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Optimization and Stability Analysis of Waterway System and Underground Powerhouse Cavern for Himchuli Dordi HPP, Nepal

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Hydropower Development
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TGB4910 Rock Engineering - MSc thesis
for
Bikram Thapa

OPTIMIZATION AND STABILITY ANALYSIS OF WATERWAY SYSTEM AND UNDERGROUND POWERHOUSE CAVERN FOR HIMCHULI DORDI HPP, NEPAL

Background

The Government has considerable ambition to develop the hydropower potential that lies in Nepal. The investment environment in the hydropower sector is improving every year with more favourable conditions for the hydropower developers and investors. As a result, in recent years, the planning and development of hydropower projects got quantum momentum in the country. Maximum efforts are being made to develop the hydropower projects with local financing and initiative, especially for the small and medium scale projects (less than 200MW capacity). The 56MW Himchuli – Dordi Hydropower project is one of such initiatives, which is being currently under prefeasibility study by the project owner People's Hydropower Company Pvt. Ltd.

This MSc-thesis is related to the lay-out design, optimization and stability analysis of the underground works of the Himchuli Dordhi HPP. 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 Himchuli Dordi HPP
- Investigate and assess the engineering geological aspects along the waterway system and at the powerhouse cavern
- Compare the layout design, carry out optimization study of the two different alternatives and suggest alternative alignment if you feel necessary.

- Carry out stability assessment of the powerhouse cavern using both analytical and numerical analysis
- Discussion on the long term stability of the pressure tunnel system and conclude the work

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 received from the supervisors and 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 12, 2018 and to be completed by June 11, 2018.

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

January 12, 2018

Krishna Panthi

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

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ABSTRACT

Potential for hydropower development in Nepal is well established, and exploitation of the resource with economical and sustainable design is the need of the time. Unlined pressure tunnel is proven safe and economical waterway system in hydropower if it can be resided in good geological location. Project case for the thesis is Himchuli-Dordi HPP (56 MW) located in Higher Himalayan region of Nepal, near the regional thrust known as Main central thrust (MCT). Different alternatives for unlined waterway is devised considering the geological and accessibility perspective. Cavern orientation is also prepared for each alignment in connection with the jointing. Rock mass available in this zone are intensely sheared and mylontized. Weathering effect and tectonic forces are equally active in this region. Some of the common key issues like squeezing, leakage, tunnel collapse is identified from the completed projects from these regions. The purpose of this thesis is to analyze the unlined pressure tunnel waterway system and powerhouse cavern for the case project.

In this thesis, to understand the mechanism of unlined tunneling literature review on various methods is done. Norwegian Rule of thumb, Numerical methods are some of the used and discussed methods in this thesis. Another aspect of this thesis is to understand the unlined tunneling in context of Himalayan geology. Review of some of the completed projects which are similar to case project is discussed in order to narrow the analysis area. Using the Numerical model RS^2, valley model of the critical locations identified from analytical and semi-analytical methods are developed. The same software is used to analyze the stability of the tunnel opening for different support systems. The support system of systematic bolting and Shotcrete lining devised as per the rock class type for the project case proves to be sufficient in most of the chainage sections considered. Some of the section located in geologically challenging places like; a valley, has higher hydrostatic pressure than the available minimum principal stress and, so could not stabilize with the established support system. Such zones are suggested to go for full concrete lining until the safe zone is reached. During optimization, it was found that the combination of an unlined tunnel in relatively good geological formation and fully concrete lined tunnel in the critical section can lower the overall cost of the system. However, the case will turn opposite if full concrete lining requirement is increased. The cheapest option then will be to limit fully concrete lined tunnel as short as possible. Despite the limited geological information on the case, results can still be used as the indicator to the key issues that can be expected in such geological conditions. By increasing the database of information from precise and staged geological investigation will open the doors for an optimum solution in future

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1 Introduction

1.1 General

Nepal is a small country in South Asia which has a width ranging from 150 to 250 km north-south extending along 890 km along east-west. In such limited area, it has the varying altitude of 60 m in the south to 8848 m (Mt. Everest) in the north above sea level (Alam et al., 2017; Panthi, 2006). So, Nepal is full of rough terrain and mountainous topography all over the country.

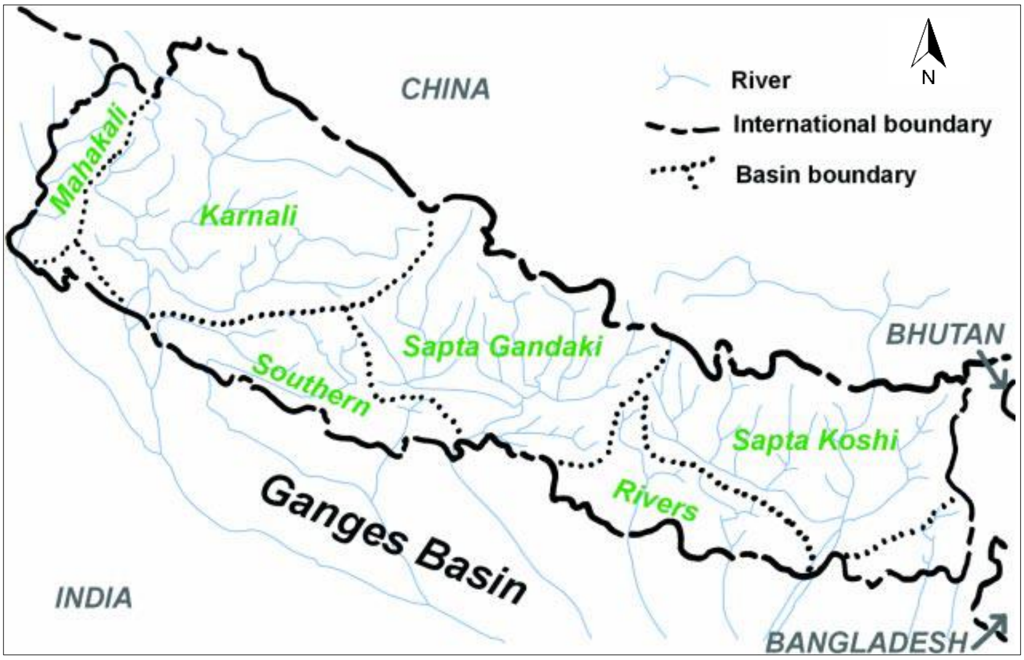


Figure 1.1 Major Rivers, tributaries and the basin covered (Pokharel, 2001)

Nepal largely relies on energy sources like forest and water (Pokharel, 2001). Forest being easily accessible near human civilization is being used from generation despite the many effects it generates. Whereas water is less utilized besides some obvious sectors like drinking water, irrigation etc. The abundant water flowing from the Himalayas (Figure 1.1), and the potential it carries along with (Table 1.1) makes hydropower as one of the best possibility. In recent years, Hydropower development is constantly increasing, despite the challenges it has with rough terrain and complex geology. With the constant shift in the technology, method of execution and ongoing research in similar geology, working conditions in Himalaya is also changing and becoming more navigational.

Table 1.1 Estimated Hydropower Potential (Pokharel, 2001)

Basin	Annual flow (in billions of m^3)	Catchment area (in km^2)	Potential (in MW)
Sapta Koshi	33	28,140	22,350
Sapta Gandaki	50	31,600	20,650
Karnali	42	41,890	32,010
Mahakali	7	5410	4160
Southern Rivers	42	40,141	4110
Total	174	147,181	83,280

According to NEA (2017), Nepal has hydropower of installed capacity 850 MW, in addition to that many projects are either in construction or study phase. With growing demand for energy and competitive markets, it is a challenge to come out with economical and structurally stable solutions. Another important characteristic of Nepal Geology is its location in a seismically active region. In recent earthquake events, many surface structures in Hydropower were affected but the underground structures were less damaged. Besides underground opening as per Persson (1987) has benefits with respect to economic reasons, good protection against war, sabotage, ice problems, landslide etc., and limited impact on the environment. So, use of underground space technology due to cost optimization, proper scheduling, and flexibility in design has become an important and necessary innovation in a recent time in Nepal and all over the world. Himchuli-Dordi Hydropower Project is one of many projects that is being constructed in Nepal considering the challenges with foreseeable benefits.

1.2 The need of underground space in Nepal

According to Panthi (2006), water conveying tunnels, transport tunnels, mining, and food storage facilities are the aspects of development that can be explored with underground technology in Nepal. From 1911 when first hydropower was installed to present time, many technologies have shifted from surface steel penstock, surface channels, underground waterways to fully underground powerhouse system. However, hydropower development is growing rapidly, which includes headworks, waterways, powerhouse, and transmission. Among other sections waterway system is a major investment for the project, so looking for the solution that could optimize the cost is the primary issue. Fully lined is a solution but costlier one. Unlined/ shotcrete lined pressure tunnels has turned out as an economic solution if the rock mass and the support provided will incur necessary safety and stability in the long run(Basnet and Panthi, 2017).

Whatever may be the option, instabilities are anticipated when going underground. So, it needs to be addressed properly and be safe and prepared during project execution. According to (Panthi, 2006), in Nepal, factors determining the instabilities are generally categorized as geological and non-geological factors. Former one points to the frequent occurrence of geological complexity like weak rock mass, discontinuities, a high degree of weathering and fracturing, rock stresses and groundwater effect. Whereas non-geological factor is related to skill and experience achieved during the process, which will affect the decision and analysis on any project.

Multiple study and identification of potential hydropower generation from numerous rivers of Nepal (Table 1.1) are in progress. The need for underground space is about to excel in near future. The benefit of unlined technology from a simple design, reduction of the construction adits, reduction in capital and to earlier completion of the project (Palmstrom, 1987) cannot be ignored. Also, underground cavern provides the flexibility and space for the equipment to execute environmentally friendly design. Use of conventional lined tunnel system in difficult geology in combination with unlined tunnel and cavern in suitable geology opens for wholesome design. In compliance to that, Himchuli Dordi HPP comprises of unlined headrace tunnel, lined steel pressure shaft, and power-house cavern.

1.3 Objective of study

The main objective of the thesis is to analyze the layout design, optimization and stability analysis of waterway system and underground powerhouse cavern of the Himchuli-Dordi Hydropower project. The scope of the project is listed below.

- Review on planning and design aspects of the conventional and unlined waterway system
- Investigate the engineering geological aspect along the waterway system and powerhouse cavern
- Carry out the comparison of layout design for headrace tunnels and powerhouse cavern
- Carry out the stability assessment of the tunnel and powerhouse cavern
- Discussion on the long-term stability of the pressure tunnel system

1.4 Organization of the Thesis

Altogether there are 10 chapters and each chapter is developed in accordance with methodology applied in the process. Chapter 0 is on the introduction of the thesis topic its scope and objectives. Then from chapter 2 to 4 gives an account of literature review done for the underground openings. The process involves reviewing the design consideration for underground structures which is included in chapter 2. Rock Mass and Rock stress from chapter 3includes rock mass properties and estimating methods. Types of rock stresses and failure concept developed is also included in chapter 3. Chapter 4 presents the various assessment methods from analytical, semi-analytical to numerical that are used during analysis of unlined tunnels and power house cavern. Chapter 5 then investigates some of the completed and case related projects, which comply the designing principles and geological scenarios. A full description of the project for thesis topic is provided in chapter 6. Form this chapter onwards focus is fully on the case project, like the description of investigation methods, results obtained from it with the interpretation etc. Study of Layout alternatives for the case project is described in chapter 7. Chapter 0 gives us the account on powerhouse cavern for the chosen alternatives. Long-term stability issues regarding the alternatives selected from the point of study topic is put in chapter 9. And finally, the conclusion of the thesis with some recommendation that could be used for further study is provided in chapter 10.

1.5 Available information

Data for the project are limited to pre-feasibility study that is still being carried out, which includes: preliminary information, Auto cad drawings, geological maps of Nepal and the project area. The information on rock types and the test data was lacking for the site, so data has been estimated using the lab test report of the Super Dordi Hydropower Poject located just downstream of the proposed site. In addition, many information were gathered with the help of literature based on similar geology and similar projects, thesis reports etc. Some of the key information like rock class type and support estimation depending on those rock class is prepared after the discussion with supervisor and co-supervisor. The data thus generated could not be verified from the actual site because project is still in investigation stage and so they lack investigation data as well.

2 Description of Underground Opening

2.1 Background

There are many ways for water to be conveyed from headworks to power station in hydropower project. This concept is constantly changing to incorporate the new idea and needed feasibility. In hydropower, surface waterways and underground waterways are common ways, which are selected based on the type of project and the geological conditions. Surface waterways become unfeasible as project size and geological complexities increases. The relevance of underground opening comes into play then. In underground openings, both lined and unlined tunneling is a common practice, which largely relies on the type of geological condition it is positioned. Benefits like flexibility in construction, environmentally friendly, and safety against war makes it even more relevant to changing world conditions.

Norway has been practical in conceptualizing modern ideas and technology to improve the way conventional hydropower is developed. From the period 1917/1920 right after the first world war, when there was a shortage of the steel to use in waterway system of hydropower, they chose to go partially underground as an alternative. Later they went underground fully in the 1940s and got acquainted with the advantages of unlined pressure tunnels for the first time (Nilsen and Thidemann, 1993). In the journey of exploration, they found the basic requirement for going unlined waterways; i.e. geological condition and geo-tectonic environment (Panthi, 2014). Luckily Norwegian geology is comprised of hard rock from the old rock formation. In addition, due to deglaciation process that took place 10000 years ago took away all the weathered rock mass exposing the stable hard rock to the surface. Thus, eliminating the challenges that might have occurred due to the weathered condition of the rock. This favorable situation convinced them to design a certain criterion like minimum rock cover, consideration for topographic effect and other issues which are famously known as Norwegian design principles for unlined tunnels. The principles have proven successful and have been applied in other parts of the world outside Norway, with certain modification in the method depending on the geological complexity.

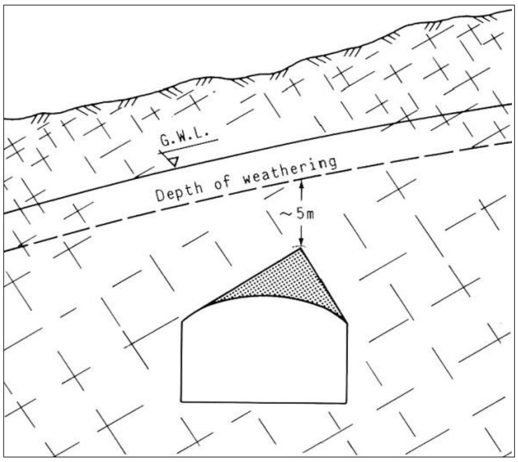
On the other hand, Nepal also has old rock formation except for Siwalik region, older than Paleozoic age. But surface weathering is intensive in most of the geological conditions; both topographical slopes and valleys in Himalaya. Thus, deep weathering is anticipated in the most projects. In the Himalayan region like Nepal, where the earthquakes de-stress the accumulated stresses due to the tectonic push between the plates forming it, makes the surrounding area even challenging to explore. So, a close study of the geotectonic environment is required to apply the Norwegian design principal in Himalayan region (Panthi, 2014). When applying Norwegian design principles primary concern is vertical gravitational stresses, but equal attention is also required on horizontal stress that is important in making an unlined pressure tunnel work. Thus, according to Panthi (2006) it is advisable to consider minimum principal stress as the deciding factor in designing unlined tunnels.

2.2 Design Consideration

According to Edvardsson and Broch (2002) an underground hydropower plant will consist of (i) headrace system which includes intake, headrace tunnel, brook intakes, surge chamber and pressure shaft tailrace system consisting of tailrace tunnel, outlet structures and in some cases surge chamber (iii) powerhouse with one or more caverns with system of tunnels and shafts for definitive purpose (iv) facilities to surface. Safe design of tunnels and cavern in every section depends on the results of the geological investigation carried out on the project site. Which ultimately decides the cost for overall underground construction. Key to successful tunneling is reflected from cost-effectiveness, selection of appropriate tunneling method and managing geological uncertainties (Panthi, 2017). Thus, careful attention in design is a primary need. Nilsen and Thidemann (1993) has pointed out four such primary areas for good design of underground openings which are explained further.

2.2.1 Site selection

The choice of location will be based on the type of rock that is to be encountered during excavation. In turn, it will govern the stability and feasibility of the project. The proper location will ease the decision maker during the construction and operation of the project. Like in shallow seated openings decision on minimum rock cover is a challenge because first designer must know the depth of weathering (Figure 2.1 Left) and then investigate for probable over break above the opening. 5 m of rock cover is accepted in hard rock for span limit of 20m (Nilsen and Thidemann, 1993). In shallow seated opening small rock stresses result in weak interlocking of the blocks. Whereas in deep-seated due to high stresses and anisotropic nature stability problems like squeezing, rock bursting and other stress-induced problems are anticipated. In deep-seated opening (Figure 2.1 right) the challenge is to locate a position that is distressed as in deep valleys, to avoid more rock stress problems. Another important point to consider is to make sure that opening is far away from the weakness zone and if the crossing is needed it is chosen with smallest one and as short as possible. So, while selecting the location for the underground complex intermediate position is advantageous.



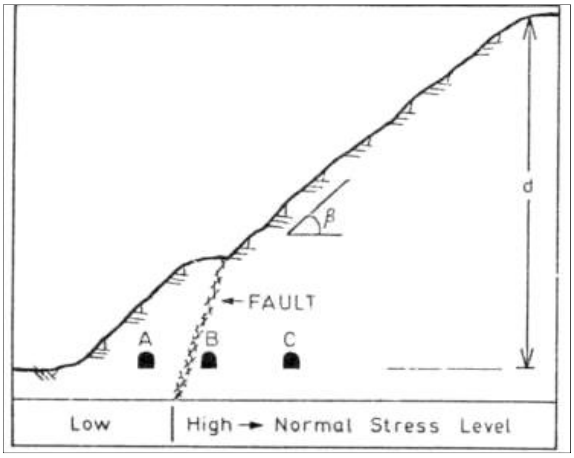


Figure 2.1 Minimum rock cover for shallow seated underground opening(left) and stress situation in valley side with fault zone (right)

2.2.2 Orientation of alignment and length axis

Optimization of the opening is more appropriate when comprehensive joint mapping and its location is selected. Orientation is proposed in such a way that stability problems and over break must be minimum as possible. For that major discontinuities in the rock mass must be identified and made sure it has less or no effect on the orientation of the opening. For the shallow opening, it is a basic rule to orient the alignment of opening at the maximum intersection angle between two predominant joint directions. However, in case of high rock stresses, the direction of principal stress also plays a role. The parallel orientation of tunnel alignment and length axis of the cavern with a major joint should be avoided and favorable orientation of tangential stress with major joint set should also be concerned. Making sure that tangential stress is oriented favorably with major joint sets resulting less over break.

2.2.3 Shaping

The design concept for underground openings is to distribute the compressive stresses evenly along the periphery. But the rock, in general, is discontinuous, ability to withstand tensile stress is low and largely depend on the shear strength of the discontinuities. So, we can only overpass such difficulty by sticking to a simple design with an arched roof, avoiding sharp intruding corners (Figure 2.2 bottom left). In shallow and intermediate openings shape is determined based on orientation, number of character of joints and foliation, and bedding partings. Whereas in deep-seated opening tensile stress might exceed the strength of rock, so small curvature radii are better avoided, and the shape of the opening should be designed in such a way that stress will concentrate locally. In doing so, areas that need support is reduced and special attention can be given to locally concentrated zone (Figure 2.2 Top). Overall Figure 2.2 summarizes the principles to design shape depending on their location and stress situation.

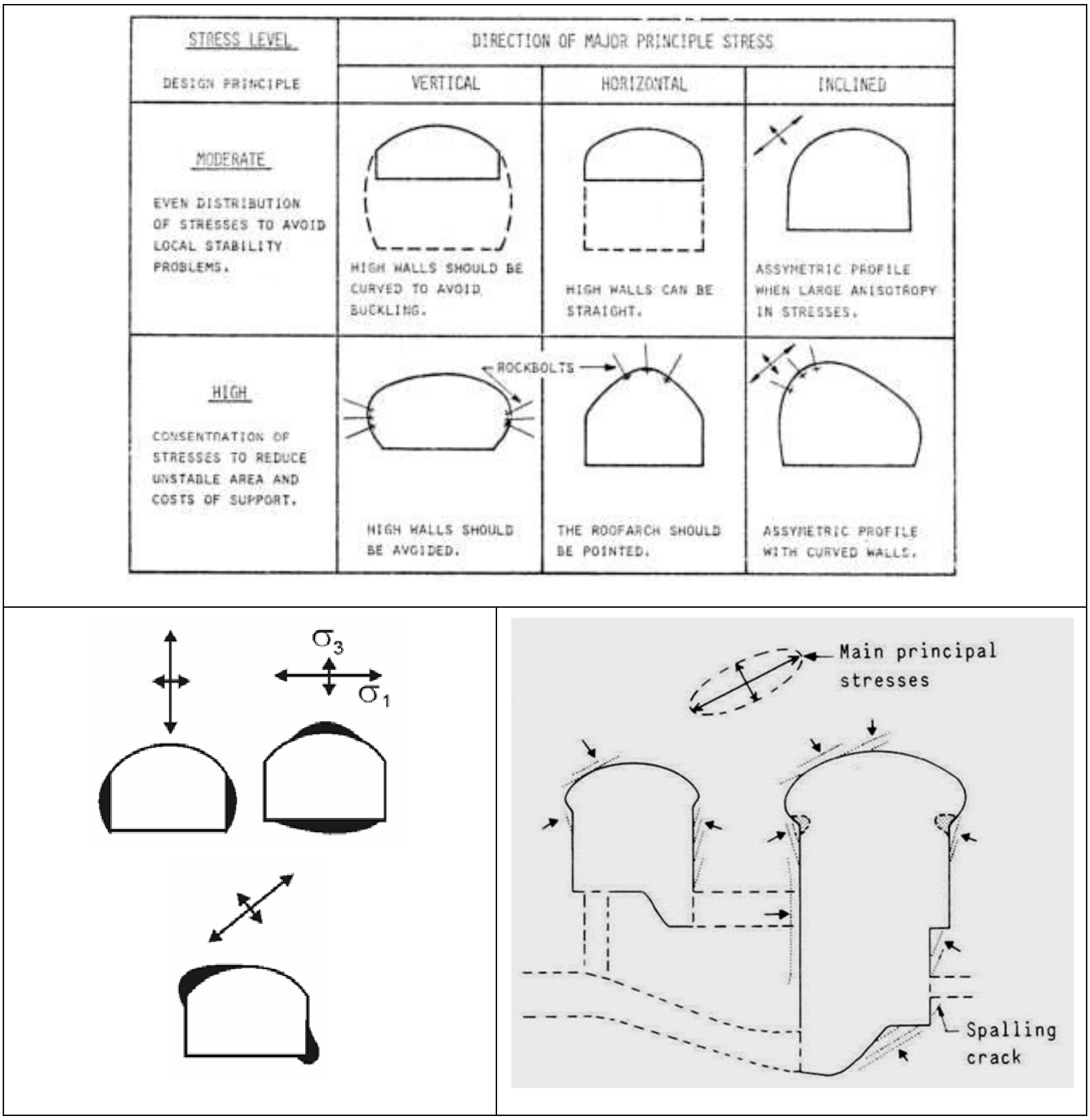


Figure 2.2 Design Principals for underground Openings with varying stress and directions (Nilsen and Thidemann, 1993; Panthi, 2016)

2.2.4 Dimensioning

Every rock mass quality has their limits regarding self-supporting capacity. Since tunneling is known as negative construction method, it’s hard to limit the size of underground openings which largely depends on geology and other factors mentioned above. From the experience some approachable limits depending on the size of power plants has been set. If it was not the case, constant increasing span would bring more stability problems. For example, it is reasonable to fulfill the demand of needed volume by increasing the opening along length axis. In a case with span increment, curvature must be maintained same for stability reasons, which is only possible with increasing arch height with every span increase. The new challenge comes up with extra space thus created. On the other hand, height of underground openings (caverns) determines the thickness between adjacent openings, beside rock mas condition and local stress situation. From general rule of thumb and in good quality rock types with simple design walls between two cavern (s) should be equal to the height of cavern (H) as shown in figure 2.3 (Nilsen and Thidemann, 1993). Generally, with complex design, use of analytical and numerical model is often selected for better understanding of the situation.

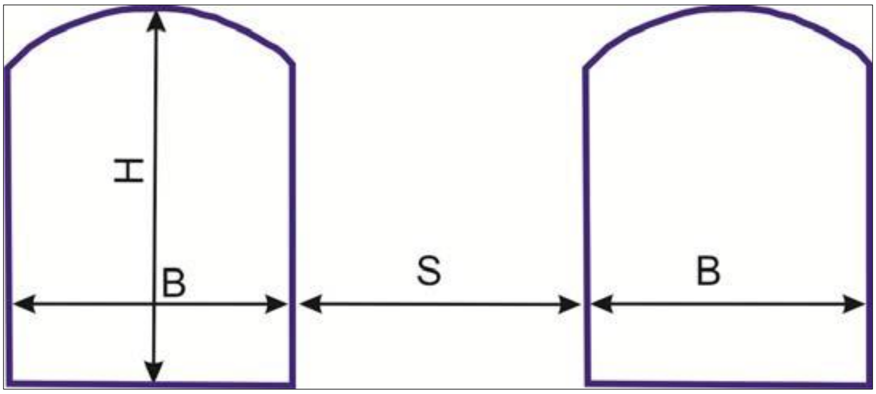


Figure 2.3 Dimensioning of two adjacent Cavern (Panthi, 2006)

With the above requirements met underground openings can be executed as fully lined tunnel known as conventional tunnel, or unlined pressure tunnel. Both tunnels are extensively used regarding the project need and geological features available. In case of hydropower, headrace tunnels are mostly low pressure to high pressure. In case of free-flowing to low pressure tunnels when good rock condition is available, it is not always required to go for intensive protection except for local regions near weakness zone and faults. Regarding the factors mentioned above (Section 2.2), it is wise to avoid rock types of unfavorable characteristics in any case. For section of mediums to high pressure tunnels including pressure shafts it must be made sure that hydrostatic pressure in the section does not exceed minor principal stress. If the pressure exceeds the stress situation then will lead to failure like hydraulic fracturing, lifting etc. So, concrete or fully steel linings are considered as the option to safeguard such situations.

2.3 Conventional lined tunnel

Conventional lined tunnels are built with an extra layer of lining over the excavated surface. Lining types depend from case to case. According to Benson (1989), purpose of lining system in the tunnel is aimed for following purposes.

- Minimize or acceptable head loss in the conduit
- Protection against excessive leakage by factors like seepage or hydraulic fracturing
- Long-term stability of the tunnel in case of watering up, operation and dewatering

However, we need additional cost and time to achieve such purpose with lining design. Even though it is proposed as lined, minimum geological restrictions must be fulfilled. Some rock types like young sedimentary ones need to be avoided to be safe from unfavorable conditions that will occur during construction. Another factor that should be known beforehand is the orientation of major weakness zone. Designer must be sure to decide the location of the opening where those weakness is not intersecting (Nilsen and Thidemann, 1993). Even though steel lining and reinforcement is a conservative approach due to the cost it entails, in compare to project delays, loss of revenue and cost for actual repairs and other anxiety while dealing with unfavorable situations, this option appears as a cheap solution.

2.4 Unlined pressure tunnel

In general, if the tectonic and geological conditions are favorable, unlined pressure serves as an economical, simple, and fast design method. It is based on the theory that rock itself is sufficed to withstand the water pressure during the transfer. When going for unlined tunnel some acceptable leakage is always anticipated but within acceptable limits. To comply with that the minimum principal stress should always be greater than the hydrostatic pressure, in addition care should be taken for critical locations i.e. connection of unlined and steel lining, and penstock connection to the powerhouse.

Not using steel and concrete lining results in a reduction of construction time meaning early production, which reduces capital cost. keeping a simple design results in less number of adits which are most expensive in unreachable mountains. Anyway, continuous geological logging and early test on the rock mass through methods like hydraulic jacking might give us an idea to relocate the critical points and avoid the unforeseeable cost and delay in the project.

2.4.1 Prerequisite for unlined tunnels

To decide with unlined tunneling, certain conditions need to be fulfilled. According to Nilsen and Thidemann (1993), following locations are not favorable for orientation of unlined tunnels.

1. High porosity rocks that may include some volcanic mass and sandstones
2. karstic areas
3. Heavily jointed rock masses and open, intercommunication joints
4. The unfavorable orientation of faults and weakness zones

5. Impermeable rock layers or clay zones that may create a pressure in critical locations. It becomes hard to eliminate some weakness and fracture zones created either by tectonic activity or other geological factors. So best results will occur when it can be eliminated during planning phase. Otherwise the length of the tunnel must be made as short as possible while passing through such zones, and the angle with tunnel alignment must be aimed higher (Panthi, 2006).

2.4.2 Lessons from Underground openings in the Himalayas

In Himalayan Geology, the discrepancy is huge in predicted to actual rock mass quality from the project that has been completed or in the construction phase. Due to constant tectonic movement and intensive monsoon in Nepal along the Himalayan zone, geology is made up of highly fractured, faulted, intercalated, and weathered, and soil covered (Panthi, 2006). One of the reasons behind most delayed projects is a limited level of investigation before the construction. Thus, uncertainty is obvious, so, minimizing the level of uncertainty as early as possible is the solution to it. Thus, it is reasonable to have uncertainty analysis check beforehand because the degree of uncertainty and risk are time-dependent with respect to rock quality knowledge (Panthi, 2006;Panthi, 2017).

To reduce the difference in the level of uncertainty, from actual to predicted, Beacher and Christian (2003) have pointed out two main ways to perform; relative frequency and degree of belief. Former one refers to the number of times the events or properties might occur in series of observation, whereas latter one refers to the judgment capacity developed from long years of experience in such regions. During pre-construction phase of engineering geological investigations, a control quality deviation can be proposed. Yet it is hard to comply with pre-construction phase study for a complete picture ahead, which is also clear from completed projects in Himalayan. Therefore, for realistic prediction of variation, stepwise investigation during pre-construction phase is an option for a safe progress in the project.

2.4.3 Design Principal

Meticulous geological and technical investigation is required while designing underground openings mostly in case of unlined tunnel and caverns. Knowledge of unfavorable conditions and their positions in early stages will prepare for uncertainties. In doing so it would help the designer to fulfill the necessary requirement of structural resistance, durability, and serviceability (Brekke and Ripley, 1987) and be safe from the surrounding rock mass deformation that is possible when dealing with high water pressure. Major issues to be addressed during such underground openings are briefly described below.

a) Vertical and side cover/ confinement criteria

From the early years of designing, accepted principal when going underground is that minimum principal stress must be higher than the water pressure throughout the tunnel at any point so that hydraulic splitting / jacking does not occur. In the conception days of this technique, it was considered that rock cover with half of the water head was sufficient because the specific weight of water is generally half of that to commonly found rock types. Valleys which are quite common in hydropower development showed the complexities when only this design approach was applied. Then originally provided rock cover was unable to fulfill the task. This resulted in consideration of both vertical and valley side cover at the same time with a factor of safety as mentioned in Table 2.1. In simple terms, underground opening with sufficient rock overburden should counter the water pressure in the tunnel.

b) Leakage analysis

When there is permeable rock mass even though necessary overburden is provided, leakage will occur. The properties of discontinuities, material infill in the faults and joints largely affect the probability of leakage. Erodible material like calcite and large opening will lead to water leakage path resulting in surface spring formation or creation of pressure within the rock mass making the overburden unstable. It is an indeed costlier task to make unlined tunnel fully watertight as characteristics of rock mass like permeability and discontinuity is almost unavoidable. So, a small amount of leakage must be tolerated, when actual leakage is known from the initially controlled fillings (Nilsen and Thidemann, 1993). When dealing with high static head in uncertain geological conditions, in-situ test must be performed like hydraulic fracturing so that critical section can be positioned into safe regions.

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Table 2.1 Recommended factor of safety against hydraulic jacking or uplift (Benson, 1989)

Design condition	Normal operating		Water hammer
	Static	Surge	
the lifting of the rock above horizontal unlined or concrete-lined tunnel.	1.3*	1.1	N/A
Along sloping portion near valleys, and at end of steel liner, with proper allowance for slope, topography, and possible landslides removing soil cover.	1.3	1.1	N/A

* Maybe reduced to 1.2 if geological conditions are well-known.

2.5 Underground powerhouse Cavern

The underground powerhouse is one of many uses of the cavern in hydropower system, besides sedimentation chamber, transformer hall, air cushion chamber etc. The reasons going underground over other alternative can be summarized in following perspectives (Persson, 1987).

