NTNU – Trondheim
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Science and Technology

STABILTY ASSESSMENT OF HEADRACE TUNNEL SYSTEM FOR PUNATSANGCHHU II HYDROPOWER PROJECT, BHUTAN

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Hydropower Development
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TGB4910 Rock Engineering - MSc thesis
for
Karma Tshering

STABILTY ASSESSMENT OF HEADRACE TUNNEL SYSTEM FOR PUNATSANGCHHU II HYDROPOWER PROJECT, BHUTAN

Background

The development of hydropower potential of the Punatsangchhu river began in 1992 with the pre-feasibility study of Punatsangchhu II Hydropower Project by Royal Government of Bhutan with a financial support received from UNDEP and NORAD. The first pre-feasibility was completed by Norconsult AS in 1993. The study was again revised and updated by the government in 2004.

The revised version of the project has a design discharge of 437 cubic meters per second and has a gross head of 267 meters. The installed capacity of the plant is 992 MW with an annual average energy production of 4,666 GWh. The project mainly consists of a diversion dam, six settling basin chambers, three parallel headrace tunnels with each 88 square meter cross-section and 11.5 kilo-meters long, underground powerhouse and transformer caverns with solid rock excavation of 130,000 cubic meters, a 250 meters long access tunnel with 50 square meters cross-section and three 350 meters long tailrace tunnels with similar cross-section as headrace tunnel.

The candidate has carried out field work during the summer 2011 and collected necessary data and information for the MSc thesis work.

MSc thesis task

This MSc thesis is related to the stability assessment of the headrace tunnel system of Punatsangchhu II Hydropower Project. Evaluation on the existing layout, possible alternative layout of the project and stability assessment of the headrace tunnel system will be of main focus, and shall include:

- Brief description of the project and engineering geological investigations at planning.
- Evaluation of the existing layout plan and discussion on the challenges associated to the implementation of existing layout in connection with rock engineering and tunnelling perspectives.
- Suggestion on the alternative layout if existing layout plan has several weaknesses.
- Review on the rock mass quality along the headrace tunnel alignment and assessment of the rock mechanical properties, which could be used as inputs for stability assessment.
- Stability assessment of the headrace tunnel system using analytical, empirical and numerical approaches and suggest optimum final rock support requirement.

Relevant computer software packages

Candidate shall use *roc-science package* and other relevant computer soft wear for the master study.

Background information for the study

- Relevant information about the project such as reports, maps, information and data collected by the candidate.
- 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.

The project work is to start on January 16, 2012 and to be completed by June 11, 2012.

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

January 12, 2012

Krishna Panthi

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

FOREWORD

This thesis titled "**Stability Assessment of Head Race Tunnel for Punatsangchu-II Hydropower Project in Bhutan**" is submitted to the Department of Geology and Mineral Resources Engineering, Norwegian University of Science & technology (NTNU), Trondheim, Norway as an obligatory requirement for partial fulfillment of Masters of Science Degree in Hydropower Development course 2010-2012.

The thesis is an outcome of the authors work carried out during the final semester of the study period. It deals with review of engineering geological conditions of the rock mass along the Head Race Tunnel. After assessing the geological and geotechnical features along the HRT design layout, explores possible alternate alignment optimizing on the geological conditions along the existing layout. The engineering property of the rock mass is studied from stability perspective and possible instabilities discussed. Finally the designed support system is checked for adequacy both empirically by NGIs Q standards and with the use of Phase2 numerical modeling.

The required datas and information on the project were collected by the candidate during the summer break of June, 2011. The field visit to the project site was also made during the same period. The thesis work is carried out under the supervision of Associate Professor Dr. Krishna K. Panthi from the Department of Geology and Mineral Resources Engineering, NTNU during the period from January to June, 2012. This thesis work is an academic in nature and the analysis presented herein is combination of knowledge gained from lessons taught during this course and my personal experience working as a hydropower construction engineer. The outside contributions and inputs are duly acknowledged.

Karma Tshering
Msc. in HPD
NTNU, Trondheim
11th June, 2012.

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I'm grateful to the Department of Hydraulics & Environmental Engineering, NTNU for giving me this opportunity to pursue my master's degree under this department. I thank all the faculty members who have taught us during the course period and others who have given all logistic supports and made the study period conducive and enjoyable. My sincere thanks to Professor Anund Killingtveit , professor in-charge for all the facilities and supports extended during the whole course period. Mrs. Hilbjorg Sandvik the course coordinator for this Hydropower development course has been very kind and supportive arranging everything positively and made everything comfortable. My gratitude is due to Mrs. Hilbjorg Sandvick for all the moral and administrative support.

