints are close to widely spaced, low to high persistence, very tight to moderately open with rough to smooth surface and silt as the infilling material. The RQD is 61 % and Q-value varies in between 2.86, 5.29 and 7.15. Hence, the quality of the rock is fair according to both RMR and Q system.

Tailrace Tunnel

The proposed tailrace tunnel in a present design is inverted horse shoe shape with diameter 6 m and 580 m long directed on the NW-SW direction. The outlet is at 20 m upstream from the suspension bridge to Dal.

The tailrace tunnel passes through quartzite. The quartzite is dark grey, very strong, medium- to thick-banded, and medium- to coarse-grained and slightly weathered having three to four sets of joints. Joints are close to wide spaced, low to medium persistence, very tight to open with smooth to rough surface and silt as infilling material. The RQD of quartzite ranges from 37% to 60%, RMR and Q ranges from 19 to 61 and 056 to 8.33 respectively which represent the rock is poor to good i.e. Class II to IV.

CHAPTER 4

4 Rock Mass Properties of the case

According to engineering geological investigation report of the case prepared by RHEP (2011) migmatitic gneisses are at headwork site. Headrace tunnel passes through migmatitic gneisses, quartzite, schists, banded gneisses, and weak or shear zones are present along the tunnel alignment.

Figure 4-1: Sample collected from field for lab investigation

Most of the tunnel alignment passes through the quartzite zone. Quartzite rocks is at the power house area also. It was difficult to collect and carry all type of rock samples along the tunnel alignment to Norway so only one type of rock sample as in Figure 4-1 was collected and brought to Norway from inside the power house area which is presented in most of the tunnel alignment as well as powerhouse.

4.1 Lab investigation of Sample

Sample collected as Figure 4-1 from tunnel was tested in laboratory at Department of Geology and Mineral Resources Engineering. Mineralogical analysis, uniaxial compressive strength, sonic velocity test, Brazilian tensile strength test were done which are described detail below.

Table 4-1: Summary of Lab test results.

Description	Value	Unit
Specific Weight	2.65	gm/cc
UCS	172.9	MPa
Modulus of Elasticity	53.626	GPa
Poisson's ratio	0.248	
Tensile Strength	17.35	MPa

All of the test were conducted according to ISRM standards. Result of the lab test is shown in Table 4-1.

4.1.1 Sample Preparation

Due to the limitation of size of the rock sample, it was difficult to get all core of 50mm standard size for UCS test. Distinct foliation plane was not seen in the rock sample. It was tried to drill the sample perpendicular to the foliation plane. Rock sample was drilled with drill bit of 35mm, 40mm and 50mm as shown in Figure 4-2. 35mm and 40mm cores are used to determine Uniaxial Compressive Strength (UCS). 50mm core were cut into pieces in a length of 25mm and used for Brazilian tensile strength test.

Figure 4-2: Samples prepare for UCS test (above) and Brazilian test (below)

Six cores were drilled for UCS test. Out of six cores five were drilled with 35mm drill bit and one was drilled with 40mm drill bit. These core were first used for sonic velocity test and to find specific weight and finally for UCS test. Eight discs of 50mm diameter and 25mm length were also drilled and prepared as per ISRM standard for Brazilian tensile strength test.

4.1.2 Mineralogical analysis

The specimen was tested for mineralogical analysis in IGB Laboratory at NTNU. The mineralogical analysis of rock shows 68.02% of quartz. Detail result of analysis is shown in Table 4-2.

Table 4-2: Mineralogical description of sample rock.

Description	Percentage
Quartz	68.02%
Plagioclase	16.26%
Mica	6.96%
Pykoxene	<1%
Alkali Feldspars	7.06%
Chlorite	<1%

The result of analysis shows that Quartz contain is dominated in the rock. According to geological report of RHEP (2011) Petrological analysis shows that in Quartzite rock contain of quartz is more than 90%.

4.1.3 Uniaxial compressive strength (UCS)

According to ISRM 1979 part 2 (Bieniawski and Bernede, 1979), UCS is intended to determine stress strain curves, Young's modulus (E) and Poisson's ratio in uniaxial compression (UCS) of a rock specimen of regular geometry. The test is mainly intended for classification and characterization of intact rock mass. Six core samples prepared as described in section 4.1.1 are used for UCS test. Size of the drilled cores are smaller than standard size and according to ISRM size correction should be done. But no size correction is needed in the modern testing machine. The core was covered completely by rubber material so that the broken pieces of sample after failure will not scatter away. Preparation for test is shown in Figure 4-3 below

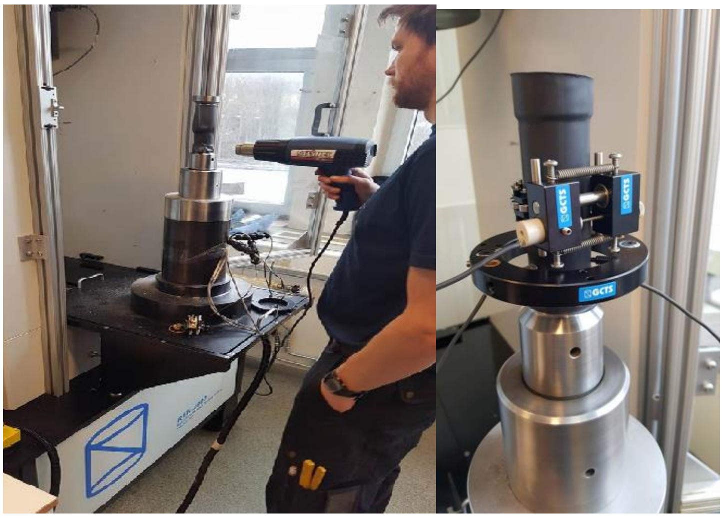


Figure 4-3: Sample preparation for UCS test (IGB Laboratory at NTNU)

Six samples were tested separately in a saturated state. Among the six samples five samples were failed at a middle with a failure angle of around 28 degree.

Figure 4-4: Core sample after UCS test. Left is sample no. 3 and right is sample no. 4

One of the samples, sample number three as shown in Figure 4-4 (left) is failed in a weakness zone on the side before reaching failure stress. So it is discarded in calculation of UCS value. UCS value is calculated from five samples only.

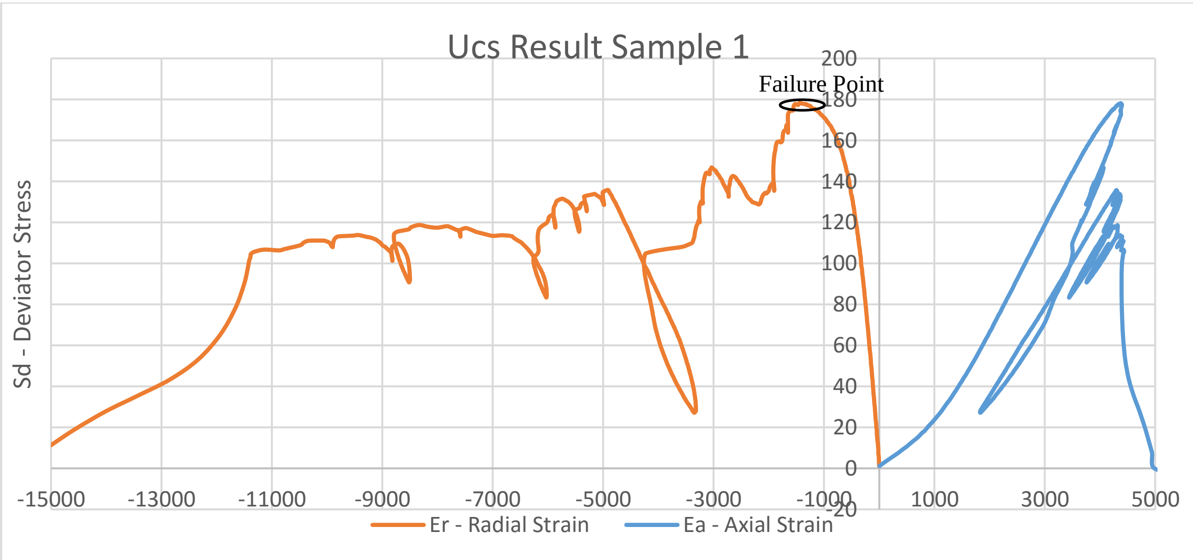


Figure 4-5: stress-strain curve of test sample one.

Young's modulus of elasticity and Poisson's ratio both have been found from the UCS testing machine. Young's modulus of elasticity is the slope of the tangent on the stress-strain curve. Tangent point is taken exactly half of the UCS value and the slope of that point is the young's modulus of elasticity of that sample.

*Table 4-3: Summary of UCS test result. * Not included in calculation*

Sample	UCS (MPa)	E-modulus (GPa)	Poisson's Ratio
R1	178.2	53.02	0.21
R2	173.4	52.29	0.26
R3	130.5*	58.52	0.29
R4	166.4	52.87	0.28
R5	172.9	54.96	0.23
R6	173.6	54.99	0.26
Average	**172.9**	**53.626**	**0.246**

Poisson's ratio is calculated by software itself. Table 4-3 shows the value of both Young's modulus of elasticity and poison's ratio. Average value of Young's Modulus of elasticity and Poisson's ratio found from these six samples are used for further calculation.

4.1.4 Sonic velocity Test

Sonic velocity test is intended as a method to determine the velocity of propagation of elastic wave in rock samples. Six samples which are used for UCS test have been used to determine sonic velocity. These samples are tested in three states. At first they were tested in room temperature just after the preparation of sample. After that they are kept in a drying oven and are tested in dry condition. Then they are submerged in water for saturation. After complete saturation they have been tested in the same way according to ISRM standard.

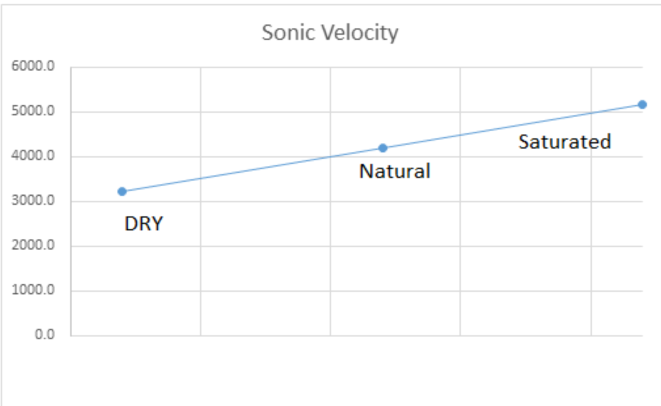


Figure 4-6: Result of sonic velocity test of sample one

These three different states of samples gave different values of sonic velocity. It was low in dry state and high in saturated state. Figure 4-6 shows the result of sample one.

4.1.5 Brazilian tensile strength test

Brazilian tensile strength test was conducted according to ISRM (Franklin, 1985). Rock specimens in the form of core disc prepared in the laboratory as described in section 4.1.1 have been used for this test. Eight samples as shown in Figure 4-2 (below) of diameter approximately 50mm and length approximately 25mm have been tested in saturated condition. All samples have been broken by application of concentrated load through a pair of spherically truncated, conical platens. Detail results of the testing is presented in Appendix A2 and the summary of the result is presented in .

Table 4-4.

Table 4-4: Summary table of Brazilian tensile test

Sample	Diameter (mm)	Width (mm)	Load (KN)	Tensile Strength (MKN/m^2)
D1	49.74	25.35	34.02	17.18
D2	49.62	28.18	39.36	17.92
D3	49.66	25.21	36.22	18.42
D4	49.67	25.41	32.84	16.56
D5	49.63	25.28	36.37	18.45
D6	49.78	25.41	34.77	17.50
D7	49.69	25.4	31.97	16.13
D8	49.64	25.29	32.82	16.64
AVERAGE				**17.35**

Load at which sample is broken is divided by the area of the sample gives the value of tensile strength with the equation 4.1. Core sample was approximately 50mm diameter so according to ISRM, size correction is not needed.

$$\sigma = \frac{2P}{\pi \, D \, W} \qquad (4.1)$$

Where,

σ = Tensile Strength

P = Breaking load

D = Diameter of the sample

W = Width of the sample

4.2 Rock Mass Classification

Both RMR system and Barton's Q systems have been used for rock mass classification along the head race tunnel alignment shown in Table 4-5. Bieniawski's RMR system is based on a combination of six parameters (eq. 4-2). Each parameter is expressed in a point rating and final RMR ranges between 0 (very poor rock for tunneling) to 100 (very good rock for tunneling).

$$RMR = (IRS + RQD + spacing + condition + groundwater) + reduction\ factor \qquad (4.2)$$

Where,

IRS = Intact rock strength

RQD = Rock Quality Designation

Spacing = discontinuity spacing of one set

Condition = expression for condition of (shear strength)

Groundwater = expression for groundwater inflow (pressure)

Reduction Factor = depending on orientation of engineering structure to the main discontinuity set

Table 4-5: Rock mass classification along the alignment

SN	Chainage	Length (m)	RMR value	Q-Value	Rock mass Classification	Description
1	0 + 000 - 1 + 400	1400	57	11	II - III according to RMR and II - IV according to Q	Fair - good
			62	9.86		
			61	3.29		
			48	5.07		
2	1 + 400 - 2 + 150	750	59	5.29	III - IV according to RMR and II - IV according to Q	Poor-Fair-good
			60	7.15		
			42	2.86		
3	2+ 150 - 2 + 550	500	55	10.67	III - IV according to RMR and II - IV according to Q	Poor-Fair-good
			46	3.96		
			61	6.8		
4	2+ 550 - 2 + 650	100	34	4.033	II - III according to RMR and according to Q	Poor - Fair
			42	3.9		
			38	2.11		
5	2 + 650 - 4 + 203	1553	51	7.15	II - III according to RMR and II - IV according to Q	Poor-Fair-good
			52	4.78		
			62	3.96		
			53	1.68		
			46	6.33		
			66	13.65		

Q - system gives the quality of the rock mass in the parameter called Q-value. The Q-value is determined with Equation 2.13. The first term, RQD divided by J_n is related to the size of the intact rock blocks in the rock mass. Similarly the second term, J_r divided by J_a is related to the shear strength along the discontinuity planes and the third term, J_w divided by SRF is related to the stress environment for the discontinuities around the tunnel opening.

The discontinuity survey has been carried out along and across the head race tunnel alignment by NEA. The surface mapping shows that except a small stretch near the intake portal, rest of the headrace tunnel length passes through the bedrock. The rock mass classification using Bieniawski's "RMR" system and Barton's "Q" system shows that the RMR value ranges from 19 to 66 and Q-value ranges between 0.58 and 13.65.

4.3 Weakness zone

According to the Geological report of RHEP (2011), few weak or shear zones parallel to the foliation plane are delineated in the project area in the Simlung Khola area. These weak zones are characterized mainly by alternating of closely spaced (1–8 cm) fractured gneiss and schist with occasionally 5 to 50 cm thick fault gouges. The thickness of weak zones varies from 10 m to 20 m. Apart from the above major weak zones, 10–50 cm thick weak or shear zones along the foliation and 1 to 2 m thick steeply dipping faults along joint planes are common throughout the area. The total effected length of shear or weak zones along the headrace tunnel alignment is about 3%. These zones may create over break and rock squeezing problems during tunnelling.

Around 1000m upstream of confluence of Simlung Khola and Bhote Koshi river there is hot spring and the tunnel passes below the Simlung Khola. Another hot spring is situated near the power house area downstream of Timure. These two hot springs may come out from the faults or joints. There could be a chances of tunnel encountering weakness zone. If hot springs are encountered along the tunnel alignment, the stretch should be properly supported and the water should be drained out completely with drainage facility.

4.4 Insitu Rock stresses

Theoretical background of the rock stresses is already discussed in section 0. Gravitational stress and tectonic stress plays a vital role in the stability of tunnel as well as surge shaft in this case study.

Gravitational stress

Gravity induce both vertical stress and horizontal stress. Major principle stress is represented by gravity induced vertical stress alone. Vertical stress is calculated from the product of rock cover (h) and specific weight (ϒ) from the equation 2.7. Horizontal stress induced by gravity is part of vertical stress and calculated from the equation 2.8 Summary of the stress in shown in Table 4-6.

Table 4-6: Gravity induced vertical and horizontal stress

Chainage (m)	Rock Cover (m)	Vertical Stress (σv) Mpa	Horizontal Stress (σh) Mpa
1+819	146.55	3.89	1.28
2+368	262.88	6.98	2.30
3+556	330.12	8.76	2.89
4+064	343.75	9.12	3.01

Tectonic Stress

Undergoing movement of Indian plates and Tibetan plate is the reason behind Himalaya being most active seismic zone. Continuous plate subduction process in this Himalayan region is causing all kind of small to large scale earthquake. The compressional tectonic deformation and active reverse faulting mechanism have considerable influence on the tectonic stress in the Himalaya.

The estimation of tectonic stress at the particular site needs stress measurement data. It is very difficult to find the measured data in Himalayan region especially in Nepal. Rasuwagadhi HEP is located in the higher Himalayan zone so according to Shrestha (2014) the orientation of the tectonic stress is very close to North-South. For this case study 8˚NE is chosen as the orientation of tectonic stress.

Total horizontal stress (σ_H) is summation of gravity induced horizontal stress (σ_h) and tectonic induced horizontal stress (σ_{tec}). Magnitude of the tectonic stress of the Rasuwagadhi HEP was not available so it is taken as 7.5 Mpa from Parbati II hydro-electric project (Panthi, 2012a). Both Parbati II hydro-electric project and Rasuwagadhi HEP lie in higher Himalayan rock formation. Both projects are structurally bounded by a major fault system in the Himalayas which is Main Central Thrust (MCT). They consist of same quartzite rock as a main rock in the project.

Table 4-7: Calculated value of tectonic stress in plane and out of plane

Chainage	Bearing of tunnel alignment	Direction of tectonic stress	plane angle between Tunnel alignment and tectonic stress	Tectonic stress (Mpa)	
m	degree	degree	degree	In plane	Out plane
1+819.6	30	8	22	6.45	1.05
2+368.56	41	8	33	5.28	2.22
3+556.6	21	8	13	7.12	0.38
4+064.31	20	8	28	5.85	1.65

Tectonic stress in the tunnel section are described in section 6.2.1as input values. Calculated value of tectonic stress in plane and out of plane are shown in Table 4-7. Tectonic stress in plane is calculated with equation 6.1 and out plane is calculated with equation 6.2 below. Bearing of the tunnel differ within the alignment along the section and it is tabulated in table above.

Chapter 5

5 Assessment of the case

5.1 Present design

Figure 5.1 shows the layout profile of the present design of Rasuwagadhi HEP. Detail features of the project is described in section 3.1. Headrace race tunnel is 4.2 km long with horse shoe shape of 6.3m x 6.3m. The geology of the alignment is discussed in section 3.2.2. Mostly quartzite rock is present along the alignment. The location of power house is also in quartzite. Quartzite is generally classified as typical hard and massive rock with high strength. The laboratory testing of the rock gives UCS value of 172.9 Mpa (section 4.1.3.). Surge shaft is located at the end of the headrace tunnel. 162m vertical shaft is designed to convey water from the end of the headrace tunnel to powerhouse.

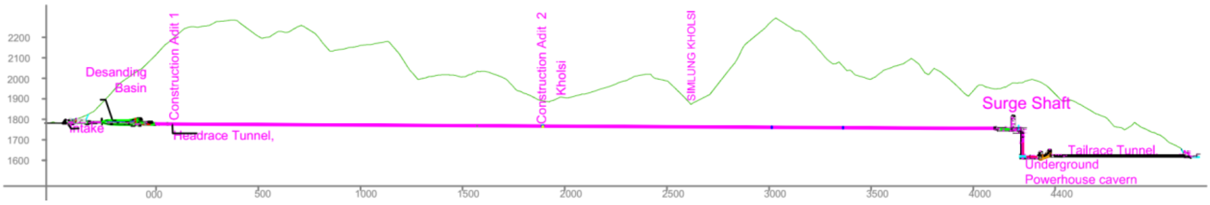


Figure 5-1: Present layout of Rasuwagadhi HEP.

According to Broch (1982) the main risk for unlined pressure tunnels and shafts is bad rock masses like weathered zones, joints etc. Hence the best way of avoiding leakages is to place the tunnel as deep into the rock mass as possible. Almost horizontal headrace tunnel with long vertical shaft is conventional design for hydropower plants. In Norway after 1960's horizontal headrace tunnel is converted to inclined pressure tunnel to the powerhouse which excludes the construction of vertical shaft. Most of the hydropower tunnels are constructed in steep topography and are not easy to make road upto the top of the hill which also increase the construction cost. Generally, steel lining is done in the vertical shaft which is very costly. So the present layout is analyzed with pressure tunnel after the construction audit 2.

5.2 Alternative design

Alternative layout of tunnel is shown in figure 5.2. The headrace tunnel after construction audit 2 (chainage 1+850m) is replaced by shotcrete lined pressure tunnel and vertical steel lined shaft is avoided. The alignment of the tunnel is kept as same as present design. The location of the power house is kept as the present design. The slope of the shotcrete lined pressure tunnel is

6.35% downward from construction audit 2 to power house. It is analyzed on the concept that insitu minor principle stress along the tunnel should be higher than the internal water pressure and should be stable throughout the construction and operation period.

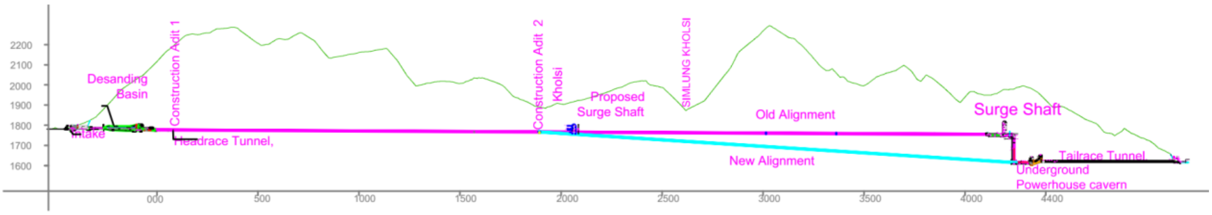


Figure 5-2: Alternative layout of Rasuwagadhi HEP

Discharge of the project is high (80 m3/s) although head of the project is 168m. Head loss inside the tunnel plays a significant role in power production. If the power plant is operated with the pressure tunnel of size 6.3m x 6.3m it will cause high head loss. Therefore the size for the pressure tunnel is reviewed. Optimization of the pressure tunnel is done based on head loss and construction cost of tunnel, which is described in section 5.2.1 below.

5.2.1 Optimization of Tunnel size

Head loss of the pressure tunnel will be high if the tunnel is constructed with the present size. Pressure tunnel is optimized making some of the assumption as shown in Table 5-1 below. The roughness inside the tunnel is high in drill and blast tunnel so roughness value (Manning's coefficient (M) is taken as 30 as per Arcement and Schneider (1989).

Table 5-1: Assumption for tunnel optimization

Price of energy	6.6 NRs/Kwh	Average of dry and wet season
Rate of return	10%	
Project life	30 Yrs	
Cost of construction	4750 NRs/m2/m	(Panthi, 2016)

Hydrological data was taken from the RHEP (2011). Duration curve for the project is shown in Figure 5-3. From the flow duration curve, design discharge of 80 m3/s is available only 40 percentage of the time in a year.

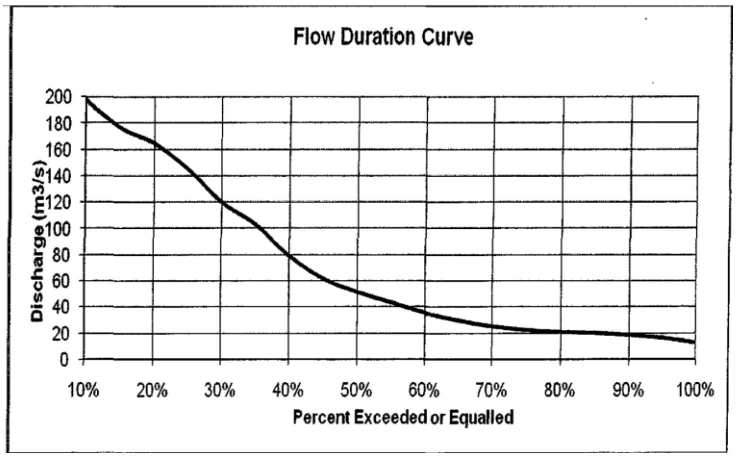


Figure 5-3: Flow duration curve (RHEP, 2011)

The tunnel has been tested with different velocity and dimension at full discharge. Increase in velocity decreases the size of the tunnel and increases head loss and vice versa.