- Economical aspects
- Safety of operation and good protection against calamities like war, sabotage, ice problems, landslides etc.
- Limited impact on environment.

Above points can be further elaborated with the help of list created by Edvardsson and Broch (2002) for underground powerhouse plants to make it even more approachable than conventional surface design.

- Underground powerhouse can be placed anywhere as far as the alignment can comply with reference to topography and geology of the project area.
- Underground layout reduces the requirement of steel lining to minimum length necessary to control water leakage from the unlined pressurized headrace to powerhouse.
- Reduction of steel lining entails lower specific losses and adds up to higher total plant efficiency
- Plants with underground powerhouse provide the operational stability which is difficult and expensive in case of surface one
- Underground powerhouse is less vulnerable to war-like scenario

- Proper rock confinement provides the necessary Structural requirements by directly transferring the reactive forces to the rock in surrounding
- Underground plants will provide safety against external calamities like landslides, avalanche flooding etc.
- Embedded steel linings, unlined lining need less or no maintenance due to no exposure to climatic conditions
- The condition for construction and erection is unaffected respective of any weather condition and time of year
- Going underground results in no or less deforestation, less use of concrete as compared to the surface structure

Site location, length axes, orientation, and shape are the primary things to consider when designing underground cavern as discussed in section 2.2. It must be clear that cavern is designed in such a way to accommodate the machinery, transformer, and switchgear in a more favorable way as possible. So, it is required to be geologically and economically feasible. There can be many possibilities in regard to the span and strengthening of rock materials in cavern along with the features discussed above. Site selection is performed early in most cases on rather uncertain information, so the possibility of alteration should always be accepted and used well.

According to Edvardsson and Broch (2002), the size of powerhouse cavern largely depends upon three factors: i) the head, ii) the unit size, and iii) the installed capacity. Each parameter has a relationship to interpret with; increase in head results in a reduction of volume per MW and increasing unit size will decrease the volume need per MW at the certain head. Beside this transformer placement in powerhouse, staff quarters, crane space requirement plays important role in deciding the volume of a powerhouse. Two of the governing parameters are discussed below.

a) The height of machine Hall

The total height of cavern from machine hall to the top ceiling is determined from the tallest component handled inside the cavern. Generally main transformer or rotor with the shaft need to be hoisted from crane installed for transportation. When the location of access tunnel to powerhouse is perpendicular, the height of the machine hall should include the height of tallest machine in the vehicle. Compact generator design and heavy items being supplied more efficiently after assembling them from safe workstation will influence in reducing erection space in machine hall.

b) Span of Cavern

It is always considered to increase the length than span. Keeping span as minimum as possible will result cost and construction schedule controlled, since risk and support application is reduced. Span generally include turbine positioning, generator and added space for construction and operation. Vertical positioning will require less width than a horizontal one. In case of Francis turbine-generator enclosure with corridor will decide on minimum width necessary whereas for Pelton turbine space for distributor pipe will decide on minimum width (Edvardsson and Broch, 2002;).

3 Rock mass and Rock Stresses

For the analysis of underground opening, reliable estimate of strength and deformation characteristic of rock masses are required (Hoek, 2007). Underground structure fully relies on the material it is located in. Stresses on the rock mass and the reaction by the rock mass to those stresses is essential to know when structure stability is concerned (Shrestha, 2014). So, surrounding rock mass must have the strength that provides the characteristic reaction when forces are subjected to it. Rock mass is then investigated under sections rock mass property and rock stresses.

3.1 Rock mass properties

According to Nilsen and Thidemann (1993) properties of rocks are defined from the following specifications.

- Physical properties

  Physical features of the rock mass like hardness, density and porosity will determine how the rock is physically. Sonic wave velocity determination is one of the methods that helps to know the rock quality as mentioned.
- Weathering of rocks

  It is the physical breakdown and chemical alteration of rock at or near earth's surface. Mechanical weathering is the result of four important processes undergone known as frost wedging, the effect of rock stress, thermal expansion & contraction, and dynamic activity. And chemical weathering is from the process of decomposition and dissolution (Panthi, 2016).
- Jointing in rock masses

  In-situ rock mass is influenced largely by the location of joints and discontinuities. Tension forces are not transferred by joints like compressive and shear forces. Behavior identification of rock mass with the type of joint, joint patterns is necessary.
- Weakness Zones and Faults

  Weakness zones typically observed in trenches and gorges on the surface has its extension deep into the bedrocks. So, it is hard to avoid such weakness zones but the study of their orientation and extent into the bedrock will give us the idea to locate the tunnel depth and position.

3.2 Estimation of Rock mass properties

According to Hoek (2007), for a Hoek-Brown criterion to be used four properties of rock mass must be estimated in accordance to know strength and deformability of jointed rock masses, which are listed below.

- Uniaxial compressive strength σ_{ci} of the intact rock pieces
- Value of the Hoek-Brown constant m_i for these intact rock pieces, and
- Value of the geological strength index GSI for the rock mass
- E_m, Deformation Modulus of the rock mass

3.2.1 Uniaxial Compressive strength

During measurement of compressive stress, intact rock is taken which is usually free from most of the discontinuities so are rather hard as compared to whole rock. Thus, there is certainty of scale effect when taking the rock mass strength form intact rock in the field. UCS-test is performed generally to estimate compressive strength. Table 3.1 lists out empirical formulas to calculate the rock mass strength when calculating in field and laboratory is difficult to install.

Table 3.1: Empirical Formulae for estimation of rock mass strength

The empirical relationship for Rock mass strength	Proposed by
$\sigma_{cm} = \sigma_{ci} * exp\left[\frac{RMR - 100}{18.75}\right]$	(Bieniawski, 1995)
$\sigma_{cm} = \sigma_{ci} * s^a = \sigma_{ci} * \left[exp\left[\frac{RMR - 100}{18.75}\right]\right]^a = \sigma_{ci} * \left[exp\left[\frac{RMR - 105}{9}\right]\right]^a$	(Hoek et al., 2007; Hoek, 1994)
$\sigma_{cm} = 5\gamma * Q_c^{1/3} = 5\gamma * \left[\frac{\sigma_{ci}}{100} * Q\right]^{1/3}$	(Barton, 2002)
$\sigma_{cm} = \frac{\sigma_{ci}^{1.5}}{60}$	(Panthi, 2006)

Where; σ_{cm} is the unconfined compressive strength of rock mass in MPa, σ_{ci} is the uniaxial compressive strength of intact rock (50 mm core diameter) in MPa, RMR is the Bieniawaski's rock mass rating, s and a are the material constant related to Hoek-Brown failure criterion (the value of 'a' ranges from 0.5 for GSI value 100 to 0.58 for GSI value 10), GSI is the geological strength index, γ is the rock density in t/m^3, Qc is the normalized rock mass quality rating and Q is the rock mass quality rating.

3.2.2 Hoek-Brown Constant, Mi

According to Hoek, constant Mi should ideally be determined by statistical analysis of the results from triaxial test of the carefully prepared core samples. Which in the actual case is related to frictional properties of the rock. In absence of this scenario Figure 3.1 proposed by Hoek can be used for respective rock mass type.

Rock type	Class	Group	Texture: Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates* (21 ± 3) Breccias (19 ± 5)	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6		
	Foliated**		Gneiss 28 ± 5	Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Diorite 25 ± 5 Granodiorite (29 ± 3)			
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	Obsidian (19 ± 3)
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

* Conglomerates and breccias may present a wide range of m_i values depending on the nature of the cementing material and the degree of cementation, so they may range from values similar to sandstone to values used for fine grained sediments.

* *These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

Figure 3.1 Values of the constant mi for intact rock (Hoek and Marinos, 2000)

3.2.3 Geological Strength Index (GSI)

It is of primary concern to know where GSI can be applied. In context to Geological strength index for jointed rock masses proposed by Hoek and Marinos (2000) aware that when a discontinuity is large in comparison to the underground openings GSI tables are not advised to use. So, when a discontinuity is small than the opening, GSI table (Appendix D) can be used. Since the strength of a jointed rock mass is not only related to properties of intact rock mass but also on the freedom of the pieces to slide rotate under different stress conditions. Type of geometrical shape and conditions of surface separating it will determine the freedom of the particle. Thus, GSI, when combined with intact rock mass properties, provides the reduced values of the intact rock mass.

3.2.4 Deformation modulus

The ratio of stress to corresponding strain during loading of rock mass including elastic and inelastic behavior is defined as deformation modulus, Em. Since jointed rock mass does not have elastic behavior, modulus of deformation is used instead of the modulus of elasticity. Laboratory test provides a higher value of deformation modulus than the actual in-situ rock mass, so in-situ test in the large specimen is required but turns out to be expensive also. In most cases results are not available beforehand so various authors have presented their empirical formula as shown in the Table 3.2, to quantify the modulus of deformation.

Table 3.2 Relationship for rock mass deformation modulus

Empirical relationship	Proposed by
$\mathrm{Em} = 2\mathrm{RMR} - 100$	(Bieniawski, 1978)
$\mathrm{E_m} = 10^{\frac{\mathrm{RMR}-10}{40}}$	(Serafim and Pereira, 1983)
$\mathrm{E_m} = 10 * \left[\frac{\mathrm{Q} * \sigma_{\mathrm{ci}}}{100}\right]^{1/3}$	(Barton, 2002)
$\mathrm{E_m} = \left[1 - \frac{\mathrm{D}}{2}\right]\sqrt{\frac{\sigma_{\mathrm{ci}}}{100}}\ 10^{(\mathrm{GSI}-10)/40}$	(Hoek et al., 2002)
$\mathrm{E_m} = \frac{1}{60} * \mathrm{E_{ci}} * \sigma_{\mathrm{ci}}^{0.5}$	(Panthi, 2006)

The relation by Hoek et al (2002) is the modified version of the original expression proposed by Hoek and Brown including feature D, that considers the effects of blast damage and stress relaxation. The value of d is assumed zero to investigate in situ condition (Hoek et al., 2002). The relation proposed by Panthi (2006), is relevant for the geology of Himalayan region specially of rock types with small compressive strength with schistose, foliated, and bedded rock types. For analysis, the values given by the Hoek et al (2002) is used.

3.3 Rock Stresses

Stress situation in the underground opening is concerned with stress condition before excavation known as virgin stresses and stresses surrounding the underground openings. Stress situation before opening is the result of different stress component (Nilsen and Thidemann, 1993), which in combination and perpendicular to principal stress planes give three principal stresses in a rock mass (Panthi, 2016).

3.3.1 In-situ Stresses

Virgin stresses can further be categorized with respect to the factors they depend on. Below is the summarized version of all the stresses that are present in the rock mass.

- Gravitational stresses

Vertical stress is induced due to the overlying strata and result of gravity alone. When strata are relatively uniform, vertical stress at the depth 'h' will be proportional to the weight of the overlying strata (Figure 3.2 left) and can be calculated by using Equation (1). Equation (2) gives the relationship between vertical and horizontal stress for elastic rock with Poisson's ratio (ν). It is worth noting that vertical stress due to gravity is a small part of the horizontal stress.

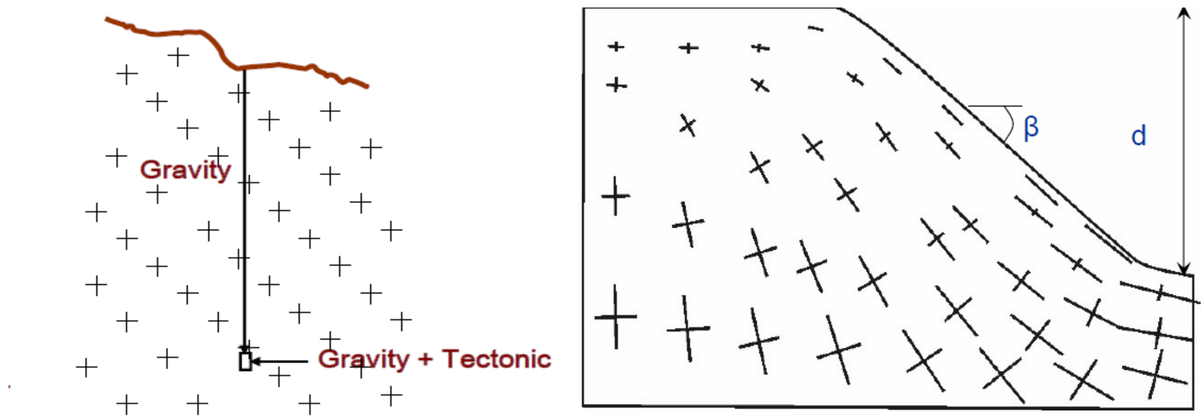


Figure 3.2 Schematic view of vertical and horizontal stress situation (left) & stress situation due to topography (Panthi, 2016)

$$\sigma_{\mathrm{v}} = \gamma \mathrm{h} \tag{1}$$

$$\sigma_{\mathrm{xv}} = \frac{\nu}{1-\nu}\sigma_{\mathrm{v}} \tag{2}$$

Where γ is the specific weight of overlying strata in MN/m^3 and h is the depth of overburden and ν is the Poisson's ratio.

- Topographic stresses

When rock formation is not uniform then influencing factor for rock stress would be the topography (inclination of valley side) and is known as topographic stresses. In case of hydropower plant where the valley is evident, in the section near the slope major principal stress (σ_1) is almost parallel to valley slope and minimum principal stress(σ_3) is perpendicular to the slope (Figure 3.2 right).

- Tectonic stresses

Tectonic plates in lithosphere that constitute the earth's outer shell, have uneven movement resulting in the generation of rock stresses known as tectonic stresses, which is the main reason for the formation of faulting and folding. Gravity and tectonic stress combined gives the horizontal stress (Figure 3.2 left), which can be computed using relation given below (Panthi, 2012).

$$\sigma_{\mathrm{h}} = \frac{\nu}{1-\nu}\sigma_{\mathrm{v}} + \sigma_{\mathrm{tec}} \tag{3}$$

Where, σ_h, σ_v and σ_{tec} are the horizontal, vertical, and tectonic stresses in MPa respectively and ν is the Poisson's ratio.

- Residual stresses

Vertical stresses when appearing extremely high are often reasoned with the presence of residual stress. Residual stresses are locked into rock material from earlier stages of its formation, like when magma is cooling down it contracts, and stress is developed of this category.

3.3.2 Stresses surrounding Underground openings

When rock mass is just unloaded due to an excavation process, there is re-distribution of stresses with changed stress magnitudes, Figure 3.3 is a schematic approach of such phenomenon, where the stress pattern has been changed around the circular opening.

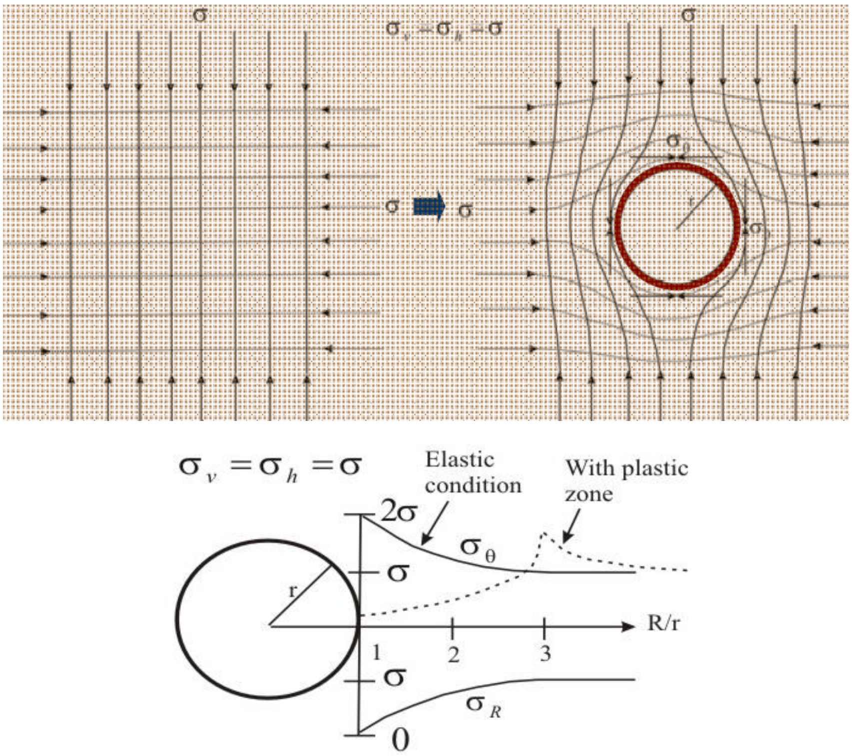


Figure 3.3 Stress situation before excavation & after excavation (top) and tangential & radial stresses surrounding the circular opening in isostatic stress state(bottom) (Panthi, 2016)

In homogenous isotropic and elastic material (Figure 3.3 bottom) tangential stress (σ_Θ)increases rapidly close to the contour. The magnitude is twice the isostatic stress around the periphery. But in often case stress situation is anisotropic, so tangential stress will vary around the periphery of the circular opening. In such cases, Kirsh's equation (4) & equation (5) are used to evaluate the tangential stresses. When Maximum principal stress (σ_1) is tangent to the contour, tangential stress reach to the point as $\sigma_{t\ (max)}$ which is given by equation (4) and whereas value reaches to a minimum when minimum principal stress (σ3) direction is a tangent to the contour and is represented by equation (7).

$$\sigma_{t(max)=}3\sigma_1-\sigma_3 \tag{4}$$

$$\sigma_{t(min)}=3\sigma_3-\sigma_1 \tag{5}$$

Beside this, shape of the tunnel opening has equal influence on stress distribution. Above case defines the condition when the opening is circular, but when it is non-symmetrical, sharp corners has an effect. Meaning more the corners are sharp due to a reduction in curvature radius, stress concentration will be higher in those corners. Then in the case of protruding points, stability will be the issue.

3.4 Failure criterion

In rock engineering design, rock is expected to be failed during excavation so that complete failure of the structure is avoided. For that study of a failure condition of intact rock plays an important role. Many failure criterions have been developed over the years but widely used failure criteria are 'Mohr-Coulomb' and 'Hoek-Brown' (Ulusay and Hudson, 2012). Both this criterion takes in account only major and minor principal stresses not considering intermediate stresses. The important thing to consider is that failure criterion is limited to intact rock material because stability in tunneling is also result of natural joints and cracks due to blasting (Nilsen and Thidemann, 1993).

3.4.1 Hoek and Brown failure criterion

In 1980 Hoek and Brown defined failure criterion for jointed rock mass looking at the interlocking of rock blocks and condition of surfaces between them. After several modification finally in 2002 generalized Hoek Brown criterion was defined for the jointed and isotropic condition:

$$\sigma`_1 = \sigma`_3 + \sigma_{ci}\left[m_b \frac{\sigma`_3}{\sigma_{ci}}\right]^a \tag{6}$$

Where σ1' and σ3' are the maximum and minimum effective principal stress at failure,

m_b is the reduced value of material constant m_i

σ_{ci} is the uniaxial compressive strength of the intact rock material

s & a are the constant which depend upon the rock mass characteristics

M_b, s and a are defined by following equations:

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14d}\right) \tag{7}$$

$$S = \exp\left(\frac{GSI-100}{9-3D}\right) \tag{8}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \tag{9}$$

Where, D is the factor that depends upon the degree of disturbance due to blast damage and stress relaxation. Hoek (2007) suggest that the D value ranges from 0 representing undisturbed rock mass to 1 for very disturbed rock mass. Different suggested values for various rock mass conditions can be found in chart created by Hoek (2007b).

3.4.2 Mohr Columb failure Criteria

In lieu with Hoek brown criteria, Mohr-Coulomb failure criteria is also common technique used to estimate rock mass properties. In this criterion, cohesive strength c' the angle of friction φ' defines the strength of the rock mass, whereas the Hoek and Brown criteria estimates the rock mass strength based on principal stresses. Hoek Bown has generated curve by fitting an average linear relationship from equation as shown in (Figure 3.4 left)

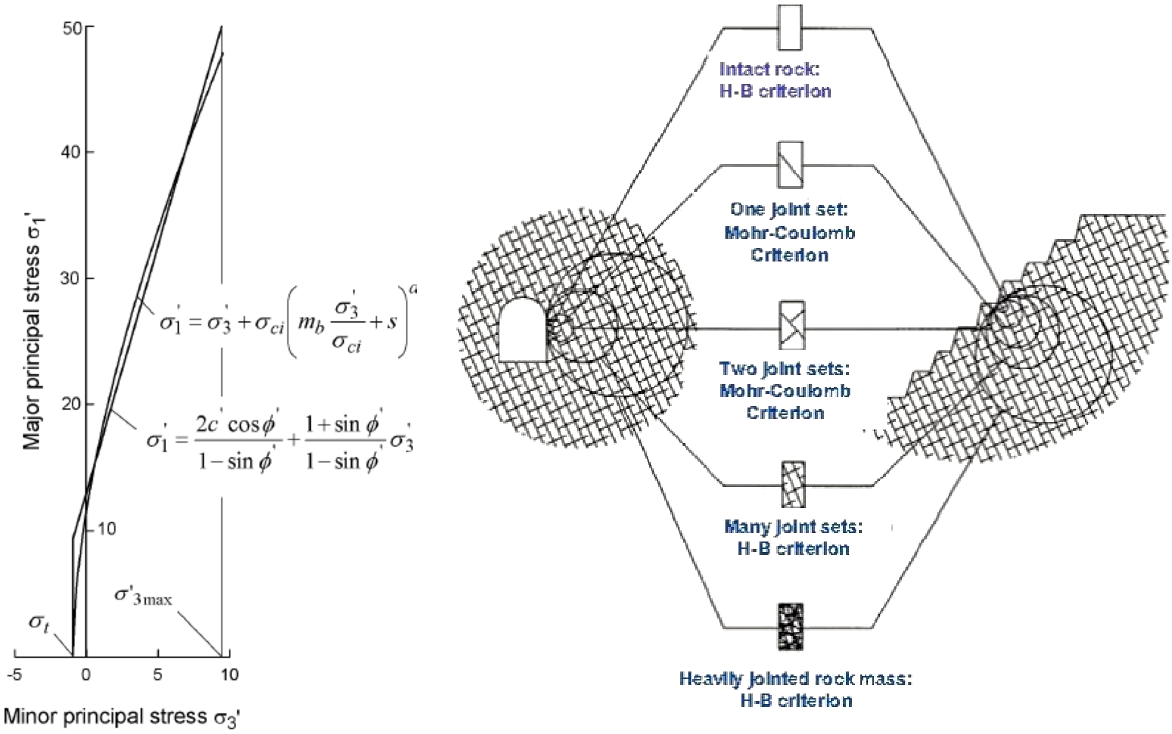


Figure 3.4 (left) Relationship between major and minor principal stresses for Hoek- Brown and Equivalent Mohr-coulomb criteria (Hoek et al 2002) & (Right)failure criteria in the transition from intact to a heavily jointed rock mass with increasing sample size(Hoek et al,, 2007; Panthi, 2016)

Hoek (2007) states that Hoek and Brown failure criteria should be used in underground excavation only when structure being analyzed in respect to the block size is large. If spacing of the discontinuity in respect to structure dimension is larger, than Mohr column failure criterion should be used. Mohr-Coulomb is based on the normal stress and shear strength. Shear strength of the rock mass can be defined by angle of friction (ø') and cohesive strength (C'). Figure 3.4 (right) shows an idealized diagram for use of failure criterion with transition of rock mass from highly anisotropic rock to an isotropic heavily jointed rock mass.

4 Assessment Methods

In the design of underground opening, to encompass most of the theoretical aspect different methods has been introduced. Methods are categorized in analytical, semi-analytical and numerical. All three categories have their own scope and limitation, so considering all three approaches will broaden the analysis. All three methods are briefly described below.

4.1 Analytical methods

Analytical method mostly considers the geometrical formation of the site. The position of the alignment with respect to ground surface, water head, valley slope, shortest distance from the valley are the key factors for the analysis. Snowy mountain criteria and Norwegian criteria are some of the analytical techniques. In the thesis Norwegian confinement criteria is discussed due to its relevance and heavy usage.

4.1.1 Norwegian Confinement Criteria

Unlined tunnels built in Norway before 1968 were designed considering minimum rock cover with constant for the valley inclination. The general understanding was that tunnel should be deep enough so that the weight of overlying rock will balance the internal water pressure, equation (10) is based on this theory.

$$h > C.H \tag{10}$$

where,

H is the hydrostatic head acting over the shaft alignment,

C has constant of 0.6 for valley inclination up to 35^0 and 1 for exceeding 35^0.

After the failure of the unlined tunnel at Byrte in 1968 equation (10) was revised to equation (11). In which inclination of the shaft was introduced along with the density of both water and rock (Broch, 1984; Panthi and Basnet, 2016). Still, it was only considering vertical rock cover.

$$\gamma_r.h.\cos\alpha > H\ \gamma_w \tag{11}$$

Equation (11) was applied to most of the design until the failure of Åskåra in 1973, which led to the establishment of a new approach by Berg- Christensen and Dannevig as illustrated in Equation (12) & is schematically represented in Figure 4.1. In this approach shortest distance from the valley, side was considered.

$$\gamma_r.L.\cos\beta > H\ \gamma_w \tag{12}$$

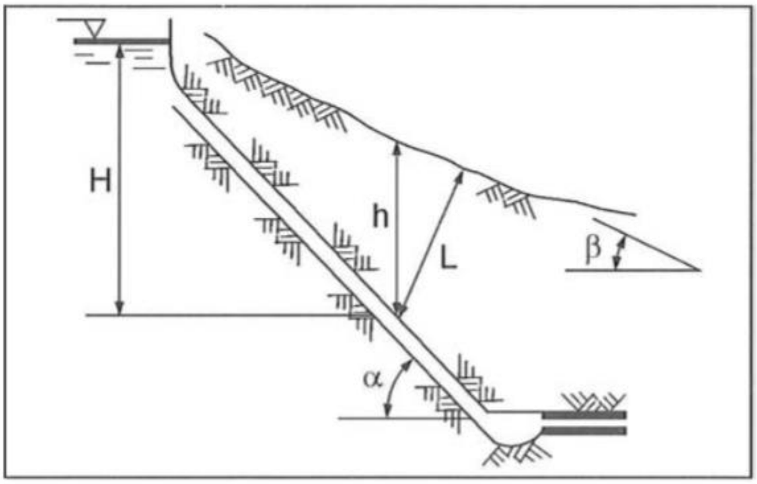


Figure 4.1 Definition for the rule of Thumb for High-pressure tunnels and shafts (Edvardsson and Broch, 2002)

Where,

L= shortest distance between the surface and the point of the shaft-tunnel studied(m)

H=maximum static water head at the point studied(m)

γ_w=density of water

γ_r= density of rock mass

β=average inclination of the valley side

α= inclination of the tunnel-shaft

Equation (11) and (12) later combined were known as the state-of-art Norwegian design criterion for unlined pressure shafts and tunnels. After some years later, Broch suggested the need for correction in complicated and irregular topography before introducing the equation (11) & equation (12). Broch (1984), suggested that special attention should be given to topography that is in the mountainous region, where deep ravines between the noses or ridges are formed by the streams and creeks flowing and eroding the profile. The point to be noted is that noses and ridges are stress relieved, so it must be neglected when estimating overburden of rock unless stress field is verified from in-situ measurements. To avoid such overestimation, we must plot the simplified valley side profile making sure that no more ridges and noses are lying there because these equations were designed for Norwegian rock formation which has smooth and simple topography. He was indicating topography like in Himalaya and other parts of the world.

Table 4.1 Applicability of Norwegian Criteria for various ground conditions (Basnet and Panthi, 2018)

Category	Favorable conditions	Unfavorable conditions
Topography Rock mass and jointing	Relatively gentle valley slope topography Homogeneous and strong rock mass formations with no or single joint set having a tight joint wall, wide spacing and anti-dip against valley slope	Deep, steep, and complex valley slope topography Weak rock mass with a high degree of schistosity; Highly porous rock mass of volcanic and sedimentary origin; Jointed rock mass having more than two systematic and long-persisting joint sets with one or more joint sets dipping steeply towards valley slope; Pre-existing open joints or the joints filled with sand and silt, which could easily be washed away; and Sub-horizontal joints at low overburden area
Faults and weak/crushed zones	No nearby major faults and zones of weakness	Nearby fault and zones of weakness that are parallel or cross-cutting to the valley slopes
In situ stress state	The minimum principal stress always higher than the static water head	De-stressed area and location not far away from steep valley slope topography; Not sufficiently far away from the locally overstressed areas
Hydrogeology	Hydrostatic water line below natural groundwater table or tunnel aligned deep into the rock mass and far away from the steep valley slope restricting flow paths to reach valley slope topography	The hydro-static line above the groundwater table and relatively near from the valley side slope; and Highly permeable and communicating joint sets

Basnet and Panthi (2018), after careful examination of major cases from Norway has developed a list of factors that classifies different ground conditions and suggests either it is favorable or not to apply Norwegian Confinement criteria as listed in Table 4.1. The Table turns out to be very useful when Norwegian method is to be applied away from Scandinavia like in Himalayan geology.

4.2 Semi-analytical method

Results due to reducing rock mass strength and properties entails a probable chance of instability problems. Tunnel squeezing is one of the stress-induced instability along with rock burst/rock spalling. Rock spalling is observed typically in strong and brittle rocks around the parallel to tunnel periphery. In case of weak rocks, time dependent & independent inward moment commonly known as tunnel squeezing of rock material occurs due to resulting of higher tangential stress than rock strength (Panthi, 2006). Geology in Himalayan is typically combined with deep weathering, irregular valley formation and encountering of many discontinuities. Which adds up to reducing rock mass strength and squeezing as instability problems. Hoek and Marinos state that variability in the strength and deformability properties of rock plays a crucial role than the overburden pressure. Considering rock mass strength and overburden pressure Hoek and Marinos (2000) has developed a relationship that gives total tunnel strain (Figure 4.2). Tunnel strain is defined as the ratio of tunnel closure to tunnel diameter.

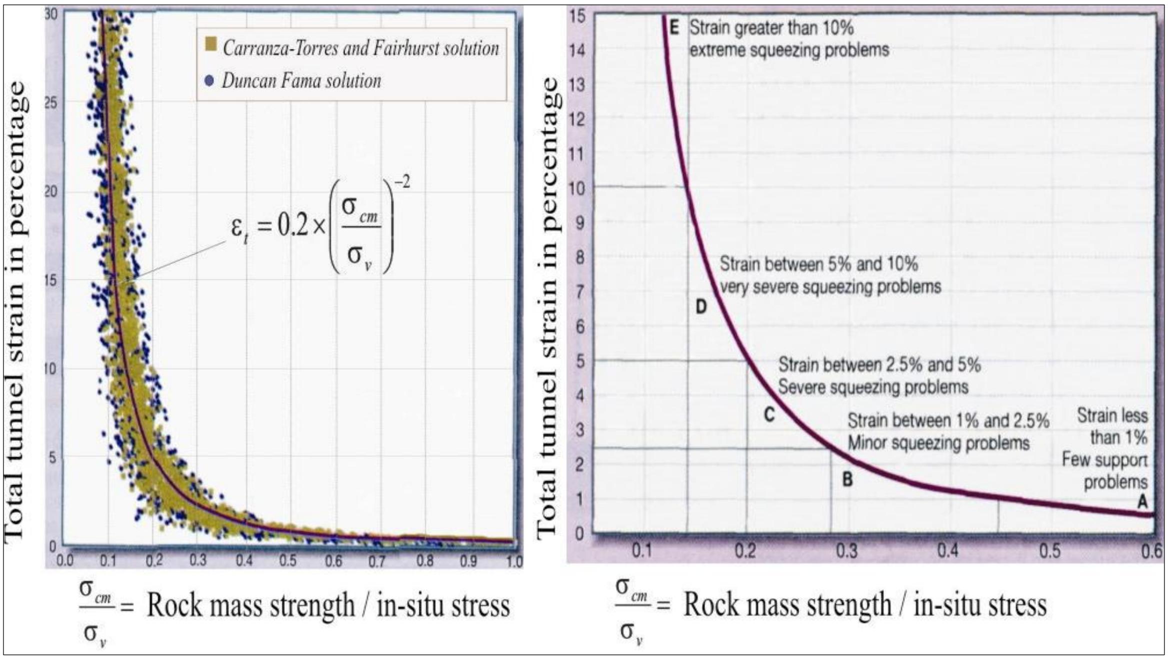


Figure 4.2 Plot of tunnel convergence against the ratio of rock mass strength and in-situ stress(left) and convergence against the degree of difficulty associated with tunnel squeezing(right) (Hoek and Marinos, 2000; Panthi, 2006).