I express my gratitude to the Norwegian state educational loan fund for providing me scholarship under the quota scheme to pursue my study at NTNU.

I would like to express my gratitude to the Managing Director, Punatsangchu-II Hydropower project in Bhutan for providing and giving me unhindered access all the data's and information on the project. Especial thanks are due to Dr. Santosh Sathi, consultant geologist with Water and Power Consultant (WAPCoS) who has provided all the important information and investigation data on the project.

I have come across so many wonderful persons during my stay here in Trondheim. I thank you all for memorable time we spent together. My friends back home who have given me immense encouragement and support which made things easy for me. Thank you all for everything.

Finally, I would like to express my emotional gratitude to my wife who has taken lovely care our daughter and son during my absence from them. The constant faith and encouragement from my family have made things easy for me. Lastly I pray my humble gratitude to my parents. I'm what I'm today all because of my parents. Thanks you for being lovely and caring parents.

Karma Tshering
June, 2012

EXECUTIVE SUMMARY

Bhutan is a small Buddhist country nested in the cradles of the eastern Himalayas. It has rugged mountainous terrains with altitudes ranging from 500 masl in its southern plains to 4500masl in the snowy capped mountains in north. This rugged mountainous topography blessed Bhutan with huge hydropower potential. The hydropower potential of Bhutan is estimated at 30000MW with projects above 10MW capacity as per the power system master plan. This is one of the highest in the world considering its small size. In absence of other valuable natural resources, hydropower has become the main national resources in the country. Out of huge potential, Bhutan today has 1488MW of installed hydropower project under generation illuminating about 90% of all houses in the country and feeding power to its developing industries. Because of its small size and even smaller population, the internal electricity consumption of Bhutan is small roughly 30% of its present generation. The balance power is exported to India generating the most needed revenue for the economic development of the country. Bhutan plan to construct 10000MW of hydropower projects within year 2020 among which some mega projects are already under construction.

Bhutan being located in the tectonically active Himalayan geological formation faces lots of geological challenges in tunneling works. The Himalayan geology intruded by numerous geological discontinuities, and frequented by the tectonic activities poses lots of challenges for tunneling works. The Himalayan geology is very complex and rock mass properties changes greatly within small distances.

The engineering geological investigation for an underground work is very important. Detailed investigation is important during the prefeasibility and feasibility study stage of the project. But the availability of funds and time are the constraining factor in performing detailed engineering geological investigations. Balance need to be made to cover most important geotechnical studies within the availability time and fund to get a reliable design.

Punatsangchu II hydropower project is a runoff river scheme project being constructed along this river basin in central western part of Bhutan. In this thesis, the engineering geological study of the HRT was carried out with an aim to carry out stability assessment and support requirement for HRT. The engineering properties of the rock mass along the HRT are reviewed from the stability aspect with discussion on orientation of main foliation, joints and weakness zones with the tunnel alignment. An alternate alignment is proposed, optimizing on orientation of main foliation, shear zones and main jointing with the tunnel alignment, restricting the exercise within the same limits of the start and end point of the HRT as given in designed layout.

It has been found that the topography along the HRT permits very limited alternatives for the tunnel alignment layout and the location of construction adits. The main foliation and joint orientations were taken care for the tunnel alignment. However, there are some rooms for optimization within the same limits, if taken care of could bring better benefit to the project. This possible optimization was used in the proposed alternate alignment.

The rock mass characteristics and possible instability problems along the HRT alignment were also reviewed in stability study. There are very less possibility of squeezing problems along the tunnel alignment but minor instability problems could not be fully ruled out due to the low rock mass strength. With the tunnel alignment crossing two nalas, some minor seepage problems were also expected at those nala crossing areas. Possible solutions for instability and seepage problems were also recommended in the relevant cases.

The NGIs Q method and Bieniawskis RMR methods were used for the rock mass classification and the support design thereof. The designed rock supports are cross checked with the standards NGIs support chart and Bieniawskis RMR guide charts. It is found that a typical conservative support approach practiced in the Indian sub continent is followed for the support system. The supports designed were on much conservative side compared with support requirement from Q support charts.

Finally the supports were further checked by using Phase2 numerical modeling. The result of the numerical analysis suggests lighter supports compared with the designed support for the HRT. The adoption of the conservative methods may keep the instability problems at bay, but the cost affect can be on the higher especially when the projects suffer from fund availability.