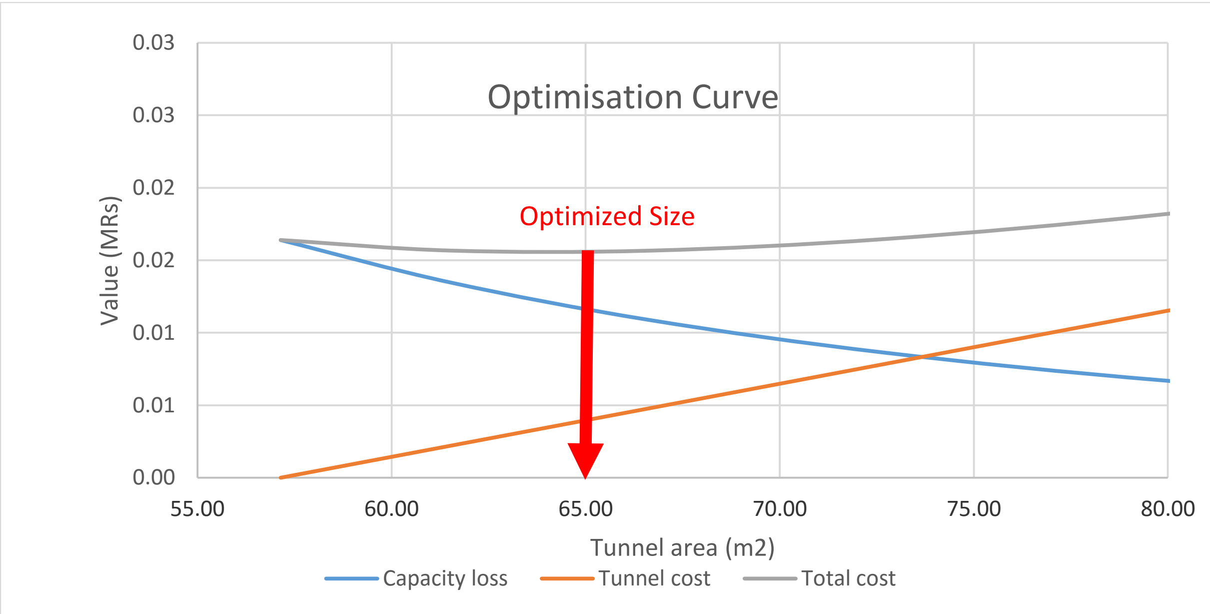


Figure 5-4: Optimization of pressure tunnel size

Increase in dimension of the tunnel reduces the friction loss and increases the revenue and at the same time increases the cost of construction. Between these two factors optimum dimension of tunnel was calculated at the lowest point of the total cost curve as shown in Figure 5-4. The optimized size of the tunnel with horse shoe shape is 8.5m x 8.5m with velocity of 1.23 m/s at full design discharge. Detail calculation of optimization is presented in Appendix B1.

5.2.2 Design of Surge shaft

Time taken to accelerate the generator from zero to normal speed with full load torque (Ta) varies between 5sec-8sec. Ta is independent of size and type of turbine. The time taken to accelerate the water in the penstock and the draft tube from zero to design discharge under influence of the head is calculated by the equation 7.1.

$$T_w = \frac{Q}{gH\sum\frac{L}{A}} \tag{7.1}$$

Where,

Tw = Penstock time constant

Q = Design discharge

H = Gross head

∑L/A = Sum L/A from nearest free water surface upstream to the nearest free water surface downstream of the turbine

Smaller the value of Tw, it is better. It is recommended by the prospective of governor stability Ta/Tw should be greater than six. In order to have damped mass oscillation between shaft and reservoir it requires a minimum free water surface area in the surge shaft and the minimum area is called Thoma area (A_{thoma}) which is calculated by the equation 7.2. Minimum area of surge shaft should be greater than 1.5 times Thoma area.

$$A_{thoma} = \frac{0.0083 * M^2 * {A_{tunnel}}^{5/3}}{H} \tag{7.2}$$

When the power plant is stopped immediately, there will be rejection of load and pressure will be created inside the tunnel. The maximum pressure of the system is given by the equation 7.3.

$$\Delta Z = \Delta Q \sqrt{\frac{\sum\frac{L}{A}}{g * A_{Shaft}}} \tag{7.3}$$

Where,

ΔZ = Upsurge pressure

ΔQ = Change in discharge from maximum to zero

∑L/A= Sum L/A from nearest free water surface upstream to the nearest free water surface downstream of the turbine g = gravity

A_{shaft} = Surge shaft area

Summary of the result is shown in Table 5-2 below.

Table 5-2: Surge tank design

Calculation			
Thoma Area	16.99	m^2	
Area of shaft	27.18	m^2	
Diameter of shaft	5.88	m	
Take Diameter of shaft	**16.00**	m	
Area of shaft	200.96	m^2	
Governor Stability			
Ta	6.00	Sec	
Tw	1.65	Sec	Should be less than 1
Upsurge			
ΔZ	13.76	m	
Down surge			
ΔZ	6.88	m	
Head loss			
Δh	6.39	m	
Submergence			
S	6.39	m	
Freeboard	2.47	m	
Bottom elevation of shaft	1760.07	m	
water level at intake	1790.00	m	
water level at shaft	1783.61	m	
water level with surge	1797.37	m	
Top elevation of shaft	1803.34	m	
Total height of surge shaft	**43.5**	**m**	

From the result, minimum Thoma cross section area has been calculated as 27.18m^2 which gives diameter of 5.88m. This value will give very long surge shaft and we have flexibility to increase the size. Diameter of surge shaft is increased and taken 16m so that the height of shaft will decrease. Total height of the surge shaft is calculated 43.5m.

5.2.3 Selection of section for analysis

It is time consuming to analyze the whole section along the tunnel alignment. Therefore, all together ten sections have been taken along the alignment based on overburden height, rock types, and shortest distance from the river which are analyzed for hydraulic fracturing and lowest value of safety factor as shown in Table 5-3. Three sections from chainage 0+000 to chainage 1+819 has been selected in straight headrace tunnel and other sections are choosen from shotcrete lined pressure tunnel.

Table 5-3: selection of tunnel section for analysis

SN	**Chainage**	**Available Head (H)**	Factor of Safety		Remark
			vertical cover (h)	**Shortest distance (l)**	
1	0+050m	11.15	74.29	43.99	
2	1+700m	20.28	27.03	11.07	
3	1+819.6m	20.92	18.59	8.93	Selected for numerical analysis
4	1+916.1m	21.45	14.19	9.39	
5	2+064.8 m	45.5	18.59	5.72	Proposed Surge shaft
6	2+368.56m	51.7	13.47	9.72	Selected for numerical analysis
7	3+048.1m	95.29	16.36	9.33	
8	3+556.6m	127.91	6.84	2.46	Selected for numerical analysis
9	3+910.78m	150.62	5.78	3.76	
10	4+064.31m	160.48	5.67	3.64	Selected for numerical analysis

Among these sections only four sections were selected for numerical analysis based on lowest factor of safety against vertical cover and shortest length between study point and river as shown in **Error! Reference source not found.**.

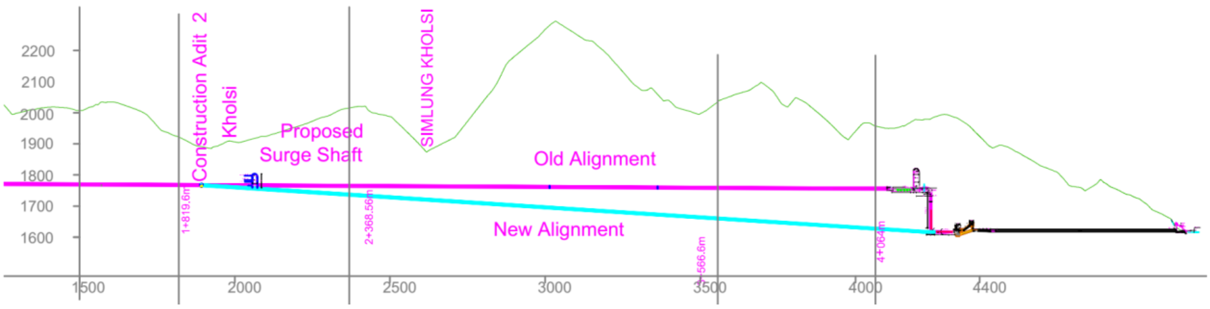


Figure 5-5: Selected sections along pressure tunnel for numerical analysis

First section have been taken at the stretch having Quartzite as rock type and there is low overburden depth, short horizontal distance from river bend. The first section is just above the starting point of shotcrete lined pressure tunnel in headrace tunnel. After that three sections have been taken at the stretch having quartzite as rock type in shotcrete lined pressure tunnel.

5.3 Rock mass classes and Support type

Rock mass classification system, i.e. Q-system proposed by (Barton, 2002), has been used to classify the rock mass classes and support type based on Q-values for the selected tunnel sections. Values of the Q-value was taken from ((RHC), 2011) along the alignment. RHC report have values between the sections.

In Rasuwagadhi HEP case, span of the tunnel is 9 m and ESR can be taken as 1.6 because it is water tunnel for hydropower. The ratio between span and ESR can be calculated and it comes out to be equal to 5.63. Rock mass classification and selection of support type is done based on the chart provided by (Grimstad and Barton, 1993).

Table 5-4: Rock mass classification and support type

Chainage	**Rock type**	**Q-value**	**Span/ESR**	**Rock mass Classification**	**Support type**
1+819.6	Quarzite	3.96	5.63	Fair	Systematic bolting of 1.6m spacing and unreinforced shotcrete (5-6cm)
		6.80		Fair	Systematic bolting of 2m spacing and unreinforced shotcrete (5-6cm)
		10.67		Good	Systematic bolting of 2.5m spacing and unreinforced shotcrete (5-6cm)
2+368.56-4+064.31	Quartzite	1.68	5.63	Poor	Systematic bolting of 1.4m spacing and unreinforced shotcrete (5-6cm)
		6.33		Fair	Systematic bolting of 1.8m spacing and unreinforced shotcrete (5-6cm)
		13.65		Good	Unsupported

In the selected tunnel sections, three classes of rock mass have been noticed i.e. poor, fair and good shown in Table 5-4. Minimum value, maximum value and average values have been chosen for the assessment of a particular chainage along the alignment. With all three Q-values, rock mass classification and support selection have been done according to calculation and charts. Minimum Q-value gives poor rock mass and maximum value gives good rock mass. All the average value show fair rock mass with systematic bolting with shotcrete lining. Based on this calculation quality of rock seem to be favorable for shotcrete lined pressure tunnel.

Chapter 6

6 Analysis of Pressure Tunnel

Shotcrete lined pressure tunnel has been analyzed for hydraulic fracturing and stability. Hydraulic fracturing has been done with Norwegian rule of thumb and numerical analysis using phase^2. Stability analysis has been done by Numerical analysis only.

6.1 Norwegian rule of thumb

Norwegian rule of thumb developed by Selmer Olsen (1970) is widely used on the analysis of pressure shaft/ tunnels. It considers vertical pressure due to both overburden and inclination of the shaft. To avoid the risk of hydraulic fracturing, the minimum stress around the tunnels/shafts should be greater than water pressure at the same point. Factor of safety is calculated as a ratio of available vertical cover and required vertical cover. This method only accounts for gravitational stress but not tectonic stress and residual stress.

After the failure of Askora tunnel, new rule of thumb was introduced by Bergh-Christensen and Dannevig (1987) taking account of the inclination of the valley directly (Broch, 1982). It is expressed as in equation 6.1 and equation 6.2

$$h > \frac{H * \Upsilon w}{\Upsilon_r * \cos\alpha} \tag{6.1}$$

$$l > \frac{H * \Upsilon w}{\Upsilon_r * \cos\beta} \tag{6.2}$$

Where,

H = static water head at the study point

h = required vertical cover at the study point

Υ_w = density of water

Υ_r =density of rock

α = inclination of the shaft

l = shortest length between study point and surface

β = inclination of the valley side

For the analysis of hydraulic fracturing, ten different sections have been chosen along the tunnel alignment. Result of the calculation of hydraulic fracturing in selected sections is presented in Table 5-3. Detail calculation is presented in Appendix B2. Selection of tunnel sections are based on the rock cover (h), shortest distance (l), surrounding geology and bends of river from the tunnel.

Table 6-1: Analysis of hydraulic fracturing

SN	Chainage	Head (H)	α	β	Calculated		Available at site		Factor of Safety		Remark
			deg	deg	h (m)	l (m)	h (m)	l (m)	h (m)	l (m)	
1	0+050	11.15	0.28	40	4.20	5.48	312.11	241.23	74.29	43.99	Safe
2	1+700	20.28	0.28	50	7.64	11.89	206.56	131.65	27.03	11.07	Safe
3	1+819.6	20.92	0.28	46	7.88	11.35	146.55	101.33	18.59	8.93	Safe
4	1+916.1	21.45	3.7	37	8.10	10.12	114.96	95.03	14.19	9.39	Safe
5	2+064.8	45.5	3.7	56	17.18	30.66	319.35	175.44	18.59	5.72	Safe
6	2+368.5	51.7	3.7	32	19.52	22.97	262.88	223.26	13.47	9.72	Safe
7	3+048.1	95.29	3.7	42	35.98	48.31	588.78	450.96	16.36	9.33	Safe
8	3+556.6	127.9	3.7	39	48.30	62.02	330.12	263.3	6.84	4.25	Safe
9	3+910.7	150.6	3.7	37	56.87	71.06	328.47	267.33	5.78	3.76	Safe
10	4+064.3	160.4	3.7	37	60.59	75.71	343.75	275.48	5.67	3.64	Safe

Tunnel section having Factor of safety (FOS) greater than 2 is considered as safe in our case. FOS is taken as 1.3 in general condition with better geological area. Rasuwagadhi HEP is located in the young mountain of Nepal where it is bit difficult to predict rock mass condition of the area. Hydraulic fracturing has been checked in all the sections. It can be seen from the calculation above in Table 6-1 that all sections have FOS more than 2. From this analysis it can be said that pressure inside the tunnel is lesser than the pressure from overburden so all the tunnel section are safe from hydraulic fracturing.

But this rule of thumb use simple equilibrium principle based on the gravitational stress only. In this kind of topography the stress regime is mainly dominated by topographic stresses. Tectonic as well as residual stress is also not taken into account. So only considering the gravitational effect oversimplifies the case. Hence for analyzing the insitu stresses condition and stress regime, numerical analysis is done and presented in section 6.2 below.

6.2 Numerical Modelling

Analytical method gives good result for simple geometries in a homogeneous medium but most of underground excavations have a complex geometry and located in an inhomogeneous rock mass. Number of computer-based numerical methods have been developed to provide means for obtaining approximate solutions to these problems. Among the different computer Programs, the Finite Element Method, Phase^2 has been selected. Numerical analysis in this thesis is to analyze the tunnel sections to determine the deformation of rock mass around tunnel, major and minor stresses before and after excavation and yielded element. Phase^2 is a 2-dimentional finite element program and is very popular for the analysis of underground/surface excavation in rock mass or soil and their support systems. The program consists of three modules: modelling, computing and interpreting. Phase^2 offer a wide variety of options when it comes to modelling, meshing, material properties and behavior, support, far-field stress, loads, joints and data interpretation. This program is user friendly, easy to operate and easy to understand.

Phase^2 computer program is used for analyzing stability of pressure tunnel and surge shafts as well as hydraulic fracturing of pressure tunnel. It is carried out as a plane strain analysis. Both elastic and plastic material properties are used for analysis. Elastic material is used to analyze the redistribution of stresses and strength factor for the material. Plastic material allows the material to yield, and is useful to examine displacements and rock mass failure. Results from the lab are used in Roc Data to generate input Parameter for Phase^2.

Roc Data is a software program for determining rock mass strength parameters, based on the Generalized Hoek-Brown failure criterion (Rocscience, 2014b). From the input parameters: UCS, GSI, intact rock property (mi) and disturbance factor (D), Roc Data calculates the Hoek-Brown parameters mb, s and a.

6.2.1 Model Set up and input data

Geometry and excavation stages

Actual ground surface is plotted as an external boundary. Tunnel is placed exactly as it is on the field so that the stresses induced by gravity is same as on the field. Overall stability and hydraulic fracturing is the scope of the analysis. Excavation has been done in single stage due to the fact that quartzite is quite strong rock and have good stand up time.

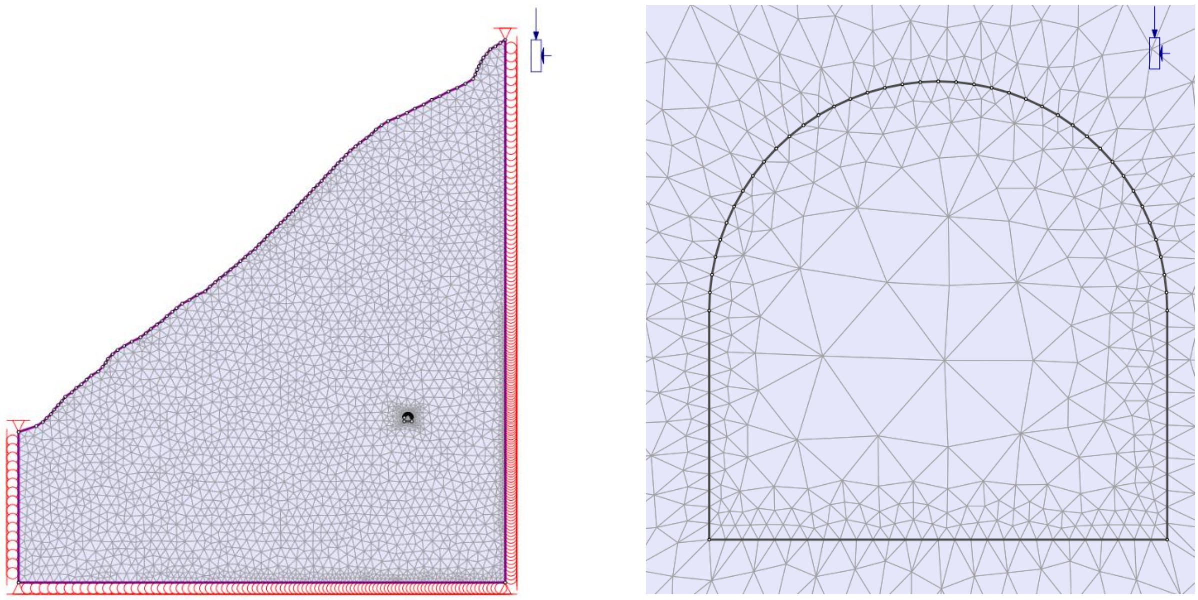


Figure 6-1: Model in Phase2 For tunnel section at 3+556m (Left) and closure view of tunnel (Right)

Typical model for tunnel alignment with topography and closure view of tunnel section is shown in Figure 6-1.

Mesh and displacement

A graded mesh type with 3 nodded triangles are used in the model, with a gradation factor of 0.03 for four cross section and 10 for longitudinal section and the number of excavation nodes are 100. Exact topography is plotted as an upper boundary and lower boundary is below the tunnel level to avoid end effects. Displacements are handled by restraining the lower external boundary in the vertical direction (y-direction) and left and right boundaries in the horizontal direction (x-direction) and the upper boundary is set free as in real free surface.

Material properties

Input parameters are obtained from laboratory test of the sample and are discussed in section 4.1 and results are presented in Table 6-2.

Table 6-2: Input values in Numerical Modelling

Rock Type	Quartz	**Poisson's ratio**	0.0248
Field Stress	Gravity	**Young's modulus**	53626 Gpa
Initial Loading Element	Field stress and body force	**Rock mass strength**	8.87 Mpa
Failure Criteria	Generalized Hoek-Brown	**Mb**	1.963
Material Type	Elastic/Plastic	**S**	0.03
Unit Weight	0.026 MN/m3	**a**	0.503

Generalized Hook - Brown failure criteria is selected to calculate input data for material properties and also for analysis. Analysis has been done considering both elastic and plastic material properties. Roc Data software is used to calculate the input parameter for material properties. Input data for Hoek-Brown parameters, GSI is set to 61, mi is set to 20 and disturbance factor (D) is set to 0.8 from an internal overview in RocData.

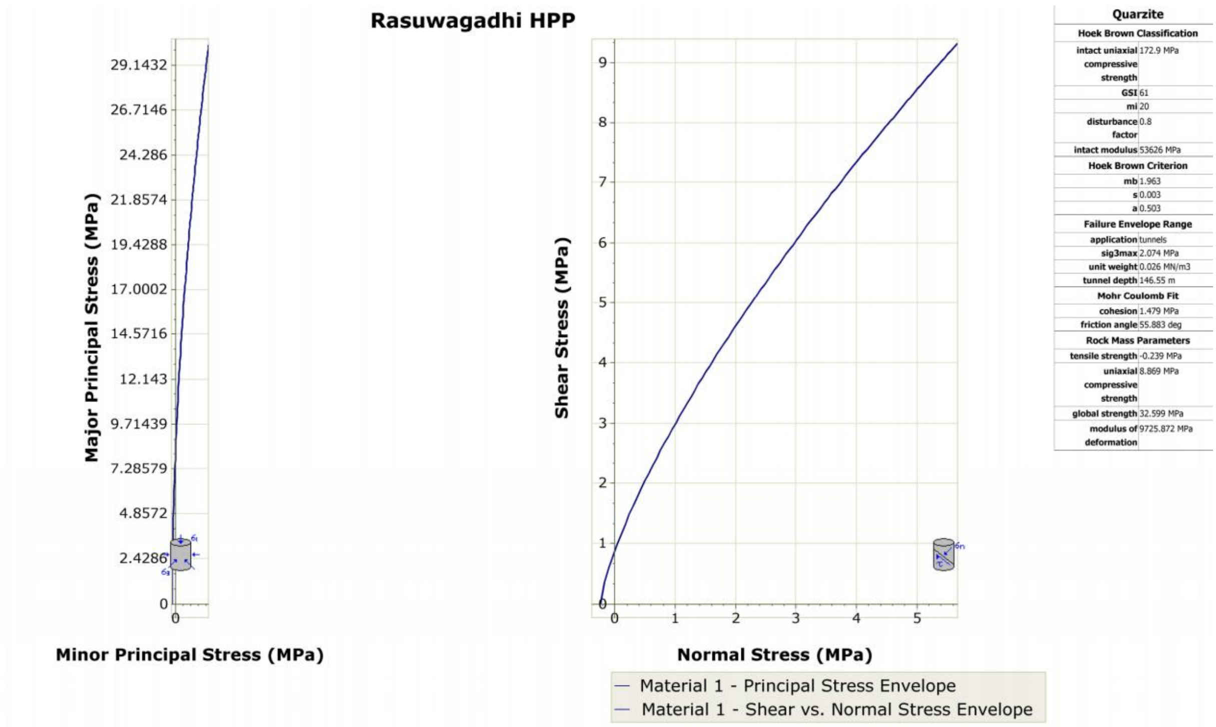


Figure 6-2: Input parameter for numerical modelling calculated from Roc Data

For plastic analysis, the residual material parameters has been taken as same as that for elastic analysis. The dilation parameter has been considered zero because of the undisturbed rock mass.

In-situ stresses

Actual ground surface has been selected to account the effect of topography in stress development. Gravity field stress is chosen for loading. Detail of stress situation is discussed in section 4.4. Magnitude of stress and stress ratio in all section are presented in Appendix C2. Due to the fact that phase[2] is a two dimensional program, the horizontal stresses must be projected into the relevant cross-section for the model. This can be done from equation 6.3 and equation 6.4 derived from an equilibrium state in a two dimensional stress plane (Figure 6.2).