For this analysis, rock mass properties are calculated using Hoek and Brown criterion as discussed in section 3.4. Since most project in Nepal is situated in the Himalayan region it is useful to use equation proposed by Panthi (Table 3.1). Based on Figure 4.2 following Table 4.2 summarizes the Geotechnical issues and support types required for respective tunnel strain range. When excavation is initiated into the rock mass and if it is weak and jointed one, deformation is evident. According to Grimstad et al. (2002), when deformation is in range 0.2 to 1 % for a span of 10 m, flexible primary layers of sprayed concrete with a combination of rock bolts helps to control the opening from collapse. Taking above facts into consideration this principle is applied for analysis of unlined tunnels and cavern openings.

Table 4.2 Relationship between strain and degree of difficulty associated with tunneling through squeezing rock (Hoek and Marinos, 2000).

	Strain %	Geotechnical Issues	Support types
A	<1	Few stability problems and very simple tunnel support design methods can be used. Tunnel support recommendations based upon rock mass classifications provide an adequate basis for design.	Very simple tunneling conditions, with rock bolts and shotcrete typically used for support.
B	1to 2.5	Convergence confinement methods are used to predict the formation of a ‘plastic’ zone in the rock mass surrounding a tunnel and of the interaction between the progressive development of this zone and different types of support.	Minor squeezing problems which are generally dealt with by rock bolts and shotcrete; sometimes with light steel sets or lattice girders are added for additional security.
C	2.5 to 5	Two-dimensional finite element analysis, incorporating support elements and excavation sequence, are normally used for this type of problem. Face stability is generally not a major problem.	Severe squeezing problems requiring rapid installation of support and careful control of construction quality. Heavy steel sets embedded in shotcrete are generally required.
D	5 to 10	The design of the tunnel is dominated by face stability issues and, while two-dimensional finite analyses are generally carried out, some estimates of the effects of forepoling and face reinforcement are required.	Very severe squeezing and face stability problems. Forepoling and face reinforcement with steel sets embedded in shotcrete are usually necessary.
E	More than 10	Severe face instability as well as squeezing of the tunnel make this an extremely difficult three-dimensional problem for which no effective design methods are currently available. Most solutions are based on experience.	Extreme squeezing problems. Forepoling and face reinforcement are usually applied and yielding support may be required in extreme cases.

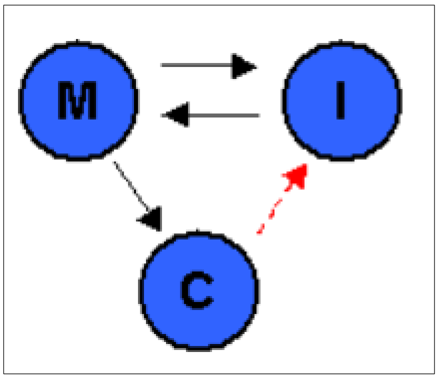
4.3 Numerical Modeling

As observed from Section 4.1, deterministic, or theoretical principals looks only into gravitational stresses accounted from confinement due to rock cover. But in most hydropower tunnels it must pass through various kinds of valleys depending on the topography. So, besides the gravitational stresses, we will also have topographical and tectonic stresses that will in large extent dominate the stress regime. Due to such shortcomings results from above-mentioned methods will make our analysis conservative.

To address such scenario numerical models are generated in discontinuous and continuous categories. Discontinuous class models the rock mass in a single block. Continuous class, on the other hand, discretize the rock mass into a large number of individual elements and are checked individually for rock stresses and deformation (Nilsen and Thidemann, 1993). The common method with this category is Finite element method (FEM) and Boundary element method (BEM). Both methods are used to define a geological model of actual site with inputs from rock properties and boundary conditions. In general, numerical models are performed to address stress state analysis and stability analysis.

4.3.1 Use of RS^2

RS^2 (Phase ^{2}) is 2D finite element program used in wide range of projects that include excavation design, slope stability, groundwater seepage, probabilistic analysis, consolidation and dynamic capabilities (Rocscience, 2018). A task like stage excavation, tunneling in weak or jointed rocks, failure interaction and support interaction can be performed with the help of three individual program modes as represented in Figure 4.3. In the figure red arrow, indicates that compute (C) must be done before interpreting (I) the model(M), whereas black arrows represent the ability to work from within model(M). More on individual modes are briefly described below:



M: MODEL, C: COMPUTE, I: INTERPRET

Figure 4.3 Schematics showing the interaction happening between three modes (Rocscience, 2018)

Model Generation (M)

To analyze the result in RS^2, there are certain steps that we need to go through to create the scenario that can be interpreted as near as real one. Below are the steps that are performed for our project analysis and are equally applicable to other related projects as well.

- **Project settings**

Basic information like project name, stages of analysis and groundwater method can be registered in this section. In case of underground cavern and big tunnel, there are stages of excavation, which is also addressed using project staged tab. Two analyses are available in this program. Plane strain analysis which assumes that the excavations are of infinite length in the out-of-plane direction, and therefore the strain in the out-of-plane direction is zero. Whereas Axisymmetric analysis allows you to analyze a 3-dimensional model which is rotationally symmetric about an axis (for example, the end of a circular tunnel).

- **Boundaries**

To define the model creating a boundary is the first step. It can be added manually, importing Dxf file or from predefined tunnel shape. In the model, excavation represents an area that is to undergo excavation, for that external boundary must be created. The model then can be staged, including joints, structural interface, and piezometric line.

- **Meshing**

when all the boundaries are defined, RS^2 automatically generates the meshes based on either triangular or quadrilateral finite elements. In addition, Custom Discretization is always available to create uniformly spaced discretization to the selected boundary segments. uniform discretization selects line segments, using the user-supplied number of discretization's, or multiplication factor, per line segment.

- **Loading**

This tab provides the input options on in situ stress condition before the excavation. Constant field stress investigates the stress field which remains same regardless of varying topography. Gravity field stress takes into the consideration of ground surface elevation. More on input parameters are discussed below separately.

- **Material Properties**

RS^2 allows defining physical and hydraulic properties of the rock or soil that define our model. To specify the material properties in general two steps are performed; Defining material properties and assigning the properties to the various regions of the model. Physical properties such as unit weight, strength, and elastic parameters are defined under material properties and if ground pore pressure is to be included that can be done through hydraulic properties.

- **Support**

An extensive range of support modeling options for geotechnical and mining applications are available in RS^2. Two main support systems known as bolts and liners are available. It is available to define the properties of the support system and staging of their placement. After defining it can be assigned to the respective zones as per requirement. Separate or combined lining system is also possible to stabilize the behavioral changes in the excavation zone. Below are the summary of support types and included in RS^2:

- **Bolts**

Program RS^2 has bolt properties categorized in five different types: End anchored, fully bonded, plain strand cable, split sets/swells, tieback. In our case, we have decided to choose fully bonded bolt.

Fully bonded bolt

According to (Rocscience, 2018), in RS^2 analysis Fully bonded bolts behave as bolt elements when it crosses the finite elements, so they are independent of each other and has no influence except indirectly form the effect they create through soil or rock. There is no interface effect to account for as it is considered fully bonded and remains like that through the analysis phase. This kind of bolt fails in tension as axial stress exceeds its capacity of the material. But due to its element nature, same bolt may have part of it as yielded but other as firm as it was. It still has a residual capacity after it has exceeded its peak capacity.

- **Liners**

For the analysis purpose, RS^2 has defined four different liner types. Table 4.3 provides a summary of all liner type and their attributes.

Standard Beam

A standard beam liner can respond to flexural, axial (compressive or tensile) shear loads. It is used to model for resistance to bending. By altering area and moment of inertia it can be used in the varied and complex area. It can be combined with reinforced concrete. Elastic liner type will respond elastically to loading but will not consider strength parameters. Whereas plastic liner has the limit set for peak and residual parameters, so is considered yield after it is reached its peak value. It is simple to design option if a single layer of shotcrete is only needed.

Reinforced concrete

Reinforced concrete is two component liner system of concrete and steel elements. This system has also same beam elements like in standard beam, that will respond to flexural, axial and shear loads. The best thing about this liner is to specify the properties for reinforcement and concrete separately. And in analysis results plot can be created for individual material performance.

Geosynthetic

In $RS2^2$ Geotextiles or Geogrids are used as reinforcement under Geosynthetics, used in the form of fabrics, meshes, grids, strips, membrane etc.

Cable truss

Cable truss provides support in tension, so it is not useful for supporting under flexural, shear and compressive loads.

Table 4.3 Summary of Liner type attributes (Rocscience, 2018)

Properties	Standard Beam	Reinforced Concrete	Geosynthetic	Cable Truss
Axial Force	Yes	Yes	tensile only	tensile only
Bending Moment	Yes	Yes	No	No
Transverse Shear Force	Yes	Yes	No	No
Plastic Yielding	Yes	Yes*	Yes	Yes
Support Capacity Plots	No	Yes	No	No

- **Compute (C)**

when the model is generated with rock property and stress situation, compute option allows for the finite element analysis of the current model. Depending on the computer used processing can be carried out through parallel or sequential model. By default, RS^2 uses a compressed file format to store all input and output files for a given model. After the analysis, all the output files will be contained in a single compressed (zip) file with extension *.fez.

- **Interpret (I)**

Interpret is the post-processing module used for data visualization and interpretation of the RS^2 analysis results. Data Contours can be viewed (e.g. stress, displacement, strength factor), and results can be displayed on the model or graphed for material queries, bolts, liners, joints etc. According to RS^2 (Rocscience, 2018), principal stresses obtained as output from the software has the following connotation.

Sigma 1

The Sigma 1 option will plot contours of the major in-plane principal stress. Whenever a file is opened, or when a new window is opened for an already open file, the default plot will always be a Sigma 1 contour plot. Remember that the in-plane Sigma 1 may not always be the major principal stress in 3-dimensions – if the value of Sigma Z is greater than Sigma 1 at a given point, then the in-plane Sigma 1 will actually be the 3-d intermediate principal stress.

Sigma 3

The Sigma 3 option will plot contours of the minor in-plane principal stress. Remember that the in-plane Sigma 3 may not always be the minor principal stress in 3-dimensions – if the value of Sigma Z is less than Sigma 3 at a given point, then the in-plane Sigma 3 will actually be the 3-d intermediate principal stress.

Sigma Z

The Sigma Z option will plot contours of the out-of-plane principal stress. Remember that Sigma Z is not necessarily the intermediate principal stress – depending on the in-plane values of Sigma 1 and Sigma 3 at a given point, Sigma Z could be the 3-dimensional major, intermediate, or minor principal stress.

Axisymmetric Sigma Z

If we are analyzing an Axisymmetric problem, then Sigma Z will represent the circumferential stress around the excavation.

4.3.2 Stress state Analysis

Geometrical and hydraulic consideration is performed with respect to the geometry of the alignment, which includes rock cover, water head, length of the tunnel etc. In addition to that, there are other factors which equally contribute to instability issues encountered during construction and operation. Stress present inside the rock mass also known as in-situ stress has an important role. The oversimplification of the rule of thumb, and ignorance of Stresses like residual and tectonic which are common in many cases led to generate the compensating technique based on finite element method that was introduced by Department of Geology, NTH (Nilsen and Thidemann, 1993). The principal is focused in finding the appropriate location for the tunnel by satisfying the criteria as expressed in equation 15. What it is trying to imply is that nowhere in the unlined pressure tunnel, internal water pressure should exceed the in situ minimum principal stress.

$$\sigma_3 > H.\gamma_w \tag{13}$$

Where,

σ_3=minor principal stress

γ_w=density of water

H=Static water head

Based on the equation (13), Broch (1984) has used the standard two-dimensional FEM-diagram (Figure 4.4), which cover the valley side inclination between 14 and 75 with a variety of rock stress configuration and rock mass properties(Nilsen and Thidemann, 1993). Required rock cover is estimated when the model is transferred to actual topographical models. Special care must be taken in case of unique geological features in respective case project.

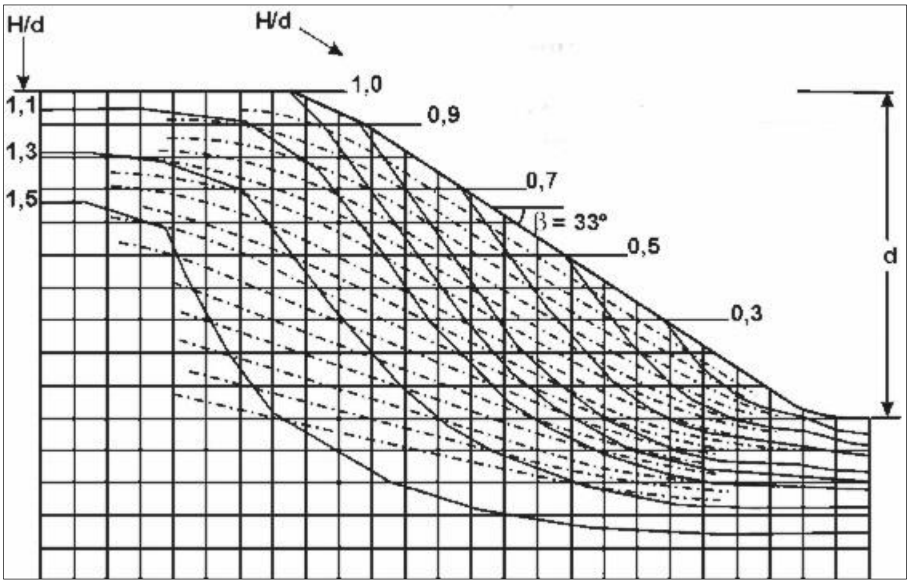


Figure 4.4 Standard Design Chart based on FEM-analysis (Nilsen and Thidemann, 1993)

Now the FEM model can also be generated with help of RS^2 (Section 4.3.2), for different sections. Stresses and orientation angle obtained from the valley model developed from the RS^2 is taken as input for the analysis of the tunnel opening.

4.3.3 Stability Analysis

Just after understanding the stress situation from the valley model, the section must be analyzed for an excavation state. Section 3.3.2 present us the scenario that will take place ones the excavation is performed. So, with the stress available from the valley model, stress situation after excavation is analyzed. The intention is to know that design section with proposed geometrical confinement and hydraulic property is fully satisfied or not. Once the critical section is known installation of support system is performed, to protect the opening from displacement and total collapse. The support is applied based on the rock types it lies on. Support is selected once the rock types are decided.

5 Review of Case related Projects

As discussed in section 2.4.2, Nepal has a typical geological formation with major faults known as main central thrust (MCT), main boundary thrust (MBT) and main frontal thrust (MFT). Many hydro powers are located around the lesser Himalayan zone (Figure 6.1) which are located in between MBT and MCT. So, the rock masses found in this area are highly sheared, fractured and weathered ((Panthi, 2006). Scenarios as such makes the geology in that area complex. As Beacher and Christian (2003) have pointed out that degree of belief (which they were relating to the knowledge gained from the experience) needed is more in such cases when deciding the choices and necessary reports are not available. Our project Himchuli-Dordi Hydropower lies in Lesser Himalayan region with the high head (810 m) design. So, it is wise to gather knowledge from relevant projects. Thus, selection of cases to review is based on above-mentioned facts. Middle Marsyangdi Hydroelectric Project is in the region with similar geology as Himchuli, Khimti is in the area dominated by MCT, and Nye Tin project because of its one of kind features as unlined tunnel withstanding head of 1047 m.

5.1 Middle Marsyangdi Hydroelectric Project (MMHEP)

The middle Marsyandi hydroelectric project is located near Besisahar in Lamjung District on the left bank of Marsyandi river. The project has installed capacity of 72 MW which is a medium sized run-of-river scheme. It has the medium head of 110 m and design discharge of 80 m3/s and has the production capacity of 400 GWh energy annually. The project comprises 5.2 km headrace tunnel of 6.4 m diameter horseshoe shaped. The project also has the additional features like 3 underground desanding caverns (140 x 15 x 25) m, surge shaft 45 m high and 22 m diameter, including 385 m long penstock and semi-underground powerhouse.

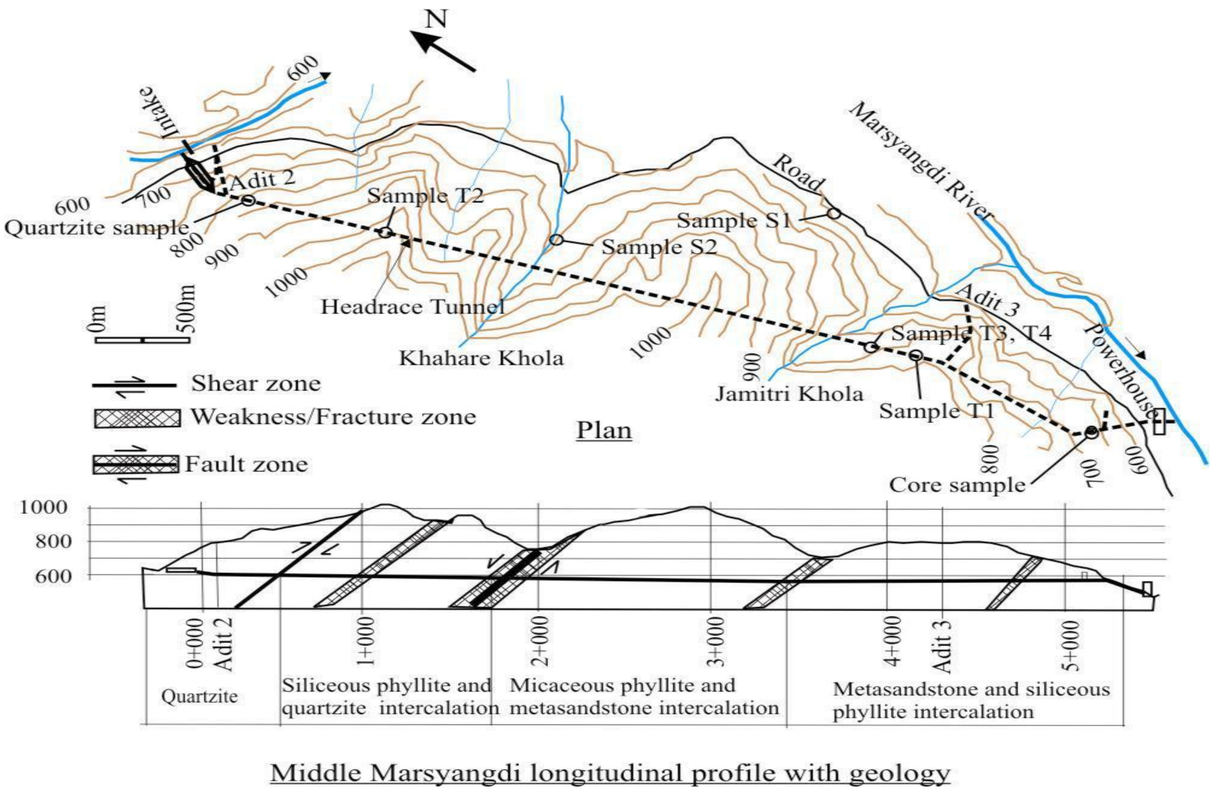


Figure 5.1 Project topography and longitudinal profile with geology for MMHEP (Panthi, 2006)

Project Geology

The project is positioned in lesser Himalayan meta sedimentary rock formation. Dominant rock types found in this project are quartzite, phyllite, and metasandstone. In Figure 5.1, the upper section of head race tunnel including intake, settling basins and other diversion rocks are in quartzite rocks and rest of the downstream section is in micaceous and siliceous phyllite. Rock mass along headrace tunnel is highly fractured, tectonically disturbed and sheared. Also, it is intersecting through some weakness and fault zones including major fault system called Madi fault passing along Khahare Khola. In the certain section where convergence test was carried out, according to Q-system rock mass were ranging from very poor to extremely poor category (Panthi and Shrestha, 2018).

Stability Problem and rock support

Major tunnel stability problem was anticipated between Jamitri and Khahare Kholas due to the dominance of micaceous phyllite (Panthi, 2006). Also due to the high overburden of 400 m in that section and weak rock types squeezing problem was probable. Due to Madi fault crossing the tunnel, the challenge was always present to deal with uncertainties.

Support system for MMHEP has been designed using RMR system. During excavation geological conditions and rock mass properties were assessed according to RMR system but there was a deviation from what was expected. So, the support class 3,4 and 5 chosen beforehand was changed as per the new classification and site conditions (Panthi and Shrestha, 2018).

5.2 Khimti-I Hydroelectric Project (KHEP)

Khimti I hydroelectric Project is also located in the lesser Himalayan region towards 100 km east of Kathmandu. The project is a high head scheme with a gross head of 684 m and design discharge of 10.75 m3/s. Installed capacity for the project is 60 MW with annual energy production of 350 GWh. Major structure involving in this project are, 7.9 km pressurize head race tunnel with inverted D section (14 m^2 cross section), 913 m steel lined penstock with an inclination of 45^0, and surge shaft of 75 m high, the underground powerhouse of dimension (70 x 11 x 10) m.

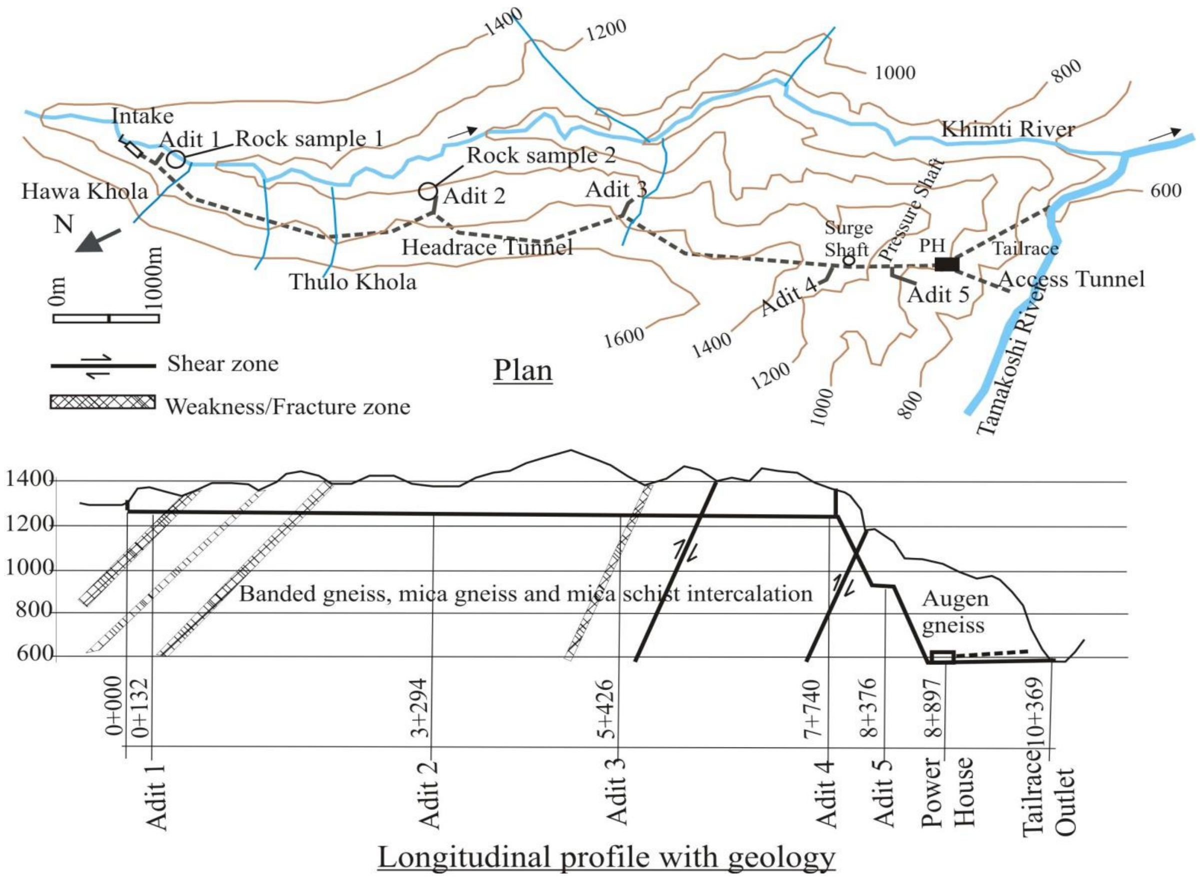


Figure 5.2 Project topography and Longitudinal profile with geology for Khimti-I Hydroelectric (Panthi, 2006)

Project Geology

The project lies in the crystalline Tamakoshi gneisses complex of the lesser Himalayan region. Major rock formations are banded gneiss and Augen mica gneiss which is represented in Figure 5.2. Also, major fault system of Himalayan surrounds the project area commonly known as “Main Central Thrust (MCT)”. Rock mass along the headrace tunnel is highly jointed, sheared, deeply weathered, and deformed, also are influenced by several minor faults and weakness zones (Panthi, 2006). Similarly, according to Q-system rock mass which were ranging from extremely to exceptionally poor, convergence analysis was carried out (Panthi and Shrestha, 2018).

Stability Problems and Rock support

The project had two significant tunnel stability problems. First was tunnel collapse due to the presence of thick bands of highly weathered and sheared chlorite intercalated with talcose mica schist, the second was large leakage through permeable joints in gneisses (Panthi, 2006). Apart from these two instability problems, minor tunnel squeezing was observed due to high overburden in parts of headrace tunnel which had dominance of chlorite and mica schist.

From the planning phase, it was made sure that tunnel will be made unlined and the lining was restricted for surge shaft and weakness areas. Rock mass was classified along Q system and rock support was based on that classification. But the huge discrepancy in rock mass quality was observed during excavation which required huge rock support for the unlined section which increased the construction cost and time.

5.3 Nye Tyin HEP Project

Nye Tyin Hydropower Project is in central southern Norway in between high mountains of Sognefjord Known as Årdal. The plant was constructed right after the end of a second world war. later in 1960s it was upgraded to an annual average capacity of 1398 GW annually. It is a world record project in case of the unlined high-pressure tunnel with a hydrostatic head of 1047m. The project has two units with a total installed capacity of 360 MW with a design discharge of 2 x 20 m3/s. Since it is an upgraded project, it still uses the water from Tyin reservoir but from new intake site created in Torolmen. The system has 11.5 km long unlined high-pressure tunnel (Figure 5.3) and six 1 m diameter brook intake of length ranging from 40-50 m in connection with a tunnel system. Old headrace tunnel is used as a part of surge system and intake to multiple brook inlets (Dawit, 2013).

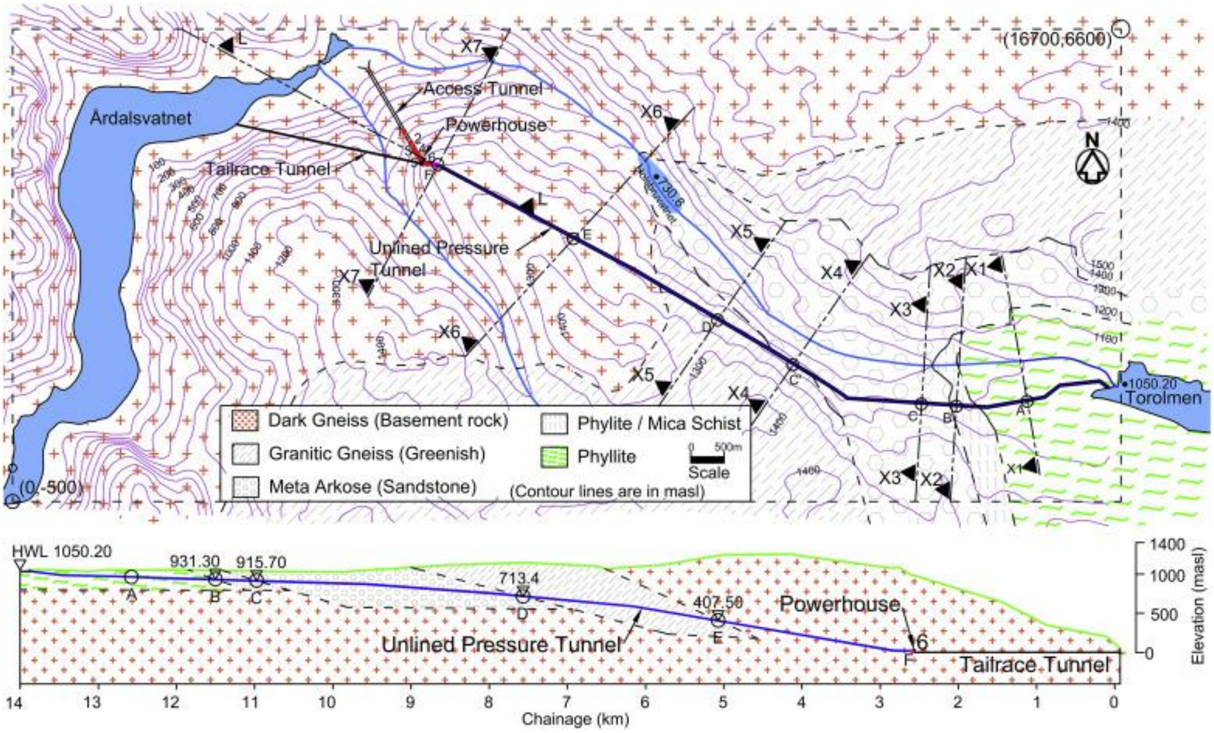


Figure 5.3 Overview of Tyin HEP with topographical ad profile view of tunnel alignment (Basnet and Panthi, 2018)

Headrace tunnel is in area where mainly Cambro Silurian Phyllite that is resulted from overthrust by a large complex known as Jotun-Valdres Nappa. A small amount of homogeneous gneiss, quartzite, and mica schist are present just above this complex. Power station area and the lower part of the pressure tunnel consist of gneiss, gabbro, amphibolite, and pyroxene-granulite (Dawit, 2013). Several minor weakness zones were observed in headrace tunnel before high pressure part starts. However, horizontal stresses and relatively gentle topography were main reasons for the success of such high head unlined tunnel in Nye Tyin Project (Basnet and Panthi, 2018).

6 Himchuli-Dordi Hydro Power Project

6.1 General Information

Himchuli Hydropower is run of river scheme located in Faleni Village area in lamjung district of Gandaki zone (Figure 6.1). It is a project located just upstream of Super Dordi hydropower Project. Both the projects are developed under Peoples Hydropower Company (P.) Ltd. Himchuli-Dordi is one of the many projects that company is initiating with an approach to attain economically feasible solution using renewable energy sources. Himchuli-Dordi mainly utilizes water from Dordi Khola (river) which is perennial in nature and is one of the tributaries of Marsayandi River. It has two intermediate adits at Kyuta Khola and Yulo Khola (Figure 6.2). Discharge from both the river would be collected in headrace tunnel from common access tunnel. The discharge will be released back to Dordi Khola just before the intake of Super-Dordi Hydro project-Kha.

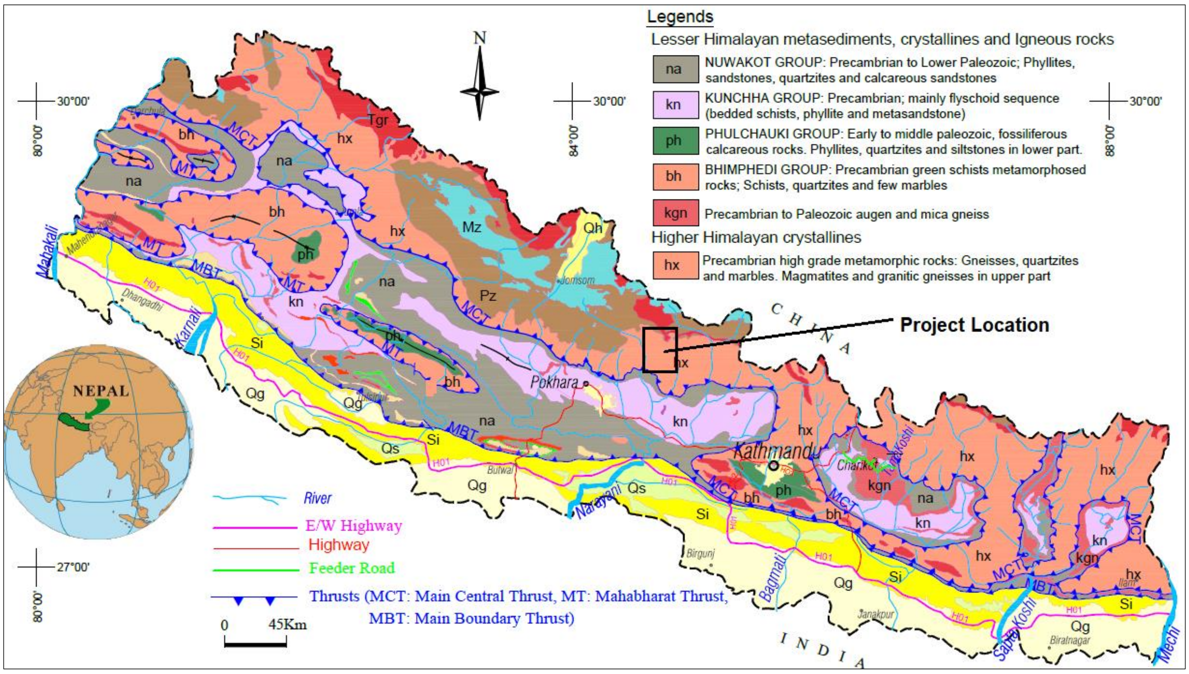


Figure 6.1 Geological map of Nepal Himalaya with project location (Basnet and Panthi, 2017a)

6.2 Project Overview

Himchuli Dordi HPP will utilize total head of 810 m with a design discharge of 5.75 m3/s giving installed capacity of 56 MW. Preliminary design has been conducted and the thesis relies largely on the raw data obtained during this phase. The main hydraulic structures of the project are; diversion weir across Dordi Khola, surface desanding basin, headrace tunnel, surge shaft, underground pressure shaft, and powerhouse which is represented in Figure 6.2. The diversion weir has been proposed across the Dordi Khola at an elevation of about 1847masl, rest of the structures are located on the right bank. The powerhouse is approximately in the elevation of 1240masl. The tailwater will be released into the same river below Taje Village area.