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Regional map of Bhutan with the project location

1 INTRODUCTION

1.1 General

Electricity is an important requirement for socio economic development of any country. It is produced from different sources including nuclear, thermal and hydropower etc. Electricity produced from nuclear and thermal source is associated with huge negative impact on environment, climate and human life. The recent Fukushima nuclear plant accident in Japan, 2011 has highlighted the severity of the risk posed by nuclear plant mishaps on environment and human life. The electricity produced from renewable sources like solar, hydro, wind and tidal are clean and environmental friendly. The geographic location of the country however defines the potential capacity to produce energy from these sources. The agents of renewable sources are spatially distributed over time and space. So every country does not have the access to clean energy source. Besides except from hydro, production of energy from clean renewable sources such as solar, tidal and wind is yet to be proven technically on massive commercial scale. In absence of better alternate and yearly increasing demand for energy, the nuclear and thermal plants will continue meeting the energy needs for years to come.

Hydropower produces clean energy but is not free from all negative impacts. The hydropower projects bring about submergence of large fertile agricultural lands, displacement of peoples and wild animals, and negative impact on environment adding to global warming. Large reservoir schemes have huge negative impacts both on society and environment during and after construction compared with run of river projects. It occupies large corridor of fertile lands and disturbs habitat of wild animals. Building of weirs across the river stops the flow of sediments to the downstream of dam location changing the hydraulic of the river system. It restricts the free movement of fishes and other aquatic animals to upstream and downstream of the weir. Increased incidences of methane and carbon dioxide production were also reported in the large reservoirs (Hakon Sundt, SINTEF, 2011) from the decomposition of vegetation growths and debris under the dammed water in the reservoir area.

For development activities, energy is required and it is the poor undeveloped countries that need the energy most. Nothing comes free and in totality. To embarrass development certain decisions have to be made. The best decision is to balance the advantage and disadvantage side of the technology, and find a balance between the two. Hydropower is one of the best renewable sources of energy. Strong rules should be enacted to prevent excess negative activities which can be avoided. A balanced development approach to hydropower development should be the way out. The run of river projects are one of the best options available from hydropower sector and this technology should be taken advantage to reduce poverty and underdevelopment from the face of the world.

1.2 Hydropower development in Bhutan

Bhutan is located in the eastern Himalayas, sandwiched between India in south, east and west and Tibetan region of China in the north. It has total area of 38394 km^2 spanning from 26.7^{0}N to 28.4^{0}N latitude and 88.7^{0}E to 92.2^{0}E longitude. The rugged mountainous terrain of the country with perennial rivers fed by the snowy mountains in the north make the country attractive for hydropower potential.

Bhutan ventured into development of hydropower resources only in late 1970s. Till then, the energy requirements were met from small mini and micro power plants in the urban towns, wood and kerosene lamp in the rural areas which were the only source of energy. The construction of country's first mega hydropower project, 336MW Chukha power plant started in 1978 under the financial and technical aid from government of India. The plant started generating power since 1986. Only after the commissioning of Chukha power plant, Bhutanese realized the importance of hydropower industry. At present Bhutan has an installed capacity of 1488MW which is $\approx$ 5% of its estimated 30000MW total potential. This total potential includes only the projects with capacity greater than 10MW potential capacity. The power plants under generation includes 336MW Chukha (1986), 60MW Kurichu (2001), 64MW Basochu (2003), 1020MW Tala (2006) and rest 8MW from mini projects. The present internal demand of the country is only 30% of its present generation capacity and the rest 70% is exported to India making it the country's highest export commodity.

The hydropower sector in Bhutan got the much needed momentum after the first democratically elected government came to power in 2008. The new government has given preference for hydropower development to boost its economic development. Soon after it came to power, the memorandum of understanding was signed with the government of India to jointly develop 10000MW hydropower project within year 2020. This move has accelerated the feasibility study reports and finalization of DPR of many projects then under planning. Of the targeted 10000MW by 2020, the construction works of 1200MW Punatsangchu I project started in 2008 and is scheduled for completed by year 2016. Works for 990MW Punatsangchu II started in fall 2010 and is scheduled for completion by year 2017. The works for 720 MW Mangdechu project was awarded in March, 2012 and construction works have already started. The DPR for 600MW Kholongchu, 180MW Bunakha, 770MW Chamkharchu and 570MW Wangchu are already finalized and the construction works are expected to start by the fall of 2012 (Kuensel, March, 2012).

1.3 Hydropower potential of Bhutan

The total hydropower potential of Bhutan is estimated at 30000MW with technically feasible potential estimated at 23760MW (Power system master plan, 2003) for projects above 10MW from 76 project sites. Since hydropower is capital intensive project, the country could not take advantage of its benefit due to technical and financial constraints. Besides, the rugged terrains make the transportation of material and machineries difficult further increasing the cost of construction. Most of the potential project sites are located deep into the country side along the five river basins of Bhutam namely Wangchu and Punatsangchu in west, Mangduechu and Chamkharchu in central and Dangmechu in the east.