In plane $$\sigma_\alpha = \sigma_H \cos^2\alpha + \sigma_h \sin^2\alpha \qquad (6.3)$$

Out of plane $$\sigma_\alpha' = \sigma_H \sin^2\alpha + \sigma_h \cos^2\alpha \qquad (6.4)$$

In plane $\sigma_\alpha = \sigma_H cos^2\alpha + \sigma_h sin^2\alpha$

Out of plane $\sigma_\alpha' = \sigma_H \sin^2\alpha + \sigma_h \cos^2\alpha$

σ_α is the normal stress on a plane, which in this case will be the excavation contour.

σ_α' is the normal stress out of plane.

σ_H and σ_h are the total horizontal stresses and gravity induced horizontal stress.

α is the angle between σ_h and the length axis of the excavation

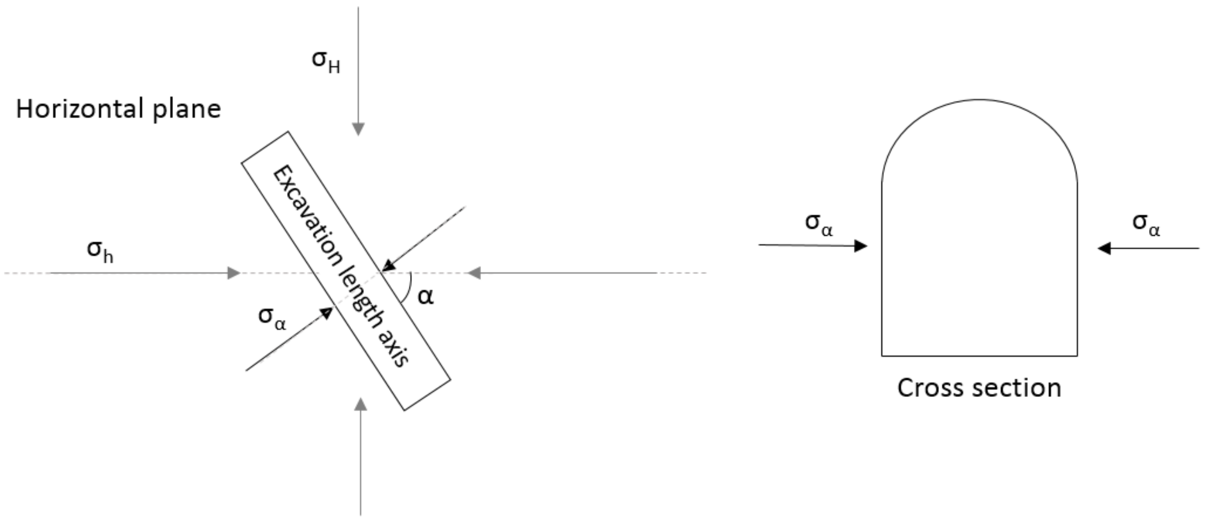


Figure 6-3: Illustation of equqtion 6.3 and equation 6.4 (Flåten, 2015)

By applying above equation the locked-in stresses in the in plane direction and in the out of plane direction is calculated which is given in Table 4-7. Horizontal and Vertical stress ratio is calculated from total horizontal stress and gravity induced vertical stress.

6.2.2 Hydraulic Fracturing Analysis in shotcrete lined pressure tunnel

Hydraulic fracturing in shotcrete lined pressure tunnel occurs when insitu minor principle stress around the tunnel is less than hydraulic pressure imposed from internal water pressure inside the tunnel. When in-situ stress acting on the rock mass is less, the existing joints of the rock mass where the water pressure is acting will open and the flow of water will occur through the periphery of the tunnel contour. For checking the possibilities of hydraulic fracturing of the tunnel, four different sections as shown in Figure 5-5 are chosen as presented in section 5.2.3.

6.2.2.1 Approach of analysis

Insitu stress situation along the shotcrete lined pressure tunnel

The insitu stress condition along the shotcrete lined pressure tunnel has been determined by running the Phase^2 model along the longitudinal direction. Whole shotcrete lined pressure tunnel is simulated in model at once as shown in Figure 6-4. The purpose of this analysis is to compare the minor principle stress at insitu condition with hydrostatic pressure at the same point inside the tunnel. The topography and direction of tunnel alignment both varies and the value of residual stresses also varies with them. Stress as input for the analysis is taken from the Appendix C2.

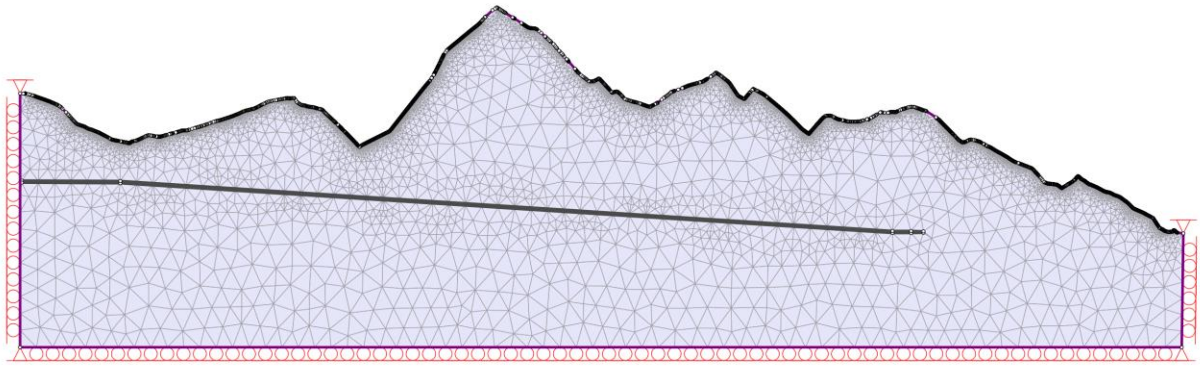


Figure 6-4: Model for virgin stress analysis

Hydraulic fracturing in selected cross sections

All the four selected sections have been analyzed with hydraulic fracturing. Valley slope model has been created to simulate the insitu stress which is influenced by topography of the area. According to the overburden depth, confinement of the joints against the water pressure varies. High overburden depth have higher confinement and vise-versa. Furthermore, tectonic stress and residual stress also contribute in the confinement of the rock joints. Major principle stress is vertical and minor principle stress is horizontal when the topography is horizontal. But in most of the cases, hydropower sites are located in the mountainous topography where the stresses regime is dominated by topography. That means near topography major principle stress is nearly parallel to the topography and minor principle stress is nearly perpendicular to topography.

6.2.2.2 Analysis results

Insitu stress situation along the shotcrete lined pressure tunnel

Insitu minor principal stress along the shotcrete lined pressure tunnel is shown in Figure 6-5 below.

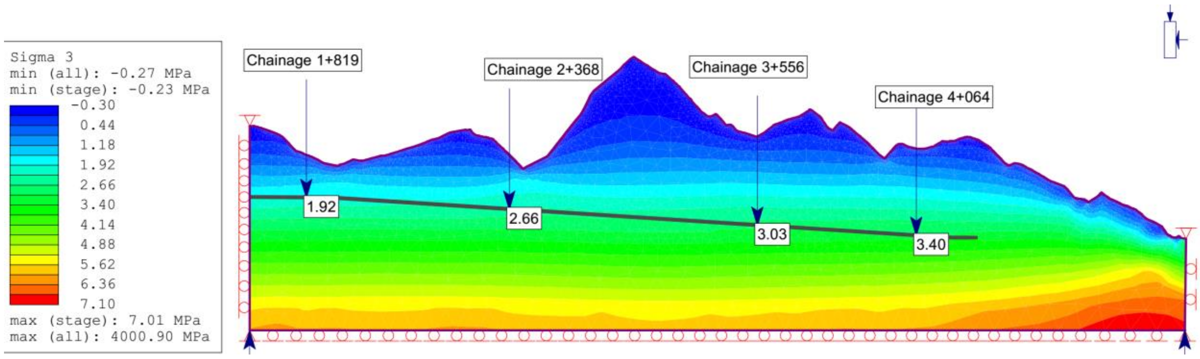


Figure 6-5: Shotcrete lined pressure tunnel alignment with insitu minor principal stress

Values of the insitu minor principle stress at various point in the tunnel alignment are plotted with hydrostatic pressure at the same point and shown in Figure 6-6.

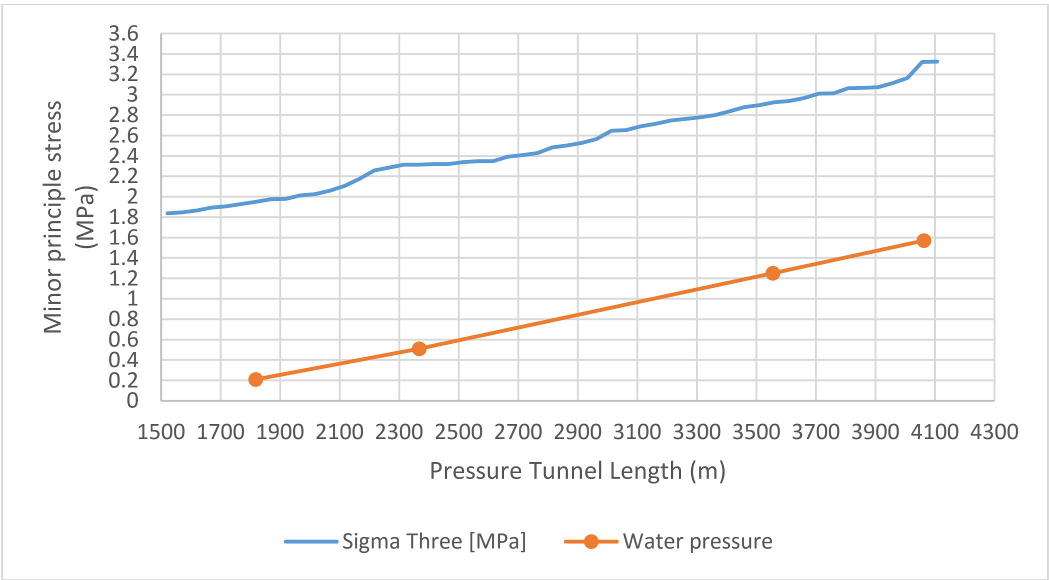


Figure 6-6: minor principle stress along the alignment.

Result shows that minor principle stress along the whole tunnel is higher than water pressure along the tunnel with FOS 2. Minor principle stress along the tunnel alignment varies from 1.92MPa to 3.4MPa, where highest water pressure is 3.4MPa near the point where steel lined penstock starts. From this result it can be concluded that, there is no risk of hydraulic fracturing in insitu stress condition before excavation. But after excavation, stress will be redistributed around the tunnel but far field stress remains same. The redistribution around the tunnel will not be more than few meters from tunnel contour and is analyzed in selected cross sections further.

Hydraulic fracturing in selected cross sections

Section 1+819m

The hydrostatic pressure at this section is 0.21MPa. Figure 6-7 depicts the model before and after excavation. Insitu minor principle stress along the contour of the tunnel is 2.9 MPa, which is greater than hydrostatic pressure at that point. After the excavation, the rock around the tunnel is destressed and minor principle stress has been reduced.

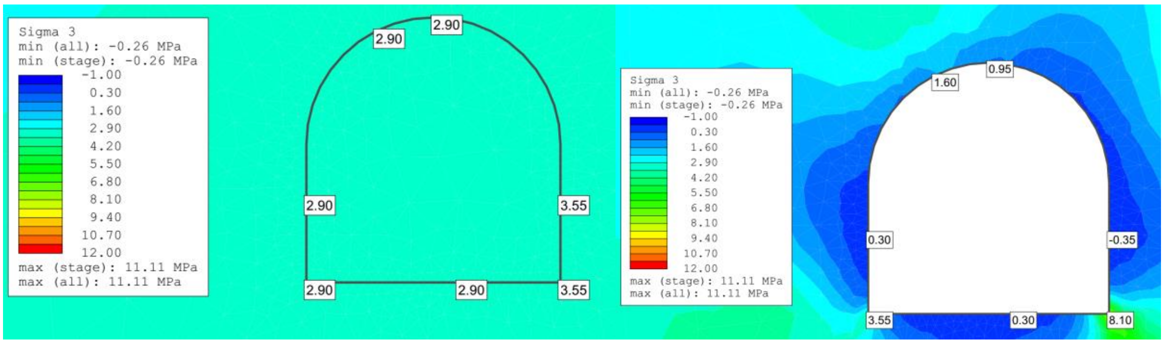


Figure 6-7: Minor principle stress before (left) and after (right) excavation at section 1+819m

Even though minor principle stress is reduced, it is greater than hydrostatic pressure along the contour of the tunnel. After the excavation possibility of water leakage through the sides of the tunnel is very low due to the higher pressure around the contour of tunnel. However, on the right wall of the tunnel minor principle stress value is negative and there may be the chances of tensile crack. If there is pre-existing joints or cracks in the vicinity of tensile crack possibility of water leakage may occur.

Section 2+368m

Internal water pressure at this section is 0.51MPa. Figure 6-8 shows the model before and after excavation. Before excavation insitu minor principle stress is same along the tunnel contour with 6.15 MPa, which is greater than hydrostatic pressure at that point. After the excavation, the rock around the tunnel is destressed and minor principle stress is reduced significantly. Minor principle stress is lesser than hydrostatic pressure along three sides of the tunnel and it is higher at the crown.

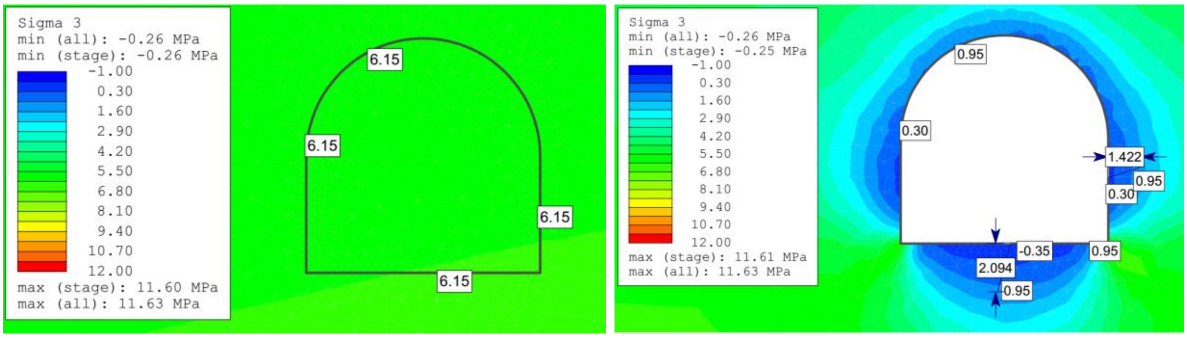


Figure 6-8: Minor principle stress before (left) and after (right) excavation at section 2+368m

After the excavation, results show that tension stress has been developed at the invert level which may result tension crack within that area. Minor principle stress is less than water pressure within 2.1m from the tunnel contour below the invert and around 1.4m away on the both side of tunnel wall as shown in Figure 6-8 (right). If there is any pre-existing joints within few meter around the contour water may leak out from these zone. But if no pre-existing joints are present in this area, water may leak within the lower stress area and become saturated due to the higher pressure beyond that point and further leakage will be prevented.

Section 3+566mand Section 4+064m

Internal water pressure at section 3+566 and 4+064 is 1.25MPa and 1.57MPa respectively and Figure 6-9 shows the model before and after excavation.

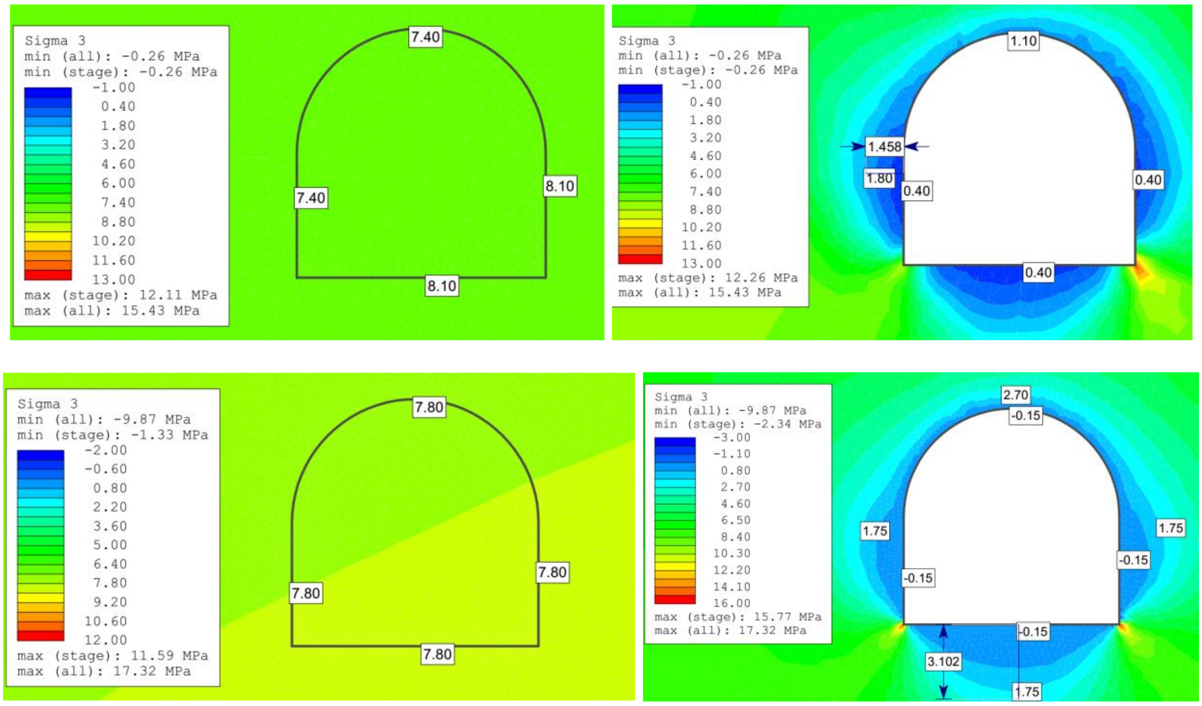


Figure 6-9: Minor principle stress before (left) and after (right) excavation at section 3+556m (above) and section 4+064m (below)

Insitu Minor principle stress is more than internal water pressure. After the excavation, the minor stress is reduced and less than water pressure within few meters as shown in Figure 6-9. In section 4+064, results shows negative stress which may develop tensile crack around the tunnel. Figure shows that stress is gradually increasing as we move outer from tunnel contour till it reaches insitu stress in far field. Hydraulic fracturing in these zones depends upon the pre-existing joints. The destressed zone will be saturated and far field stress doesn’t allow water leakage if there are no pre-existing open joints which works as a flow path.

6.2.3 Stability analysis of the headrace tunnel

The sections which have been checked for hydraulic fracturing have also been analyzed for possibilities of instability caused by stress regime.

a) Section 3+556m

Elastic Analysis

In elastic analysis, the material type is considered as elastic that means, the rock mass behaves elastically. The major concern of this analysis is to find the strength factor around tunnel periphery. According to rocscience, if the strength factor is less than one, failure is expected in model and the material behaves as plastic material. In addition to strength factor, major principle stress, minor principle stress and total displacement around tunnel contour have also been analyzed.

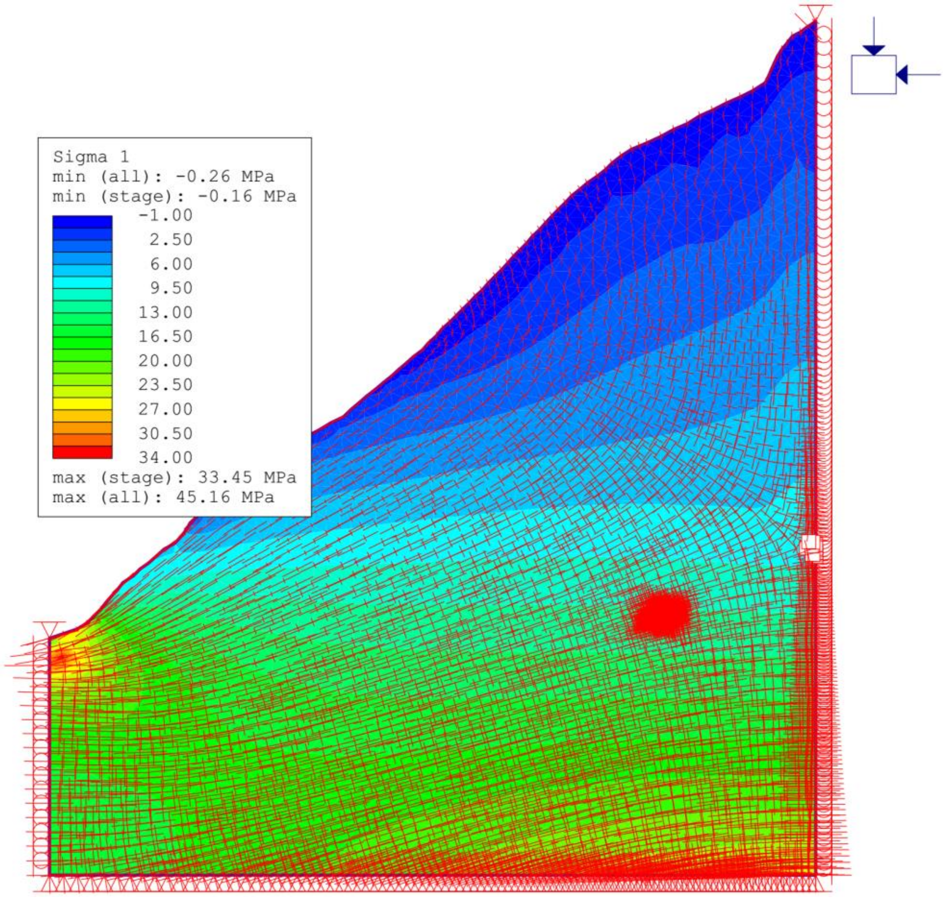


Figure 6-10: Model with stress trajectories at tunnel section 3+556m

Major principle stress

In Figure 6-11 it can be seen that major principle stress is almost parallel topography even at the tunnel level that means there is effect of topography in stress development.

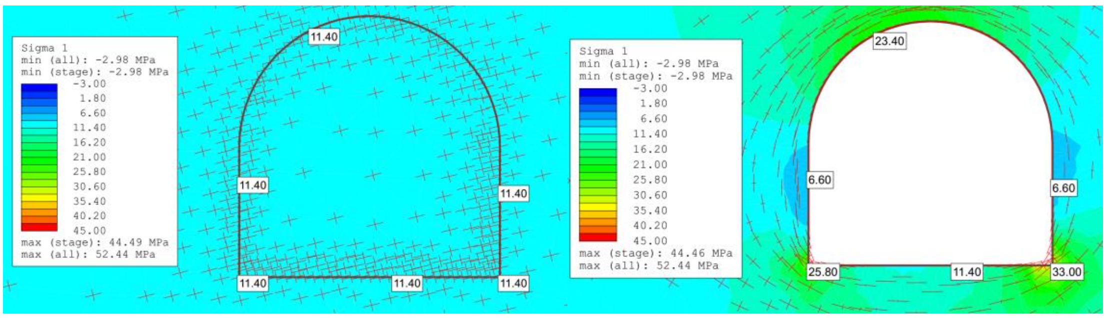


Figure 6-11: Major principle stress before Excavation and after excavation at section 3+556m.

Before excavation, major principle stress (σ_1) has same value 11.4 MPa at the tunnel location. After excavation, stress value varies around the tunnel and maximum stress of 33 MPa is developed at the corner of tunnel and minimum of 6.6 MPa at the side of tunnel as shown in Figure 6-11.

Displacement and strength factor

Displacement of tunnel section at 3+556m is shown in Figure 6-12 (left). Deformation vector shows that tunnel is displaced towards the slope of the valley. Displacement value is same on the three side of the tunnel and maximum displacement is on the ridge side with value of 18mm as shown in figure below.