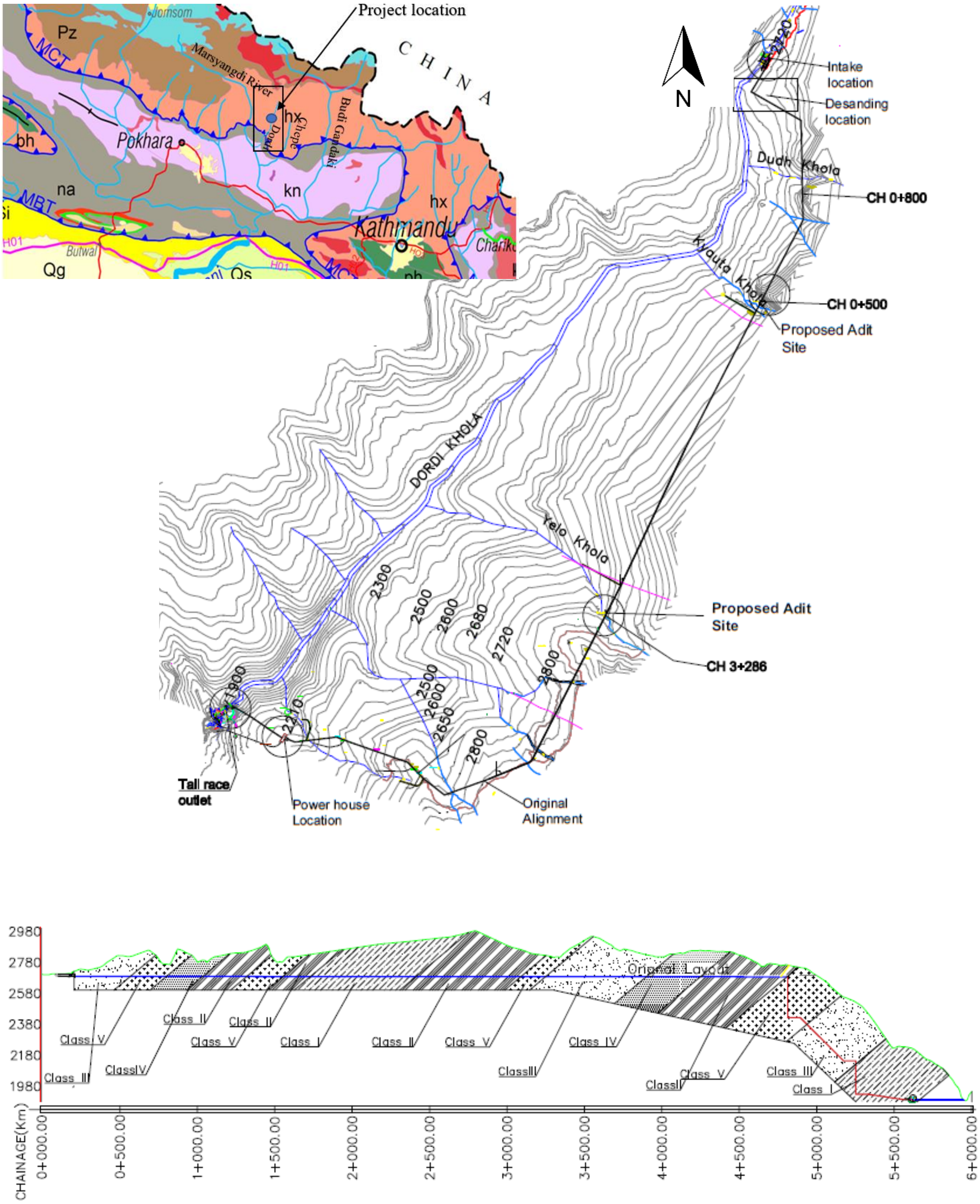


Figure 6.2 Original layout with geological location (top left corner) and a section view (down) for Himchuli-Dordi HPP

6.3 Project Geology and Rock mass condition

Most of the hydropower projects and especially the underground structures are situated in lesser Himalayan or lower part of higher Himalayan (Panthi, 2006). Geological aspects around these regions based on the completed projects would give us the proper visualization with respect to what should we expect as we go deeper. Figure 6.1 gives us the visual division on the geology of Nepal, Figure 6.2 (top left) geology of project area. From the figures the main thrusts are present in between such divisions. The three main thrust is Main central thrust (MCT), main boundary thrust (MBT), and main frontal thrust (MFT). This division presents five main tectonic zones with distinct and unique geological features. Salient features of each tectonic zones are summarized in Table 6.1, which is based on different geological features and rock types available among each region.

Table 6.1 Geomorphic units of Nepal (Upreti, 1999)

Tectonic zone	Altitude (m)	Width (Km)	Main rock types	Geological age
Gangetic plain and terai	100-200	20-50	Alluvial deposits, coarse gravel at foothills of Siwaliks.	Recent
Siwalik/sub Himalayan zone	200-1000	15-30	Sandstone, mudstone, siltstone, shale, conglomerates etc.	Mid-Miocene to Pleistocene
Lesser Himalayan zone	200*-5000	70-165	Shale, slate, phyllite, limestone, dolomite, marble, schist, quartzite, gneiss and granite.	Precambrian to Mesozoic
Higher Himalayan zone	>5000	10-60	Gneiss, schist, marble, granite, quarzite, amphibole etc.	Precambrian
Tibetan Tethys zone	>2500	….	Gneissic schist, marble, shale, slate, limestone, sandstone etc.	Late Proterozoic to early Cambrian

*In the lesser Himalayan Valleys, the elevation ranges from 200-2000 meters

The Himchuli-Dordi Hydropower project (Figure 6.1 & Figure 6.2) lies in the Lesser Himalayan zone. It is located to the north of Main Central Thrust which can be identified as Himalayan Gneisses Zone. Generally, rocks along the MCT are intensely sheared and mylontized. Other rocks are supposed to be metamorphic rocks with crystalline Gneisses, with typical types like kyanite gneisses, augun mica gneisses, and granite gneisses. Same can be expected while excavating in this region (Panthi, 2006) . From the geological report for Super Dordi HPP, various gneisses and augen gneisses were found in this zone. The general trend of the rock mass foliation is dipping towards north east upstream. The Himalayan gneiss masses are possible of Midland meta-sediments and traced as a basement of the Tibetan Tethys sediments. They are intruded and reactivated by the later intrusion of granites and thrust over to the Midland meta-sediment terrain in the south (Peoples Hydro, 2018).

Table 6.2 Rock mass Classification for the Project Area (Panthi, 2018)

Rock class	Quality description	Q Value	% of Rock available	
			A 1	A 2
I	Good to very good	>10	11 %	16 %
II	Fair to good	4-10	22 %	30 %
III	Poor	1-4	33 %	16 %
IV	very poor	0.1-1	12 %	26 %
V	Extremly to exceptionally poor	<0.1	22 %	12 %

Form the data available so far and geological maps in hand the geological formation of rock mass along the project area can be classified as in Table 6.2. The main rock types found in the project area is banded gneiss with schist partings. These formations are extensively present in the Lesser Himalayan unit which extend east-west throughout the Himalayan range. Slightly – moderately weathered, medium foliated, strong gneiss rock is present in most of the downstream of tunnel and powerhouse. Altogether two types of soil are expected like in super Dordi project area, colluvial and alluvial soil. The colluvial soil made of the high content of fine silt and clay are present in cultivated land and slopes. Similarly, alluvial soil made of clean, predominantly coarse-grained material (sand, gravel, cobble, and boulder) are present in headworks and powerhouse area.

7 layout Alternatives of the Case

Original layout for the project can be seen in Figure 6.2. Layout alternatives are explored so that cost and time can be saved with a simple design. Design alternatives chosen for the thesis was decided from the discussion with supervisor and co-supervisor.

7.1 Layout Alternatives

There are two main alternatives Alignment 1 & Alignment 2 considered for this project. Each one is then categorized with probable alternatives with the unlined sections. Which caters 6 alternatives of distinctive features. They are named A 1.1, A 1.2 & A 1.3 for alignment 1 similarly A 2.1, A2.2 & A 2.3 for alignment 2. The common section has been pointed in the figure with CH 3+286, which refers that section upstream of that point will be same for all alignment so, the comparison is maintained for the alternatives behaving separately downstream from that point on. The profile shown below also covers the alternatives from common section onwards. The dark blue lines in cross sections refer to unlined headrace and tailrace, red line indicate steel penstock lining.

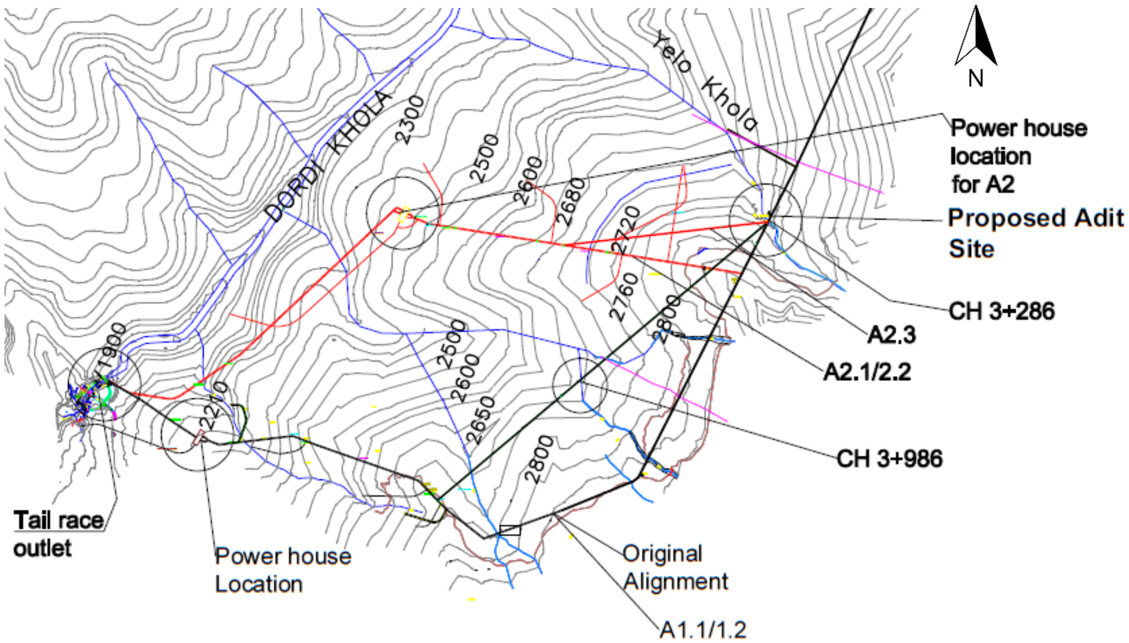


Figure 7.1 Layout of Alternative Alignments

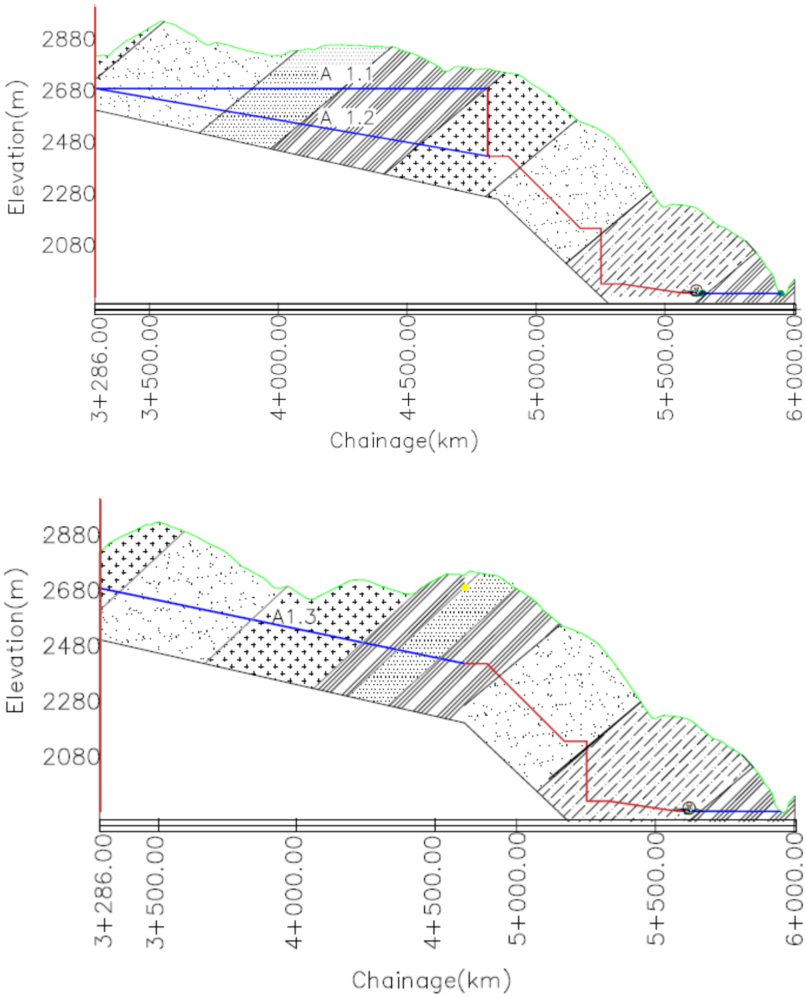


Figure 7.2 Profile view for alternative alignment A1.1, A1.2 & A1.3 (Bottom)

Alternatives are analyzed with the methods as described in Chapter 4. The methodology involves first hydraulic criteria, which investigates into the surge effect and need of positioning of the surge shaft. Second steps would be to check the geometrical confinement for each section with respect to unlined tunnel criteria. If both the assessment is successful rock quality class are designated based on the lineaments and weakness location area present. When rock quality/class is established, assessment is carried out based on semi-analytical and numerical methods.

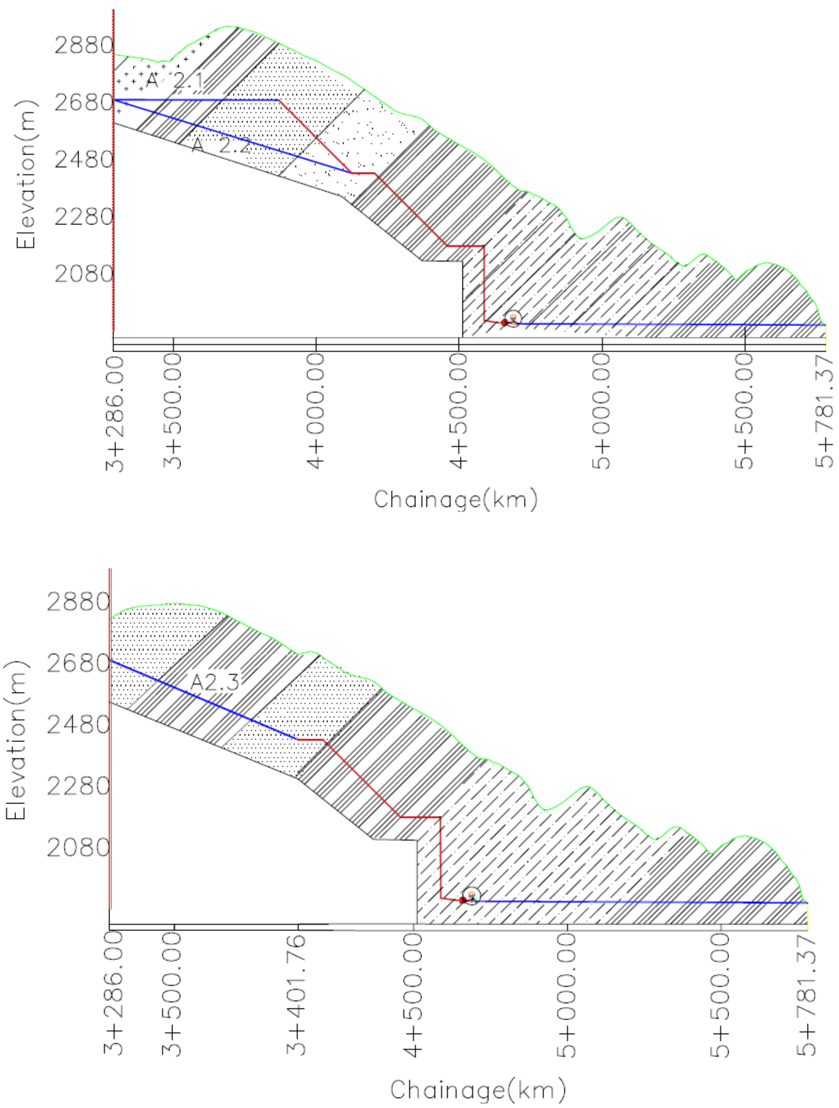


Figure 7.3 Profile alternative for Alignment A2.1, A2.2 & A2.3(Bottom)

7.2 Hydraulic criteria

Schemes considered in this stage were underground and flowing in pressure. According to Chaudhry (2013) as a rough rule of thumb for governing stability and good regulation of hydropower system, the provision of a surge tank should be investigated from the expression in Equation. (14). Six alternatives investigated were A1.1, A1.2, A1.3, which represents original layout, new unlined layout in the same alignment, and the last one is new alignment away from main layout (Refer Figure 7.1). Same is true for alignment A2.1, A2.2 & A2.3.

Each alternative is first investigated for the need of surge shaft based on the criteria stated in equation (14). If it satisfies the criteria as mentioned surge shaft need is ascertained. Then penstock time constant need to be calculated from equation (15). Penstock time constant is defined as the time required for the water in penstock and draft tube to reach at full design discharge Qo from zero, under the design head of Ho.

$$\frac{\sum L_i \cdot V_i}{H_n} > 5 \tag{14}$$

Where:

L_i: Waterways length from intake to tuthe rbine (m).

V_i: Flow velocity in waterways from intake to tuthe rbine (m/s).

H_n: Net head for design flow (m).

$$\mathrm{Tw} = \frac{\mathrm{Qo}}{\mathrm{gHo}} \sum \frac{\mathrm{L}}{\mathrm{A}} \tag{15}$$

Where,

L: length of the waterway

A: area of the waterway

Q_o: Design discharge

H_o: Design head

Then criteria

$$\frac{\mathrm{Ta}}{\mathrm{Tw}} > 6 \tag{16}$$

Where,

Ta: time is taken by the generator to accelerate from zero to normal speed with full torque (5 sec <Ta< 8 sec)

When a requirement in Equation (16) is maintained it is understood that length of the tunnel from the turbine to design surge location is in accordance. If the requirement is not provided either length between surge point to turbine must be decreased or area of the tunnel must be increased. Table 7.1 summarizes the calculation which satisfies the requirement. After that, we can design the surge tank area and correspondingly the upsurge and down surge from equation (19). Now we know that both projects alignments require a surge tank. The minimum free water surface area in surge shaft is required to damp the mass oscillation between shaft and reservoir, which is calculated using the Thoma Criteria (Equation (17) and Equation (18), refer Table 7.1 for the summary of results.

$$\mathrm{A_{Thoma}} = 0.0083 \frac{\mathrm{M^2 A_{tunnel}}^{5/3}}{\mathrm{H_o}} \tag{17}$$

But it is recommended to maintain following criteria

$$\mathrm{A}_{\text{surge shaft}} > 1.5\ \mathrm{A}_{\text{Thoma}} \tag{18}$$

Where:

A_{st}: Surge shaft area

M: Manning Roughness coefficient

A_{tunnel}: Area of the tunnel

Upsurge calculation is performed using the equation below:

$$\Delta z = \pm \Delta Q \sqrt{\frac{L/A_T}{gA_{st}}} \quad (19)$$

Where,

ΔZ: Surge in the shaft

L: length of the tunnel from the nearest free water surface

Table 7.1 Summary of Surge Analysis for different alternatives

		Alignment 1			Alignment 2		
Parameters	Units	A 1.1	A 1.2	A 1.3	A 2.1	A 2.2	A 2.3
Design discharge Q_0	m3/s	5.75	5.75	5.75	5.75	5.75	5.75
Net head for design flow	m	801.64	801.91	802.25	803.44	803.39	803.71
Design diameter of surge	m	3.00	3.00	3.00	3.00	3.00	3.00
Designed Ast	m^2	7.07	7.07	7.07	7.07	7.07	7.07
ΔQ ($\pm$)	m	12.07	10.82	10.82	11.69	13.50	12.67
water level at surge tank	masl	2692.90	2692.87	2693.22	2694.08	2694.39	2699.75
water level in intake of surge tank	masl	2680.84	2682.04	2682.39	2682.39	2680.89	2687.08
Maximum upsurge	masl	2704.97	2703.69	2704.04	2705.77	2707.90	2712.42

7.3 Confinement Criteria for unlined tunnels

From the layout design (Figure 7.1) chainage 3+734 m in headrace tunnel is common for all the alignments. For alignment A1.2 & A 2.2, rock cover is same as in original alignment A1.1 & A2.1. Since A1.3 & A2.3 are deviating from the main alignment its calculation is performed separately. Summary of all the alternative calculation is presented in Table 7.2 & Table 7.3.

Total 6 alternatives thus investigated where A1.1, A1.2, A1.3, representing original layout, unlined layout in the same alignment, and changed alternative with unlined orientation. Same is true for alignment A2.1, A2.2 & A2.3. In the first alternative of A1 & A2, investigation was carried out for the stability of unlined section within the same design. The alignment passes through the Norwegian rule of thumb and then looks for the availability of minimum principal stress in the critical section like ridges and probable lineament present areas.

Table 7.2: Norwegian Rule of thumb criteria for common sections for Alignment 1 & Alignment 2

Section	Chainage	H (m)	h1 (m)	h2 (m)	*F	L1 (m)	L2 (m)	**F
First ridge section	0+830	12.13	66.33	4.54	14.62	66.33	9.90	6.70
	0+780	11.99	73.71	4.48	16.44	73.71	6.64	11.11
	0+730	11.85	133.67	4.43	30.16	95.76	6.56	14.60
second ridge section	1+580	14.22	109.83	5.32	20.65	84.99	7.20	11.80
	1+530	14.07	82.87	5.26	15.75	66.46	5.85	11.36
	1+480	13.94	137.54	5.21	26.38	66.87	6.16	10.86
Third ridge section	3+347.7	19.13	122.44	7.15	17.11	122.44	7.82	15.66
	3+297.6	18.99	124.95	7.10	17.59	124.95	7.53	16.59
	3+247.6	18.86	135.26	7.05	19.18	135.26	7.43	18.20
	3+197.6	18.71	144.77	7.00	20.69	144.77	10.36	13.98
	3+146.	18.52	155.22	6.93	22.41	155.22	8.09	19.19

*F= Available Rock cover (h1)/Rock cover required (h2), **F=Available horizontal shortest length (L1) to length required (L2)

All the common section satisfies the criteria, the similar check has been performed for every alternative, which can be seen from Table 7.3. All the alignment alternative begins with 3+286.47 but ends at their respective length. A1.3 & A2.3 ends at different length because of higher inclination. The factor of safety for every section is above the acceptable range. Alignment A1.3 Ch 4+056.47 has the lowest factor of safety of 1.52, which is also above the range. Which concludes that geometrical location of the tunnel inside the ground surface has the sufficient overburden.

Table 7.3: Summary of Norwegian Rule of thumb criteria for all the alternatives

Alignment	chainage	H	h1	h2	*F	L1	L2	**F
A1.1	3+286.5	11.58	116.48	4.33	26.89	116.48	4.40	26.46
	4+141.3	12.02	133.13	4.50	29.61	133.13	5.00	26.65
	4+241.3	12.09	136.8	4.52	30.25	136.8	5.17	26.46
	4+341.3	12.12	147.72	4.53	32.59	147.72	4.72	31.30
	4+820	12.59	73.37	4.71	15.58	73.37	4.77	15.39
A1.2	3+286.47	11.58	116.48	4.33	26.89	116.48	4.40	26.46
	4+141.33	123.91	245.02	46.60	5.26	245.02	51.51	4.76
	4+241.33	140.36	265.07	52.79	5.02	265.07	60.03	4.42
	4+341.33	157.62	293.22	59.28	4.95	293.22	61.38	4.78
	4+820	276.5	337.28	103.99	3.24	337.28	104.70	3.22
A 1.3	3+286.47	11.58	337.28	4.33	29.57	128.06	5.11	25.04
	3+986.47	156.42	140.5	59.23	2.37	140.5	69.88	2.01
	4+056.47	170.92	115.78	64.72	1.79	115.78	76.36	1.52
	4+572.12	276.5	324.06	104.70	3.22	337.28	104.70	3.22
A2.1	3+286.47	18.42	140.5	6.89	23.09	159.07	7.36	21.62
	3+300	18.86	115.78	7.05	16.41	115.78	7.77	14.90
	3+733.07	20.17	197.12	7.54	26.13	173.53	8.91	19.48
A2.2	3+286.47	18.42	159.07	6.89	23.09	159.07	7.36	21.62
	3+300	63.65	179.19	24.65	7.27	179.19	26.23	6.83
	3+989.3	276	321.87	106.89	3.01	246.14	121.89	2.02
A 2.3	3+286.47	18.42	113.15	113.15	15.27	113.15	8.04	14.07
	3+586.47	144.99	155.96	155.96	2.67	155.96	56.15	2.78
	3+896.47	276	321.87	321.87	2.9	275	121.89	2.26

7.4 Rock mass quality

Data available from geological maps and stress situation from the geological formation along the project section has rock classification types as presented in Table 6.2.

Table 7.4 shows the material properties that are used in the overall section of the underground system based on the rock mass classification stated in Table 6.2. It is to be noted that the calculated properties are based on limited information of the geological features of the project area. Individual parametric features have been discussed in Section 3.2. This table is generated using software Rocdata a Rocsciene package, which specialize in analysis of rock and soil strength envelop using different strength criteria (Rocscience, 2018). It is used as model input for RS^2 analysis in every section observed.

Table 7.4 Rock mass properties for the project site calculated using Rocdata software

Rock type Genisses	Rock Class					
Hoek-Brown Classification	I	II	III	IV	V	Units
Sigci	39	39	39	39	39	MPa
GSI	60	50	45	30	25	
mi	23	23	23	23	23	
Poissions ratio	0.2	0.2	0.2	0.2	0.2	
Disturbance factor (D)	0	0	0	0	0	
Ei	19565	19565	19565	19565	19565	
Hoek-Brown Criterion						
mb	5,512	3.86	3.226	1.89	1.58	
s	0.012	0.004	0.002	0.0004	0.0002	
a	0.503	0.506	0.508	0.52	0.53	
Failure Envelope Range						
Application	General	General	General	General	General	
sig3max	9.75	9.75	9.75	9.75	9.75	MPa
Mohr-Coulomb Fit						
Cohesion (c)	2.861	2.497	2.33	1.84	1.66	MPa
Friction Angle (ϴ)	40.724	37.72	36.21	31.62	30.06	degree
Rock Mass Parameters						
Tensile strenght	-0.083	-0.04	-0.03	-0.01	-0.01	MPa
Uniaxial compressive strength	4.173	2.35	1.75	0.67	0.47	MPa
Global strength	12.477	10.17	9.18	6.58	5.77	MPa
Modulus of deformation	10173.8	6010.09	4375.71	1592.26	1171.08	MPa

For the Himchuli project disturbance factor is taken as 0.5 assuming on the basis that rock type Genesses is of good quality and will have moderate effects on surrounding. Headrace tunnel for the project has size of 3.2 m, powerhouse cavern size is of (18x46x33) m. Mean joint spacing is assumed to be less than these openings so Generalized Hoek Brown Failure criterion is used in all the numerical calculations. Rock Data is used to calculate the input parameters for the analysis. Since our project is in Himalayan region which has significant weathering effect, therefore it must be considered beforehand because rock mass properties like strength, deformability and other properties are likely to get altered with it. From the investigation performed at Dordi HPP downstream of the Himchuli-Dordi HPP value for σ_{ci} of 39 MPa is used as representative figure. The major rock formation identified is Gneisses, from Figure

3.1.the range is presented from 28 ± 5 for that rock type. This value is used during the numerical analysis of the project. From the investigation performed at Dordi HPP downstream, the rock formation for Himchuli-Dordi was selected as very blocky- interlocked, partially disturbed rock mass, multi-faceted angular blocks formed by 4 or more joint sets with very good to fair surface condition i.e. GSI is taken in range 50-60.

Values for the density, modulus of elasticity and Poisson's ratio has been taken from the laboratory reports of the Dordi HPP as shown in Table 7.5. Also, form the case study MMHEP has similar features as in our project so value considerations are done with respect to that project to be in the probabilistic zone.

Table 7.5 Other mechanical Properties

Property	Himchuli HPP
Density	2.7 g/cm3
Modulus of Elasticity	19.6 GPa
Poisson's ratio	0.2

7.5 Stress criteria

The main task is to know the stress situation in the tunnel section using numerical analysis. It is well-accepted theory that water pressure in any part of the tunnel must be lower than the minimum principal stress. Value of minor principal stress in the critical section even though it has sufficient overburden may be smaller than the hydrostatic pressure. If that is the case unlined tunneling in that section needs special consideration. RS ^{2} uses two different filed stress application types. When virgin stresses are being analyzed in valley model, gravity field stress type is selected so that it can incorporate the changing ground elevation. For that total stress ratio of 0.25 (calculated using equation ν /1-ν) is assigned for all the sections. Since the alignment is changing from section to section, the locked in horizontal stress in-plane and out-plane also changes accordingly. But for analytical calculation tectonic stress in headrace section is taken as 1.5 Mpa and for powerhouse cavern 3.5 Mpa (Panthi, 2018).

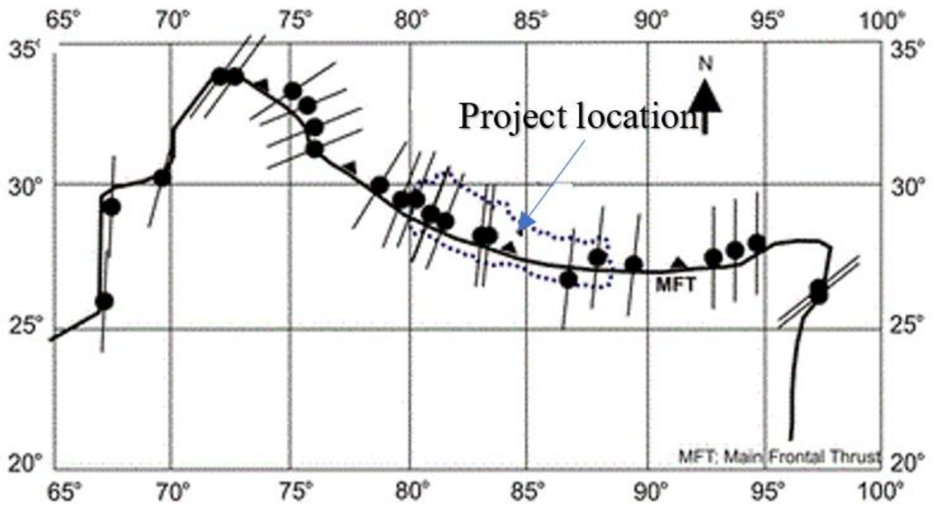
7.5.1 Input For the model

Valley formation in the Himalayan region is distinct. The cross-section should thus be chosen to have the clear view of valley section and effect of stress distribution along the tunnel location. Locked in stresses are calculated by using the equation (20) and equation (21). The value alters along with the angle Θ (angle between tectonic stress and the out of a plane from the tunnel orientation) so we get different locked in pressure in each case which adds to new scenarios to the analysis. Another variable data is the horizontal stress along each section due to the different rock cover. In case of alignment 1 and alignment 2 particularly in common section, we have taken the average horizontal stress due to small variation in rock cover. In case of critical section viz. junction area, valley individual horizontal stresses are considered.