Department of renewable energy under the ministry of Economic Affairs is responsible for policy framing and planning of the projects. Druk Green Power Corporation is a government owned entity entrusted with operation and maintenance of the commissioned projects. It is also responsible to encourage and attract foreign investors into hydropower construction in Bhutan. The country's FDI policy was formulated and came into effect in 2012.

Bhutan has huge hydropower potential beyond its need but lags technical and financial ability, India on other side suffers from huge energy crisis. This situation is transformed into win-win opportunity for the two countries when a memorandum of understanding was signed between the two countries in 2008. According to this, the government of India will help facilitate construction of these projects by rendering technical and financial support while the Bhutan government will sell the surplus power to India from these projects. Till 2012, the hydropower industry in Bhutan was Indian centered for its construction except for 114 MW Dagachu which is under construction with investment from Austrian government, Tata Company from India and DGPC from Bhutan with different shares holdings. The RGoB has come up with the foreign direct investment policy in hydropower in 2012. It is hoped that with this policy in place, more foreign investors will come forward to develop the vast resource of hydropower in Bhutan.

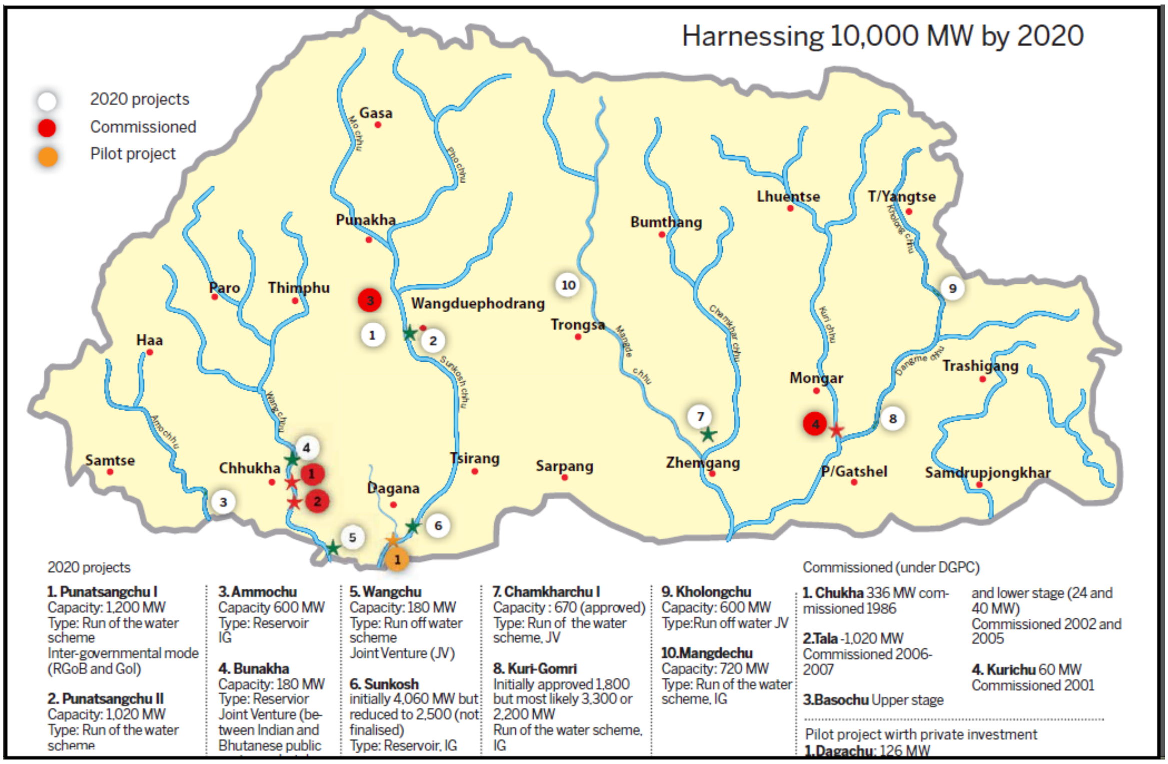


Figure 1.1 location of 10000MW projects to be executed within year 2020 (Kuensel, 28/1/2012)

1.4 Object and Scope of study

The main objective of this thesis is related to the stability assessment of the headrace tunnel system of Punatsangchhu II Hydropower Project. It discusses the geotechnical evaluation of rock mass along the existing HRT layout, explore possible alternative layout, discusses on stability assessment and support system of the headrace tunnel system. The object of the work includes

- Brief description of the project and engineering geological investigations at planning phase.