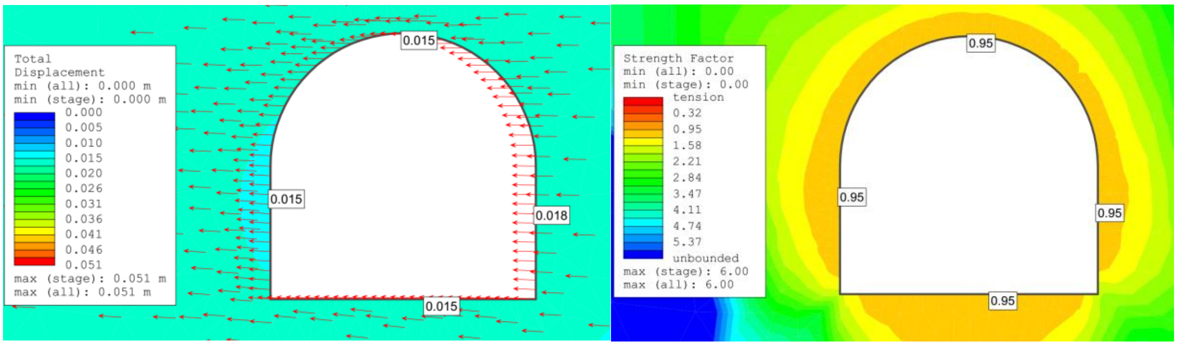


Figure 6-12: Displacement after excavation (left) and strength factor (right) at tunnel section 3+556m.

Figure 6-12 (right), Shows that Strength factor is less than one in the tunnel contour. Strength factor is less than one so rock will not behave elastically, there may be plastic failure on the material so plastic analysis is necessary for additional information.

Plastic Analysis without support

The plastic analysis has been done for the tunnel section to find the deformation, principle stresses, strength factor and yielding element around the tunnel with and without support.

Major principle stress and Displacement

In Figure 6-13 before excavation major principle stress has same value of 11.25Mpa around the contour of tunnel.

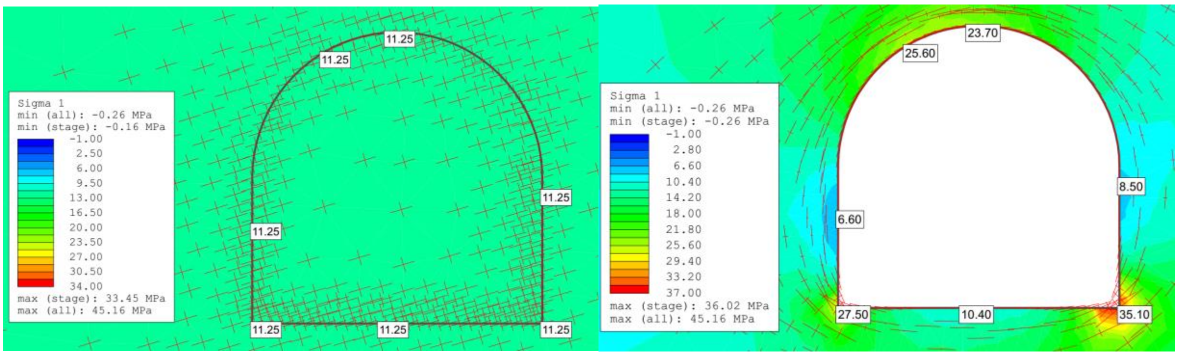


Figure 6-13: Major principle stress before Excavation and after excavation at section 3+556m.

Comparing to Elastic analysis, stress values are slightly changed but the pattern of stress distribution is same. After excavation maximum stress of 35.1 MPa is developed in right corner of tunnel and minimum stress of 6.6 MPa at the left side.

Displacement, Strength factor and Yielded element

Displacement of tunnel section is shown in Figure 6-14 (left). Deformation vector shows that tunnel is displaced towards the slope of the valley. Maximum displacement is 16mm on the right side of tunnel and 13mm on the left side.

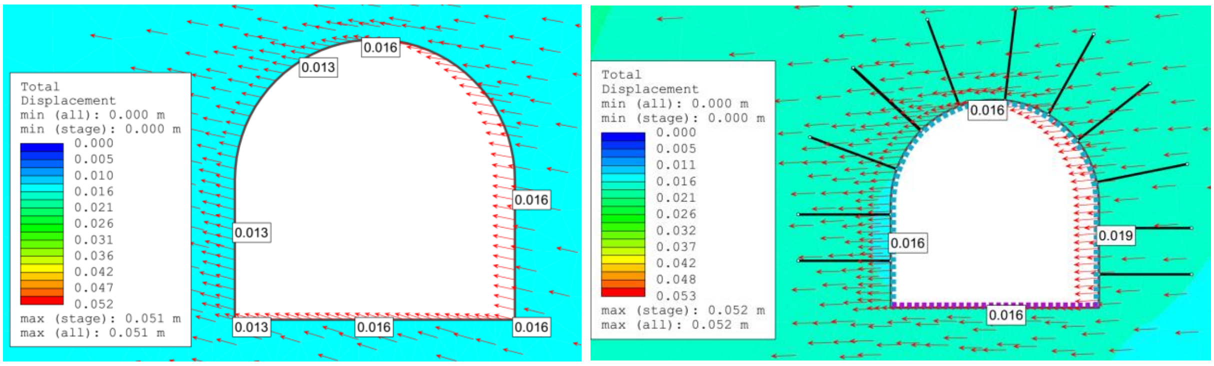


Figure 6-14: Displacement after excavation (left) without support and with support (right) at tunnel section 3+556m

Figure 6-15 left, Shows Strength factor is less than one in all sides of the tunnel same as elastic analysis.

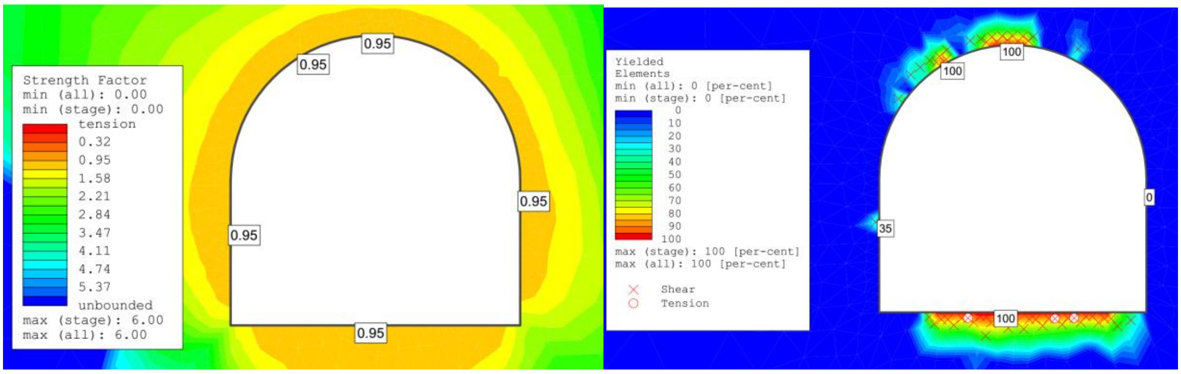


Figure 6-15: Strength Factor (left) and yielded element (right) at tunnel section 3+556m

Elements have been yielded in crown and invert without support which is shown in Figure 6-15 (right). Only very small section on the left wall is yielded partly as in Figure 6-15 (right). Most of the element is yielded due to shear.

Plastic analysis with support

Detail properties of rock support provided are given in Table 6-3.

Table 6-3: Properties of rock support provided.

	Shotcrete	**Concrete**	**Bolt**	
Type	Standard Beam	Standard Beam	Type	Fully Bonded
Material Type	Plastic	Plastic	Diameter	25mm
Thickness	50mm	300mm	Length	2m,4m,6m
Young's modulus	3000 MPa	30000 MPa	Spacing	Differs
Poisson’s ratio	0.2	0.2	Bolt Modulus	200000 MPa
Compressive strength (Peak)	35	35	Tensile Capacity	0.1 MN
Compressive strength (Residual)	5	5	Residual Tensile Capacity	0.01 MN
Tensile Strength (Peak)	5	5		
Tensile Strength (Residual)	0	0		
Beam Element Formulation	Timoshenko	Timoshenko		

From stability analysis without support it is noticed that there is 16mm maximum displacement and very little elements are yielded due to shear and tension. Keeping these facts in mind, minimum support are used. Rock bolt of 4m length is installed systematically at spacing of 2m x 2m. Then 50mm shotcrete is lined along the two sides of the tunnel and crown. 300mm concrete is lined at the invert level for transportation purpose.

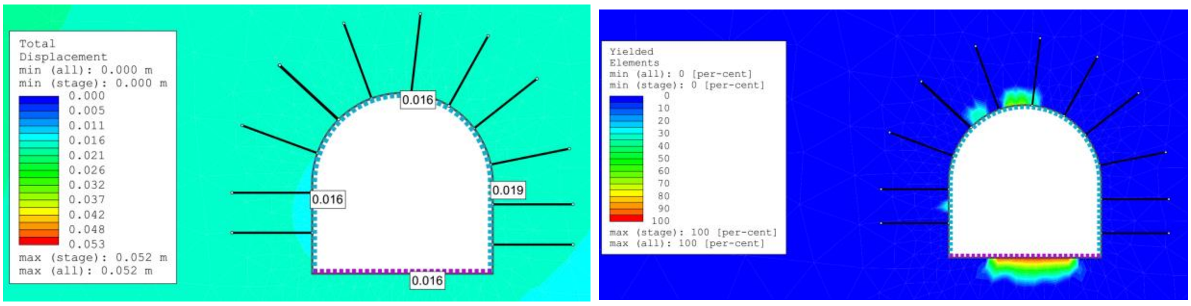


Figure 6-16: Major Principle stress (left) and yielded element (right) after support installation at tunnel section 3+556m

From the result of the support analysis we can see that total displacement is more than plastic analysis without support as shown in Figure 6-14 (right) above. Stresses value have changed with and without support but the distribution of stresses are similar. After the application of support, some part of stress is taken by support too so yielding of the rock mass is reduced. Yielding of supports is also analyzed and the result shows that there is no yielding in provided supports.

b) Section 1+819m

Elastic Analysis

Results of analysis shows Strength factor is less than one around the tunnel contour. According to rocscience tutorial, strength factor less than unity means it is not necessary for plastic analysis.

Plastic Analysis without support

Strength factor in the elastic analysis is less than one so plastic analysis has been done to find the yielding element around the tunnel.

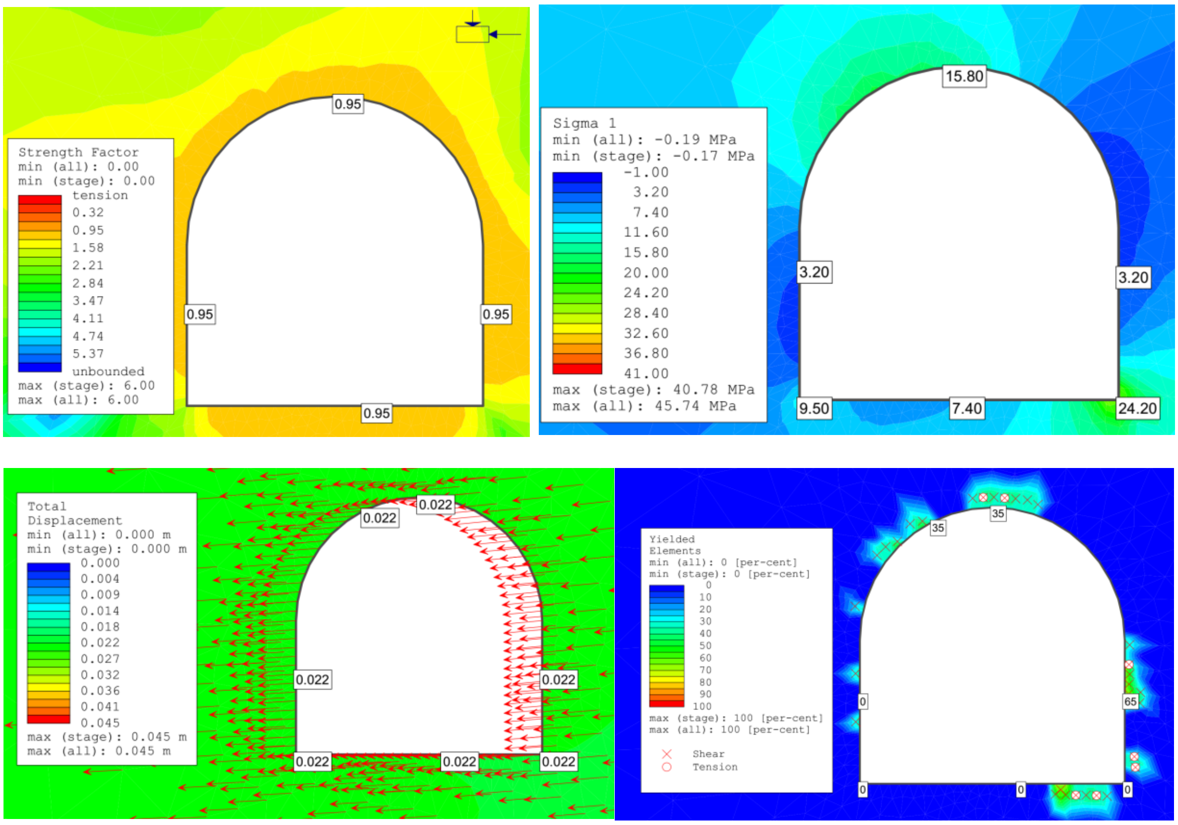


Figure 6-17: Strength Factor (Top left) and Major Principle Stress (top right). Total Displacement (bottom left) and Yielded element (bottom right) at tunnel section 1+819m.

Figure 6-17 shows the result of the plastic analysis. The strength factor around the periphery of the tunnel is still less than one. Major principle stress before excavation is concentrated on the left corner with maximum value of 24.2MPa and in crown with 15.8Mpa.. Displacement is same around the tunnel contour with 22mm. Very few element is yielded along the contour of tunnel.

Plastic analysis with support

Detail properties of support are given in Table 6-3 above. From plastic analysis without support it is noticed that there is 22mm displacement and very few elements are yielded due to shear and tension. Rock bolts of 4m is used at spacing of 3m. 50mm shotcrete is lined along the on the two sides of tunnel and crown. 300mm concrete is lined at the invert level for transportation purpose. Systematic bolting have been done around the contour of the tunnel. Displacement before and after support installation is almost same. In the crown of the tunnel displacement is 2mm more than without support. Displacement will occur right after it is destressed and there will be time gap between excavation and installation of support so support will act after the displacement has occurred.

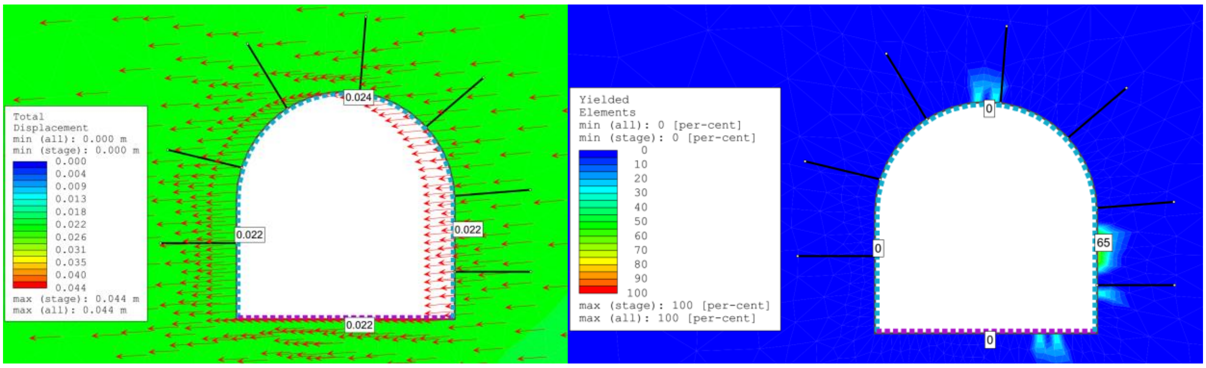


Figure 6-18: Total displacement (left) and yielded element (right) after support installation at tunnel section 1+819m

c) Section 2+368m

Elastic Analysis

Analysis results shows Strength factor is less than one along the contour of the tunnel. As strength factor is less than one so we have to go for plastic analysis

Plastic Analysis without support

Strength factor in the elastic analysis is less than one so plastic analysis has been done to find the major stress, displacement, yielding element around the tunnel

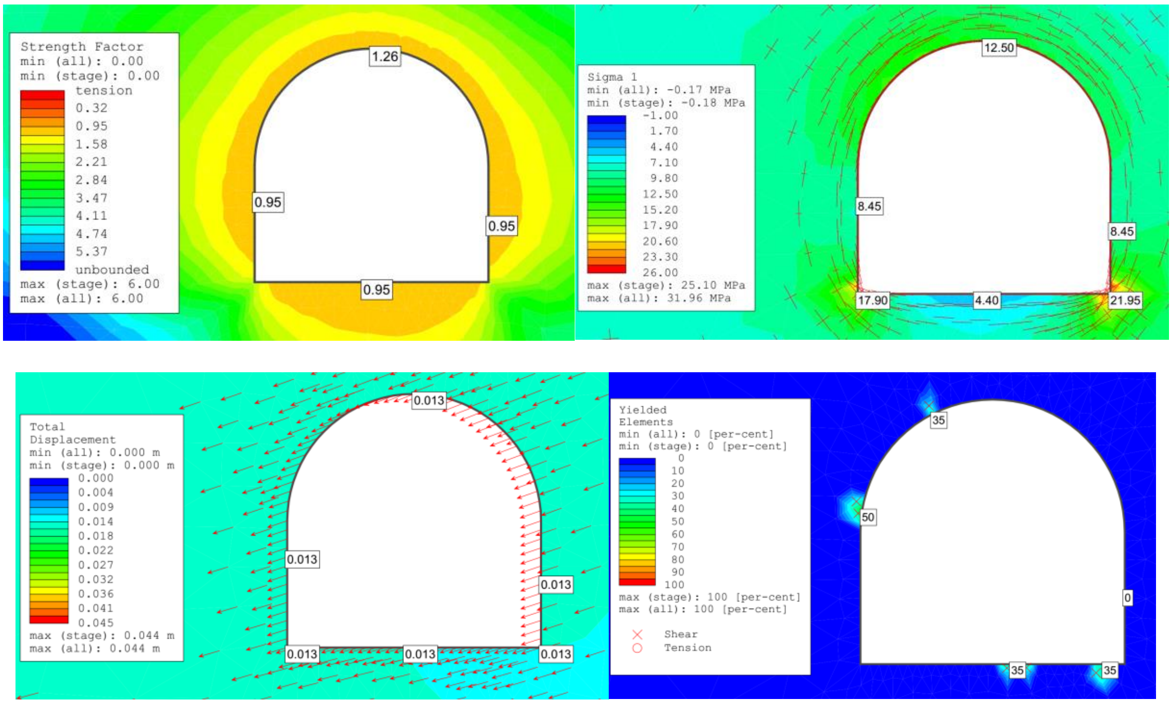


Figure 6-19: Strength Factor (Top left) and Major Principle stress (top right). Total Displacement (bottom left) and Yielded element (bottom right) at tunnel section 2+368m.

Figure 6-19 shows the result of the plastic analysis without support. The strength factor around the periphery other than crown of the tunnel has same value as elastic analysis and on crown it is more than one. Major principle stress is maximum at the crown. Displacement is 13mm at the tunnel location. Small section is yielded on the left side and on the invert level of the tunnel section. Results of the analysis shows that there is very less yielding of rock in the tunnel section.

Plastic analysis with support

Detail properties of rock support are given in Table 6-3 above. From stability analysis without support it is noticed that there is small displacement of 13mm and very little elements are yielded due to shear only. Rock bolt of 4m is installed systematically around the tunnel contour at spacing of 3m x3m. Then 50mm shotcrete is lined along the contour of the tunnel two sides and crown. 300mm concrete is lined at the invert level for transportation purpose.

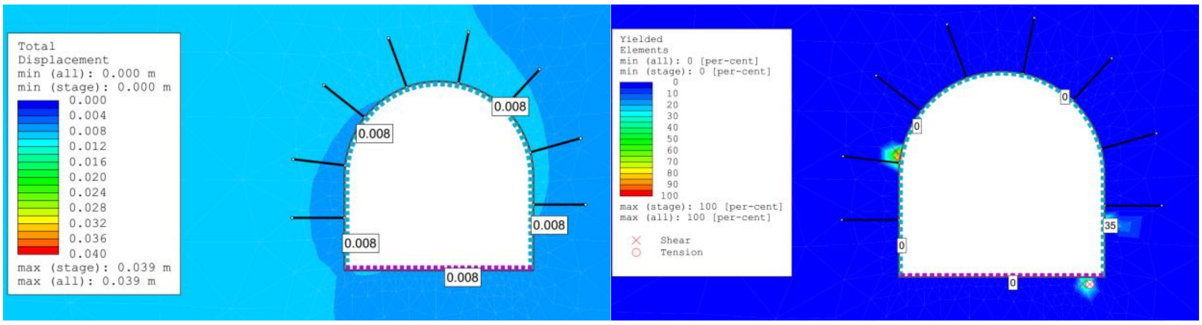


Figure 6-20: Total displacement (left) and yielded element (right) after support installation at tunnel section 2+368m

From the result of the support analysis we can see in the Figure 6-20 (left) total displacement has reduced from 13mm to 8mm after application of support. This means that the support have resist some deformation. The analysis shows that there has been small change in the stress situation after application of support. Yielding of the supports have also been tested and the results show no yielding.

d) Section 4+064m

Elastic Analysis

Analysis result shows that strength factor is less than one in crown and two sides of the tunnel wall. As strength factor is less than one so we have to go for plastic analysis.

Plastic Analysis without support

Strength factor in the elastic analysis is less than one so plastic analysis has been done to find the major stress, displacement, yielding element around the tunnel with and without support and support estimation.

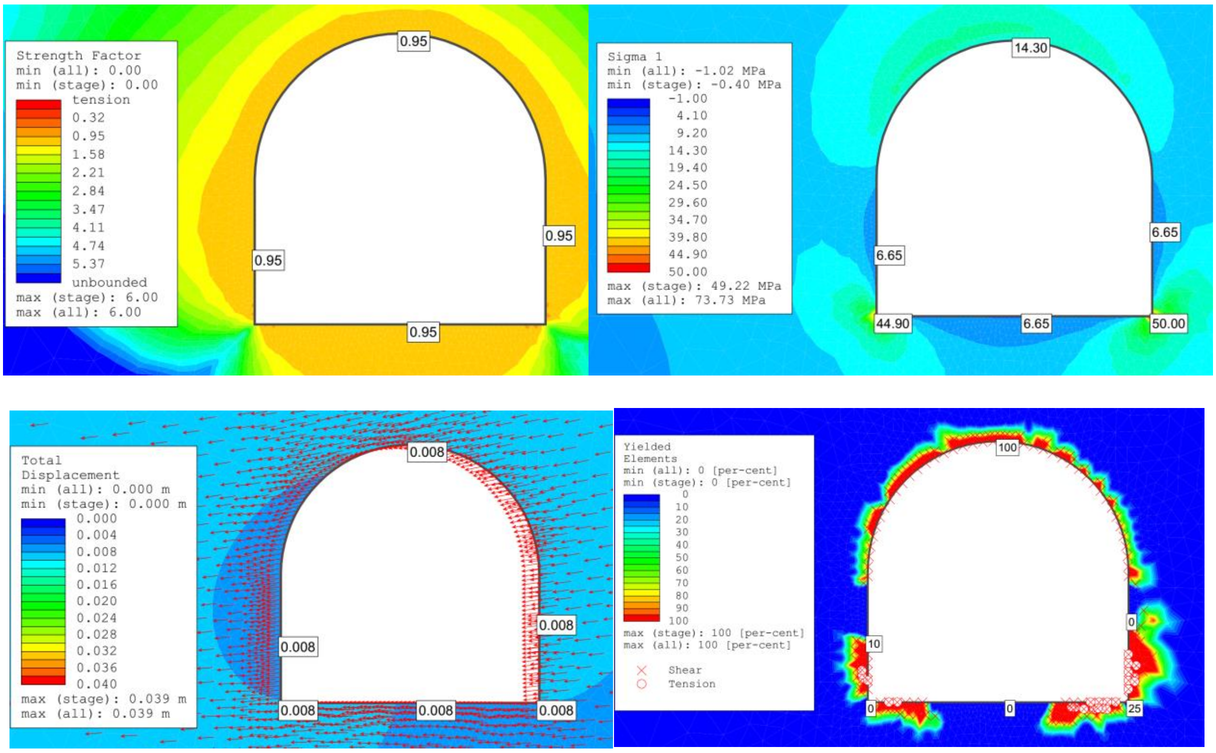


Figure 6-21: Strength Factor (Top left) and Major Principle stress (top right). Total Displacement (bottom left) and Yielded element (bottom right) at tunnel section 4+064m

Figure 6-21 shows the result of the plastic analysis. The strength factor around the periphery of the tunnel has same value as elastic analysis. Major principle stress is maximum at the corners Displacement is 8mm at the tunnel location. Element are yielded on the crown and both side of the tunnel and also in invert level.