In-situ stresses

In-situ stresses in rock mass are the result of overlying strata, plate tectonics, and stresses due to topographic effects. Generally, in-situ stress is measured using methods like hydraulic fracturing, 3D over coring. Since it is also mentioned in the section 1.5, we don't have measured data for the selected site. So, we investigate reviewing from similar nature of projects. The primary concern is that minimum principal stress should be sufficient enough to withstand the hydrostatic pressure at that point to keep the unlined tunnel safe from hydrofracturing and other stress-related failures.

Basnet and Panthi (2017a), has used tectonic stress magnitude of 15 MPa with orientation N350E in their case. From Figure 7.4 it can be approximated that trend of tectonic stress for Himalaya is NE-SW at the north-western part and around N-S towards southeastern part. Project location has a similar orientation. Still to confirm three-dimensional stress measurement is required and is proposed as suggestion.



Note: stress tensors and international boundaries are not in true scale.

Figure 7.4 Approximate horizontal tectonic stress orientation (Panthi, 2012).

Since all the alignment does not have the same orientation as the tectonic stress, so it will develop the shear stress along the cross section. Which can be defined as in-plane and out-of-plane stress depending on section alignment considered with tectonic stresses. The following formula has been used for estimating shear stress thus generated.

$$\sigma_{yy}=\sigma_{tec}\,Cos^2(\Theta)+\sigma_h \tag{20}$$

$$\sigma_{xx}=\sigma_{tec}\,Sin^2(\Theta)+\sigma_h \tag{21}$$

Where σ_{xx} is in-plane and σ_{yy} out of plane shear stress. Θ is the angel made by tectonic stress with out of plane section and $\sigma_{h,}$ σ_{tec} are horizontal and tectonic stresses respectively.

7.5.2 Model Output:

With the use of software RS^2 and inputting the information from the location, valley model for Section A1.1 A1.2 &A1.3 is generated as shown in Figure 7.5& Figure 7.6 for all the measured section. Corresponding principal stresses from the model are presented as in Table 7.6 & Table 7.7. It is to be noted that Orientation of stress in RS^2 is measured in Counter clockwise direction(CCW). Same process is applied for the alignment A2 with valley model generation.

a) Valley model for Alignment A1.1 and A1.2 at chainage CH-2+241

Rock class type at the chainage CH-2+241 is III as can be verified from Figure 6.2 and rock mass properties is selected from the Table 7.4 into the model. The summary of stress distribution and orientation in both the alignment at common chainage CH-2+241 is summarized in Table 7.6 below.

Table 7.6 Principle stress with orientation for Alignment A1.1 & A1.2 for CH-2+241

Principle stress	A1.1	A1.2	Units
σ 1	3.8	6.9	Mpa
σ 3	0.88	1.64	Mpa
σ z	6.84	9.71	Mpa
Orientaion	273	273	*CCW

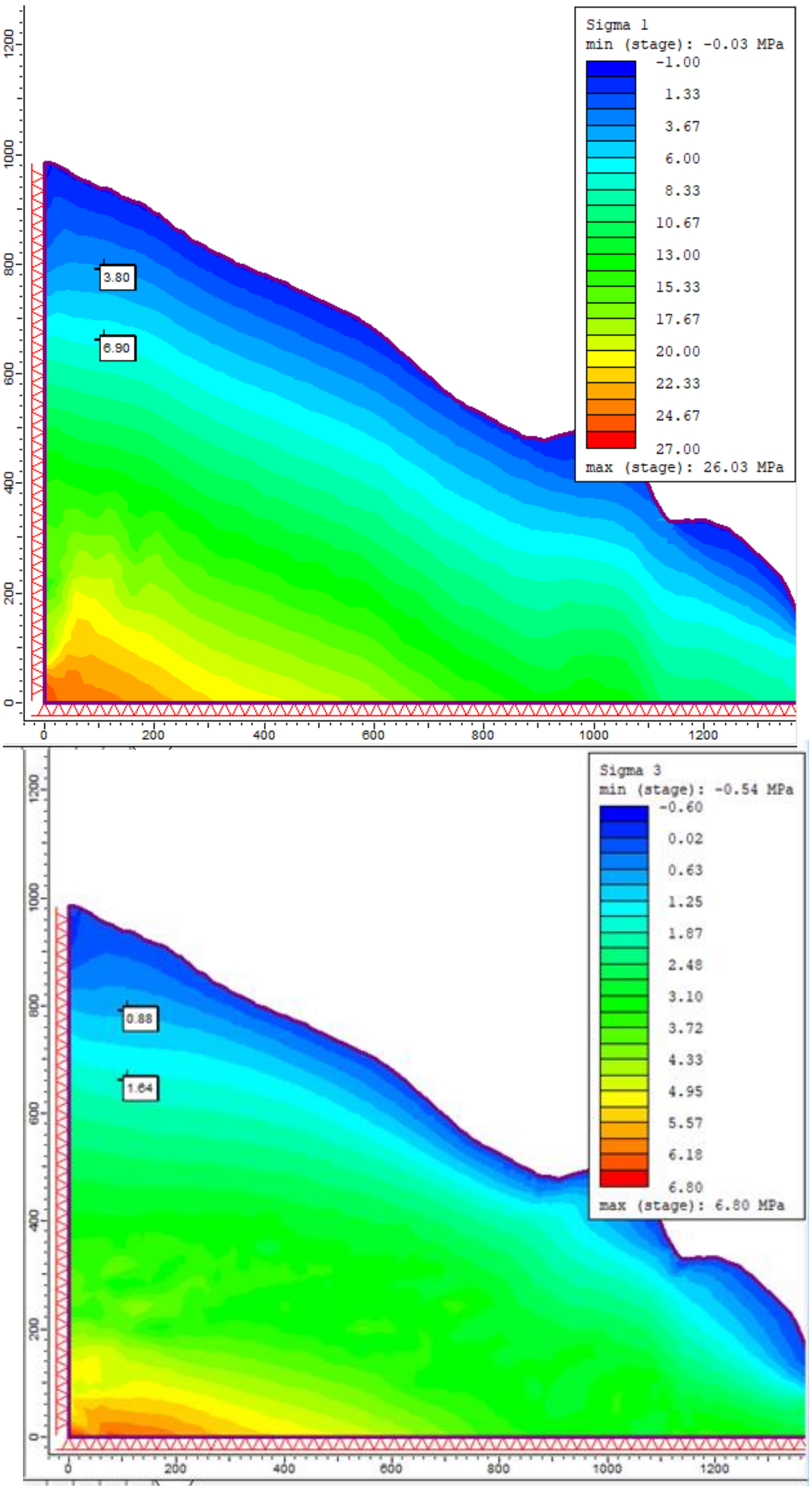


Figure 7.5 Maximum Principal stress &Minimum Principal stress at chainage CH 2+241 for A1.1 (top value) &A1.2 (bottom value)

b) Valley model for A1.3 Chainage CH-4+095

For alignment A1.3 at chainage CH-4+095 classification of rock class is IV which signify poor to very poor as presented in Table 6.2 and corresponding rock mass properties are taken from Table 7.4. The valley model is then prepared for the Alignment A1.3, which is presented in Figure 7.6. The stress distribution in that chainage can be summarized as shown in table below

Table 7.7 Principle stress and Orientation obtained from valley model For A1.

Principal stress	A1.3	Units
σ_1	3.83	Mpa
σ_3	1.11	Mpa
σ_Z	12.75	Mpa
Orientation	279	CCW

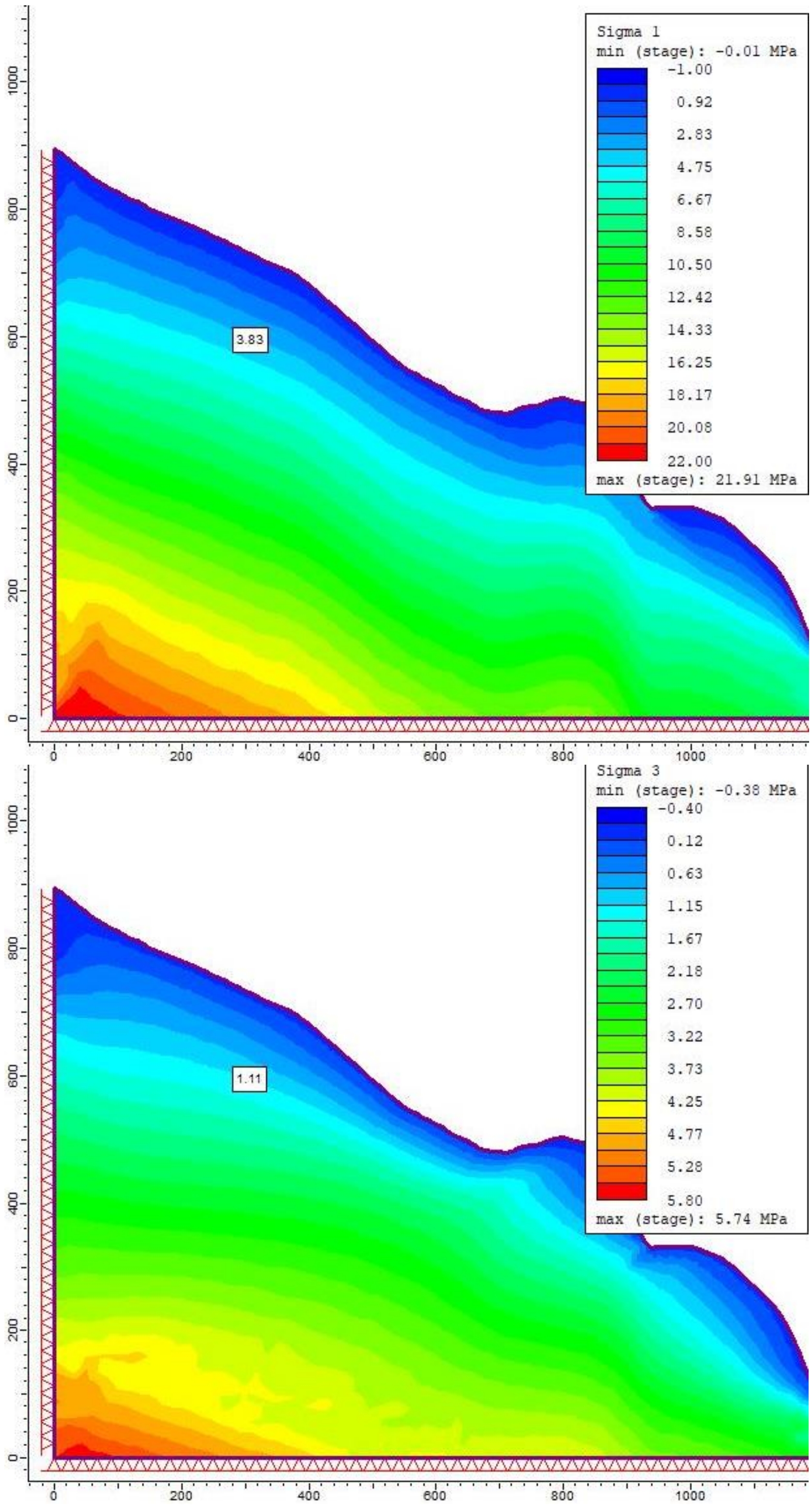


Figure 7.6 Figure A1.3 CH-4+095 Valley model with maximum principal stress and minimum principal stress(bottom) at tunnel location

7.5.3 Result interpretation

When valley model is designed for critical points along the length for all the alignment, stress distribution can be generalized along the length as presented in Figure 7.7 and Figure 7.8. The plot is presented against available minimum principle stresses which is generated from valley model and required minimum principle stress to counter net hydrostatic head available at the point. The factor of safety is taken into consideration from Table 2.1. The region where the available minimum principal stress has the factor of safety 1.3 is considered okay and if less than that it is not considered okay. So, chainage 3+986.47, 4+056.47, 4+095 from Alignment A1.3 has not passed the condition, similarly, chainage 3+989.32 of Alignment A2.2 and 3+897.47 of Alignment A2.3 has also not met the criteria. These sections are investigated in more detail for the stability reasons. Keeping it unlined with normal support criteria for stability will not fulfill the purpose in this section, so lining must be performed in these sections until the safe zone is not reached.

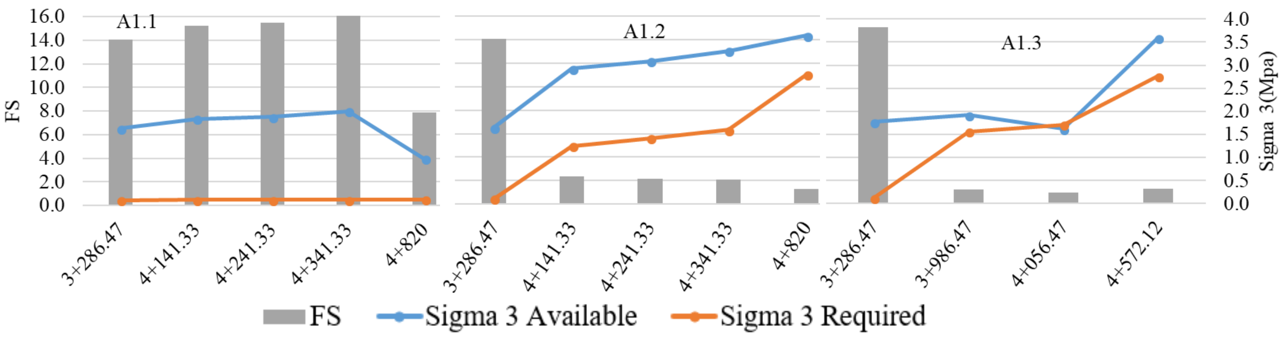


Figure 7.7. Plot showing the minimum Principal stress available and required along the alignment 1

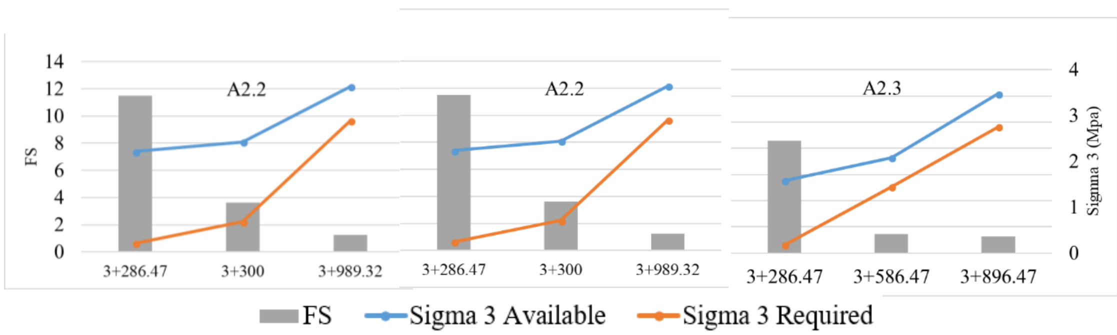


Figure 7.8 Plot showing the minimum Principal stress available and required along the alignment 2

7.6 Stability Assessment

Stability of the tunnel section and cavern section are analyzed under two headings, semi-analytical and numerical analysis. First one investigates the tunnel strain present due to the stress situation available around the periphery in the underground. Second deals with overall displacement after excavation in presence of all kind stress situation.

In this thesis after meeting with the supervisor and considering the Himalayan Geological scenario a set of support are allocated as shown in Table 7.8. The support system can also be compared with Q-chart (Figure 7.9). In weak rock mass, even when there is no considerable deformation, it is advised that minimum layer of shotcrete and bolt patterns are to be installed for safety purpose.

Table 7.8 Support specification for the Project based on the rock mass classification (Panthi, 2018)

Rock class	Support specification for the project		
	SRF(cm)	S-bolting (m)	Steel ribs (m)
I	10	1.5x1.5x3	
II	10	1.2x1.5x3	
III	15	1.2x1.2x3	
IV	20	1x1.2x3	
V	25	1x1.2x3	1.1x1

This specification has been compared with the Q charts as presented in Figure 7.9, to have the range of option to accommodate the changing scenarios. For the case project, Q value > 10 is considered good to very good and is provided with 10 cm shotcrete layer combined with systematic bolting of 1.5m in-plane spacing. From the Figure 7.9 same Q value can have spot or systematic bolting with bolt spacing ranging from 2.3-2 m depending on the use of shotcrete layer or not. Assumption for the case project has the high end of support system then proposed in the Q-table. In other rock class types also, we have a similar assumption.

Chart below provides an option of using a single (E)or Double (D) layer of Reinforced Ribs of shotcrete (RRS) for very poor to extremely poor rock class type. While using this option it is considered that displacement in rock has an immediate effect, so right after excavation temporary installation helps to slowly relax the deformation and finally RRS option can be used as permanent support in the long run (Grimstad et al., 2002). After certain thickness Double layer is preferred than single high thickness layer. This is in accordance with less or no geological investigation upon the project site. In every case giving a higher factor of safety based on recent and relevant project experience leads to an approachable design.

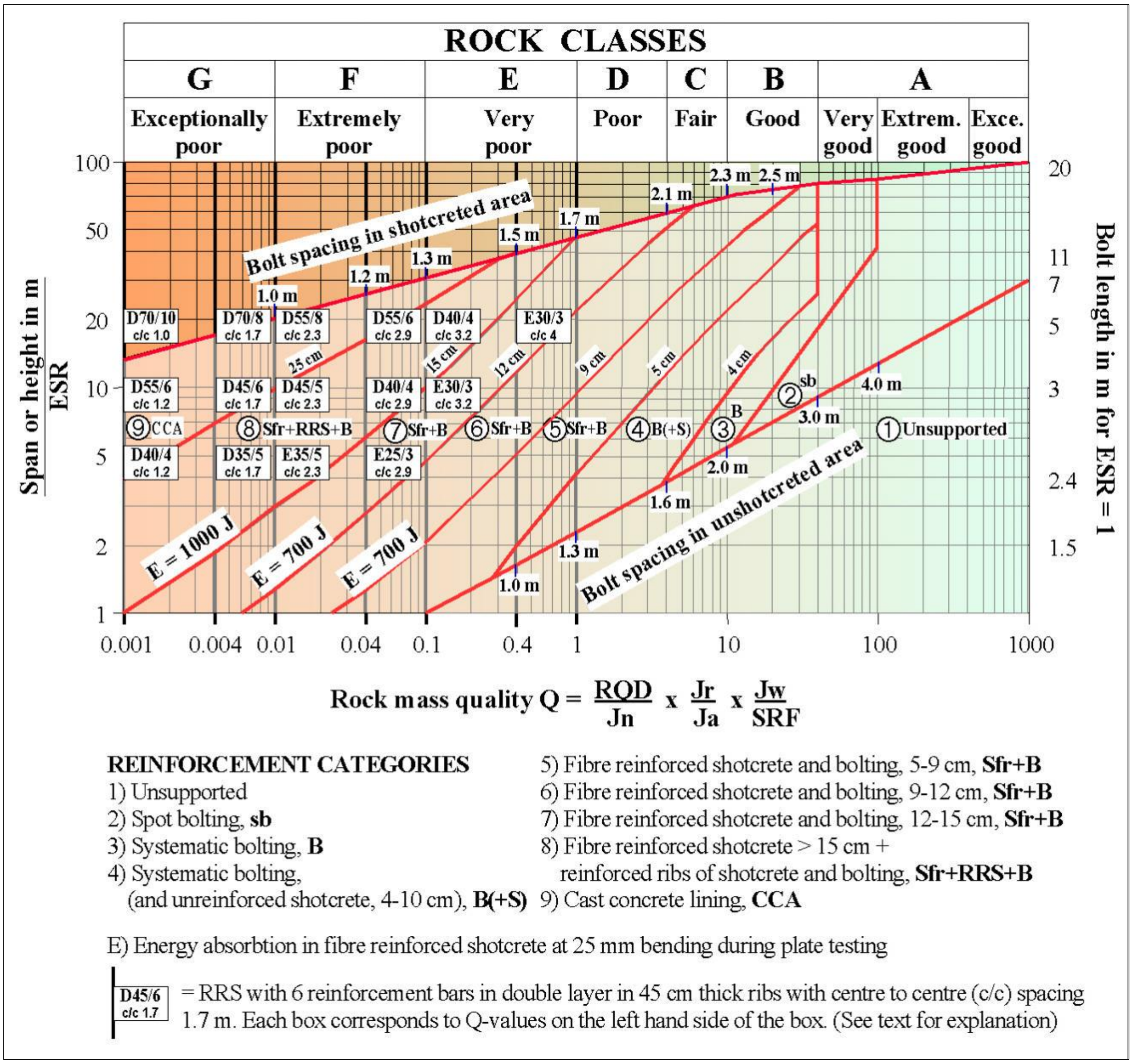


Figure 7.9 Q-chart with recommended thickness, number of rebars in a single (E) or double layers (D) and spacing between the ribs (RRS) in different rock mass qualities, Q for 5, 10 and 20m spans. (Grimstad et al., 2002)

7.6.1 Semi-analytical analysis

Rock mass properties are calculated using Hoek and Brown criterion as discussed in Chapter 4. It is to be noted again that project is situated in the Himalayan region so, it was useful to use equation proposed by (Panthi, 2006) for the unconfined compressive strength of rock mass (σ_{cm}) as expressed in Table 3.1.

Table 7.9 Tunnel strain calculation in common section of headrace tunnel.

Alignment	Chainage	σ_{cm}	σ_v	σ_{cm}/σ_V	Tunnel strain	Remarks
A1.2	4+341.33	4.06	7.92	0.51	0.76	A
A1.2	4+820	4.06	9.11	0.45	1.01	B
A1.3	1+285.65	4.06	9.11	0.45	1.01	B

From the tunnel strain calculation based on the Hoek and Marion's criteria, tunnel strain developed for all section under analysis can be categories in two classes. Table 7.9 shows the highest tunnel strain point and corresponding value. The significance of A and B are expressed in Table 4.2. Even though other sections have no significant tunnel strain, minimum support criteria as assigned earlier are still used for safety reasons.

7.6.2 Numerical Analysis

In this thesis, a comprehensive 2-dimensional finite element (RS^2) has been used to incorporate all the scenarios due to topographical conditions, tectonic influence and due to excavations. RS^2 is 2D elasto-plastic finite element program used to estimate the stresses and displacement around underground and surface excavation in both rock and soil (Rocscience, 2018).

After having the approximate idea of the expected tunnel strain pattern around the tunnel alignments (section 7.6.1), critical sections are identified based on an analytical and semi-analytical calculation performed. The chosen sections are further investigated with help of RS^2. The process then involves two basic steps.

- Accounting the virgin stresses from the valley model as performed in section 7.5 in the selected chainages
- Stability analysis into those critical sections after excavation using the virgin stresses and installing support types as designed from Table 7.8, then alter the support specification if it is not sufficient.

Input Parameters for the model

Material properties are generated using software Rocdata as presented in Table 7.4. Panthi criteria (Table 3.2) is used to calculate deformation modulus, Em since it is applicable in Himalayan geology. Other input data and application procedure is same as explained section 7.5.1.

7.6.3 Output results

Each alignment has a certain critical section. Those sections are either identified from analytical, semi-analytical calculation performed in section 7.3 and section 7.5 or Probable geological locations example valley section or other discontinuities. Below a representative model for those identified sections are analyzed and presented. The process involves generating valley profile at the critical section, that profile is then analyzed with the RS^2 to get the probable in-situ stress situation. Thus, obtained stresses and orientation angle is taken as input into the cross-section of the opening. Principal stresses in RS^2 and their connotation are explained in section 4.3.1. First, each section is performed under elastic state, then results are noted. If the strength factor is below one around the excavation boundary it is expected to fail when left without any support application. Next step is to look the scenario in the plastic state. Support is applied, and total displacement is compared with before and after state. Support applied may not be sufficient sometime, so the systematic addition of support component is done afterward.

a) Alignment A1.1 at chainage CH-4+241

loading obtained from the valley model is applied and interpreted. In Figure 7.10 the stress orientation near invert and roof are highly stressed. Total displacement is 8.2 mm in invert section. Strength factor is also below 1 Figure 7.10 (bottom)) which signifies that tunnel will collapse if the support system is not applied. Now plastic analysis is performed to look for safety issues.

From plastic mode, maximum displacement is reflected in invert and roof section see Figure 7.11. After support application maximum value is reduced from 14 mm to 10 mm. Invert section has the largest displacement of 10 mm, where other section does not show any significant displacement. Summary of support is specified in Table 7.10.

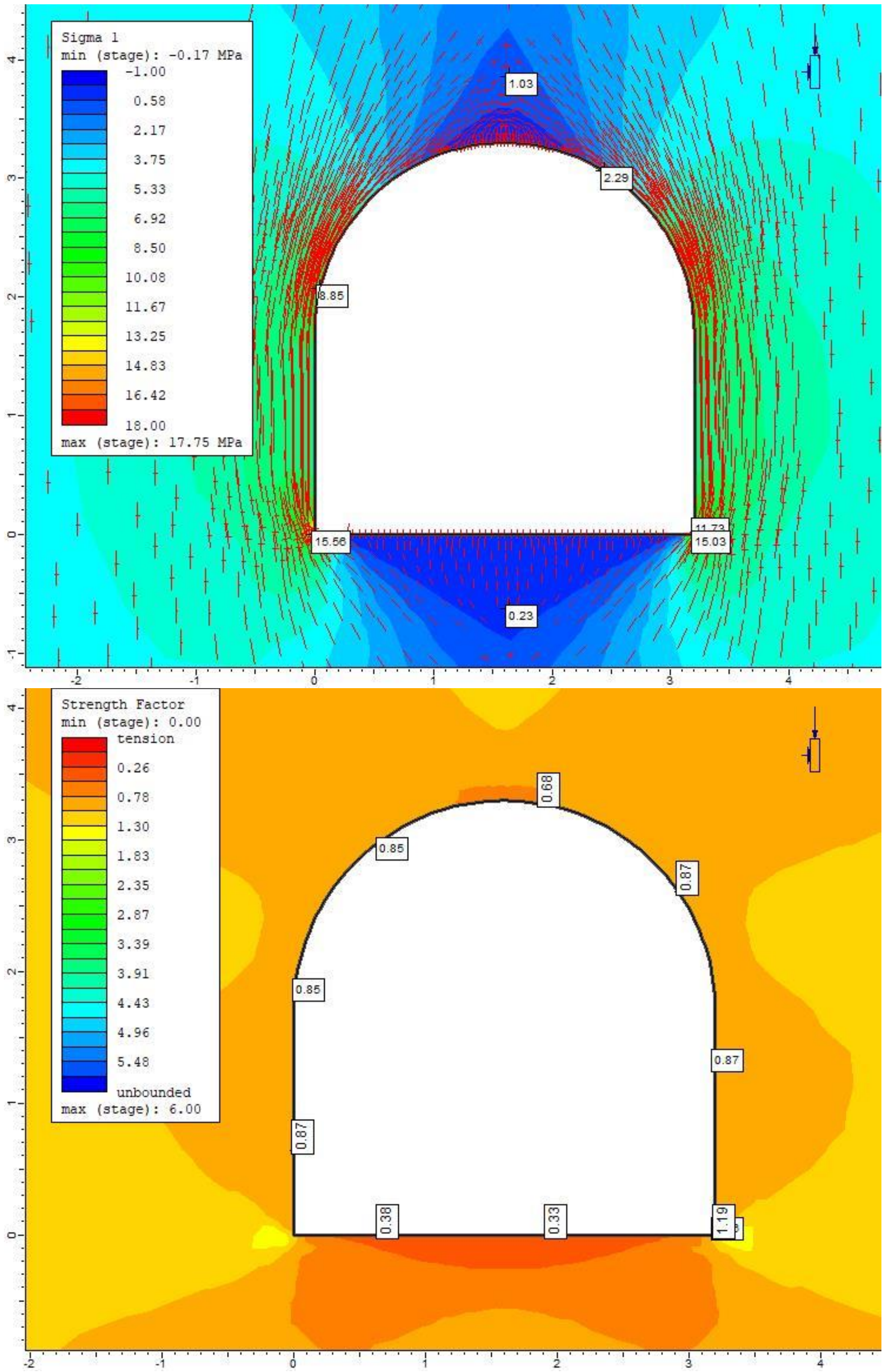


Figure 7.10 Maximum principal stress & Strength factor(Bottom) in elastic case around the opening of section A1.1

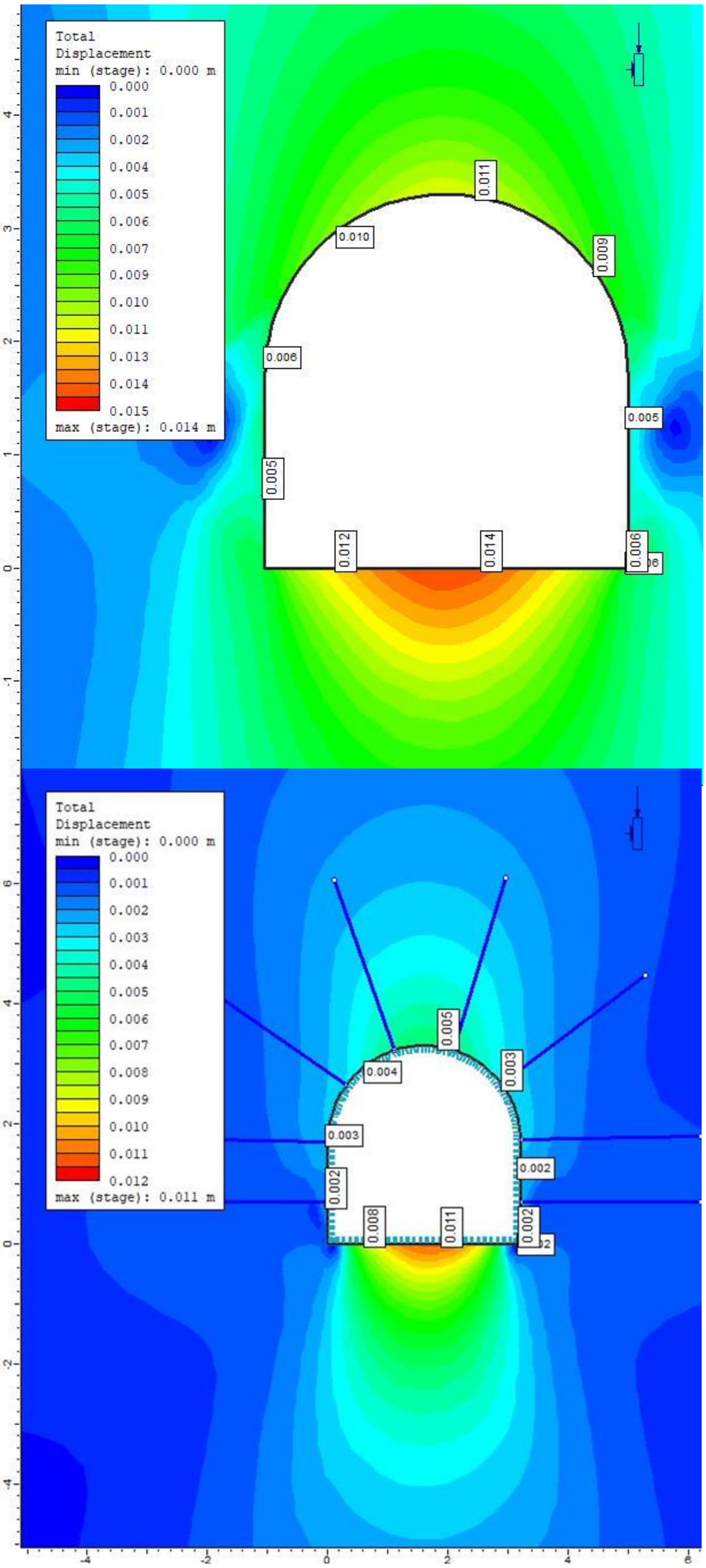


Figure 7.11 Displacement before(top) & after support application (Bottom) in the plastic state analysis for A1.1

b) Alignment A 1.2 mid chainage CH-4+241

For Section A1.2 interpretation results are presented in Figure 7.12 &Figure 7.13. Elastic strength factor is below 1 around the boundary of excavation and especially in invert, it is as low as 0.22. So, it is understood that support is needed after excavation. Maximum displacement is 15 mm in invert section and 12 mm in the roof during elastic state