- Geological evaluation of the existing layout plan and discussion on the challenges associated to the implementation in connection with rock engineering and stability perspective.

- Suggestion of an alternative layout and discussion on its features.

- Review on the rock mass quality along the headrace tunnel alignment and assessment of the rock mass mechanical properties.

- Stability assessment of the headrace tunnel system using analytical, empirical and numerical approaches and suggest optimum final rock support requirement.

1.5 Methodology

The required information and data on the project was collected from the project authority by the candidate during the summer break of 2011. Based on this available information, the engineering geology along the head race tunnel alignment was reviewed. The possible shortcomings of the HRT alignment were studies in respect to stability and other design requirements and possible alternate alignment proposed. Rock engineering theory of the existing alignment on stability situations was reviewed. The rock mass classification system and the support systems were discussed for adequacy. The stability of tunnels and the support systems is further analyzed using Phase2 numerical methods. Concluding remarks from the findings of these studies were made. However, it has to be stated that the study was totally academic in nature and the construction works of the project have already started in 2011.

2 PROJECT DESCRIPTION

2.1 General

Punatsangchhu-II hydroelectric project is one in the series of projects planned along the Punatsangchu river basin in the western part of Bhutan. It is one of 10000MW projects planned to be constructed within year 2020. It is a run of river scheme project utilizing the natural head along Punatsangchu river with an installed capacity of 990MW. The construction works of the project started in fall of 2011 and is expected to be complete by year 2017. After completion, it is expected to generate an annual energy of 4214.5 GWh and benefit Bhutan by the sale of excess energy to India.

An independent authority is created by GRoB for the implementation of the project. The authority will be responsible for the successful implementation and will act as the main representative of government for the consultants and the contractors. After commissioning the authority will be dissolved and project handed over to Druk Green Power Corporation.

2.2 Project location

The project is located on Punatsangchhu river in Wangdue Phodrang Dzongkhag in Western Bhutan. All project components are located between 22 km and 38 km downstream of Wangdue Bridge along the national highway on the right bank of the river. The dam site is about 22.50 km away from Wangdue Bridge.

All the project components are located within small radial distance from the national highway and are already connected with access roads. Paro is the nearest airport and is located about 124 km to the west. Bhutan does not have rail networks but the material can be transported till Hasimara rail way station in west Bengal state of India and transported by road to the project site. The project can be approached either from Phuntsholing via Thimphu from west or from Geylegphu via Tsirang from central south of Bhutan. The road network from either side has all weather fully widened road developed for transportation of big machineries and project equipments for Punatsanchu-I project located just upstream of this project.

The national electrical transmission lines passes near by the project area. The required electricity connections to all the constructions sites were already provided by the project authority before the award of works to the main contractors. The minimum accommodation facilities for the labors and site offices and stores for the main contractors is also constructed prior to the award of works. This is expected to expedite the project completion saving loss of time by contractors during initial mobilization period.

The main Wangdue town is located 22kms upstream of the dam site. Every basic commodity is available in the town, besides there is a daily bus service from the project area to the border town which can be used by the expatriate laborers and for extra commodities not available in the town.

A government hospital is located within radial distance of 22km in the main town and there is another bigger hospital being constructed by the Punatsangchu I project within the same vicinity. The main national hospital in Thimphu is around 90kms from the dam site. The main contractors are also mandated to have a full fledged dispensary units with qualified medical officer at all project sites.