Plastic analysis with support

Detail properties of rock support provided are given in table 6.2 above. From stability analysis without support it is noticed that there is very small displacement and elements around the tunnel are yielded due to shear and tension. Keeping these facts in mind, minimum support are used. Rock bolt of 4m length with spacing of 2m x2m have been installed around the tunnel as shown in Figure 6-22. Then 50mm shotcrete have been lined along the contour of the tunnel two sides and crown. 300mm concrete is lined at the invert level for transportation purpose.

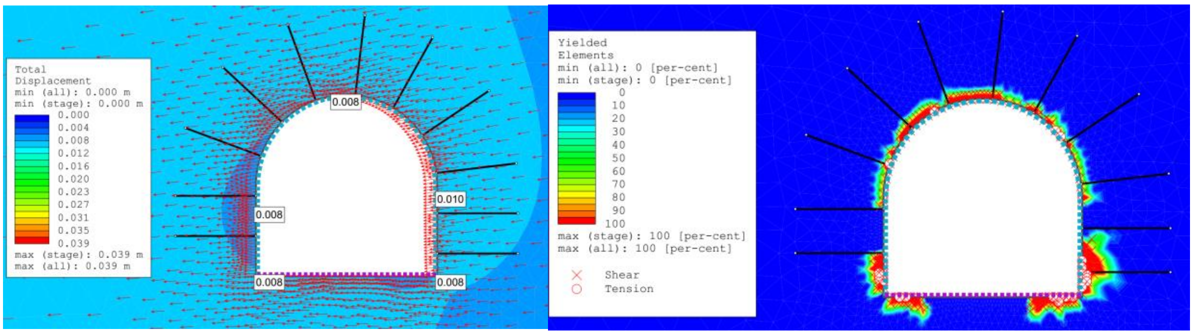


Figure 6-22: Total displacement (left) and yielded element (right) after support installation at tunnel section 4+064m

From the result of the support analysis we can see from the Figure 6-22 left total displacement has not changed much with and without support. Result shows small increment in displacement as compared to no support analysis on the left wall. This may be because the time gap between excavation and support installation. Within this time gap, rock will go some deformation. Distribution of major principle stress is also almost same with and without support. Yielding of provided support is also tested. Result doesn’t show any yielding on both shotcrete and bolts.

6.3 Concluding remarks

6.3.1 Hydraulic Fracturing

Analysis of hydraulic fracturing of shotcrete lined pressure tunnel of Rasuwagadhi HEP has been carried out with two different method. The first method is Norwegian thumb rule method which compares the water pressure with overburden. The result of thumb rule shows that Factor of safety is greater than 2 and it is safe against hydraulic fracturing. However, this method doesn’t take in account of tectonic and residual stresses. Therefore, this type of simplified technique can be used in the preliminary study phase of the project.

Second method is numerical analysis taking into account of all stresses. Tunnel alignment has been analyzed with two different approach within this method. Initially insitu minor principle stress has been analyzed along the tunnel alignment with longitudinal section. Throughout the longitudinal section of tunnel alignment insitu stress is greater than the water pressure inside the tunnel. After that four different tunnel cross sections along the alignment have been analyzed for minor principle stress. In all four sections, insitu minor principle stress is greater than the water pressure. Result of analysis with two method is summaries in Table 6-4 with FOS.

Table 6-4: Summary result of Hydraulic fracturing

Section (m)	Water pressure (Mpa)	Minor Principle stress (Mpa)			FOS with minimum value
		Longitudinal model	Cross section model before excavation	Redistributing zone after excavation	
1+819	0.21	1.92	2.9	0	9.1
2+368	0.51	2.66	6.15	1.4	5.2
3+556	1.25	3.03	7.4	1.5	2.4
4+064	1.57	3.4	7.8	3.1	2.2

After excavation minor principle stress is lower than the water pressure within few meter around the tunnel contour. Value of stress increases outside the tunnel area as we move further from the contour until it reaches insitu stress in far field. This result shows that the destressed zone will be saturated and far field stress doesn't allow water leakage. But if there are pre-existing joints it might works as a flow path. Hence, from all analysis it can be concluded that there will be no hydraulic fracturing and leakage along the pressure tunnel alignment. However if there is pre-existing open joints chances of water leakage is always there.

6.3.2 Stability Analysis

Rock mass on the whole alignment of shotcrete lined pressure tunnel alignment in Rasuwagadhi HEP could be classified as typical hard rock. Four different selected sections have been analyzed for stability and support optimization is done using Phase^2 computer software. Both elastic and plastic analysis without support and plastic analysis with support have been done for all four sections.

In all sections along the tunnel alignment, strength factor is less than one so plastic analysis has been done. In all selected sections little yielding have been found. Result of the numerical modelling showed that the support measures for the tunnel sections are:

- Chainage 1+819m: One layer of shotcrete with systematic bolting. Shotcrete layer is 50mm thick in the crown and wall, 300mm thick concrete in invert level and bolt of 25mm diameter and 4m length with 2m x 2m spacing.
- Chainage 2+368m: One layer of shotcrete with systematic bolting. Bolts of 25mm dia and 4m length with spacing 3m x 3m have been used for bolting and shotcrete layer of 50mm thick is used in crown and wall. 300mm thick concrete lining has been used in invert level.

- Chainage 3+556m and Chainage 4+064m: Systematic bolting with one layer of shotcrete. Bolts of diameter 25mm and 4 m length have been used in 2m x 2m spacing along the contour of the tunnel. Shotcrete layer of 50mm thick has been used. 300mm thick concrete lining has been used in invert level.

Rock support calculated from Q-System along these four sections suggested that systematic bolting for poor rock in 1.4m spacing and for good rock as unsupported. Thickness of unreinforced shotcrete vary between 4cm to 10cm according to poor and good rock. Support provided during numerical modelling and calculated from Q-system is nearly similar with each other.

Chapter 7

7 Analysis of Surge Shaft

In the present design of Rasuwagadhi HEP, surge tank was designed at the end of the headrace tunnel, before the starting of the vertical shaft. In the alternative design it is located near the starting of the pressure tunnel. Due to the presence of the small Kholsi it is shifted little towards the pressure tunnel. Just below or around the kholsi there will be chances of water leakage into surge tank so it has been proposed after crossing this Kholsi.

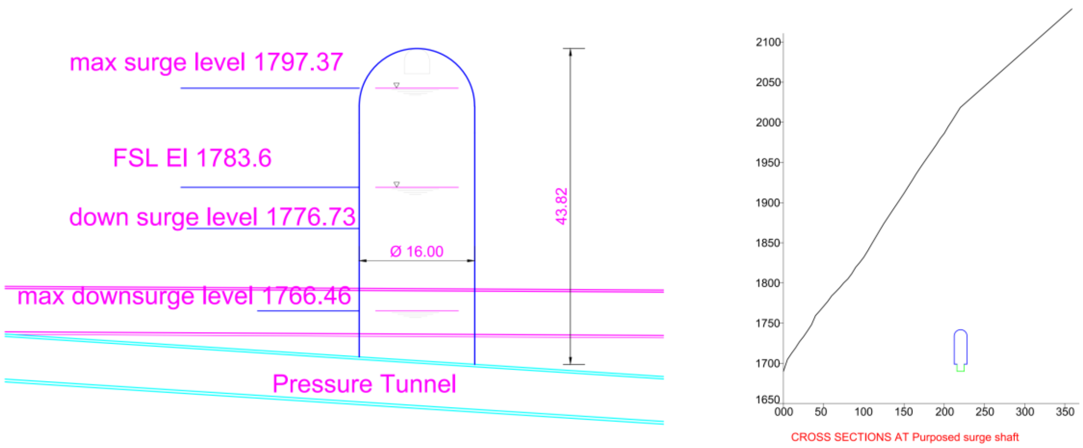


Figure 7-1: Surge shaft Location

The proposed surge shaft is located 170m from the construction audit along the pressure tunnel and 30m from the Kholsi. The entry point of the construction audit for tunnel and surge shaft is proposed at the same location. The audit length for the surge shaft is 150m with upward inclination of 10%.

7.1 Support estimation with Q-system

To define rock support for the surge shaft Q-system is used. Surge shaft is proposed at chainage 2+064m. Location of surge tank lies in between the chainage 1+400 to 2+150 and According to geological report of Rasuwagadhi HEP (RHEP, 2011), Q-values are 2.86, 5.29 and 7.15 in between these chainage. For the support estimation of surge tank Q-value has been taken same as in chainage 1+400m to 2+150m. According to Q-values and calculated span/ESR values 5.16 rock mass is classified as poor and fair.

Support estimation has been done by trying out combination of shotcrete and fully bonded rock bolts with the help of chart provided by Grimstad and Barton (1993). According to the chart, rock support required for Q-values of 2.86 and span/ESR value has been estimated systematic bolting with spacing 1.3m to 1.6m and 40mm – 50mm unreinforced shotcrete. For the Q-value of 5.29 rock support requirement has been estimated systematic bolting with spacing 1.6m to 2.0m and 40mm unreinforced shotcrete. For the highest Q-value of 7.15 rock support has been estimated systematic bolting with 2.0m spacing.

7.2 Numerical assessment

A two dimensional finite element program Phase^2 has been used for analysis of the surge shaft.

7.2.1 Description of model

Model of Vertical surge shaft as designed in section 5.2.2 is shown in Figure 7-2 and has been analyzed for stability. The model has been created to be excavated and analyzed in multiple stage. Excavation of surge shaft is started from aeration tunnel. Aeration tunnel is excavated first and the crown part is excavated. Muck is taken from the adit tunnel, which is shown in first stage in model.

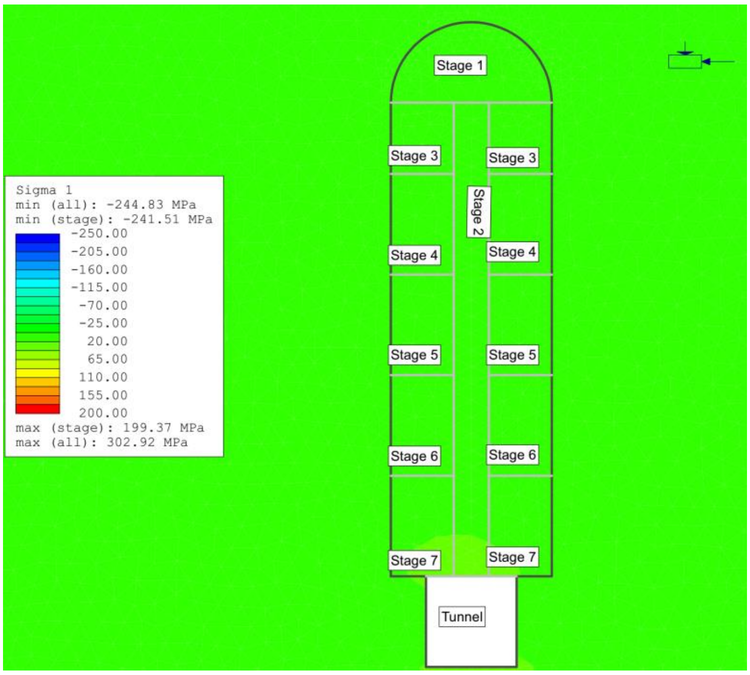


Figure 7-2: Assumed excavation stages of surge shaft

A pilot hole of dimension 2m x 2m is drilled at the center of the shaft after the crown is excavated downwards to pressure tunnel, which is modeled as stage 2. The surge shaft is excavated from top in a multiple stages after excavation of pilot hole. Total 5 stages are made for excavation of rest of the shaft. Each stages are excavated through benching of side wall, starting from pilot hole in a stages and the muck is transfer from pilot hole towards the pressure tunnel from where it is taken out.

7.2.2 Input Parameter

The material properties for the rock mass are same as explained in section 6.2.1. For this analysis stress ratio and stress values are taken from Table 4-7 and detail of calculation is presented in Appendix C2

Results of Elastic Analysis

In elastic analysis strength factor shows the degree of overstress. It represents the ratio of available rock mass strength to induced stress at the selected point.

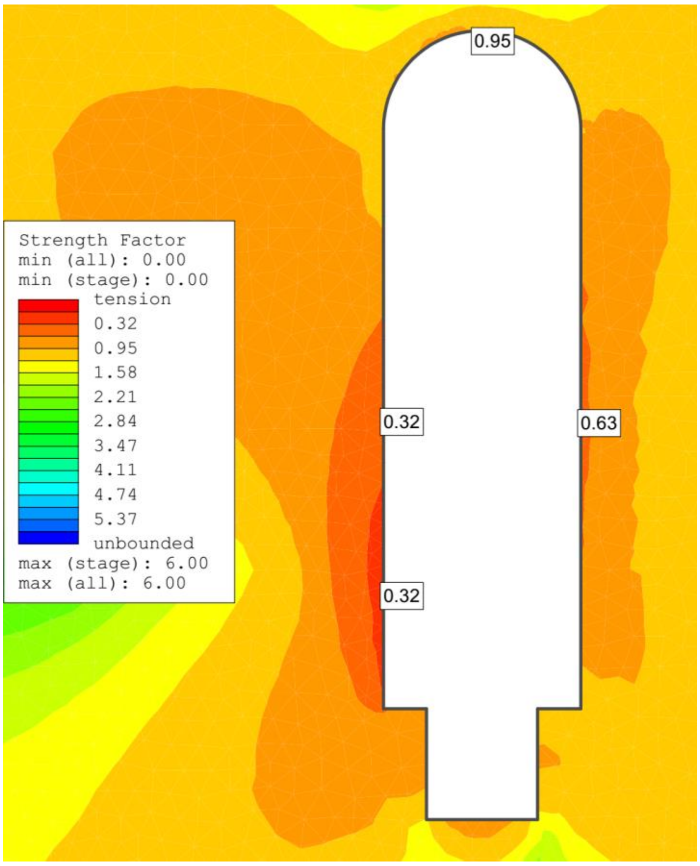


Figure 7-3: Strength factor

The analysis result in Figure 7-3 shows that strength factor is below one in crown of shaft and on all other sides of the surge shaft which suggest for plastic analysis.

7.2.3 Results of Plastic Analysis without support

Major principle stress at the crown of the surge shaft is maximum with value 24.2MPa. On the center of side of the shaft, major principle stress is minimum with value of 1.8MPa. Figure 7-4 shows the situation of major principle stress in the surge shaft. At the right corner, the junction of tunnel and surge shaft maximum principle stress is negative it shows that there is a possibilities of developing tensional crack.

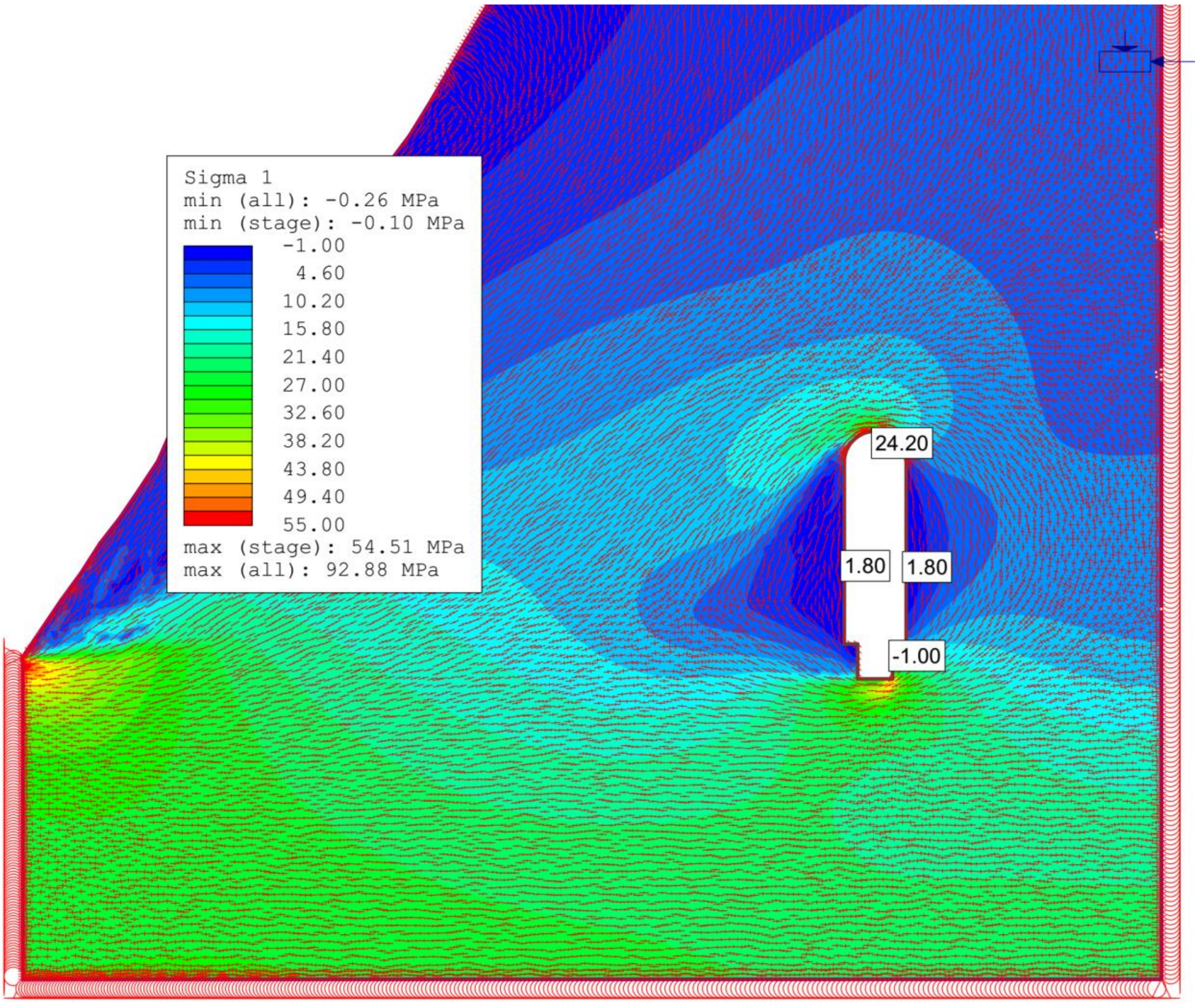


Figure 7-4: Major principle stress value of surge shaft in plastic analysis

Figure 7-5 shows that yielded element is developed during the progress of excavation downward. At stage 1 "excavation of crown" there is no mark of yielded zone on the crown. Yielding on sides start after the excavation of pilot hole. Yielding of side progress until excavation stage 5 and terminate after that. In the excavation of final stage there is no mark of yielding on side further. As we can see in Figure 7-5 yielding is concentrated on the middle section of the model and on left side below crown level. The results show little yielding on crown of the shaft. Most of the yielding is due to tension and little with shear.

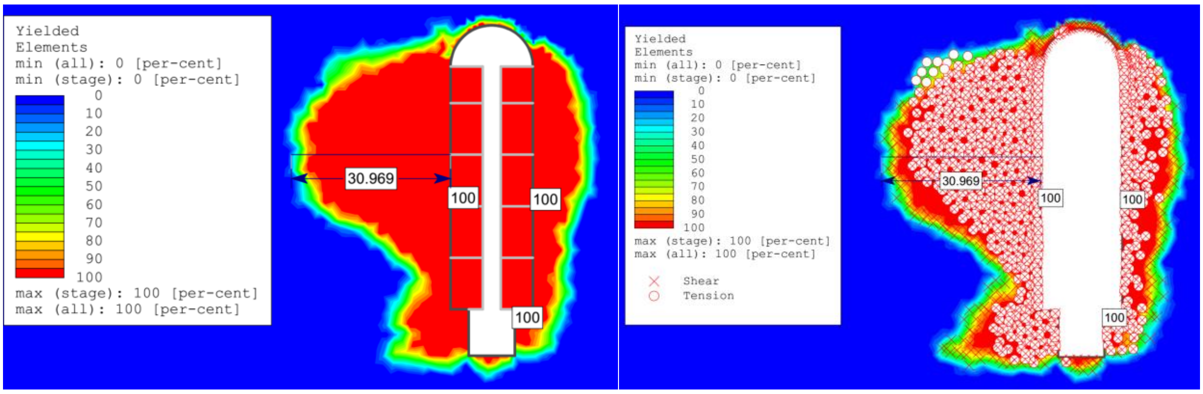


Figure 7-5: yielded element of surge shaft in excavation stage 3 (left) and excavation stage 7 (right) in plastic analysis

Figure 7-6 shows total displacement of the surge shaft after its excavation. From the figure displacement vector shows that surge shaft is displaced left towards valley side. The value of displacement is not even along the contour of the surge shaft. Displacement along left wall and right wall is different. Maximum displacement is 30mm from the ridge side towards valley and 20mm in crown towards valley.

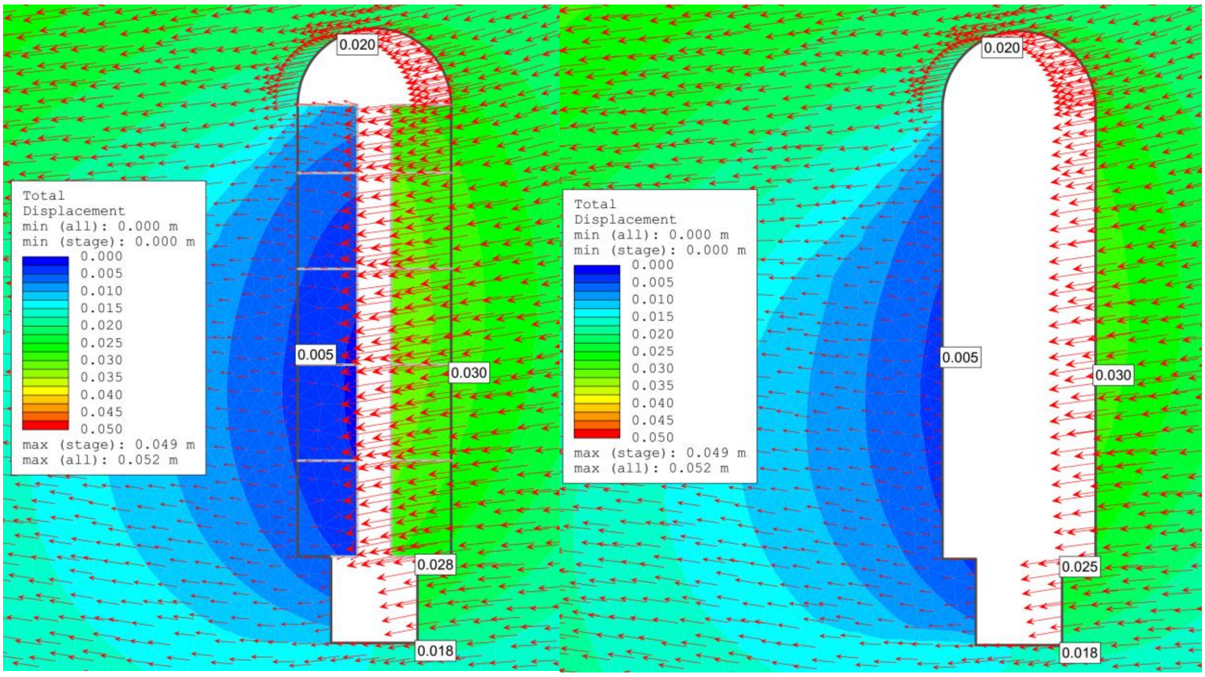


Figure 7-6: Total deformation of surge shaft in excavation stage 2 (left) and excavation stage 7 (right) in plastic analysis

Results of the yielded element shows it needs to be supported. Plastic analysis with support is shown below.