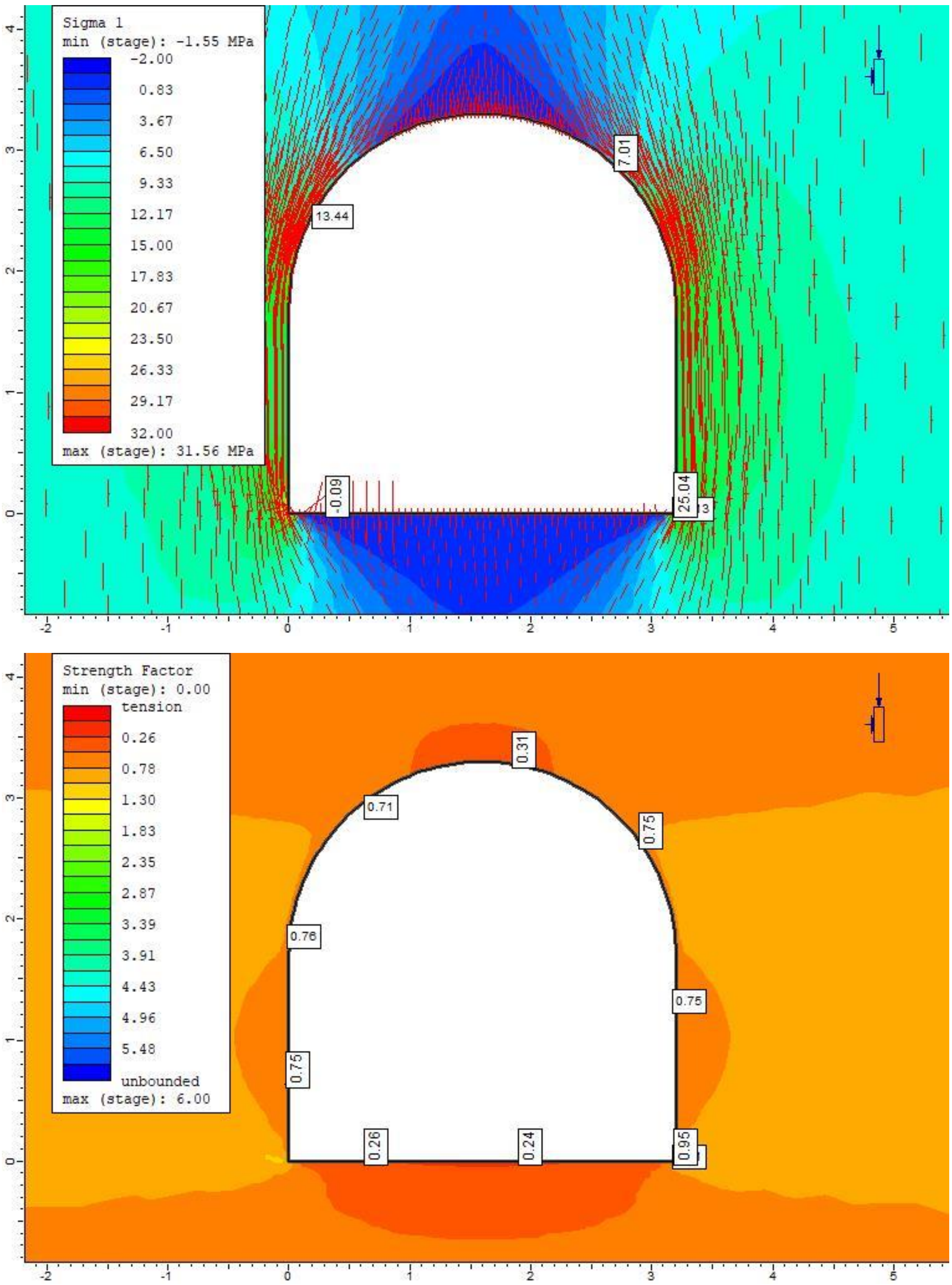


Figure 7.12 Maximum principal stress & Strength factor for the section A1.2 CH+4+241 at elastic state analysis

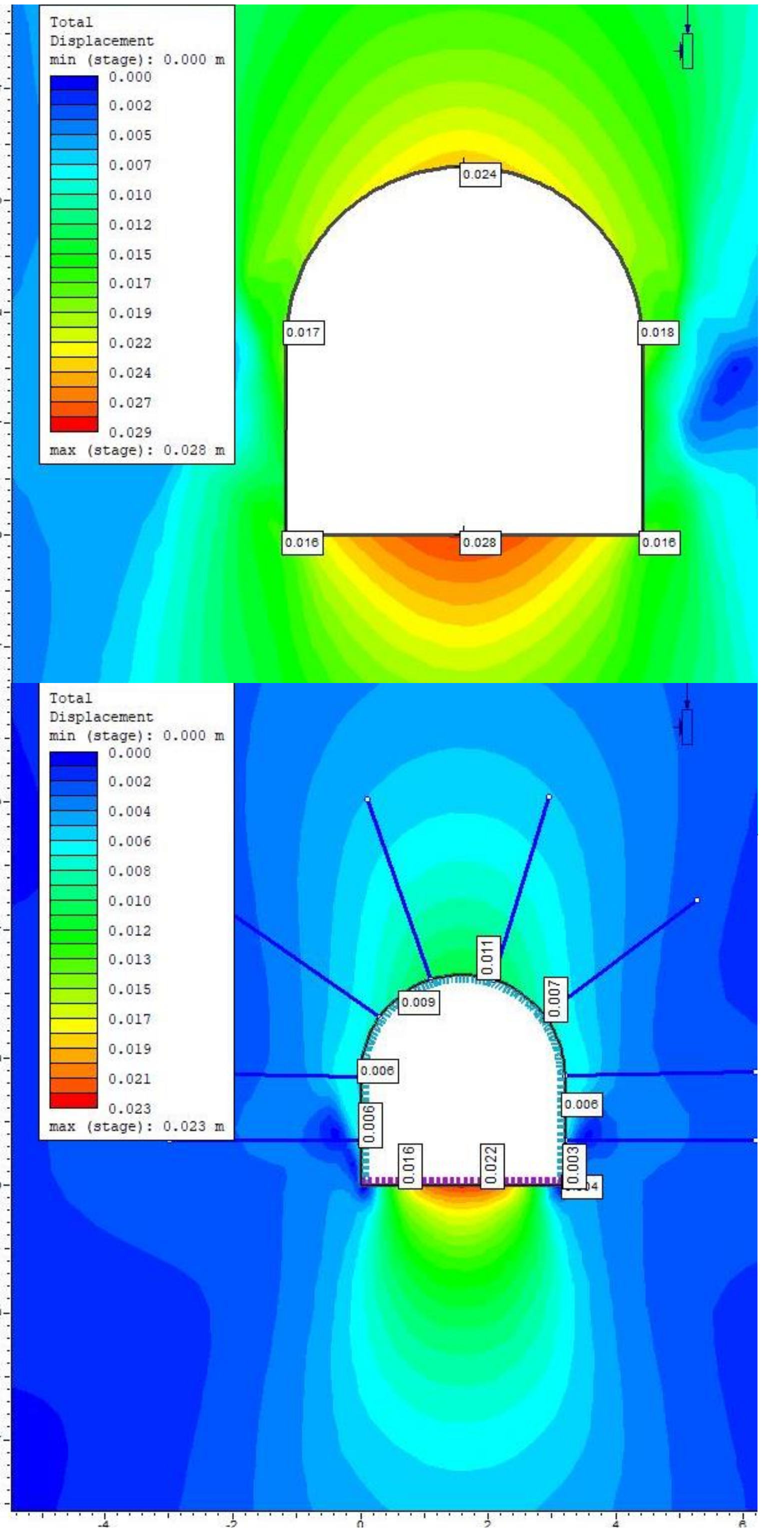


Figure 7.13 Displacement during plastic analysis & Total displacement after support installation for section A1.2

In respect to elastic state, maximum displacement has hiked form 15 mm to 29 mm in invert and 12 to 24 mm in the roof. Invert displacement has reduced to 23 mm after support installation as shown in Figure 7.13. Invert section will also be used for transportation during construction it is still suggested to use the reinforced lining of 30 cm in invert section for safety and good accessibility reasons. Table 7.10 presents the summary of support that has been applied in the section considered and where able to withstand stress situation present by allowing limited displacement.

Table 7.10 Summary of support installation for the A1.2 section

Sections	S-Bolt Roof/ wall	Sections	Shotcrete Roof/ wall
Type	FB	Type	SFR
Length (M)	3	Beam Formulation	Timoshenko
Dimeter (mm)	25	Thickness (m)	0.25
Bolt modulus (Mpa)	200000	Young's Modulus(Mpa)	30000
Spacing (M)	1.5x1.5	Poisson's Ratio	0.25
Peak tensile capacity (MN)	0.1	Peak /Residual UCS	35/5
Residual Tensile capacity (MN)	0.01	Peak /Residual tensile strength	5/0

c) Alignment A1.3 Chainage-808.92

Stresses obtained from the valley model for A1.3 alignment as presented in Figure 7.6 is applied and interpreted. In Figure 7.14 the stress orientation near invert and roof are highly stressed so higher displacement is obvious in these regions. Strength factor is also below 1 (Figure 7.14 bottom) which indicate need of support system to be safe from tunnel failure.

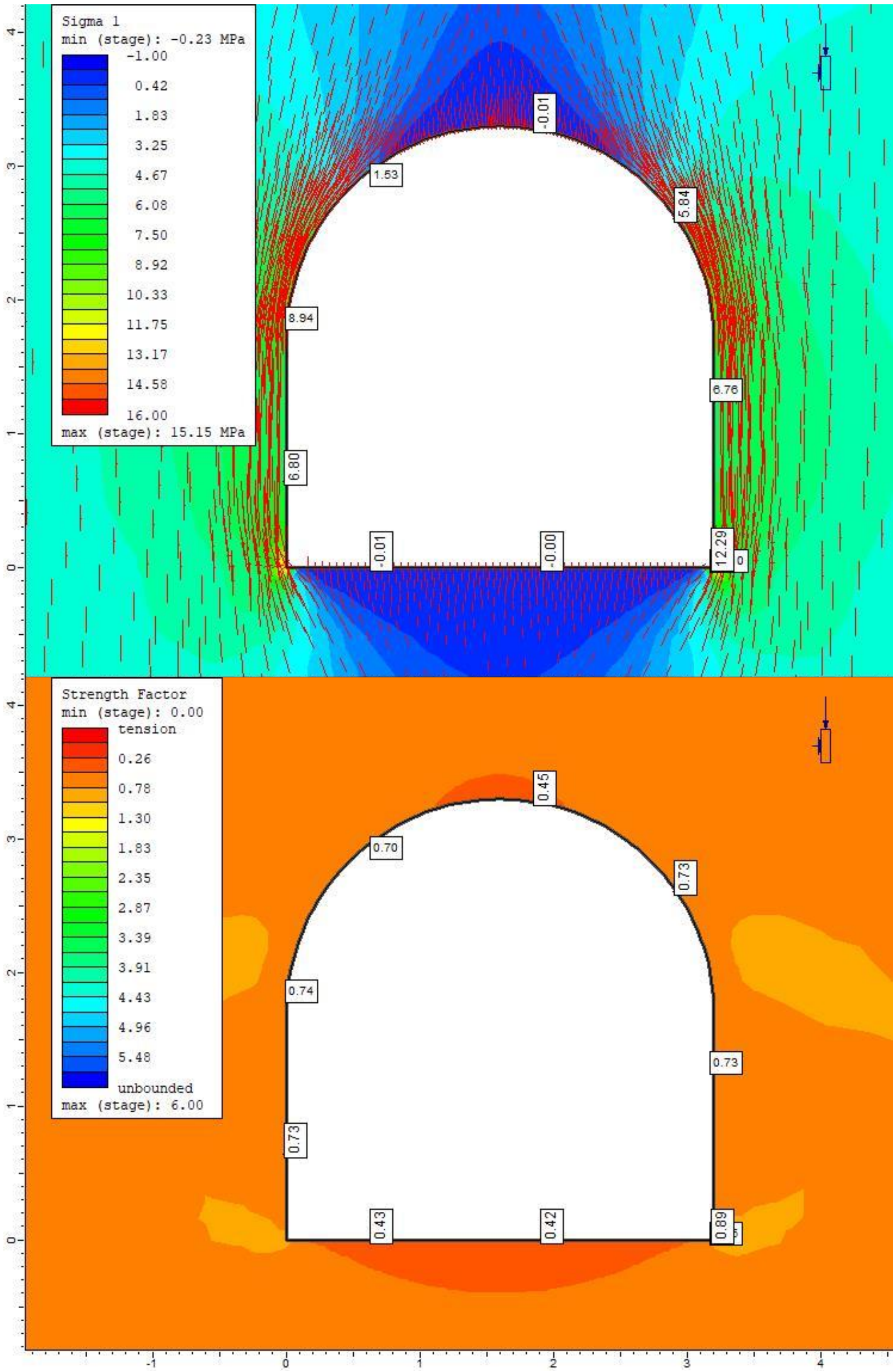


Figure 7.14 Maximum principal stress and total Strength factor (bottom) for the Section A1.3 in elastic state

From the stress situation observed in the section, minimum stress is below the hydrostatic head. So simple support system cannot withstand the instability like leakage and squeezing phenomenon. For that full concrete lining is applied in those rock class type as suggested from Table 6.2. In RS^2 core replacement approach is suggested for applying concrete lining. The first step is to determine the amount of tunnel wall deformation prior to support installation. Core replacement technique determines the modulus reduction sequence that yields the amount of tunnel wall deformation at the point of and prior to supporting installation (Rocscience, 2018). Figure 7.16 and figure 7.16 are the interpreted results from this analysis.

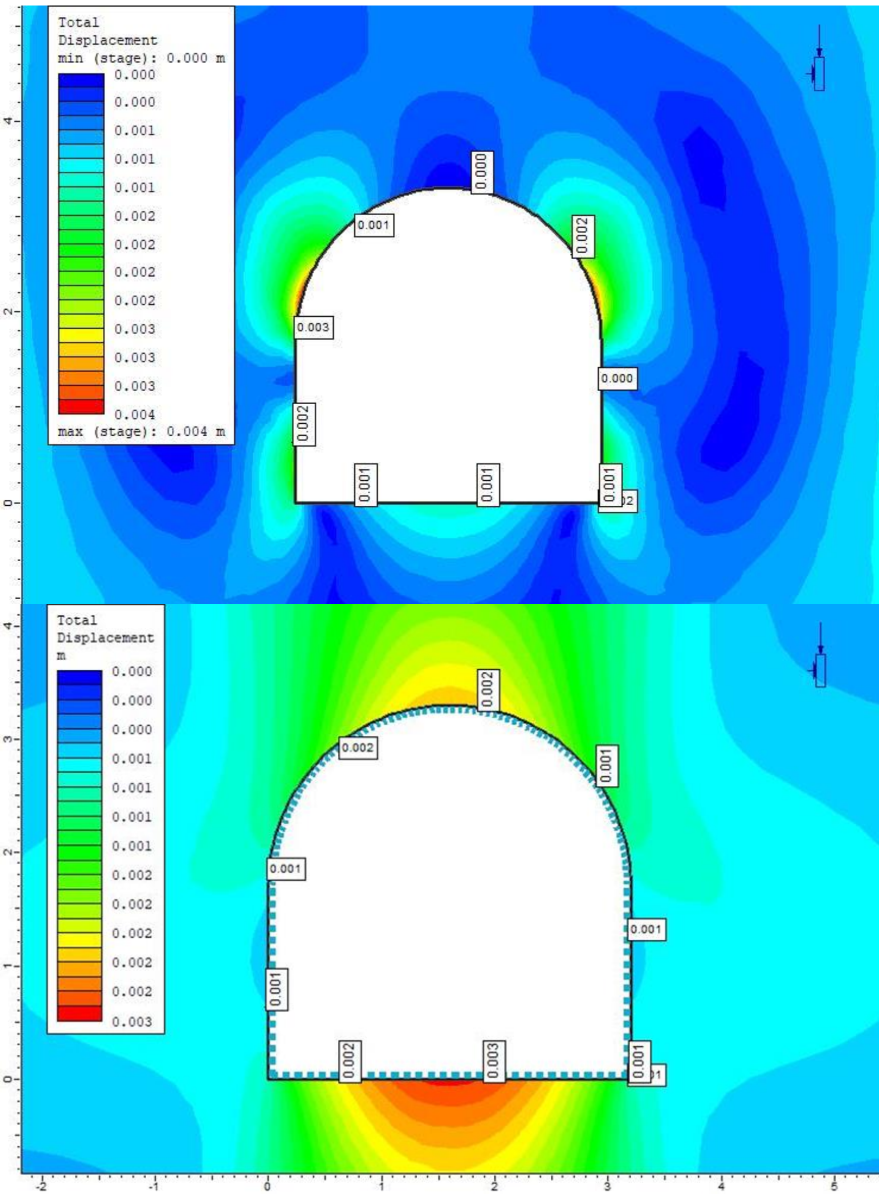


Figure 7.15 A1.3 CH-4+095 total displacement before support installation & after use of lining (bottom)

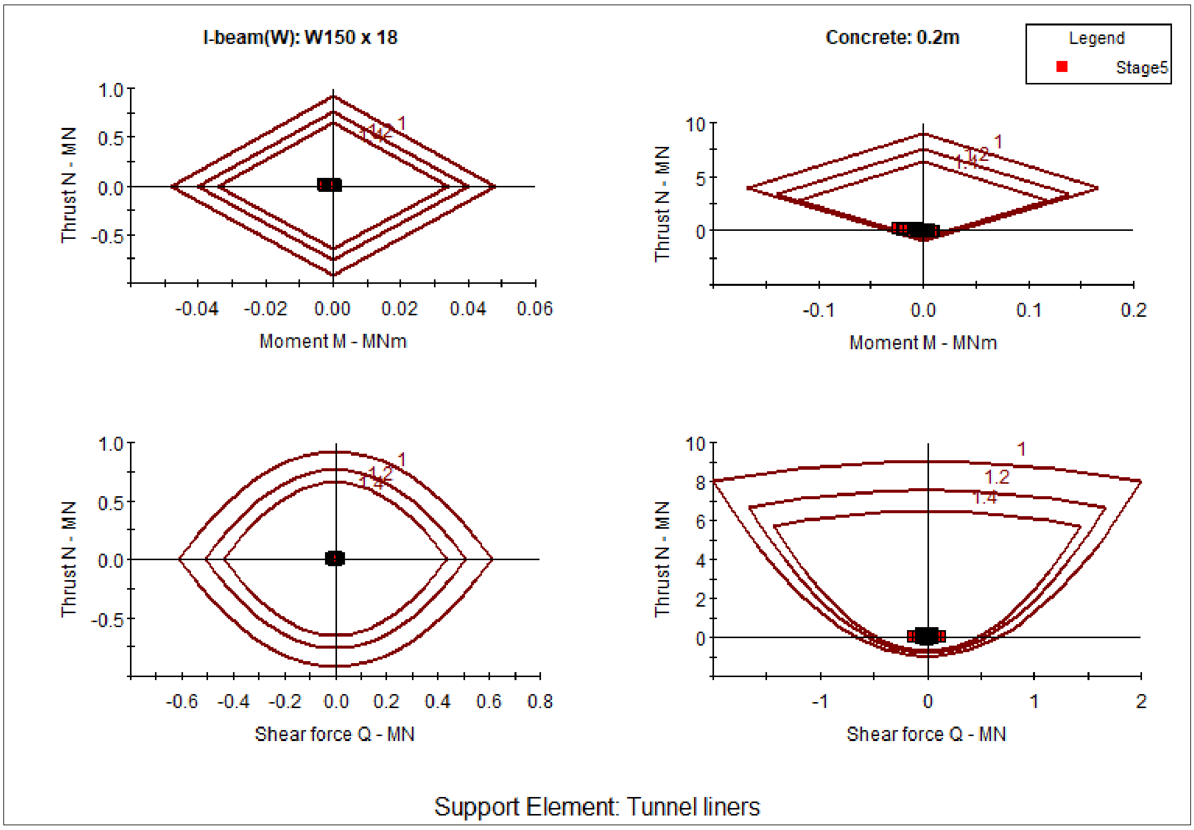


Figure 7.16 A1.3 CH-803 use of lining with the performance table

From the Stability point of view, in the model concrete lining with I beam of size 150 x 18when tested with 20 cm thick shotcrete, allowed maximum displacement of 0.003 m. The dark red lines in Figure 7.16 represent the capacity envelope for 3 factors of safety (1, 1.2, 1.4). Then it can be noticed that all the data points fall within the factor of safety=1.4 envelope, on all four plots. This means that the support system chosen has a factor of safety greater than 1.4.

7.7 Optimization

From stability analysis performed in section 7.6, full concrete lining as new support classification is suggested for sections which has insufficient minimum principal stress. Rest of the section are supposed to behave in accordance to the analysis performed unless any local or discontinuities are encountered. Taking all this in account we investigate the financial side of the implication. Through which it would be clear how much each section costs. Since there are numerous factors that cannot be enumerated the discrepancy in real scenarios is expected. keeping such aspect aside calculation is performed with respect to the price rate suggested in Table 7.11 as per market rate in Nepal. Figure 7.17 provides the variation in support system use and their total cost, which will govern the overall cost. Total cost will include excavation cost, steel penstock cost along with the support system cost presented which are presented in Figure 7.17 (a) & (b).

Table 7.11 Price for different excavation state and support component (Panthi, 2018).

Item	Unit	Rate (USD)
Excavation cost Head race, tailrace, Access & Adit	per m3	55
Excavation cost for Shaft	per m3	90
Steel fiber shotcrete	per m3	250
Steel lining	per kg	2.2
Concrete backfilling	per m3	175
Steel Rib	per set	350
Rock bolt	per m	25

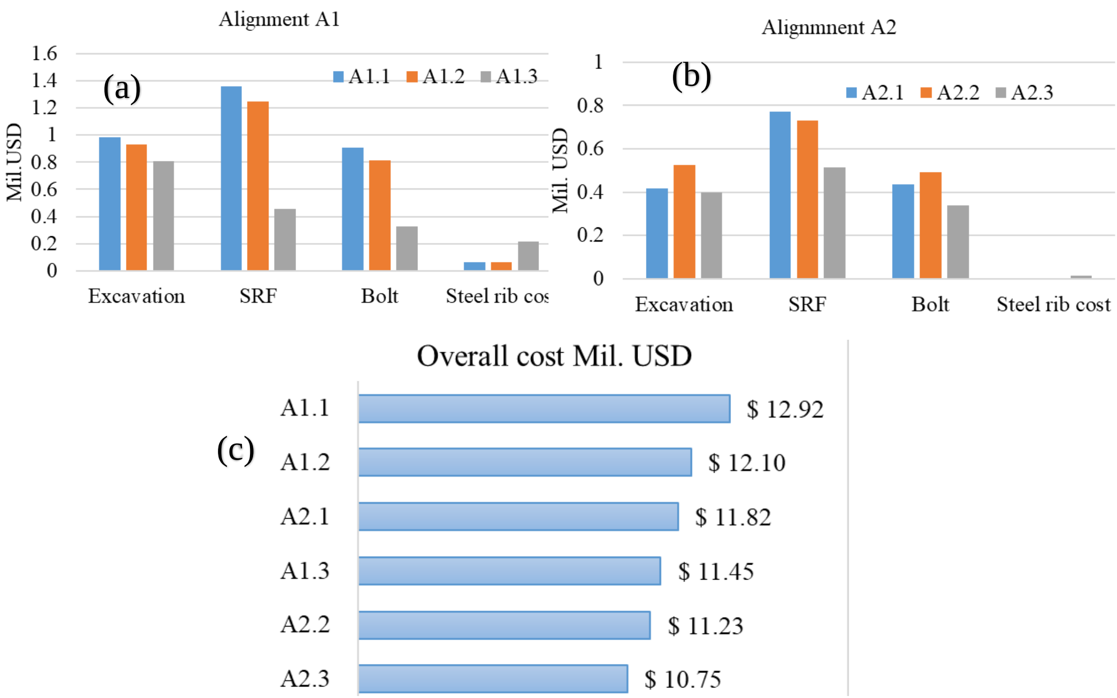


Figure 7.17 Summary of cost calculation for support system for (a) Alignment A1, (b) Alignment A2, and (c) Total overall cost with ranking for all alternatives

Calculation for the common section in all alignment is same. Variation is observed after common point onwards, which is clearly projected in the Figure 7.1. Total cost for each alignment including the common section with ranking is presented in Figure 7.17(c). Alignment A1.1 which is in-line with old proposed alignment is longest (Table 7.12) and significantly expensive one too. Alignment A2.3 is the cheapest one since it is shortest of all the alignment (Table 7.12) with the favorable geological condition. Alignment 2 has the longest tailrace tunnel which makes the cost of A2.1 alignment expensive than shorter alignment from Alignment A1.

But other two alignment A2.2 and A2.3 despite the longest tailrace tunnel are less expensive than remaining alternatives. Table 7.12 also presents the cost per meter for each option with total length and the total cost for overall length.

Table 7.12 Total cost calculation for each alignment

Section	Total length (m)	Overall cost (mil. USD)	Cost /m (USD)
A1.1	7715.24	$ 12.92	1674.16
A1.2	7346.09	$ 12.10	1647.76
A1.3	7144.42	$ 11.45	1603.10
A2.1	6666.28	$ 11.82	1773.12
A2.2	6447.68	$ 11.23	1741.72
A2.3	6237.52	$ 10.75	1723.07

7.8 Conclusion

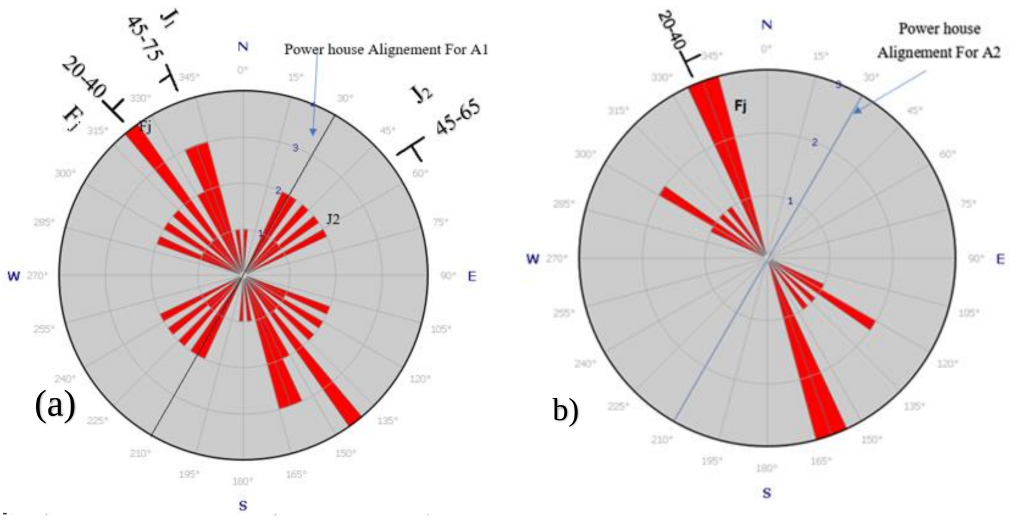
From the analytical and numerical model, Alignment A1 is filled with a lot of complexities and longest of all which also appears the costlier option as well. Concrete lining is applied in those section where minor principle stress was below the hydrostatic head. These locations where identified in rock class type ranging from III to V as classified in Table 6.2. Option A1.1 has approximately 203 m, A1.2 has 195 m and A1.3 has 690 m of concrete lining suggested. Whereas in alignment A2 only alignment A2.3 is suggested with approximately 890 m of concrete lining. These parts indicated must be looked more meticulously for further location of critical zones Alignment 2 has long tailrace tunnel but with good geological placement, as has been seen from analytical, semi-analytical, and numerical calculations. Despite the good results from this chapter, there is equal chance of differing in numbers as the project advances in field due to the presence of local discontinuities and other challenging parameters. As geological investigation is carried out visualization will become clearer. So, this calculation must be taken as suggestion for betterment of choices rather than as final result.

8 The powerhouse cavern

Underground caverns are used as a multipurpose option when it is associated with different scale and scope of projects. In case of Hydropower, Caverns are used for housing turbines, Electrical Generators, and Transformers. Cost and importance of this components in hydropower project are immense and there is a constant need for public access to these components from construction, an operation to maintenance period. Which then requires potential risk to be as minimum as possible in combining with cost-effective and practical engineering solutions (Hoek, 2007).

8.1 Placement of PH cavern

Powerhouse is designed as underground in both the alternatives. It is situated at an elevation of 1895.15 masl inside the ground surface. Alignment 1 is located in between two major joint sets striking parallel to the foliation plane. Whereas cavern 2 has no identified such major joints surrounding it.



Fj: Foliation joint, J1: Joint set 1, J2:Joint set 2

Figure 8.1 Rosette with foliation and major joint sets in Cavern location for (a)A1 and (b)A2

In the rosette plot as shown in Figure 8.1 discontinuity set striking mid NW-SE is prominent and is the foliation plane as well. Cavern axis of alignment A1 &A2 makes an angle of 68^0 with foliation plane. Beside foliation plane which is dipping NE, there are two cross joint sets dipping SW and SE Figure 8.1(a) identified for cavern A1. Since no geological test is carried out in cavern location for alignment A2 so it is set in accordance with foliation plane only. Also, it is positioned under massive rock formation away from lineation zone there is still chances of encountering some joint set once excavation is carried out.

8.2 Stability Assessment

Methods for analyzing stability assessment for the cavern is as similar to the process involved during tunnel analysis as presented in Section 7.6. It also undergoes semi-analytical assessment for tunnel strain calculation, numerical models to address the stress distribution and analyze support stability

8.2.1 Semi-analytical assessment

Rock mass properties are calculated using Hoek and Brown criterion as discussed in section 3.2. It is to be noted again that project is situated in the Himalayan region so, it was useful to use equation proposed by (Panthi, 2006) for the unconfined compressive strength of rock mass (σ_{cm}) as expressed in Table 3.1. tunnel strain calculation for both the cavern is shown in

Table 8.1. As in tunnel assessment, only two class type A&B are identified, significance to that is explained in Table 4.2.

Table 8.1 Summary of strain development in Cavern opening

Cavern	ucgs	σ_{cm} (Mpa)	σ_v (Mpa)	$\sigma_{Cm}/\sigma v$	Tunnel strain(%)	Remarks
A1	39	4.06	8.08	0.50	0.79	A
A2	39	4.06	12.20	0.33	1.81	B

8.3 Numerical Analysis

8.3.1 Model inputs

Cavern alignment is same for both alternatives as can be verified from Figure 8.1. Both the alignments are positioned in rock class of II (fair to good) based on the classification devised for the project as in Table 6.2. The rock mass properties included during analysis are presented in Table 7.4. Tectonic stress for a cavern in both alignments has been taken as 3.5 MPa (Panthi, 2018). Support systems for the cavern opening were decided from the discussion with the supervisor which is summarized in Table 8.2.

Table 8.2 Support assigned for Powerhouse cavern

Cavern	S-Bolting		SRF
	Walls (m)	Roof(m)	cm
A1	1.5x1.5x8	1.5x1.5x8	25
A2	1.5x1.5x8	1.5x1.5x8	25

Excavation in cavern is decided to be perform in 7 steps. First step is to begin with the crown area of 5m height then next steps will be in consecutive 6 m benching. Critical valley side that can have influence on the cavern orientation as mentioned in section 7.5 is generated to investigate the stress situation. The stress distribution thus obtained is used to investigate excavation in stages and deformation observed with an increase in excavating steps is noted. The final stability assessment will largely depend on how cavern reacts in presence of in-situ stress. The final support system has been decided from the stability analysis performed through numerical model -RS^2-if needed Q-table and Table 8.2 were reviewed.

8.3.2 Model Output

Based on the input parameters valley models are generated for each cavern location which are presented in respective figures below. The purpose of this step would be to investigate stress distribution and the orientation of major and minor principal stresses. Which is very useful when investigation on stability of the opening is performed. After that we can locate the critical point/section in the tunnel/cavern opening and design support system to address that issue. This process hugely depends on the experience of the designer and the familiarity with such geological condition otherwise it can be a hit and trial in numerical model based on the material available in the market and availability of budget.

a) Powerhouse Cavern for Alignment 1

Figure 8.2 represents the valley model generated with corresponding principal stresses. In top we have major principal stress as 9.02 Mpa and bottom minor principal stress as 4.02 Mpa for powerhouse location in alignment A1. Summary of principal stresses and orientation is presented in Table 8.3.