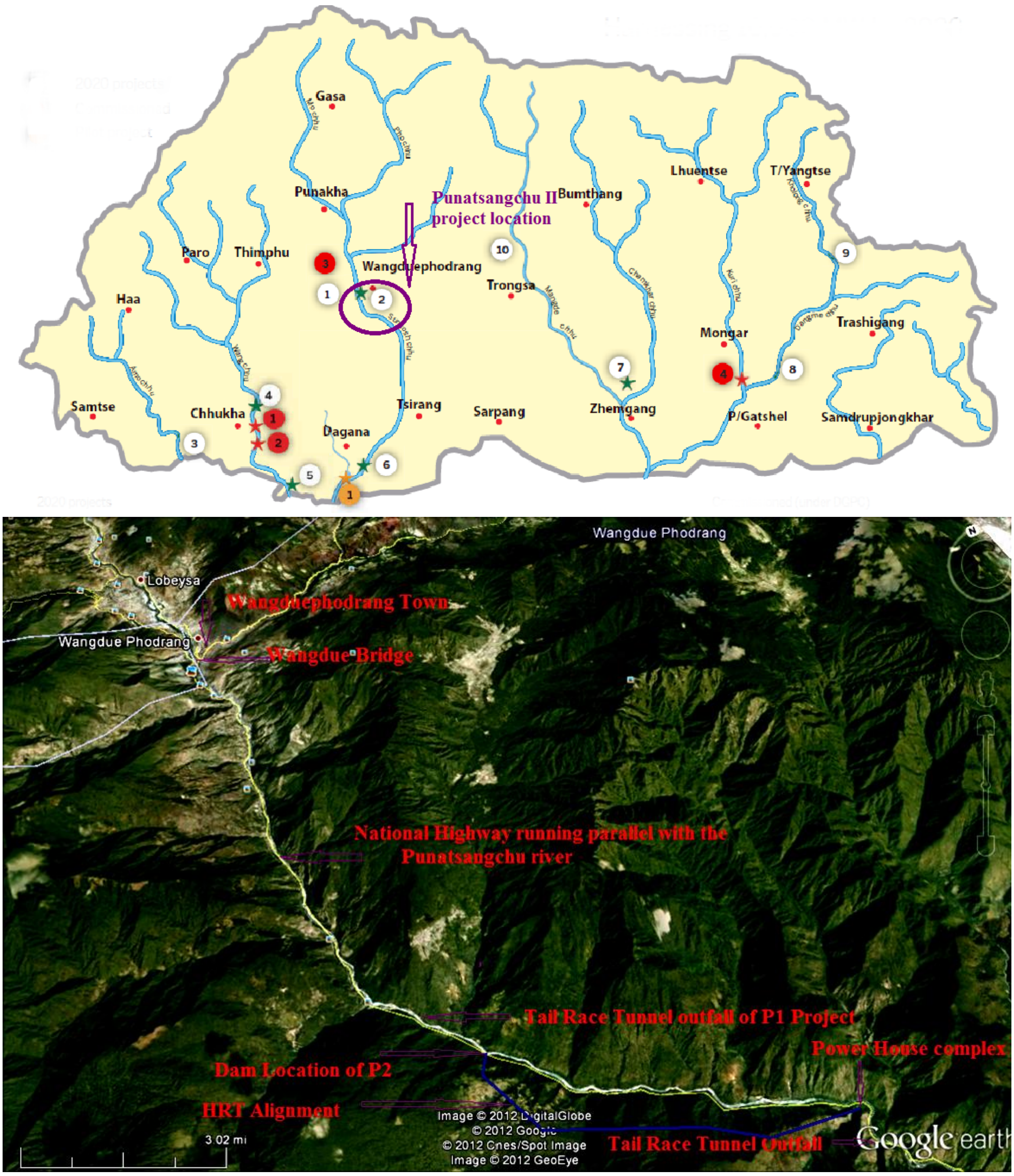


Figure 2.1 Map showing the project location (Google earth, 30/3/2012)

2.3 Regional geology

Punatsangchu project area is located within part of the Tethyan Belt of Bhutan Himalayas and at the proposed dam site; rocks of Shumar Formation of Thimphu Group of Precambrian Age are exposed. The rocks of Thimpu Group in general are characterized by coarse-grained quartzo-feldspathic biotite-muscovite gneiss, with bands of mica schist, quartzite and concordant veins of foliated leucogranite, migmatites with minor metabasics and interbedded limestone. Garnet crystals and porphyroblasts are also seen within this gneiss. The bedrock exposed in the project area (reservoir and dam) is represented by garnetiferous, biotite bearing quartzofeldspathic gneiss showing a general foliation trend N10^{o}E to N40^{o}E and dips 20^o to 40^o towards ESE to SE. At places, the rocks exhibit broad warps as evidence from the swing in foliation from N40^{o}E to N-S and even up to N 10^{o}W- S 10^{o}E.

On the basis of study of Aerial Photographs for Punatsangchhu –I HE Project, three sets of Lineament have been picked up trending (i) N-S (ii) NW-SE and (iii) NE-SW. The N-S trending lineaments aligned parallel to 90^{o}E ridge, which is reported to be neotectonically active mainly in the Bay of Bengal. The Punatsangchhu River probably flows along one such sympathetic north-south trending lineaments at the dam site. The other two sets of lineament are less in abundance. A few NE-SW/NW-SE trending lineaments picked up from the aerial photographs appear to be faults as indicated by the shifting of main river course. The traces of N-S lineaments in colluvial deposits along the valley slope marked by linear topographic elements of varying relief suggest probable active neotectonism in the area.

Seismicity

The Kingdom of Bhutan is located in the eastern part of the Himalayan Orogenic Belt. It has also been found that the recent seismicity in the Himalayas is the highest in 50 km wide zone in the Lesser Himalaya, with a concentration of earthquake epicenters just south of the Main Central Thrust (MCT) with respect to the project site, which may represent seismicity at a deeper part associated with activity of the detachment surface connecting with the Main Boundary Thrust (MBT; reported to be neotectonically active) and Himalayan Frontal Thrust (HFT; reported to be neotectonically active) of the Himalayan Front.