7.2.4 Results of Plastic analysis with support

Generally, support should be designed to achieve stable condition at every excavation stages as well as final stage. Trinh et al. (2010) states that for every excavation stages, rock support at early stage may be heavier than required in order to take heavier load resulting from later excavation stages. It may be wise to delay installing of bolt for sometime within standup time and shotcrete can be delayed a little longer time. Shotcrete can be done just after bolting or after excavation of next stage.

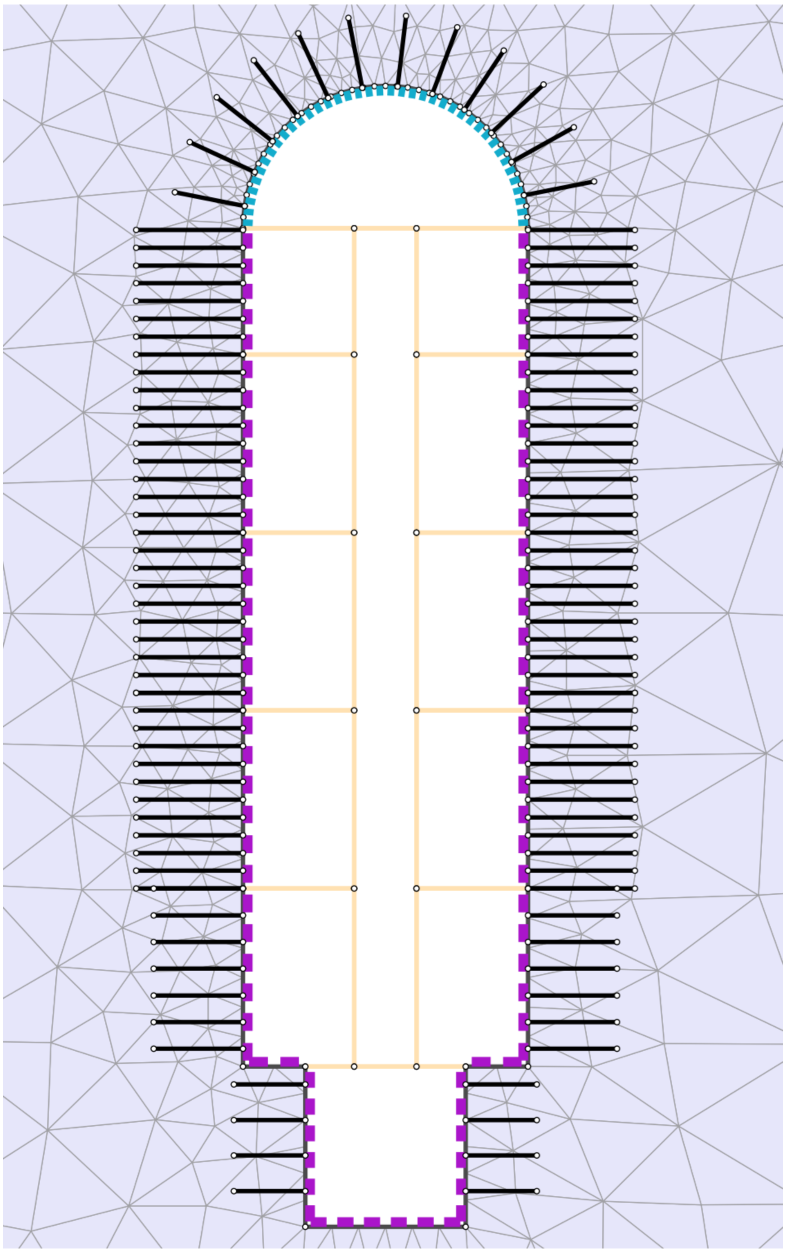


Figure 7-7: Installation of rock bolts, shotcrete and concrete

To find the reasonable support measure different support type like bolt, shotcrete and their combination are analyzed. Excavation of crown part is done and 25mm bolts are installed. After installation of bolts concreting of 100mm is done. Excavation is followed by bolting and Concreting for all other stages. Length of the bolt and spacing of the bolt is different along the different section and determined by seeing the yielded element in plastic analysis (Figure 7-5) without support as shown in Figure 7-7.

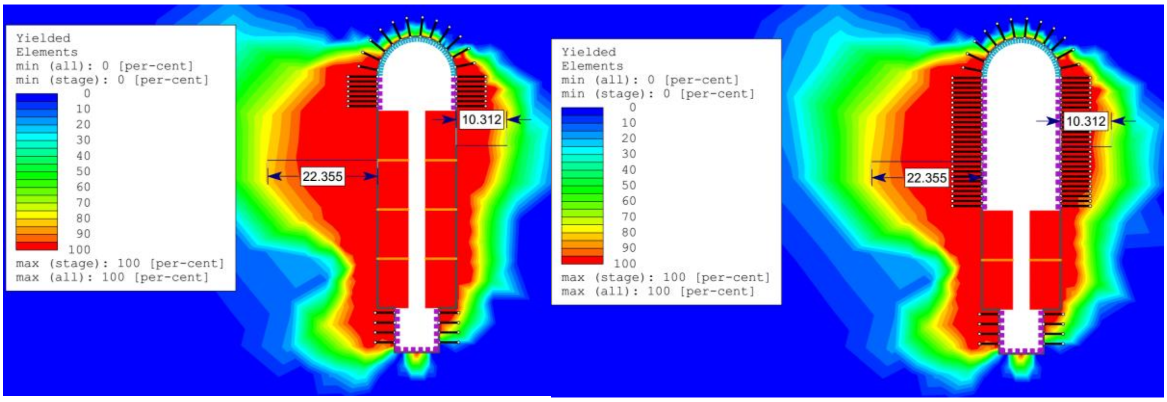


Figure 7-8: yielded zone development during excavation

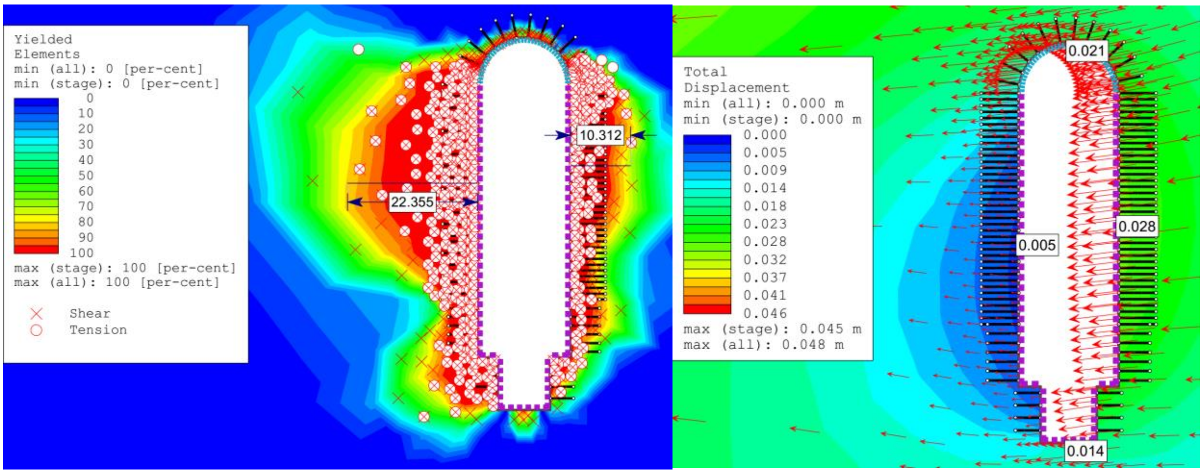


Figure 7-9: yielded zone development at final stage of excavation and displacement

As we can see in Figure 7-8 and Figure 7-9, there is no any sign of yielding of supports both bolts and shotcrete/concrete in entire stages. Supports takes load and it is not stressed beyond limit. After the application of the support yielded element is reduced from 31m to 22m on the left side as shown in Figure 7-5 and Figure 7-9. Yielding is due to tension mainly and little bit with also shear.

7.3 Concluding remarks

Rock mass at the location of surge shaft in Rasuwagadhi HEP could be classified as typical hard rock. Generally rock mass behavior shows that if a certain relaxation of rock mass around an opening is allowed, the requirement of rock support capacity will be significantly reduced. This behavior shows that it may be wise to allow some delaying in support installation. The delaying should be within a standup time that the stability is still retained.

Result of the numerical modelling showed that the support measures for the surge shaft are:

- On the roof: Systematic bolting with one layer of shotcrete. The bolt have 4m long and 2m x 2m spacing. Shotcrete layer is 100mm thick.
- On the wall: Systematic bolting with one layer of concrete. Bolts of length 6m are installed in excavation stage 3 and stage 6 in the spacing of 1m. At the final excavation stage bolt of length 4m is installed with spacing of 1.5m. The properties of bolt and shotcrete is same as in Table 6-3.

Rock support calculated from Q-System and support provided during numerical modelling are correspondence with each other. Bolt spacing used in a numerical modelling is fairly coherent to the spacing provided by the Q system chart. Maximum displacement is negligible.

8 Conclusion and recommendation

8.1 Conclusion

The rock in the study area mainly consist of Quartz. No major faults or joints are present in the study area according to the available reports and details from Rasuwagadhi HEP. Some hot springs are located on the both side of alignment in a far distance which may be connected with weakness zone. Since quartz is a good rock for tunneling and no major discontinuities are present along the alignment there is not much risk in pressure tunnel. If the project is constructed with shotcrete lined pressure tunnel instead of long headrace tunnel and vertical shaft, the length of the tunnel will have reduced which will reduce the cost. The inputs to stability analysis and hydraulic fracturing analysis are rock mass parameters and rock stresses. Therefore, quality of analysis largely depends upon the correct estimation of these input parameters. For the analysis, the tectonic stress value has been taken to be equal to 7.5MPa from the similar type of project, but stress measurement at the site will be necessary to verify this value.

Following conclusions can be drawn based on the analysis done during this thesis:

High Pressure tunnel

- The main challenge that has been faced in the analysis is correct estimation of rock mass parameters. However, the input parameters have been estimated with the help of different reports, literatures, discussion with Supervisors and some of the parameter were estimated through lab test of sample
- If the size of the present dimension is used in the pressure tunnel there will be high head loss which will eventually reduce energy production. Horse shoe tunnel section is designed with size 8.5m x 8.5m based on increase in construction cost and increase in benefit from lowering head loss.
- Norwegian rule of thumb shows that tunnel is safe against hydraulic fracturing. Overburden pressure is higher than the water pressure inside the tunnel along the tunnel alignment. Throughout the high pressure tunnel, insitu stress is more than water pressure. After the excavation of the tunnel, rock mass is destressed and minor stress along the tunnel section is reduced significantly. In some section minor principle stress is even less than water pressure but this area is limited within small length of around tunnel and far field stress is equal to insitu stress. We can conclude from this result that disturbed zone will be saturated and due to higher stress outside the area there will be

no hydraulic fracturing and leakage. But if pre-existing joints is present there, it may works as flow path and initiate water leakage. Tangential stress is also limited in a very small length of maximum 1.6m.

- Stability analysis and rock support estimation of pressure tunnel has been done with numerical modelling in the four representative sections. Small amount of yielding is shown after the excavation in all section. Rock bolt of length of 4m and 25mm size have been used. Bolts are installed in grid around the tunnel contour. Due to the minor principle stress less than water pressure around the tunnel contour, 50mm shotcrete has been provided around the tunnel. 300mm concrete lining has been done on the invert of the tunnel due to the transportation purpose during construction.
- However, rock mass condition and stress value can expected to vary between predicted and actual during construction along the alignment. The rock support estimated can only be taken as primary estimation. It is also possible to encounter unfavorable rock mass during the construction like local weakness zone so the final support installation must be decided based on the site condition during construction.

Surge shaft

- Surge shaft is proposed above the high pressure tunnel in the quartzite rock. The location of surge shaft is based on the geological condition of the site. It is proposed after the Kholsi 170 m below the construction adit due to the water leakage possibilities from the Kholsi. Surge shaft is 16m in diameter and 43.5m high, designed based on hydraulic requirement.
- The result of the stability analysis shows that during the excavation of pilot hole after crown part is excavated larger area around the surge shaft is yielded. More rock mass is yielded on the valley side (left side) than ridge side. Right side wall of the shaft is displaced maximum of 30mm where maximum displacement of crown is 21mm. displacement is not improved much after the application of the support because it is already displaced before application of support. Rock bolt is installed around the surge shaft in two different pattern according to the yielded element. On the roof of the shaft and just above the tunnel of the shaft 4m long bolts have been installed in 2m x 2m spacing with Shotcrete layer is 100mm thick. On the rest of the wall bolts of length 6m have been installed in spacing of 1m x 1m. Support installation is done after the

excavation of benching and before moving on next benching. After the application of the support yielding element on the left side is improved much.

- Stability analysis and support estimation have been done without considering the dynamic effect of moving water inside the surge shaft.

Even though there are some uncertainties in the input parameter which were taken from different reports and literature, the shotcrete lined pressure tunnel and surge shaft performs well against stability. It is concluded from the results that the pressure tunnel is safe and more economical option than the present headrace tunnel with steel lining vertical shaft.

It can be concluded that during the planning and designing phase of the project, better to see all the possible solution and go for the best option than going through the same design for all the projects.

8.2 Recommendation

There are many limitations in this thesis. These limitations can be improved with some more efforts on the analysis. Following major points have been recommended for the further analysis;

- Tectonic Stress measurement is necessary to verify the estimated value.
- Very few investigation of rock mass classification and weakness zone has been made along the tunnel alignment. More investigation is needed along the alignment with joint mapping.
- There is a high stress difference at the corner of tunnel and on the other part of the tunnel which may cause unfavorable condition. This problem can be solved by designing the pressure tunnel section without sharp corner.

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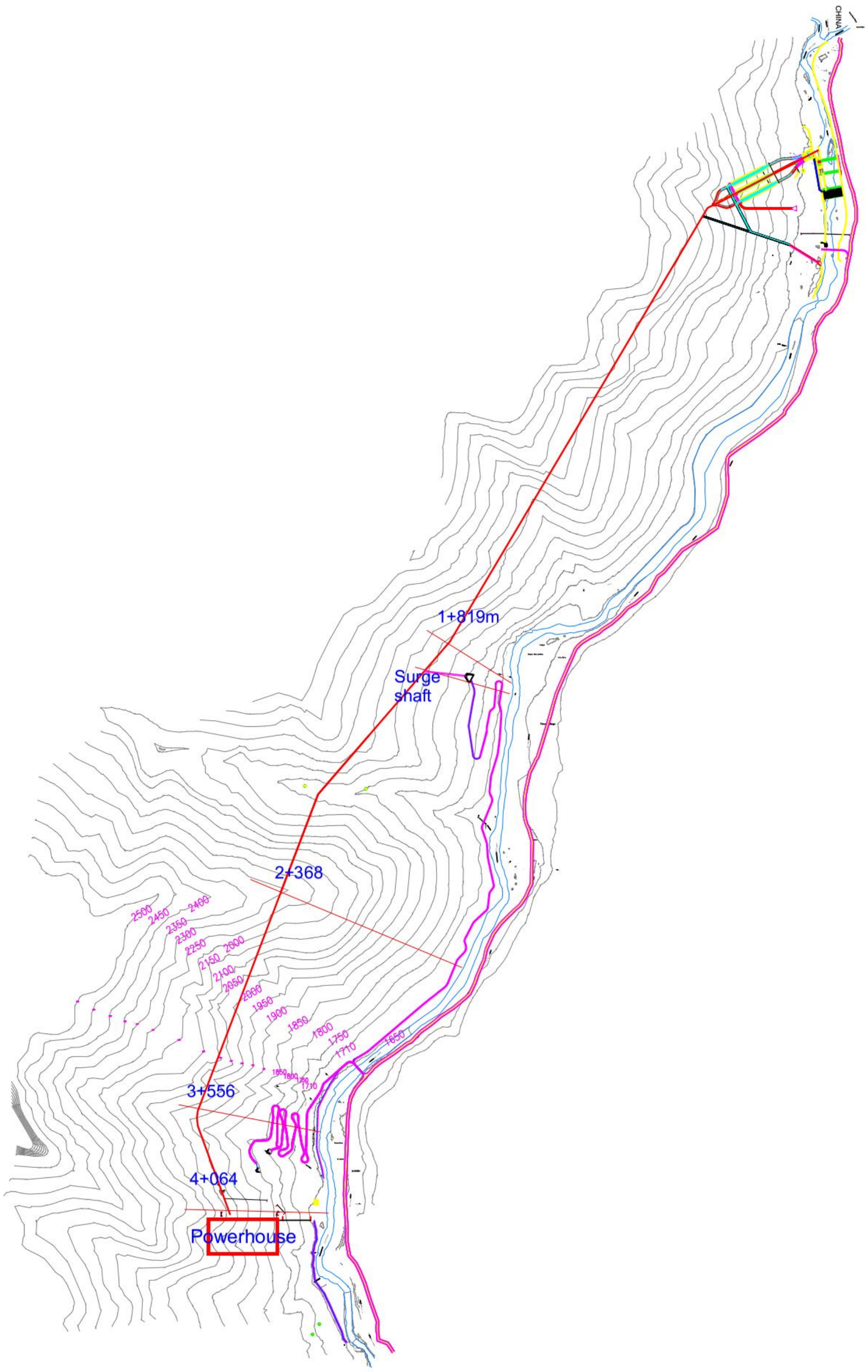
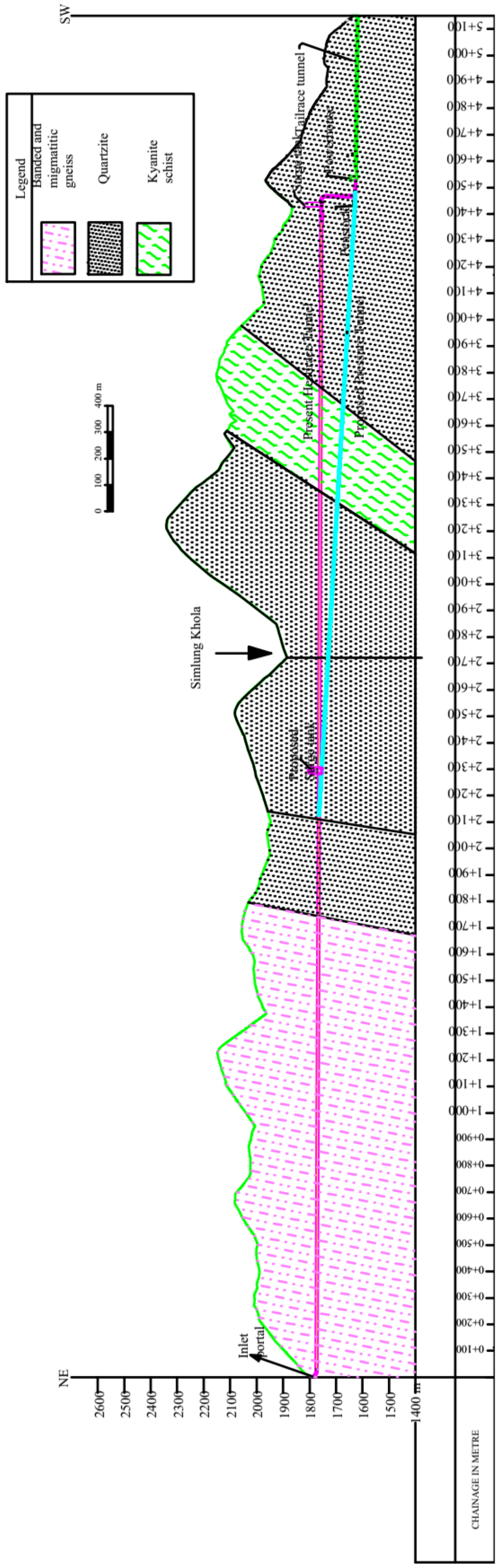
APPENDIX A: PROJECT RELATED DRAWINGS

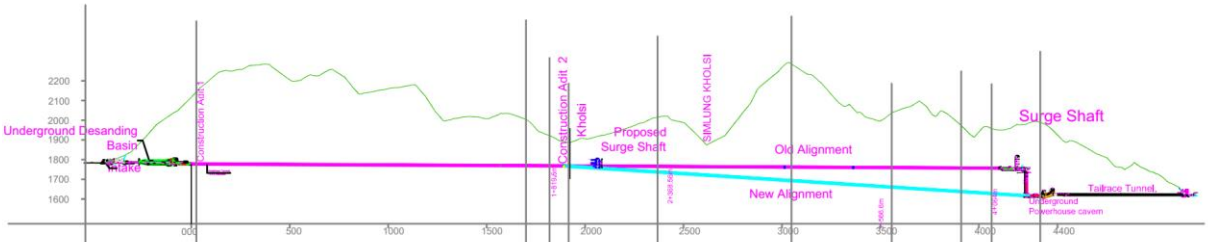
APPENDIX A1: Plan of the Rasuwagadhi Hydroelectric Project, Nepal.

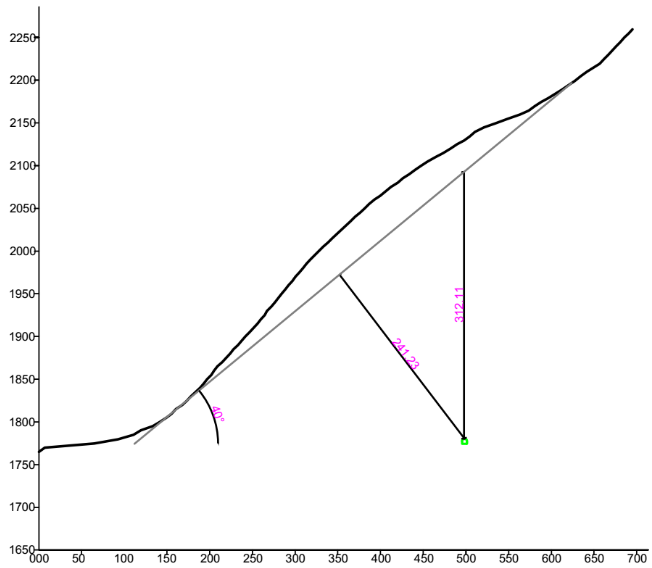


APPENDIX A2: Geological Profile of the Rasuwagadhi Hydroelectric Project, Nepal.