Table 8.3 Summary stress situation for powerhouse location in Alignment A1

Principal stress	A1.3	Remarks
σ_1	9.08	Mpa
σ_3	4.02	Mpa
σ_Z	5.42	Mpa
Orientation	277	CCW

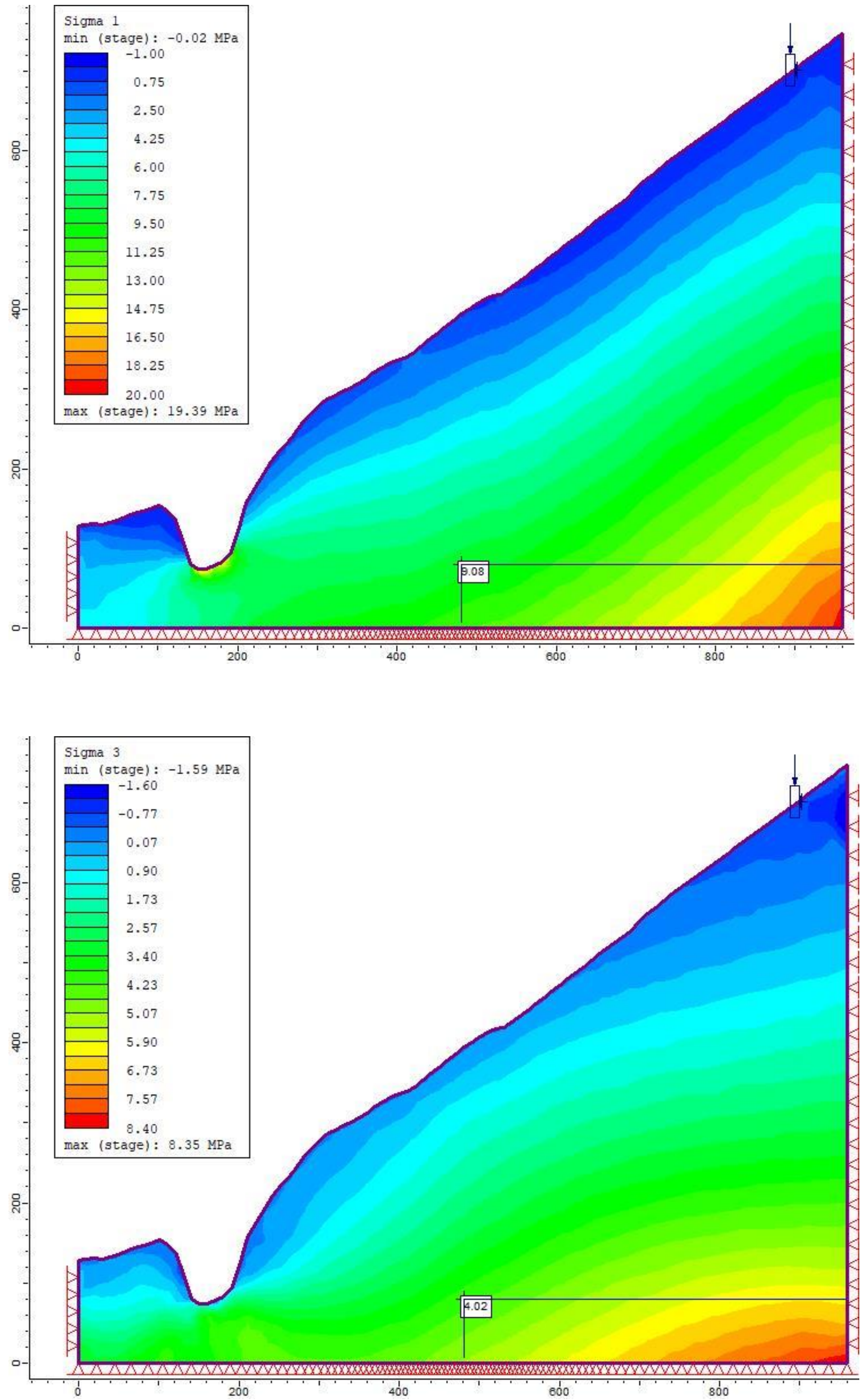


Figure 8.2 Principal stresses for Cavern in Alignment 1

Then elastic analysis was carried with each excavating stage. According to the analysis when the strain factor is less than 1 as in Figure 8.3 it is considered vulnerable to stability. Total deformation for first stage of excavation due to the stress orientation can be seen in the Figure 8.4 (a), where maximum displacement is reported as 0.054 m in roof. So, support system as presented in table 8.2 is applied. After the support application displacement in roof has reduced to 0.035 m and so is reduced in the other contour until first excavation see Figure 8.4 (b)

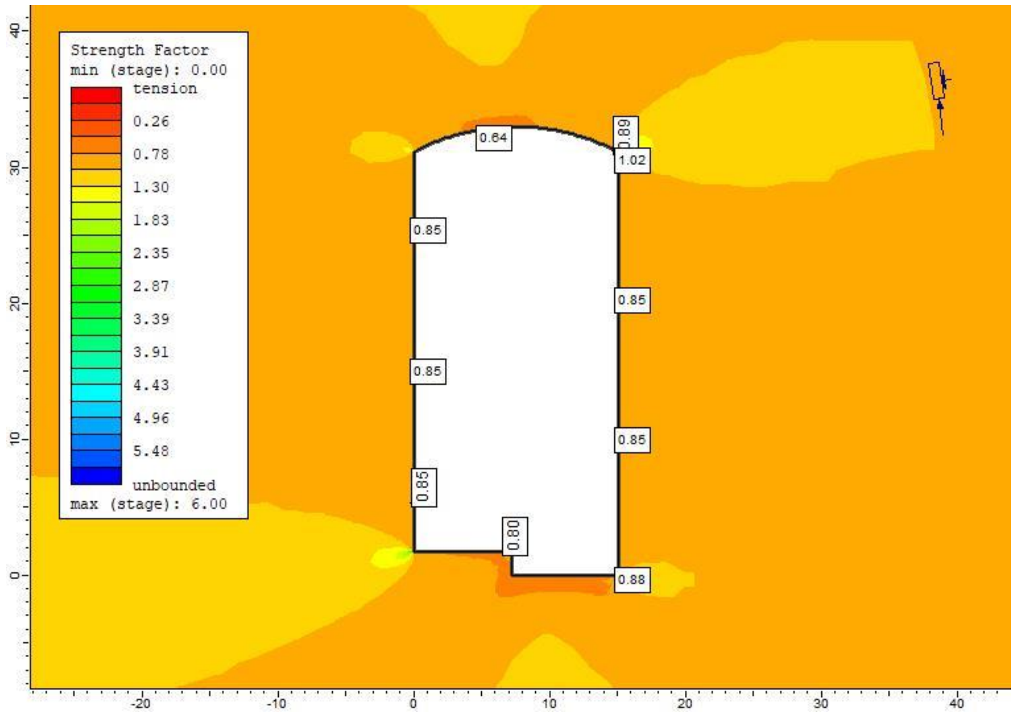


Figure 8.3 Total Strength factor from elastic state

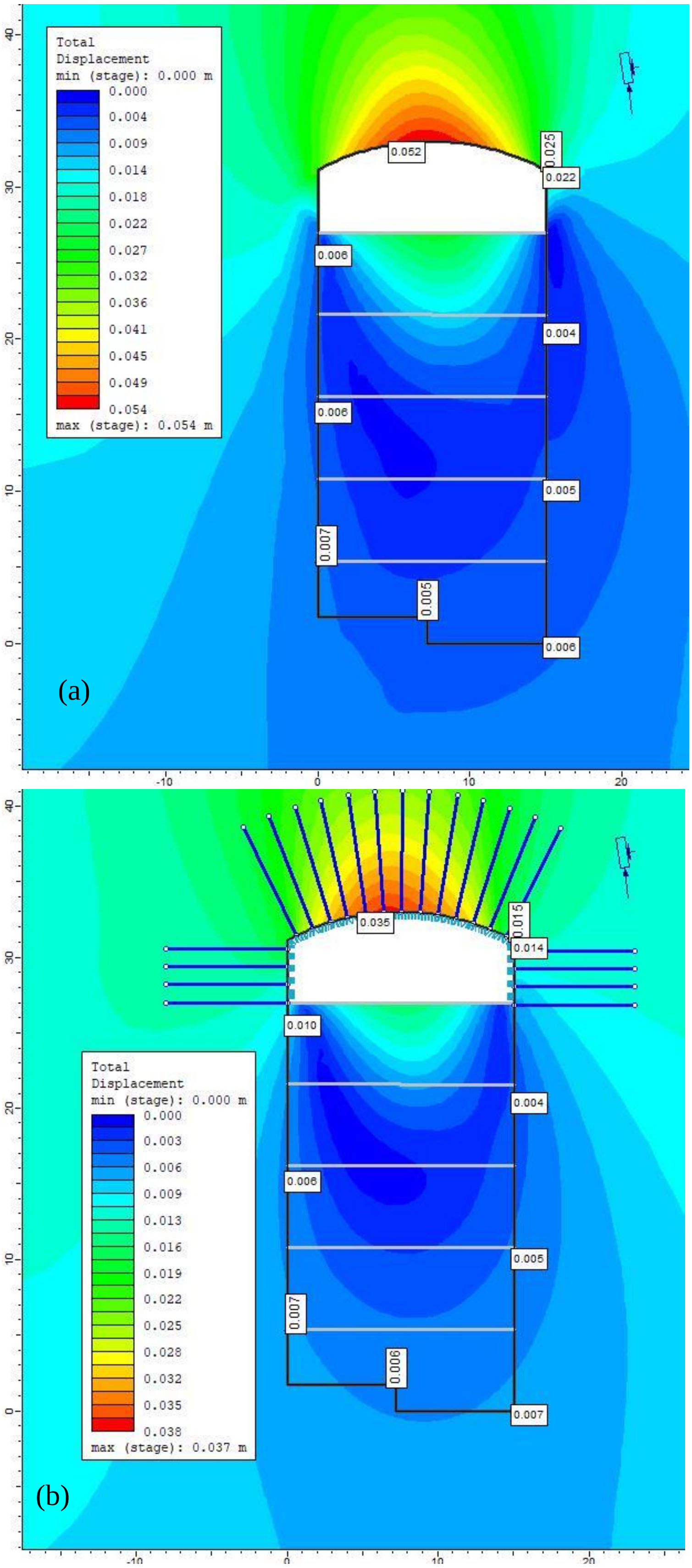


Figure 8.4 First excavation and roof displacement (a) before support installation and (b) after support installation

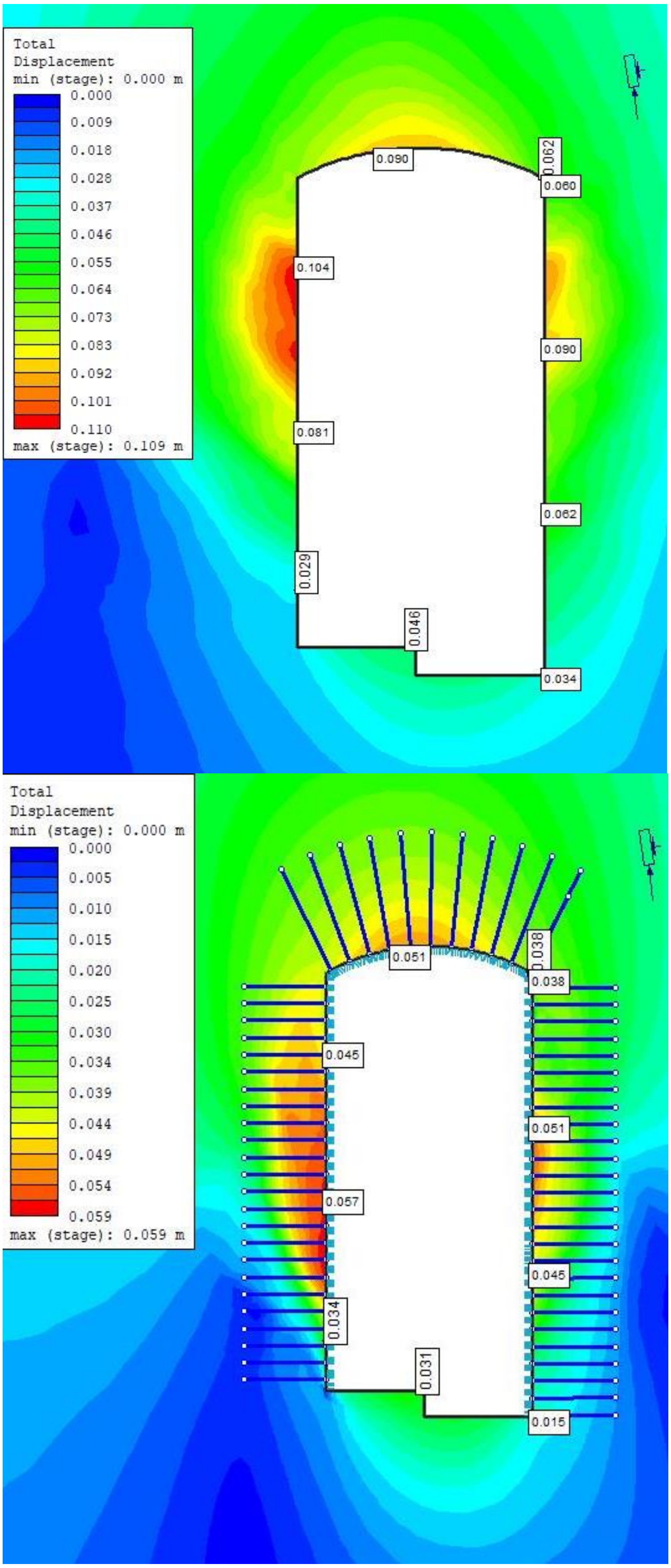


Figure 8.5 Total Displacement (a)before the support installation and (b) after support installation for a cavern in Alignment A1

Figure 8.4 and figure 8.5 also provides the visual displacement pattern happening once the support is installed. It is interesting that roof section appears less displaced then wall in full excavation mode. Maximum displacement is reported as 0.109 m in left wall. And displacement in roof increases further to 0.09 m and displacement is same in right wall as well. Once the support system is installed and in steps of excavation then maximum displacement is reduced by half as shown in Figure 8.5 (b). left Side wall is still the maximum displaced one and value as reduced in right wall and roof as well. Figure 8.4 is tested for first opening and Figure 8.5 is tested for full excavation in different models to get effects in respective stage. The values will differ when the execution takes place because every benching is observed, and support are applied accordingly so there is some relax time for the rock mass in between. But the important thing to note is that pattern of the stressed zone is same. Table 8.4 presents the summary of support types installed.

Table 8.4 Summary of support applied in the Alignment A 1 original shape cavern

	S-Bolt		Shotcrete
General	Roof/ wall	General	Roof/ wall
Type	FB	Type	SRF
Length (M)	8	Beam Formulation	Timoshenko
Dimeter (mm)	25	Thickness	0.25
Bolt modulus (Mpa)	200000	Young's Modulus	30000
Spacing (M)	1.2x1.5	Poisson's Ratio	0.25
Peak tensile capacity (MN)	0.1	Peak /Residual UCS	35/5
Residual Tensile capacity (MN)	0.01	Peak /Residual tensile strength	5/0

b) Powerhouse Cavern location for Alignment 2

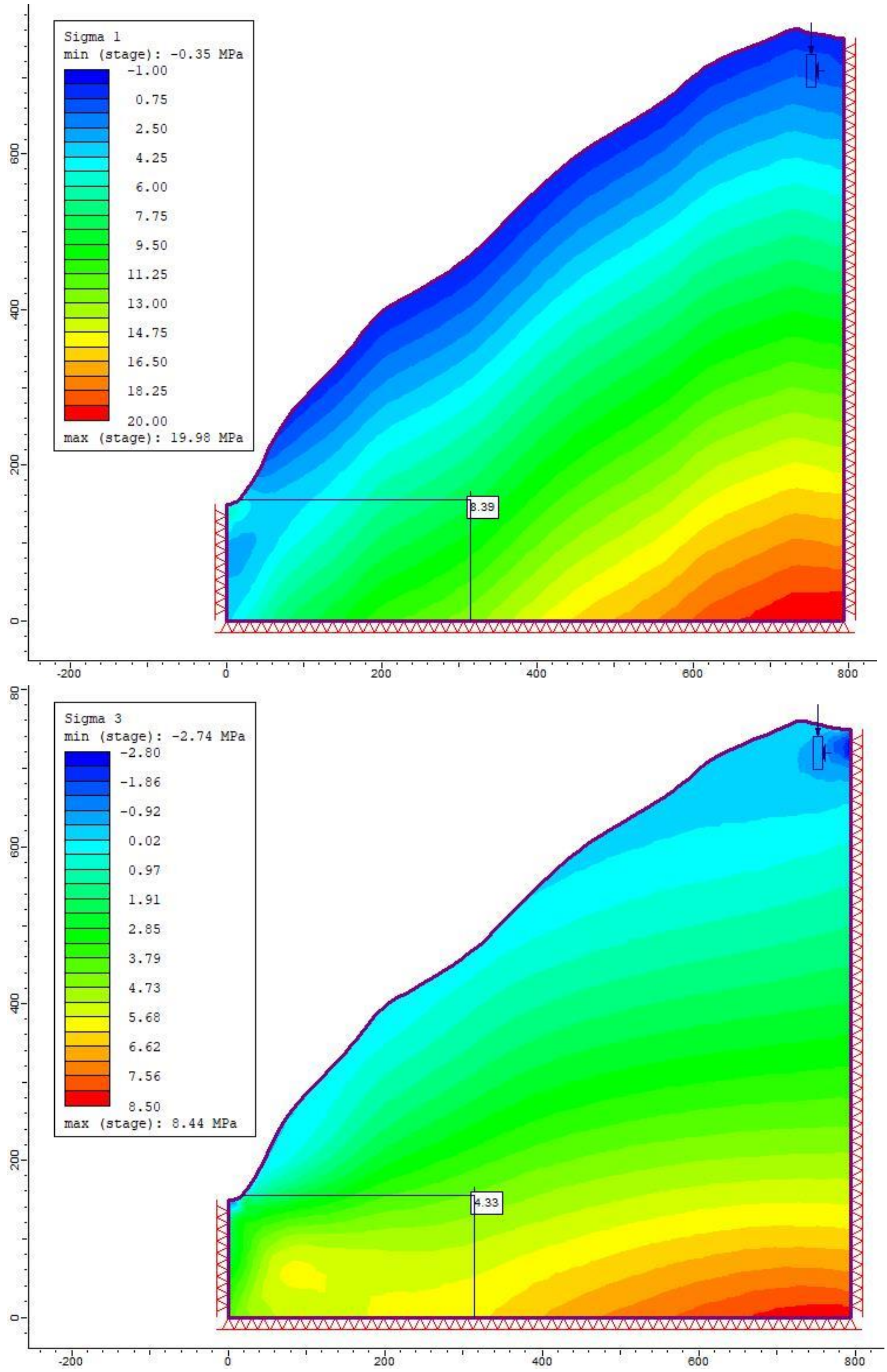


Figure 8.6 Principal stresses in Powerhouse location for alignment A2

Figure 8.6 is the representative valley section model in cavern location for alignment A2. In which top figure represents major principal stress with value 8.87 Mpa and bottom one represents minor principal stress of value 4.02 Mpa. Summary of principal stresses and orientation is presented in Table 8.5 below.

Table 8.5 Summary stress situation for powerhouse location in Alignment A1

Principal stress	A1.3	Remarks
σ_1	8.87	Mpa
σ_3	4.02	Mpa
σ_Z	5.42	Mpa
Orientation	276	CCW

Then elastic analysis was carried for the cavern opening with excavating stage. From the analysis when the strength factor is less than 1 as seen in Figure 8.7 it is considered unstable if left without support. Then plastic analysis need to be performed for the stability analysis. Here it is performed for roof stability and full excavation stability. Figure 8.8 (a) presents the maximum displacement as 0.01 m in the roof without the support installation. Even though the displacement is low we need to apply minimum support system for safety reason. Figure 8.8 (b) is the case when support is installed.

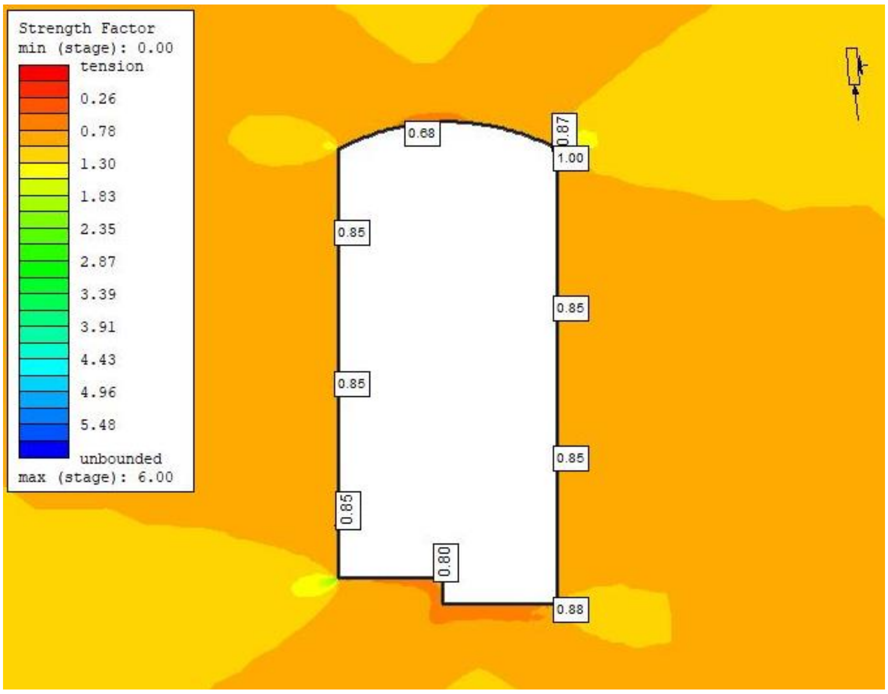


Figure 8.7 Strength factor in the elastic state for cavern in alignment A2

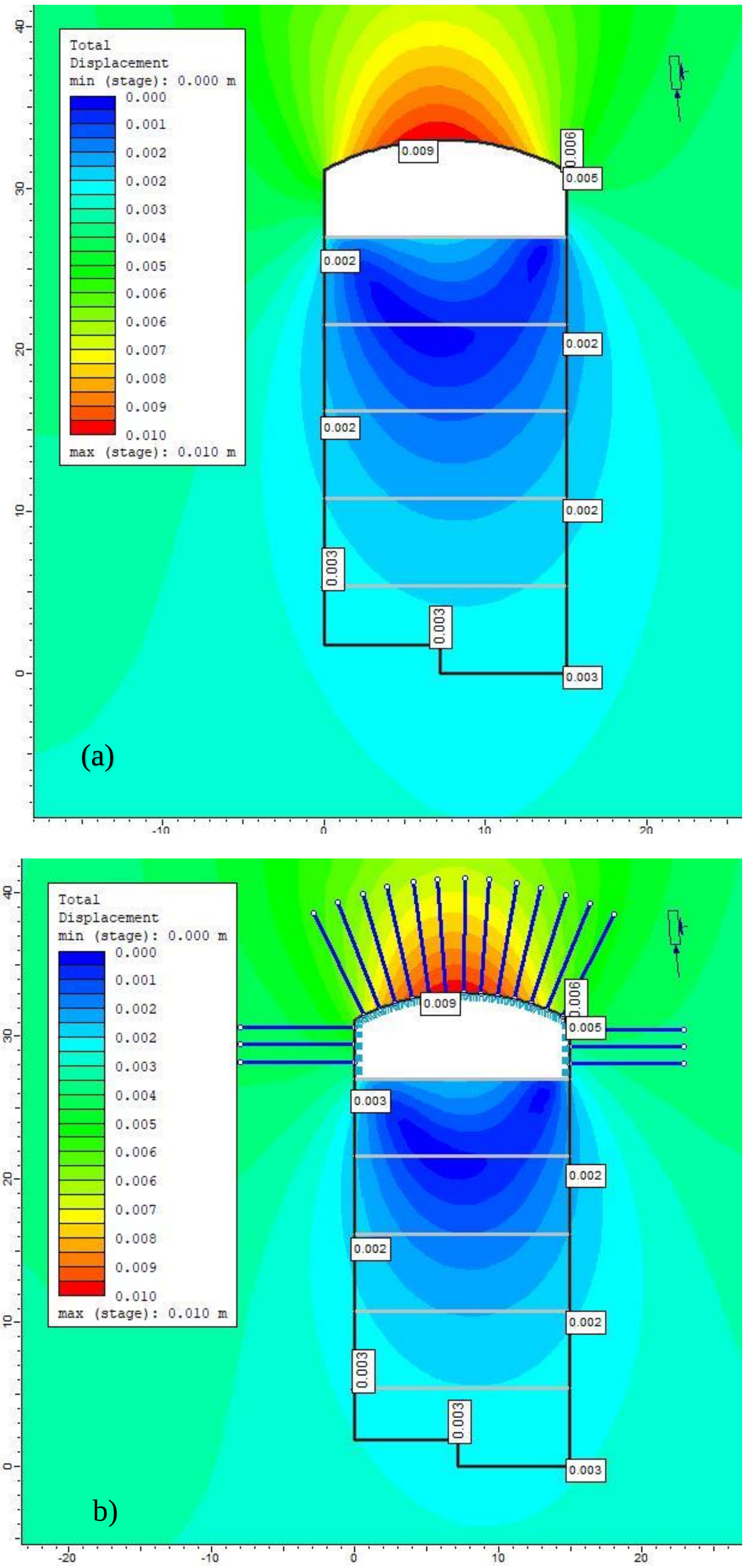


Figure 8.8 Total displacement (a) before roof support installation and (b) after roof support installation

Figure 8.8 and Figure 8.9 provides the visual displacement pattern happening before and once the support is installed. In both caverns roof section is stressed in first excavation and unlike cavern for A1 right wall is only stressed. Maximum displacement in full excavation mode without any support installation is reported as 0.025 m which is located in invert and is reduced to 0.014 m once the support is installed but maximum displacement is then observed in the right wall with value increase from 0.07 m to 0.025 m. (figure 8.9) It might be due to the stress orientation/inclination towards the right wall. In compared to the dimension of the cavern, displacement does not account much so the support system installed fulfils the job. Table 8.6 presents the summary of support system installed.

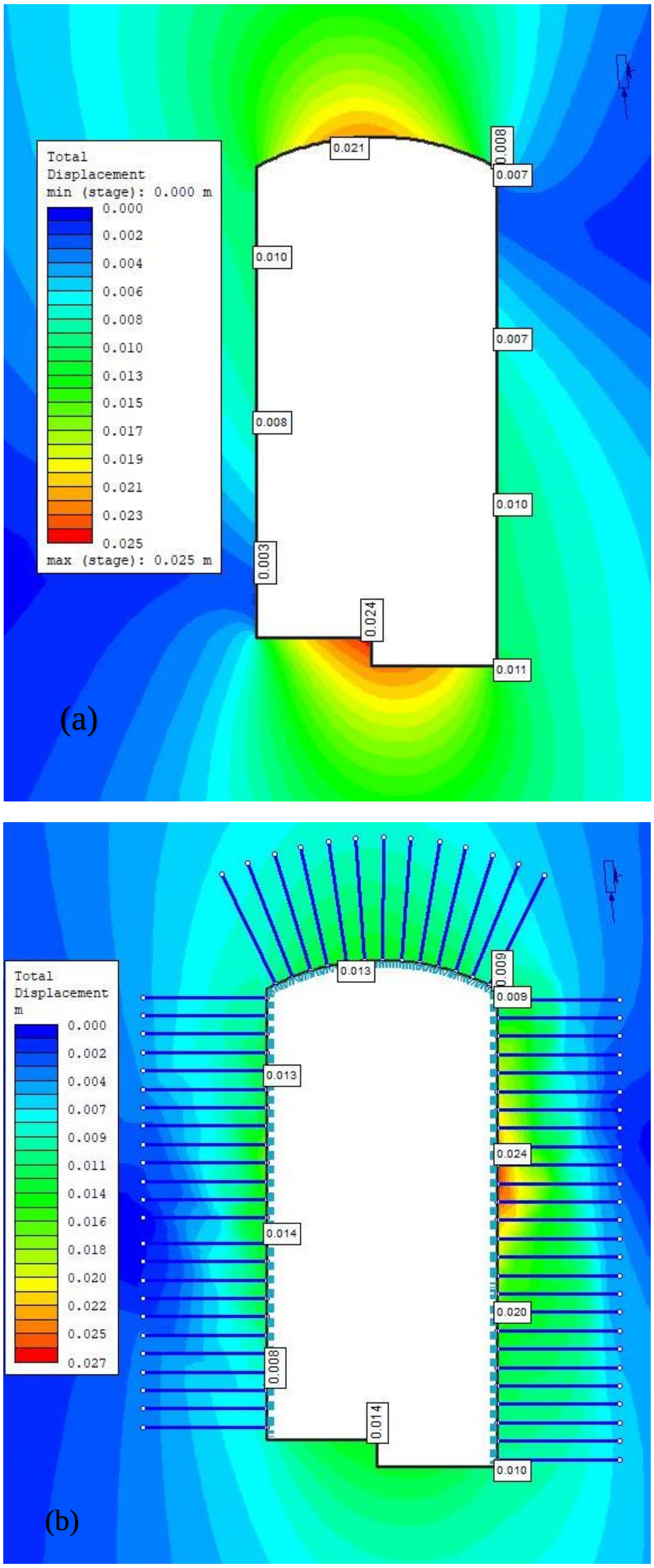


Figure 8.9 Total displacement (a) before the support installation and (b) after full stage installation of support for Cavern in Alignment A2

Table 8.6 Summary of support installation in the Alignment A 2 cavern

Sections	S-Bolt	Sections	Shotcrete
	Roof/ wall		Roof/ wall
Type	FB	Type	SRF
Length (m)	8	Beam Formulation	Timoshenko
Dimeter (mm)	25	Thickness (m9	0.25
Bolt modulus (MPa)	200000	Young's Modulus (MPA)	30000
Spacing (M)	1.2 X 1.5	Poisson's Ratio	0.25
Peak tensile capacity (MN)	0.1	Peak /Residual UCS (MPa)	35/5
Residual Tensile capacity (MN)	0.01	Peak /Residual tensile strength	5/0

8.4 comparison and Discussion

After careful examination of the powerhouse cavern for both the alternative, a support system was designed. Systematic bolting with in-plane spacing of 1.2 m and shot-crete (SFR) lining with layer of 25 cm has provided acceptable range of displacement. Since the joint pattern is known for the cavern location in alignment A1 the analysis can be reflective enough for that. In case of Alignment 2 orientation is designed in respect to foliation plane, but chances of encountering discontinuities are still there. To assure the stability of the cavern opening for the future, it is advised to test for the stress situation when the cavern face is first encountered. A constant eye on total displacement can be performed with help of extensometer so that support system can be fully tested and improved if necessary

9 Long-Term Stability of Headrace tunnel

Monitoring of the tunnel performance during construction and if possible in the long run, will provide information which will be helpful in maintaining tunnel safety and study purpose in future. It also allows us to evaluate the stability scenario established and the verification of support system installed. For that different measurements had to be collected with the progress of the project. It is possible when the construction is taking place. Measurement can be obtained from a visual inspection or installing certain devices in the sections of the openings. Some of the gathered measurement that could be useful are in-situ stress state, rock/block falls, deformation inside tunnels and caverns, crack opening in the applied support, tension in anchor/bolts and inflow/leakage (Panthi, 2017). In the competitive market constant update and verification of the old design is necessary for sustainable future. During the operating days the total head loss, and total amount of discharge/leakage can be checked frequently. The increase in the head loss and leakage in unlined tunnel will indicate some problems regarding stability of the water way system if all the mechanical equipment has performed in accordance. Then the section that were indicated as critical during investigation stage can be examined for the probable causes. This method is an economic way to identify the problem without shutting down the whole system before large stability is encountered (Panthi, 2006).

From the relevant case study so far reviewed from Himalayan, deformation could be the issue. Only solution to counter those issue is to increase the database by field instrumentation (Panthi and Nilsen, 2007). For that surface measurement and monitoring of deformation on tunnel contour or monitoring deformation inside the rock mass can be performed. Convergence measurement can be performed through installing multiple measuring points on tunnel contour. Once the surface deformation exceeds 0.5% of tunnel width/height, in-depth study of rock mass impact will be required because visco-plastic zone (Figure 3.3) aligned outside the tunnel contour will largely govern the long-term stability of tunnel (Panthi, 2017). During construction pressure monitoring on tunnel support in weak rock types as in our project will provide an economical option for support revision in future.