The MCT in the Lesser Himalayan region is situated around 50 km south of Wangdue-Phodang. However another small splay of the MCT is located around 8 km WNW of Wangdi in the Central Himalayas. Similarly, MBT and HFT are also situated ~ 68 km and ~ 70 km south of Wangdi respectively.

Three major faults are present in nearby areas. The most prominent is almost along the course of Punatsangchhu river parallel to the 90^{o}E line and traced from ~35 km south of Wangdi towards south up to the Bhutan-India border. Another minor fault runs almost parallel to Punatsangchhu river course (NW-SE) located around 25 km southeast of Wangdi. Another NW-SE trending fault located 35 km. south of Wangdi was picked up on hills right bank of Punatsangchhu river. All these faults show the manifestation by offset of different litho-packages and other related geological evidences, e.g. abrupt change in Lithology, intrusion of granite and quartz reef, presence of rock flour / mylonite etc. The seismic status of these faults is not known. Neotectonic activity has been suspected in the D/S of earlier proposed powerhouse near village Kerabari.

Therefore, detailed MEQ (Micro Earthquake) study is recommended to incorporate the data in the design of the Project.

2.4 Project features

Punatsangchu II is a run of river hydropower project planned along the main Punatsangchu river. 828.3m long and 12m diameter circular diversion tunnel diverts the river along the left bank of the river. A concrete gravity weir is constructed over the main river and the design discharge of 460m 3/sec flow is conveyed through 8584.3m long 11m diameter circular head race tunnel. The main important features of the project are shown in table. 2.1. The detailed features of the project components are given in the annexure A.

Table 2-1 Important features of Punatsangchu II project (WAPCoS, 2012)

General	
Type of scheme	Run Of River scheme
Maximum gross head	264m and a net design head of 236m
Design discharge	460m3/sec + 20% for silt flushing.
Power and Energy	
Installed capacity	990 MW
Average annual energy	4214.5 GWh
Hydrology	
Catchment Area	6835 km^2
Storage Capacity	Gross capacity 7.0 MCM and 4.6MCM live capacity
Water levels	MWL/FRL El.843 m, MDD El.825m
Design flood	11723 m^3/s PMF + 4300 m^3/s GLOF
Head works	
Dam	Concrete gravity,
Size of dam	213.5m long on top, 86m maximum height from the deepest foundation level.
Intake structure	4 Nos. Bell mouth with 6.4m finished circular
Intake centreline level	El. 814.5m
Desilting chamber	4 Nos. and all underground
Size	19m wide, 24.70m height and 420m length, designed to remove silt particles above 0.2mm size.
Head Race Tunnel	
Shape and size	11.0m diameter circular shape
length	8584.3m
Surge shaft type	Open to sky , Orifice type
size	31.0m diameter with 2.8m restricted orifice size.

height	137m
Butterfly valve chamber	120m length, 12m width and 21m height to accommodate 3 nos. 5.5m valve size.
Pressure shafts	3 nos. 400m long vertical shafts which bifurcate in horizontal reach to feed a turbine each.
Steel liner grade	ASTM 537 Cl-II in the upper reach with thickness varying from 22mm to 36mm and ASTM 517 Gr-F in lower reach with thickness varying from 28mm to 32mm.
Power House complex	
Power house	Underground type
Size	236m long 23m wide and 51m height.
Transformer hall	215.4m long, 14m wide and 26.5 m height.
Turbines	6 nos. Vertical shaft Francis turbine.
Downstream surge chamber	319 m length, 18m width and 58.5m height.
Tail race tunnel	11m diameter D shaped
length	3000m long

The whole civil components of the project construction are divided into three packages for tendering and construction, each package with separate construction adits. This is done to avoid conflict between the contractors during the construction and cause contractual problems for payment at later stage. The access roads to every construction sites were also constructed by the project authority before the award of the main contract works. Some of the major construction machineries were also procured by the authority after discussions with the prospective bidders. This decision by the project authority is expected to cut short construction time and make contractors comfortable and make them concentrate on the main work from the first day of awarding the works.

3 PLANNING AND INVESTIGATION

3.1 Prefeasibility

The hydropower industry is one of the most important engines of developmental growth for the Bhutanese economy. In absence of major manufacturing industries, the balance of the trade with its trading partners India is mainly contributed by sale of electricity to India. Punatsangchu II project is one of the 10000MW hydropower projects planned to be executed within year 2012. The project is already under construction after the contract for the works were awarded in August, 2011. A brief history of the project features from prefeasibility to final design stage is reviewed and discussed in this chapter.