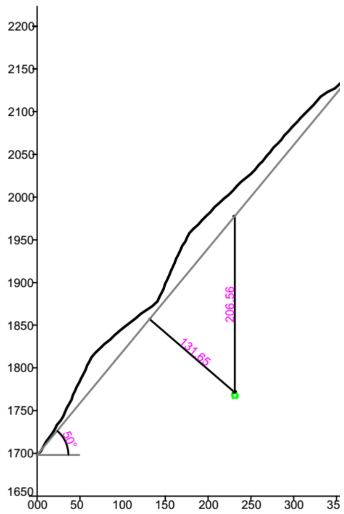


APPENDIX A2: Profile and Cross section of the Case

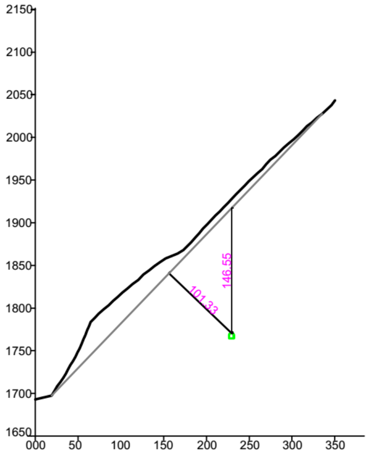




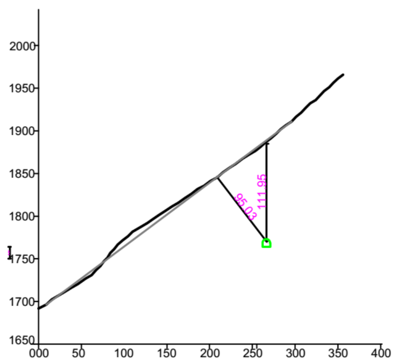
CROSS SECTIONS AT 0+040M



CROSS SECTIONS AT 1+700M

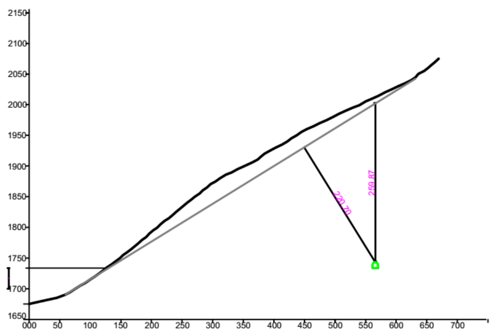


CROSS SECTIONS AT1+819.6M

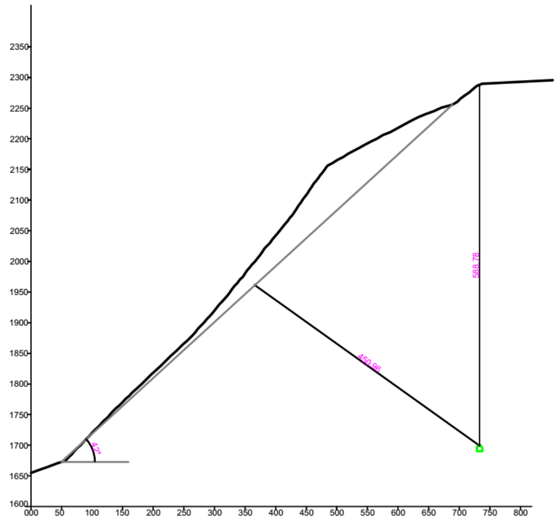


CROSS SECTIONS AT 1+916.1M

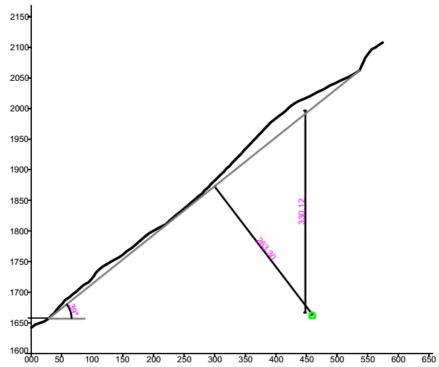
APPENDIX A2: Profile and Cross section of the Case



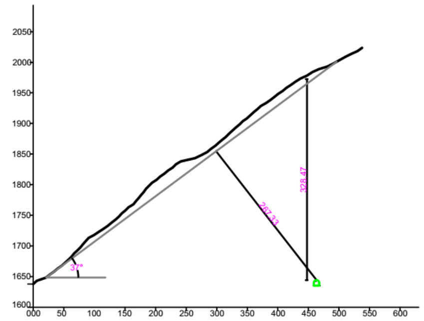
CROSS SECTIONS AT 2+368.56M



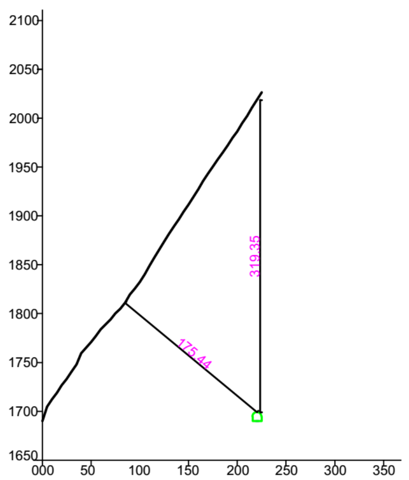
CROSS SECTIONS AT 3+048.14M



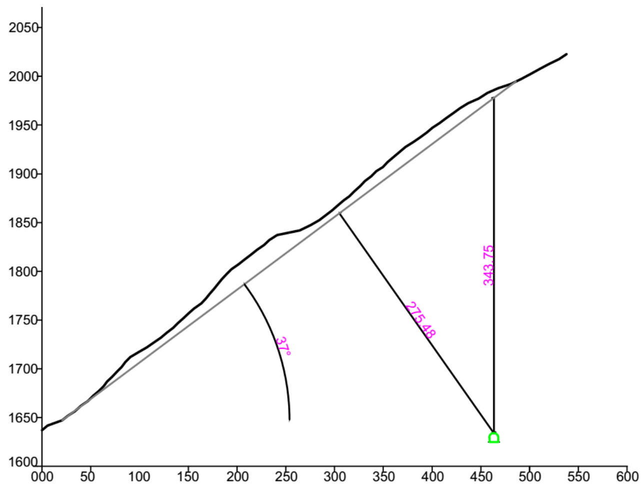
CROSS SECTIONS AT 3556.61M



CROSS SECTIONS AT 3+910.58M



CROSS SECTIONS AT Newshaft



CROSS SECTIONS AT 4+064.31M

APPENDIX B: LAB TEST RESULTS

Appendix B1: Specific weight and Sonic velocity Test Report

Specific Weight

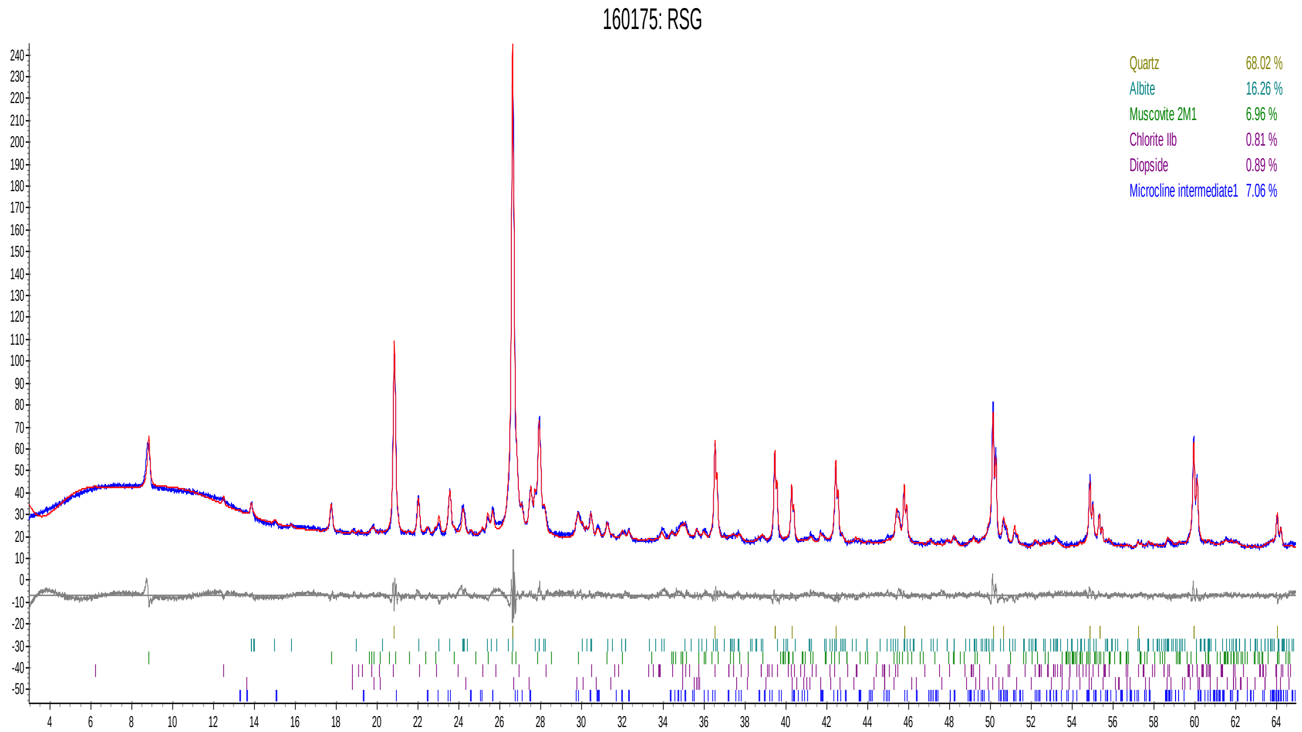
			Natural		Dry		Saturated	
Sample	Avg D (mm)	Length (mm)	Weight (gm)	Density (gm/cc)	Weight (gm)	Density (gm/cc)	Weight (gm)	Density (gm/cc)
R1	36.293	90.02	246.7	2.65	246.32	2.65	247.04	2.65
R2	36.285	90.02	246.56	2.65	246.25	2.65	246.97	2.65
R3	36.232	89.96	246.28	2.65	245.96	2.65	246.66	2.66
R4	36.252	90.02	247.03	2.66	246.66	2.66	247.39	2.66
R5	36.268	89.98	246.78	2.65	246.43	2.65	247.14	2.66
R6	39.868	102.49	339.31	2.65	338.88	2.65	339.78	2.66
Average				2.65		2.65		2.66

Sonic Velocity

Sample	Avg D (mm)	Length (mm)	Time (µs)	Sonic Velocity (m/s)	Travelling Time (µs)	Sonic Velocity (m/s)	Travelling Time (µs)	Sonic Velocity (m/s)
R1	36.293	90.02	21.4	4206.5	27.9	3226.5	17.4	5173.6
R2	36.285	90.02			28.1	3203.6	17.5	5144.0
R3	36.232	89.96	18.9	4759.8	27.7	3247.7	17.4	5170.1
R4	36.252	90.02			28.4	3169.7	17.5	5144.0
R5	36.268	89.98			28.4	3168.3	17.9	5026.8
R6	39.868	102.49			32.4	3163.3	19.9	5150.3

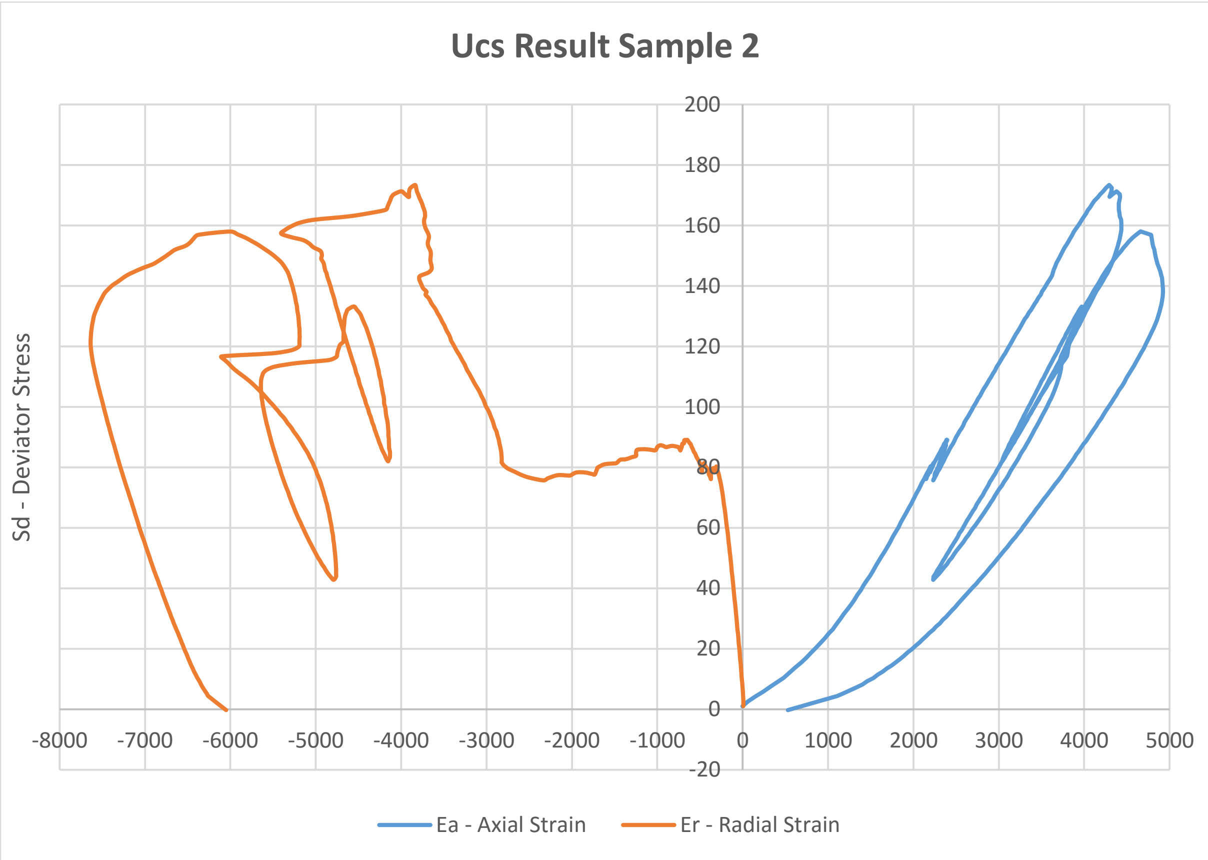
Appendix B2: Minerological Analysis test Report

Description	Percentage
Quartz	68.02%
Plagioclase	16.26%
Mica	6.96%
Pykoxene	<1%
Alkali Feldspars	7.06%
Chlorite	<1%

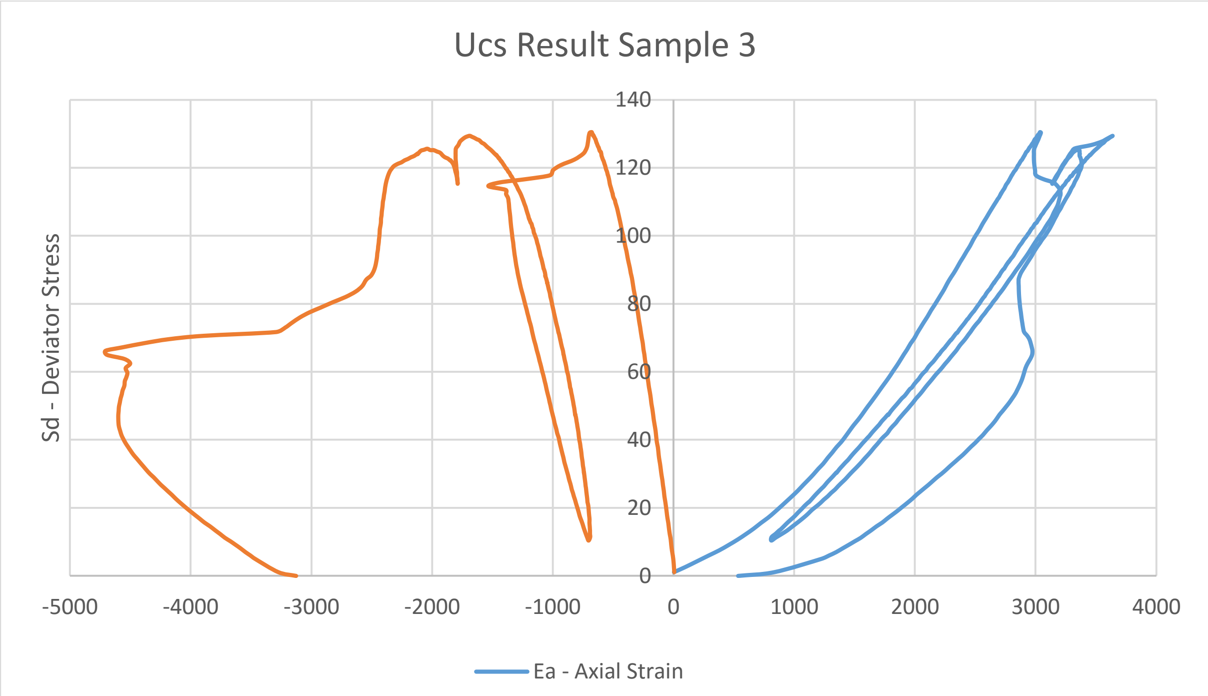


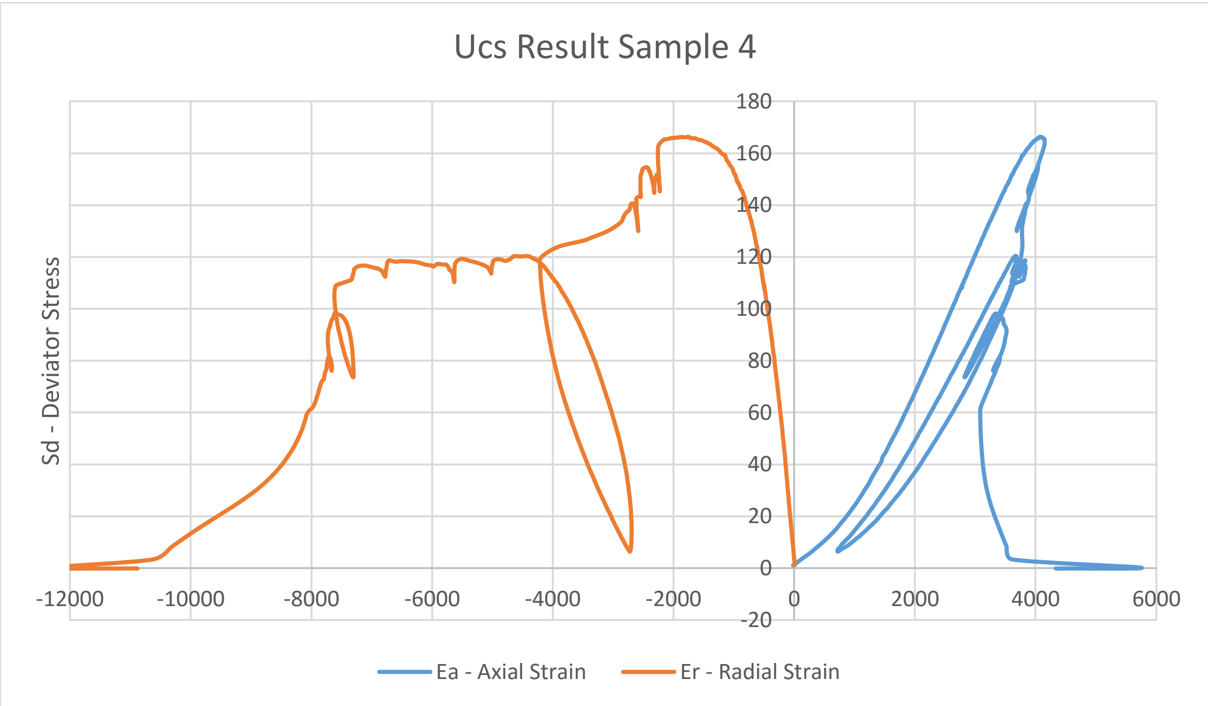
Appendix B3: UCS Test Results

Sample	**Avg D (mm)**	**Length (mm)**	**Gauge length Lo (mm)**	**UCS (MPa)**	**E-modul (GPa)**	**Poissons Ratio**	**Tangent Point (MPa)**	**Failure angle**
R1	36.293	90.02	74	178.2	53.02	0.21	90.6	28
R2	36.285	90.02	73.2	173.4	52.29	0.26	68.1	
R3	36.232	89.96	72.7	130.5	58.52	0.29	82.6	
R4	36.252	90.02	76.7	166.4	52.87	0.28	84.1	23
R5	36.268	89.98	76.4	172.9	54.96	0.23	86.9	21
R6	39.868	102.49	73.1	173.6	54.99	0.26	87.5	26
Average				172.9	53.626	0.248		

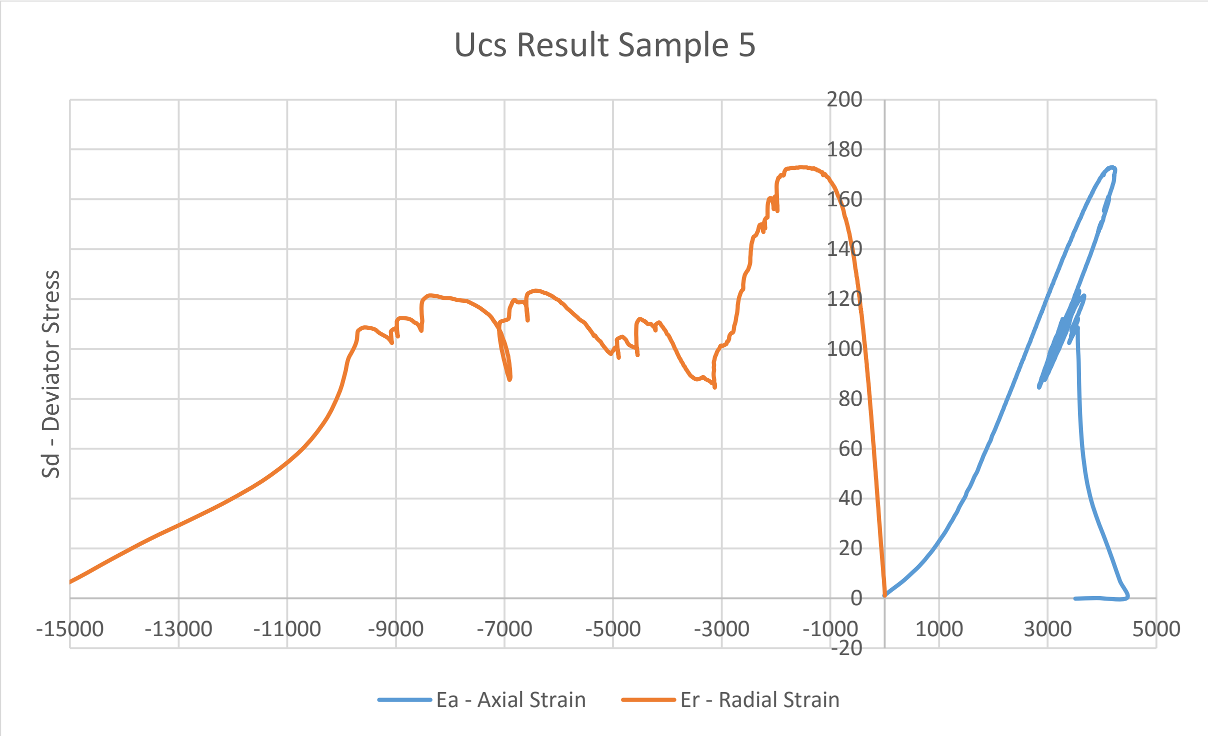


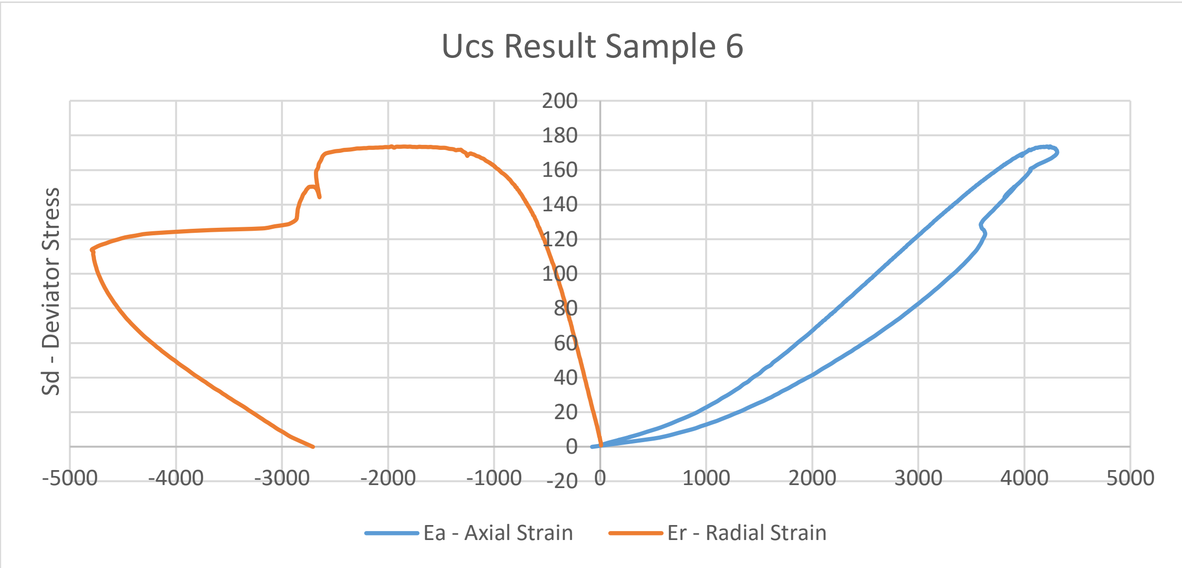
Appendix B3: UCS Test Results





Appendix B3: UCS Test Results





Appendix C: Data and Calculation

Appendix C1: Optimization of Pressure Tunnel

Pressure Tunnel Optimization of Rasuwagadhi Hydroelectric Project

Turbune Discharge (qmax)	80.00	m^3/s
mean turbine discharge	0.00	m^3/s
Length of Pressure tunnel	1.00	m
Manning Coefficient (M)	30	
Head	168	m
Efficiency (n)	0.9	
Price of energy	6.6	Rs/Kwh
Rate of return	10 %	
Capital Recovery Factor	0.11	
O& M	10 %	
Project life	30	yrs
Optimised Area	72.5	m^2
Width (D)= √((8*A)/(4+π))	9.0	m
Cost of tunnel	4750	per meter

Velocity (V) m/s	1.4	1.3	1.2	1.1	1	0.9	0.8
Area (A) m^2	57.14	61.54	66.67	72.73	80.00	88.89	100.00
Width (D)= √((8*A)/(4+π))	8.00	8.30	8.64	9.03	9.47	9.98	10.59
Hydraulic Radius R = D/4	2.00	2.08	2.16	2.26	2.37	2.49	2.65
Head loss (H)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Capacity Loss Constant Cmax (Kw)	0.61	0.50	0.40	0.32	0.25	0.19	0.14
∫(q/qmax)^3*Δt	4071.701514						
Capacity loss over a year V*∫(q/qmax)^3*Δt	26873.22999						
Value of Head loss (Cmax*V*∫(q/qmax)^3*Δt) (MRs)	0.02	0.01	0.01	0.01	0.01	0.01	0.00
Cost of tunnel (MRs/m)	0.27	0.29	0.32	0.35	0.38	0.42	0.47
Total cost (MRs)	0.27	0.29	0.32	0.35	0.38	0.42	0.47
Annual Cost (MRs)	0.03	0.03	0.03	0.04	0.04	0.04	0.05
Cost increase Due to increase in area (MRs)	0.00	0.00	0.00	0.01	0.01	0.02	0.02
Total cost (MRs)	0.0164	0.0157	0.0157	0.0165	0.0182	0.0210	0.0253

Hydrological

Percentage of Time	0 %	10 %	20 %	30 %	40 %	50 %	60 %	70 %	80 %	90 %
Time	0	876	1752	2628	3504	4380	5256	6132	7008	7884
Discharge (q)	200.00	200.00	162.00	120.00	80.00	39.45	22.64	12.66	8.37	5.76
q/qmax	1.00	1.00	1.00	1.00	1.00	0.49	0.28	0.16	0.10	0.07
(q/qmax)^3	1.00	1.00	1.00	1.00	1.00	0.12	0.02	0.00	0.00	0.00

Appendix C2: Analysis of Hydraulic Fracturing and stress calculation.