10 Conclusion and Recommendations

This thesis was proposed to do stability analysis of unlined headrace tunnel, powerhouse cavern and optimizing the layout alternatives. The main assumption or purpose that has been established from history to present is to use the rock mass itself as a self-supporting element. In early days half of hydrostatic head was considered sufficient for rock overburden. Eventually, the concept was expanded from its conservative approach to Norwegian Rule of thumb which explores other scenarios. The concept of positioning the unlined tunnels where the minimum principal stress is more than the hydrostatic head is the primary issue nowadays. All concepts described in this thesis were developed from the issues they faced during constructions days. Even though, the issues relating to ideal tunnel design have not all been discovered but the quest of identifying and renewing the concept/idea is still on. So, Geological investigation plays an important role in knowing the scenario as tunneling proceeds. For that the quality input of data are important for analysis and the reliability of results.

Stability challenges are most common when Tunneling is taking place in Himalayan region. The main reason is the presence of weak, highly schistose, highly deformed, highly weathered, and fractured rock mass condition (Panthi, 2006). Himchuli-Dordi has rock types of banded Gneisses and schist partings, which is common in lesser Himalayan. It is located near the Main Central Thrust (MCT). Generally, in this zone, Rock mass strength is less than the induced tangential stress due to rock overstress, So, problems like squeezing, leakage are common and anticipated. Rock stress problem due to tectonic stress and valley effect has a major influence on our project so, locked in stress calculation and valley model from RS^2 was very useful to address that.

10.1 Unlined head race tunnel

Two main alternatives were taken into consideration, and individual alternatives were explored into different alignments. In total six alternatives were analyzed using analytical, semi-analytical, and numerical methods. The shape of tunnel considered during analysis was inverted D, with height 3.7 m. Following points can be concluded from the analysis performed:

- Foliation plane in the region is striking between $(120\text{-}165)^0$, dipping $(20\text{-}40)^0$ NE, tunnel alignment in average is oriented $(25\text{-}68)^0$ NS

- Analytical, semi-analytical and numerical calculation shows us that A1 alternative is dealing with complexities. Though complexities in A2 alternative appears low, but local discontinuities which is obvious in this kind of project location will convince designer to address instability issues separately.
- The support element is designed based on the rock mass classification decided for the project. Most of the sections behave positively with the support system applied, but Some sections like A1.3 CH-808 have no sufficient minimum principal stress to counter the hydrostatic head. So, such section is recommended for fully lined until the safe zone is not reached.
- Alignment 2 has the longest tailrace section, and the tunnel is assumed to behave similarly to head race tunnel based on the rock type it is located in. The analysis concludes that it is the cheapest option of all. Except for A2.1 other two option are comparatively cheaper than Alignment 1.
- Alignment A2 has rock type distribution better than alignment A1 (Table 6.2). So, the support cost for A1 turns out to be more with additional support quantity. But ones the investigation during construction begins value may alter. For assurance, critical points can be investigated, and results can be updated accordingly.

10.2 Powerhouse Cavern

- Cavern orientation for both the alternative makes an angle of 68 degrees with foliation plane. Among two other major joint sets one is parallel to the cavern orientation so local instabilities is expected in Cavern A1.
- Cavern A1 has rock overburden of 300 m and cavern A2 has 451 m as overburden. Results from nearby Dordi HPP intake shows that rock strength varies from 39-50MPA which is reflective for the Cavern A1, so low stress situation and parallel joint can influence plastic deformation around the periphery of opening. Careful staging of excavation should be followed to limit the deformation, time, and cost for the project.
- Numerical analysis performed for the both the cavern shows that good staging can significantly reflect the displacement and instability problem. If more local stabilities are encountered during construction staging can be modified to counter those problem first.

10.3 Recommendation

- For Cost-effectiveness of the tunneling, predicting rock mass quality is very important. The difference between actual rock mass and what has been considered during analysis will procure disputes between stakeholders and result in project delays.
- It is recommended to perform a better geological investigation to interpret the result obtained from this thesis and assess the effectiveness of the support applied which will help to decide final support requirement.
- The degree of uncertainty still exists even though the pre-construction investigation is performed, due to difficulty in estimation of reliable rock mass strength. It would be wise to investigate the critical section identified in detail from this thesis.
- One way of dealing with such issues would be to increase the database of information by installing measuring instruments and reviewing from experience.
- During the investigation days, some location can be separated as vulnerable zone which can have long term stability issues like rock fall, water leakage and displacement. This will help us to narrow down the cause of problem if encountered once the project is in operating stage. Because constant monitoring during operating years is difficult.
- We can thus say that investigation during construction is necessary act. An option for design modification should always be allowed, because "standard investigation procedure" according to (Nilsen, 2014) does not exist due to variation in ground conditions. Results obtained from methods like tunnel mapping, probe drilling, and measurement while drilling must be incorporated into the stage progression to have an optimum result.

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Appendices

Apendix A: Himchuli Dordi Project Description

Table: A Project summary for Himchuli Dordi HPP (Ltd., 2014)

General	
Development Region	Western
Zone	Gandaki
District	Lamjung
VDC	Phaleni & Dhodeni
River	Dordi
Licensed Area	
Latitude	28018’50” N - 28020’35” N
Longitude	84033'40” E - 84035’10” E
Type of Scheme	Run-off-River
Installed Capacity	56 MW
Gross Head	810 m
Hydrology	
Catchment Area	115km2
Design Discharge Himchuli Dordi	5.75 m3/s @ Q40%
Annual Rainfall	2035 mm
Headwork	
Type of Diversion	Free flow weir with boulder lining
Type of Intake	Side intake with orifice
Approach Canal	
Type	Rectangular shaped
Length	400m
Headrace Tunnel	
Type	Horseshoe shaped
Diameter	3.5m
Powerhouse	
Type	Underground (35m x 18m)
Type of Turbine	Pelton
Number of Units	3
Rated Discharge for each	1.92 m3/s

Appendix B: Analytical Calculation for all the alignments

Table: B Alignment 1 principal stress and factor of safety

	Chainage	σ1	σ3	σ3 required	FS	Remarks
For alignment A1.1	3+286.47	3.80	1.63	0.12	14.05	Ok
	4+141.33	4.16	1.83	0.12	15.24	Ok
	4+241.33	4.24	1.87	0.12	15.50	Ok
	4+341.33	4.49	2.00	0.12	16.46	Ok
	4+820	2.99	0.99	0.13	7.85	Ok
For alignment A1.2	3+286.47	3.80	1.63	0.12	14.05	Ok
	4+141.33	6.88	2.89	1.24	2.33	Ok
	4+241.33	7.40	3.05	1.40	2.17	Ok
	4+341.33	8.13	3.26	1.58	2.07	Ok
	4+820	9.29	3.60	2.77	1.30	Ok
For Alignment A 1.3	3+286.47	4.05	1.77	0.12	15.30	Ok
	3+986.47	4.33	1.92	1.56	1.22	NOT OKAY
	4+056.47	3.79	1.62	1.71	0.95	NOT OKAY
	4+572.12	9.29	3.60	2.77	1.30	Ok

Table: C showing the minimum Principal stress available and required along the alignment 2

	Chainage	σ1	σ3	σ3 required	FS	Remarks
For alignment A2.1	3+153.2	4.75	2.11	0.18	11.48	Ok
	3+300	3.79	1.62	0.19	8.58	Ok
	3+733.07	5.67	2.48	0.20	12.29	Ok
For alignment A2.2	3+153.2	4.75	2.11	0.18	11.48	Ok
	3+300	5.23	2.31	0.64	3.63	Ok
	3+989.32	8.88	3.48	2.76	1.26	NOT OKAY
For Alignment A 2.3	3+286.47	3.73	1.58	0.18	8.60	Ok
	3+586.47	4.68	2.08	1.45	1.44	Ok
	3+896.47	8.88	3.48	2.76	1.26	NOT OKAY

Table: D Surge effect analysis

	Alignment 1				Alignment 2		
Particulars	A 1.1	A 1.2	A 1.3	Units	A 2.1	A 2.2	A 2.3
Length of steel lining	1401.52	1125.06	1125.06	m	1294.07	898.93	898.93
Length of un-lined section	4610.26	4631.91	4398.31	m	3522.26	3817.04	3604.4
Length from intake to turbine	6011.78	5756.97	5523.37	m	4816.33	4715.97	4503.3
length turbine to surge shaft	1416.64	1140.18	1140.18	m	1329.69	1773.82	1561.2
Design discharge Q_0	5.75	5.75	5.75	m3/s	5.75	5.75	5.75
velocity in unlined	1.2	1.2	1.2	m/s	1.2	1.2	1.2
Gross head	810	810	810	m	810	810	810
net head	801.64	801.91	802.25	m	803.44	803.39	803.71
mannings coeffieceint	43	43	43		43	43	43
Area of headrace tunnel	10.93	10.93	10.93	m2	10.93	10.93	10.93
Area of steel lining	5.31	5.31	5.31	m3	5.31	5.31	5.31
Surge check	11.27	10.44	10.08	OK	9.29	8.50	8.18
Tw	0.63	0.57	0.55		0.53	0.46	0.45
Ta	6.00	6.00	6.00		6.00	6.00	6.00
$\frac{Ta}{Tw} > 6$	9.55	10.60	10.90	OK	11.29	13.05	13.46
Ast required	1.53	1.53	1.53	m2	1.53	1.53	1.53
design diameter of surge	3.00	3.00	3.00	m	3.00	3.00	3.00
Designed Ast	7.07	7.07	7.07	m2	7.07	7.07	7.07
del Q($\pm$)	12.07	10.82	10.82	m	11.69	13.50	12.67
Submergence	3.27	3.27	3.27		3.27	3.27	3.27
water level at surge tank	2692.90	2692.87	2693.22	masl	2694.08	2694.39	2699.75
water level at surge intake	2680.84	2682.04	2682.39	masl	2682.39	2680.89	2687.08
maximum upsurge	2704.97	2703.69	2704.04	masl	2705.77	2707.90	2712.42

Appendix C: Geological information for the case project

A) Test data of Super Dordi used for Himchuli HPP

Table: E Lab Data for Super Dordi HPP

Sample	UCS (MPa)	E-module (GPA)	Poissons Ratio
Sd2	40.5	18.96	0.45
Sd3	43.6	24.35	0.31
Sd5	42.5	21.14	0.24
Sd6	29.4	13.81	0.25
Average	39	19.565	0.3125

B) Measured joint strike and dip near cavern location for Alignment A1

Table: F Investigated discontinuities and foliation plane in cavern A1

Foliation Joints (Jf)		Cross Joints (J1)		Cross Joints (J2)	
Strike	Dip	Strike	Dip	Strike	Dip
160	40NE	150	50SW	25	50SE
160	40NE	130	50SW	40	65SE
155	30NE	170	65SW	60	60SE
155	25NE	150	50SW	50	60SE
155	25NE	110	50SW	45	65SE
115	20NE	135	70SW	60	45SE
130	30NE	180	45SW	50	55SE
140	35NE	140	50SW	40	60SE
160	30NE	140	55SW	30	45SE
120	25NE	140	70SW	30	65SE
120	30NE	110	75SW	25	60SE

D) Horizontal Compressional Stress from world stress map

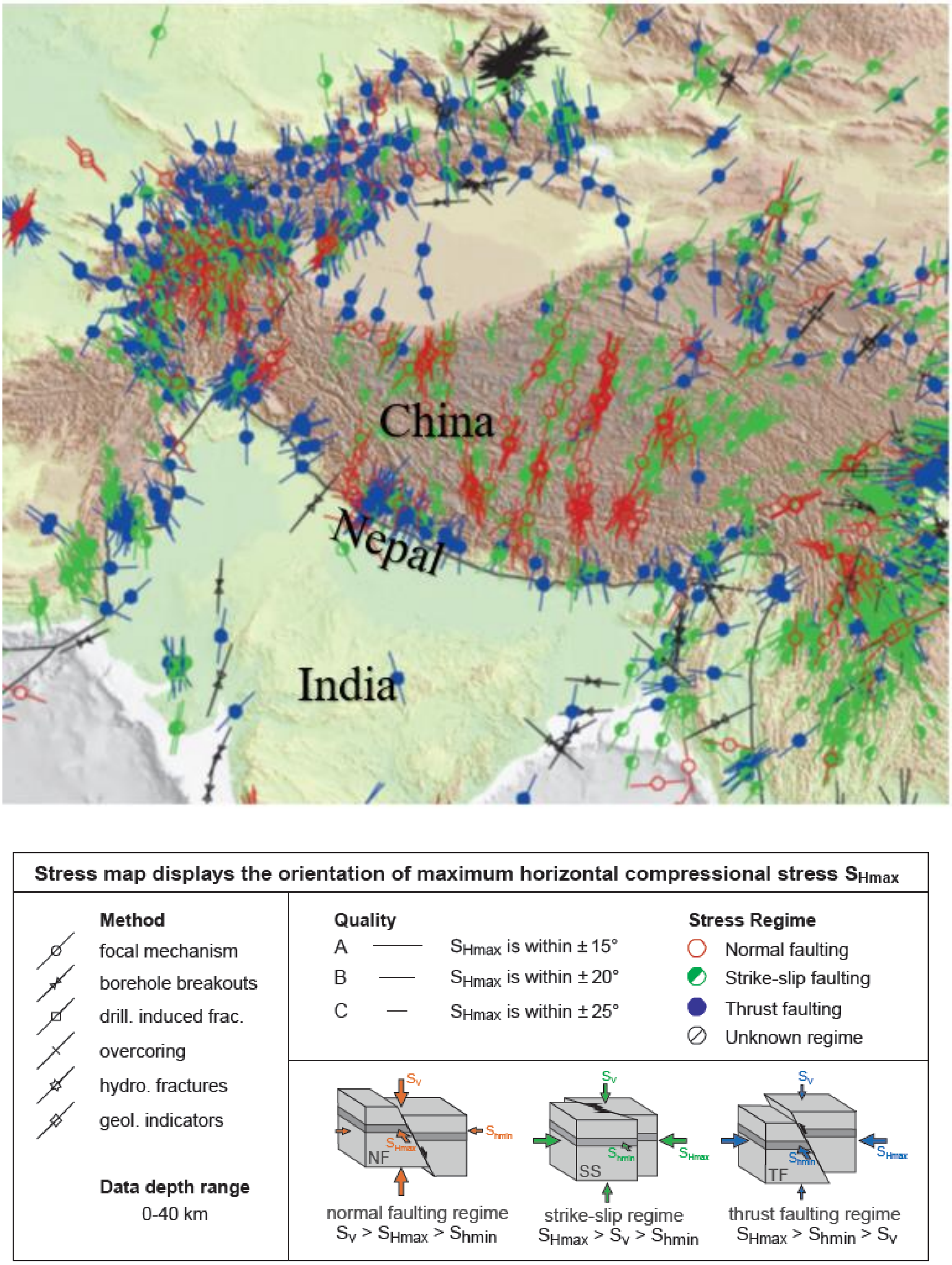


Fig: A World stress map for the Himalayan region (WSM, 2016)

Appendix D Geological charts

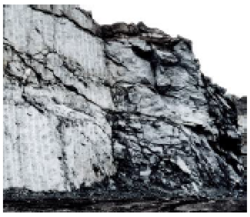
Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	D = 0
	Mechanical or hand excavation in poor quality rock masses (no blasting) 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

Fig: B suggested Values of Disturbances Factor, D (Hoek, 2007b)

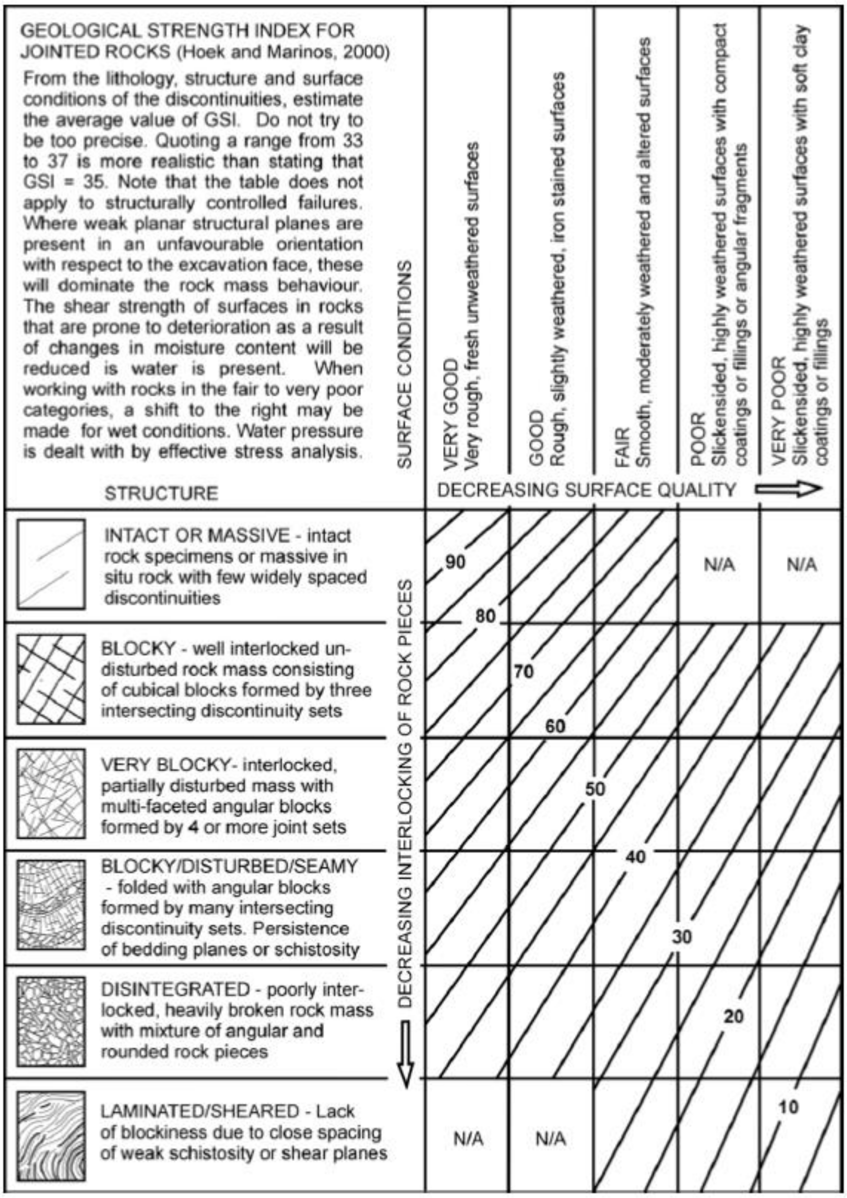


Fig: C Geological strength index for jointed rock masses (Hoek and Marinos, 2000)

Appendix E: Total cost calculation with support specification

Table: G Cost calculation for common section

common section				support Cost ($)			
Rock class	Length	volume	Excavation cost($)	SRF	S-bolt(no:)	Total no	S-Bolt
I	828.00	9052.64	497895.11	367379.64	552.00	3864.00	96600.12
II	1018.82	11138.89	612638.75	452045.01	679.21	5433.71	135842.67
III	518.85	5672.65	311995.85	345316.48	432.38	3459.00	86475.00
IV	191.63	2095.11	115231.31	170050.42	159.69	1596.92	39922.92
V	450.00	4919.91	270594.84	499156.51	375.00	3750.00	93750.00
Total	3007.30	32879.20	1808355.88	1833948.06	2198.28	18103.63	452590.70
Length of tailrace	315.35	3447.76	189626.85	139919.12	210.23	1681.87	126140.00
Access to powerhouse	359.01	3925.07	215878.70	318578.90	299.17	2991.72	224379.31
Access to vertical shaft from main access	125.22	1369.05	75297.53	111118.90	104.35	1043.50	78262.50
Access to first benchmark	178.93	1956.26	107594.52	158780.58	149.11	1491.08	111831.25
Access to second benchmark	196.17	2144.75	117961.31	174079.17	163.48	1634.75	122606.25
Access to common adit zone	231.55	2531.57	139236.08	205475.01	192.96	1929.58	144718.75
Access to second brook intake	166.17	1816.76	99921.66	147457.49	138.48	1384.75	103856.25
Access to first brook intake	96.70	1057.23	58147.83	85810.55	80.58	805.83	60437.50
Total	1669.10	18248.45	1003664.48	1341219.71	1338.36	13383.56	133835.58

Overall length	4676.40	
G Total	2478719.76	
without head race per m	1485.0664	$
total cost for common section	6573614.4	$
per m cost	1405.7004	$

Table: H Cost Calculation for A1.1

Rock class	Length	volume	Excavation cost($)	SRF	S-bolt		
					number	Total	Cost
I	-	-	-	-	-	-	-
II	-	-	-	-	-	-	-
III	914.97	10003.48	550191.48	608950.98	762.48	6099.80	457485.00
IV	238.43	2606.78	143373.17	211580.24	198.69	1986.92	149018.75
V	371.31	4059.58	223276.83	411870.68	309.43	3094.25	232068.75
Access tunnel	112.61	1231.18	67714.86	124911.14	93.84	938.42	70381.25
Total	1637.32	17901.02	984556.33	1357313.05	1364.43	12119.38	908953.75
Length of steel lining m	1401.52	13393.94	1205454.69				
Concrete filling	-	5952.863	1041750.96				
Steel lining cost per weight	-	-	202.11				
Total cost steel	-	-	623174.65				
Concrete filling with steel rib	202.94	739.86	2697.31				
Steel rib no:	625.71	no	-				
Steel rib cost	-	-	218998.18				
The total length of sec	3038.84						
Grand Total cost	6342898.94						
Cost per meter	2087.28						

Table: I Cost Calculation for A1.2

Rock class	Length	volume	Excavation cost($)	SRF	S-bolt		
					number	Total	Cost
I	-	--	-	-	-	-	-
II	380.53	4160.382	228821.01	253258.7	253.69	2029.493	152212
III	547.96	5990.915	329500.33	364690.4	456.63	3653.067	273980
IV	241.95	2645.27	145489.83	214703.9	201.63	2016.25	151218.8
V	374.19	4091.077	225009.23	415066.4	311.83	3118.258	233869.4
Total	1544.631	16887.64	928820.41	1247719	1223.77	10817.07	811280.1
	Length	volume	Cost ($)				
steel lining excavation	1125.06	10751.89	967670.00				
Concrete filling	-	4778.617	836258.02				
Steel lining cost per weight	-		202.11				
Total cost steel	-		500248.93				
Concrete filling with steel rib	194.35	708.54	177135.85				
Steel rib no:	176.68	no	-				
Steel rib cost	-		61838.64				
The total length of the section	2669.69						
Grand Total cost	5530971.32						
Cost per meter	2071.76						

Table: J Cost Calculation for A 1.3

Rock class A1.3	Length (m)	volume(m3)	Excavation cost($)	SRF($)	S-Bolt		
					number	Total	Cost
I	-	-	-	-	-	-	-
II	92.30	1009.18	55504.72	61432.52	61.54	492.291	36921.8
III	471.36	5153.44	283439.08	313709.9	392.80	3142.4	235680
IV	91.02	995.10	54730.51	80767.52	75.85	758.475	56885.63
Lining section	688.28	7525.05	413877.82	-	-	-	-
Total	1342.9	7157.71	807552.13	455909.9	530.1838	4393.17	329487.4
Additional Work	Length	volume	Cost ($)				
Length of steel lining m	1125.06	10751.89	967670				
Concrete filling		4778.62	836258.02				
Steel lining cost per weight	-	-	202.11				
Total cost steel	-	-	500248.93				
Concrete filling with steel rib	688.28	3053.86	763465.43				
Steel rib no:	625.71	no					
Steel rib cost			218998.18				
The total length of sec	2468.02						
Grand total cost	4879590.04						
Rate per meter	1977.13						

Table: K Total cost calculation for A 2.1

Rock class A 2.1	Length	volume	Excavation cost($)	SRF	S-bolt number	S-bolt Total	S-bolt Cost
II	80.72	882.52	48538.70	89537.59	67.27	672.67	50450.00
III	-	-	-	-	-	-	-
IV	348.07	3805.49	209302.11	386092.02	290.06	2900.58	217543.75
V	150	1639.97	90198.28	166385.50	125.00	1250.00	93750.00
Access tunnel	197.74	2161.92	118905.39	219340.46	164.78	1647.83	123587.50
Total	695.81	7607.38	418405.77	771817.99	579.84	5798.42	434881.25
Length of steel lining m	1294.07	12367.07	1113036.38				
Concrete filling		5496.476	961883.29				
Steel lining cost per weight	-	-	202.11				
the total cost of steel	-	-	261544.49				
The total length of the section	1989.88						
Grand Total cost	3961569.18						
Cost per meter	1990.86						

Table: L Total cost Calculation for A2.2

Rock class	Length	volume	Excavation cost($)	SRF	S-bolt number	S-bolt Total	S-bolt Cost
II	84.19	920.4598	50625.29	56031.98	56.13	449.0133	33676
III	267	2919.144	160552.94	177699.7	222.50	1780	133500
IV	364.21	3981.953	219007.44	323196.1	303.51	3035.083	227631.3
V	156.95	1715.954	94377.47	174094.7	130.79	1307.917	98093.75
Total	872.35	9537.512	524563.14	731022.5	712.9267	6572.013	492901
-	Length	volume	Cost ($)				
steel lining excavation	898.93	8590.83	773174.4				
Concrete filling	-	3818.145	668175.41				
Steel cost per weight	-	-	202.11				
Total cost steel	-	-	181682.74				
The total length of sec	1771.28						
Grand Total cost	3371519.17						
Cost per meter	1903.44						

Table: M Cost Calculation for A2.3

Rock class A2.3	Length (m)	volume(m3)	Excavation cost($)	SRF($)	S-Bolt		
					number	Total	Cost
II	336.57	3679.76	202386.90	224001.5	224.38	1795.04	134628
III	-	-	-	-	-	-	-
IV	325.62	3560.04	195802.43	288951.7	271.35	2713.5	203512.5
Total	662.19	7239.81	398189.33	512953.2	495.73	4508.54	338140.5
	Length	volume	Cost ($)				
Length of steel lining m	898.93	8590.83	773174.4				
Concrete filling		3818.15	668175.41				
Steel cost per weight	-	-	202.11				
Total cost steel	-	-	181682.74				
Concrete filling with steel rib	50	182.29	911.43				
Steel rib no:	-	no	45.45				
Steel rib cost	-	-	15909.09				
The total length of sec	1561.12						
Grand total cost	2889136.10	USD					
Rate per meter	1850.68	USD/m					

Appendix F: Numerical Model for A2.3 CH-3+896 at the junction with penstock

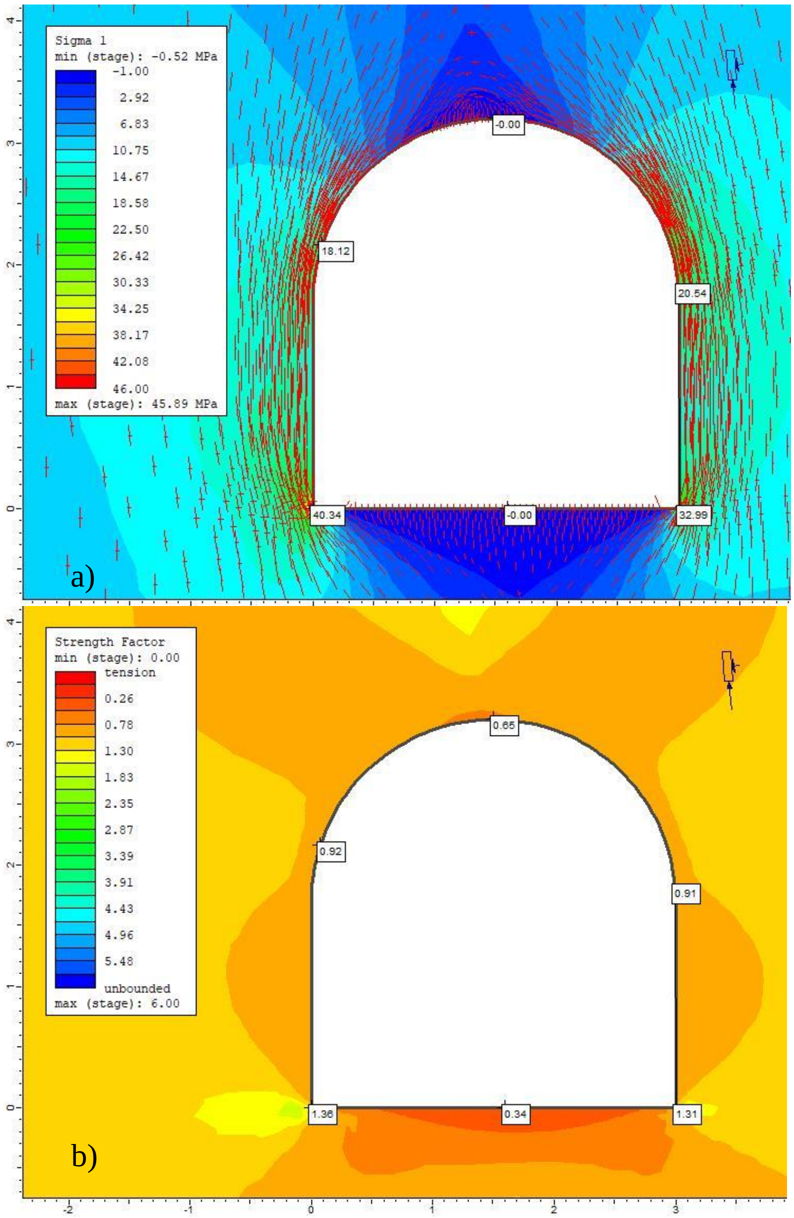


Fig: D a) Major principal stress with stress orientation, b) Strength factor for the section A2.3

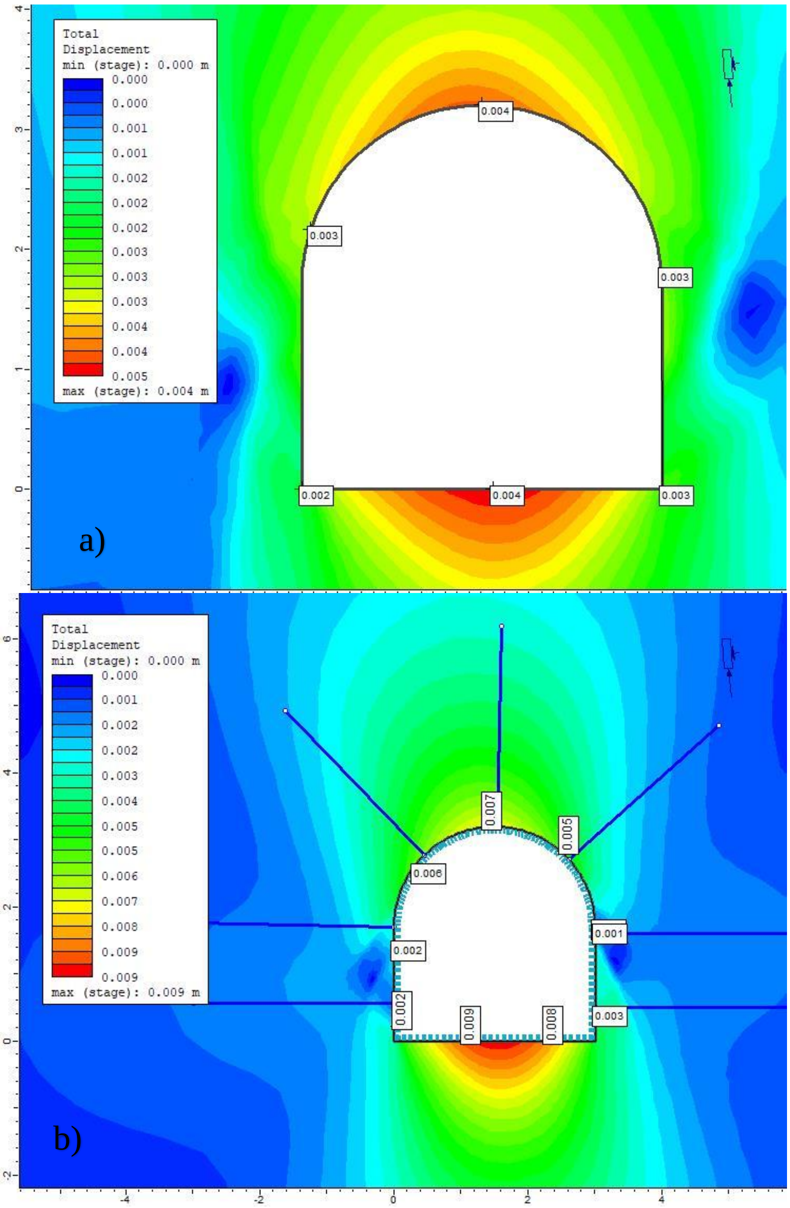


Fig: E a) Total Displacement before the support installation b) Total displacement after support installation