Hydraulic Fracturing Calculation

$$h > \frac{H * \Upsilon w}{\Upsilon r * cos\alpha} \qquad l > \frac{H * \Upsilon w}{\Upsilon r * cos\beta}$$

Density of rock 2.654

Intake elevation 1785.5

SN	Chainage	Head (H)	α	β	Calculated		Available at site		Factor of Safety		Remark
			Degree	Degree	Vertical cover h (m)	Shortest Distance l (m)	Vertical cover h (m)	Shortest Distance l (m)	Vertical cover h	Shortest Distance l	
1	0+050	11.2	0.3	40	4.2	5.5	312.1	241.2	74.3	44.0	Safe
2	1+700	20.3	0.3	50	7.6	11.9	206.6	131.7	27.0	11.1	Safe
3	1+819.6	20.9	0.3	46	7.9	11.3	146.6	101.3	18.6	8.9	Safe
4	1+916.1	21.5	3.7	37	8.1	10.1	115.0	95.0	14.2	9.4	Safe
5	2+064.8	45.5	3.7	56	17.2	30.7	319.4	175.4	18.6	5.7	Safe
6	2+368.5	51.7	3.7	32	19.5	23.0	262.9	223.3	13.5	9.7	Safe
7	3+048.1	95.3	3.7	42	36.0	48.3	588.8	451.0	16.4	9.3	Safe
8	3+556.6	127.9	3.7	39	48.3	62.0	330.1	263.3	6.8	4.2	Safe
9	3+910.7	150.6	3.7	37	56.9	71.1	328.5	267.3	5.8	3.8	Safe
10	4+064.3	160.5	3.7	37	60.6	75.7	343.8	275.5	5.67	3.64	Safe

Appendix C2: Analysis of Hydraulic Fracturing and stress calculation.

Poissions ratio (ν)		0.248												
ν/1-ν		0.33												
Chainage	Rock Cover	Stress (Mpa)				Bearing of tunnel alignment	Direction of tectonic stress	plane angle between Tunnel alignment and tectonic stress	Total (Mpa)		Locked stress (Mpa)		Stress Ratio
		Vertical (σv)	Horizontal (σh)	Tectonic **(σt)**	Total Horizontal (σH)				In plane	Out plane	In plane	Out plane	
1+819.6	146.55	3.89	1.28	7.5	8.78	30	8	22	7.73	2.33	6.45	1.05	1.99
Surge shaft	114.96	3.05	1.01	7.5	8.51	30	8	22	7.45	2.06	6.45	1.05	2.44
2+368.5	262.88	6.98	2.30	7.5	9.80	41	8	33	7.57	4.52	5.28	2.22	1.09
3+556.6	330.12	8.76	2.89	7.5	10.39	21	8	13	10.01	3.27	7.12	0.38	1.14
4+064.3	343.75	9.12	3.01	7.5	10.51	20	8	28	8.85	4.66	5.85	1.65	0.97

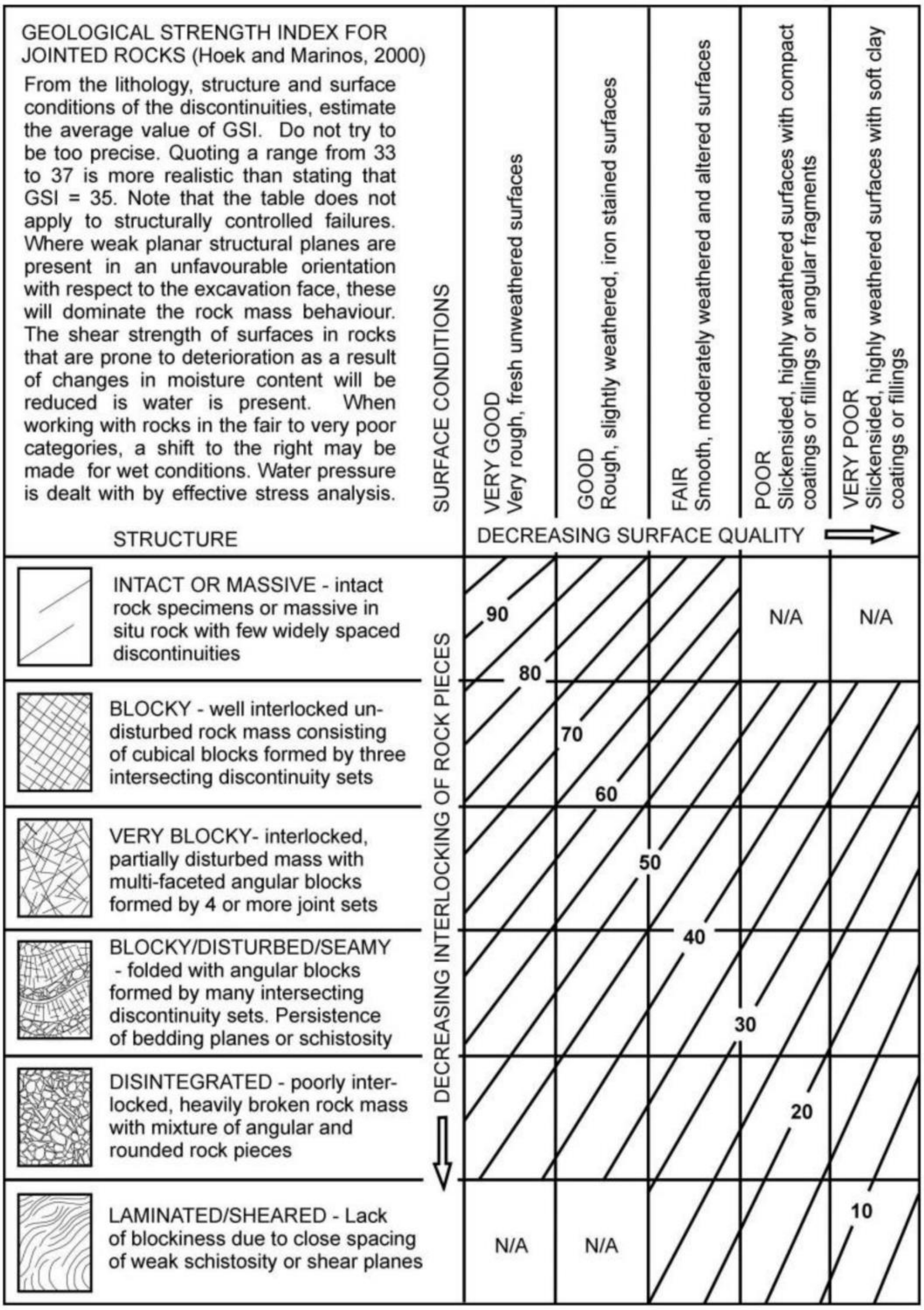
Appendix D1: Field estimates of uniaxial compressive strength of intact rock (Marinos and Hoek, 2000)

Table 1: Field estimates of uniaxial compressive strength of intact rock.[3]

Grade*	Term	Uniaxial Comp. Strength (MPa)	Point Load Index (MPa)	Field estimate of strength	Examples
R6	Extremely Strong	> 250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100 - 250	4 - 10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, peridotite , rhyolite, tuff
R4	Strong	50 - 100	2 - 4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, sandstone, schist
R3	Medium strong	25 - 50	1 - 2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Concrete, phyllite, schist, siltstone
R2	Weak	5 - 25	**	Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer	Chalk, claystone, potash, marl, siltstone, shale, rocksalt,
R1	Very weak	1 - 5	**	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife	Highly weathered or altered rock, shale
R0	Extremely Weak	0.25 - 1	**	Indented by thumbnail	Stiff fault gouge

* Grade according to Brown (1981).
** Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results.

Appendix D2: Geological Strength Index (GSI)-SYSTEM



Appendix D3: Disturbance factor, D

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

Appendix D4: RMR Classification of Rock Mass

A. Classification parameters and their ratings

	Parameters		Range of values or ratings						
1	Strength of Intact Rock	Point load strength index (MPa)	> 10	4 - 10	2 - 4	1 - 2	Low range uniaxial strength is preferred		
		Uniaxial compressive strength (MPa)	> 250	100-250	50-100	25-50	5- 25	1 - 5	< 1
		Rating	**15**	**12**	**7**	**4**	**2**	**1**	**0**
2	Drill core quality, RQD (%)		90-100	75-90	50-75	25-50	< 25		
		Rating	**20**	**17**	**13**	**8**	**5**		
3	Spacing of discontinuities (m)		> 2	0.6-2	0.2-0.6	0.06-0.2	< 0.06		
		Rating	**20**	**15**	**10**	**8**	**5**		
4	Condition of discontinuities	Length, persistence (m)	< 1	1-3	3-10	10-20	> 20		
		Rating	**6**	**4**	**2**	**1**	**0**		
		Separation (mm)	none	< 0.1	0.1-1	1-5	> 5		
		Rating	**6**	**5**	**4**	**1**	**0**		
		Roughness	very rough	rough	slightly rough	smooth	slickensided		
		Rating	**6**	**5**	**3**	**1**	**0**		
		Infilling (gouge) (mm)	none	hard filling		soft filling			
				< 5	> 5	< 5	> 5		
		Rating	**6**	**4**	**2**	**2**	**0**		
		Weathering	un-weathered	slightly weathered	moderately weathered	highly weathered	decomposed		
		Rating	**6**	**5**	**3**	**1**	**0**		
5	Ground water	Inflow per 10 meter tunnel length (l/min)	none	< 10	10-25	25-125	> 125		
		ρ_w / σ_1	0	0.0.1	0.1-0.2	0.2-0.5	> 0.5		
		General conditions	dry	damp	wet	dripping	flowing		
		Rating	**15**	**10**	**7**	**4**	**0**		
		here, ρ_w is joint water pressure and σ_1 is major principle stress							

B. Rating adjustment for discontinuity orientation

Tunnel alignment	very favorable	favorable	fair	unfavorable	very unfavorable
Rating adjustment	**0**	**-2**	**-5**	**-10**	**-12**

C. Rock mass classes determined from total ratings

Rating	100-80	80-61	60-41	40-21	< 20
Class No.	I	II	III	IV	V
Description	Very good	Good	Fair	Poor	Very poor

D. Meaning or rock mass classes

Class No.	I	II	III	IV	V
Average stand-up time	Can be estimated from Figure 4-4				
Cohesion of the rock mass (kPa)	> 400	3-400	2-300	1-200	< 00
Friction angle of the rock mass (degrees)	< 45	35-45	25-35	15-25	< 15

Appendix D5: Description of ratings for input parameters and chart of Q-system (based on Barton, 2002)

RQD (Rock quality designation, %)	
Very poor	0 - 25
Poor	25 - 50
Fair	50 – 75
Good	75 – 90
Excellent	90 - 100

Notes:
(i) where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.
(ii) RQD intervals of 5 i.e. 100, 95, 90 etc., are successfully accurate.

J_n (Joint set number)	
Massive, no or few joints	0.5 - 1
One joint set	2
One joint set + random joints	3
Two joint sets	4
Two joint sets + random	6
Three joint sets	9
Three joint sets + random	12
Four or more joint sets, heavily jointed, sugar cube etc	15
Crushed rock, earthlike	20

Note: For tunnel intersections, use (3 x J_n) and for portals use (2 x J_n)

J_r (Joint roughness number)

(a) Rock wall contact	
Discontinuous joints	4
Rough or irregular, undulating	3
Smooth, undulating	2
Slickensided, undulating	1.5

(b) Rock wall contact before 10 cm shear	
Rough or irregular, undulating	1.5
Smooth, undulating	1
Slickensided, undulating	0.5

© No rock wall contact when sheared	
Zone containing clay minerals thick enough to prevent rock wall contact	1
Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1

Notes: (i) Description refers to small-scale features and intermediate scale features, in that order (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m. (iii) Jr = 0.5 can be used for planner, slickenside joints having lineations, provided these are oriented for minimum strength. (iv) Jr and Ja classification is applied to the joint set that is least favorable for stability both from the point of view of orientation and shear resistance, $\tau \approx \sigma_n \cdot \tan^{-1} (J_r/J_a)$

J_a (Joint alteration number)

(a) Rock wall contact (no mineral fillings, only coatings)	*$ø_r$ (appr.)*	***J_a***
Tightly healed, hard, non-softening, impermeable filling i.e., quartz/epidote	-	0.75
Unaltered joint walls, surface staining only	25 - 35	1
Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay free disintegrated rock ,etc.	25 - 30	2
Silty or sandy clay coatings, small clay fractions (non-softening)	20 - 25	3
Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talk, gypsum, graphite etc., and small quantities of swelling clay	8 - 16	4
(b) Rock wall contact before 10 cm shear (thin mineral fillings)		
Sandy particles, clay free disintegrated rock etc.	25 - 30	4
Strongly over-consolidated non-softening clay mineral fillings (continuous, but < 5mm thickness)	16 - 24	6

Medium or low over-consolidated non-softening clay mineral fillings (continuous, but < 5mm thickness)	12 - 16	8
Swelling clay fillings, i.e., montmorillonite (continuous, but < 5mm thick)	6 - 12	8 - 12
(c) No rock wall contact when sheared (thick mineral fillings)		
Zones or bands of disintegrated or crushed rock and clay	6 - 24	6, 8 - 12
Zones or bands of silty or sandy clay, small clay fraction (non-softening)	-	5
Thick, continuous zones or bands of clay	6 - 24	13 - 20

***J_w* (Joint water reduction factor)**	*Approx. P (bars)*	***J_w***
Dry excavations or minor inflow, i.e., < 5 l/min locally	< 1	1
Medium inflow or pressure, occasional outwash of joint fillings	1 – 2.5	0.66
Large inflow or high pressure in competent rock with unfilled joints	2.5 - 10	0.5
Large inflow or high pressure, considerable outwash of joint fillings	2.5 - 10	0.33
Exceptionally high inflow or pressure at blasting, decaying with time	> 10	0.2 – 0.1
Exceptionally high inflow or pressure continuing without noticeable decay with time	> 10	0.2 – 0.1

Notes: (i) The last four factors are crude estimates. Increase J_w if drainage measures are installed. (ii) Special problems caused by ice formation are not considered. (iii) For general characterization of rock masses distance from excavation influences. The use of $J_w = 1$, 0.66, 0.5, 0.33, etc. as depth increases from say 0-5, 5-25, 25-250 to >250m is recommended, assuming that RQD/J_n is low enough (0.5-25) for good hydraulic connectivity.

***SRF* (Stress Reduction Factor)**

(a) Weakness zones intersecting excavation, which may cause loosening of rock mass	*SRF*
Multiple occurrence of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock at any depth	10
Single weakness zone containing clay or chemically disintegrated rock (depth ≤ 50m)	5
Single weakness zone containing clay or chemically disintegrated rock (depth > 50m)	2.5
Multiple shear zones in competent rocks (clay free), loose surrounding rock at any depth	7.5
Single shear zone in competent rocks (clay free), (depth of excavation ≤ 50m)	5
Single shear zone in competent rocks (clay free), (depth of excavation > 50m)	2.5
Loose, open joints, heavily jointed or sugar cube etc. at any depth	5

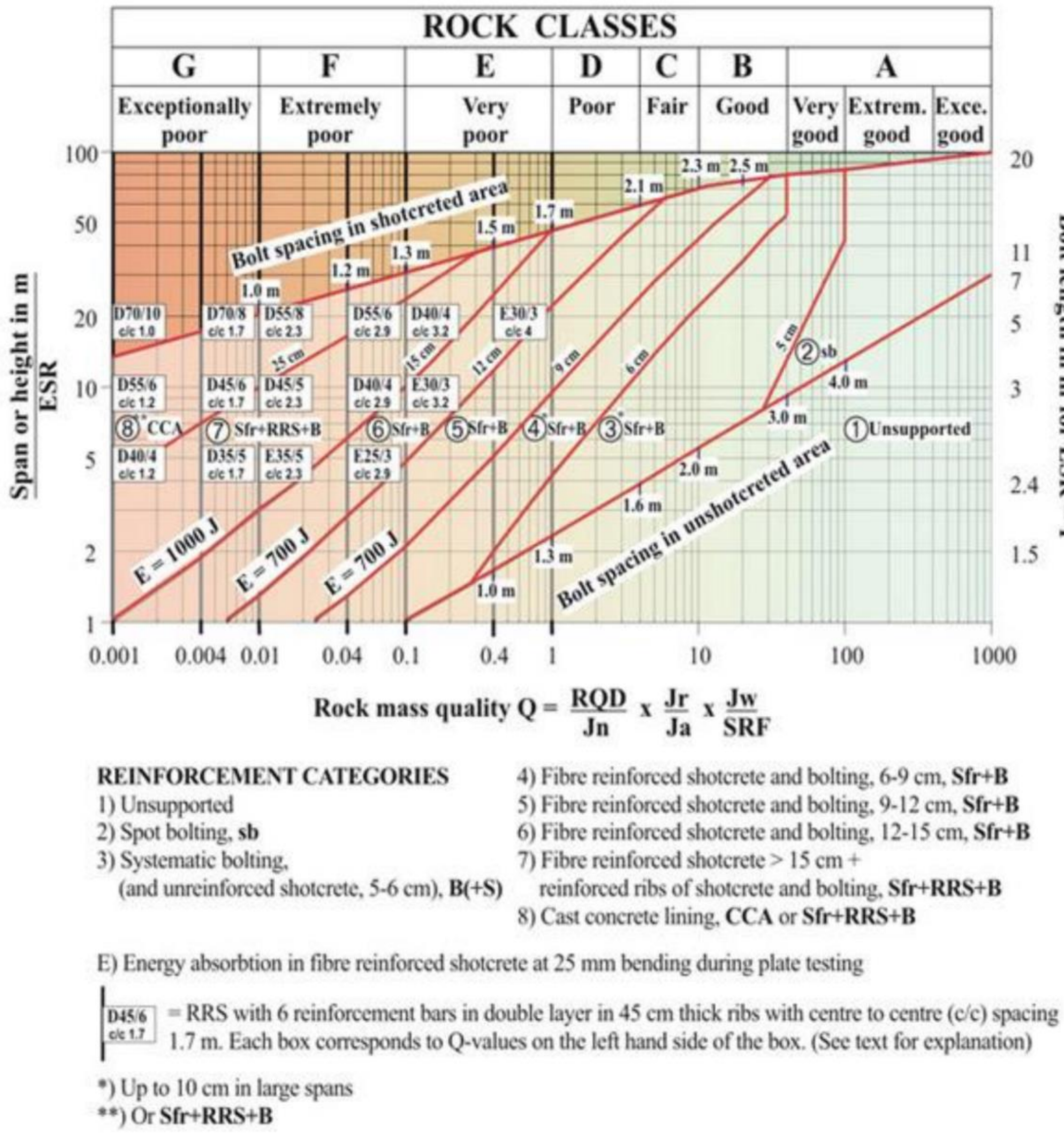
Note: Reduce these values of SRF by 25 – 50 % if the relevant shear zones only influence but do not intersect the excavation.

(b) Competent rock, rock stress problems	σ_c / σ_1	σ_t / σ_c	*SRF*
Low stress, near surface, open joints	> 200	< 0.01	2.5
Medium stress, favorable stress condition	200 - 10	0.01 – 0.3	1
High stress, very tight structures. Usually favorable to stability, may be unfavorable for wall stability	10 - 5	0.3 – 0.4	0.5 - 2
Moderate slabbing after > 1 hour in massive rock	5 - 3	0.5 – 0.65	5 - 50
Slabbing and rock burst after a few minutes of excavation	3 - 2	0.65 - 1	50 - 200
Heavy rock burst and immediate dynamic deformations	< 2	> 1	200 – 400

Notes: (i) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1 / \sigma_3 \leq 10$, reduce σ_c *to* $0.75\ \sigma_c$ and when $\sigma_1 / \sigma_3 > 10$, reduce σ_c *to* $0.5\ \sigma_c$. (ii) For general characterization of rock mass, overburden from excavation influences. The use of SRF 5, 2.5, 1 and 0.5 is recommended as depth increases from say 0-5, 5-25, 25-250 to > 250m respectively.

© Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure	σ_t / σ_c	*SRF*
Mild squeezing rock pressure	1 - 5	5 - 10
Heavy squeezing rock pressure	> 5	10 - 20

(d) Swelling rock: chemical swelling activity depending on pressure of water	*SRF*
Mild swelling rock pressure	5 - 10
Heavy swelling rock pressure	10 - 15



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

APPENDIX E: APPROVAL FROM RASUWAGADHI HEP

RASUWAGADHI
HYDROPOWER CO. LTD.
रसुवागढी हाइड्रोपावर कं.लि.

P.L. Regd. No.: 1303/068/069
Lazimpat, Kathmandu

Ref. No.: 072/73 : 231
प. सं. :

Date: 27 January, 2016

To,

The Department of Geology and Mineral Resources Engineering

Norwegian University of Science and Technology, (NTNU)

Trodheim, Norway

Ref: Data collection for MSc Thesis Work

With the request letter from NTNU for access to relevant data for the purpose of MSc Thesis works to Mr. Ramesh Kumar K.C. in NTNU that will focuses on assessment of engineering geological condition at Rasuwagadhi Hydroelectric Project, it is our great pleasure to state that he can use the relevant data for his thesis works particularly for the sole academic purpose.

We hope that Mr. K.C. will be able to come with fruitful conclusion through his thesis work on Rock Engineering focused on our Project. We hope a copy of his dissertation after completion of his thesis works.

Sincerely

....

Kiran Kumar Shrestha

Project Manager

Rasuwagadhi Hydroelectric Project