3.1.1 Prefeasibility report (1992-1993)

The prefeasibility study for the project was carried out under grant from United Nation Development Program (UNDP) and Norwegian Development Aid (NORAD) in 1992-1993 by Norconsult International AS from Norway. The initial prefeasibility study of the project was carried using topographic maps, geological maps and other relevant data. No field investigation was carried out at this stage.

The survey was carried out with reference to local coordinate system established for this purpose. A local datum was established using barometer reading only. The traverses carried out were not connected to any Great Trigonometrical Station (GTS station). The dam and powerhouse areas was surveyed and mapped in scales 1:2000 & 1:2500 scales using traditional ground survey techniques. Apart from this, 1:50000, 1:10000 and 1:5000 maps were also used. No control survey was carried out at this stage.

Geological field reconnaissance was carried out for dam and powerhouse sites using available geological maps and interpretation from aerial photographs. The slope stability at the dam sites, in the reservoir areas and above the tunnel entrances were studied. The findings from the desk study indicated the first 2-3 km of the tunnel to be in granite gneiss and the rest of the tunnel will be in intercalation of schist and granite with granite being the dominant rock in the last 4 km.

The result, a runoff river hydropower scheme utilizing a gross head of 245m was planned. An underground power house with 5 numbers of Francis turbines to generate maximum power output of 650 MW was proposed. The total catchment area considered was 6199 KM2 and the FRL and TWL were kept at El.808m and El 563m respectively. The total length of the dam at crest was envisaged to be 165m long.

3.1.2 Revised prefeasibility report (2003-2004)

After the initial prefeasibility study was conducted in 1992, no further study was continued on this project. Only in 2004, NORCONSULT AS has submitted the power system master plan along with the revised prefeasibility report to royal government of Bhutan. A brief description of various project features of the revised report is discussed as below.

Dam

The proposed diversion dam is concrete gravity structure with 190m long at the crest and dam height of 42.5m above the river bed level and 70m above foundation. The estimated design flood considered was 10128 m^3/s. Four radial gates, 12 m wide and 19m high were provided.

Intake and Desilting Basin

Six intake structures consisting 3.6mX5.7m roller gates with sills at elevation 803.8m and trash racks with cleaning equipment were provided. Six desilting chambers 250m long and with cross sectional area 243m^2 were provided with gated flushing tunnels arrangement.

Head Race Tunnel (HRT)/Penstock

The head race tunnel with length of 11.5km and cross sectional area of 88m2 was designed to carry the design discharge of 437 m3/sec. Three surge shafts are provided at the end of HRT to take care of surge problems. Vertical pressure shafts with total length of 365m including the horizontal reach were provided.

Power House and Transformer Cavern

An underground powerhouse consisting 8 generating units with 5 numbers of Francis turbines with rated output of 124MW each were proposed. The design flow was fixed at 54.6 m3/s per unit and the maximum net head at 265m.The transformer cavern was placed parallel to the power house cavern to house 8X3 single phase 420kV transformers.

Tail Race Tunnel

Three parallel tunnels of 350m length with cross sectional area of 88m2 was designed to discharge the water back into the Punatsangchu river.
The most important features of the revised prefeasibility study report are given in Table 3.1. The details of the project salient features are given under Appendix A.

Table 3-1 Important features of revised prefeasibility study report (WAPCoS, 2011)

Sl. No.	Description	Parameter
1	Type of scheme	Run off scheme
2	Catchment area	7007 Km2
3	Design flood	10128 m3/sec
4	River bed elevation at dam site	788m
5	Gross head	267m
6	Design discharge	437 m3/sec
7	Installed capacity	992 MW
8	Type of dam	Concrete gravity
9	Height of the dam	70m above foundation level
10	Crest length of dam	190m
11	Length of head race tunnel	11500m
12	Numbers of HRT	3
13	Cross sectional area of HRT	88m2
14	Length of pressure shaft.	365m, 3.5m Φ
15	Power house type	underground
16	Size of power house and machine hall	130,000.00m3
17	Tail race tunnel	2 nos. 350m long.

3.2 Feasibility study report

This chapter deals with review of the prefeasibility study report and the details of the feasibility study report.

3.2.1 Review of prefeasibility report

The Water and Power Consultancy Service (WAPCOS), India was awarded the work of feasibility study by royal government of Bhutan in 2007. The consultant reviewed the prefeasibility report after conducting site visits along the proposed project areas to study the geological features.

After the initial review of the prefeasibility study report along with the actual geological conditions at sites and possible changing circumstances to be met while implementing the project, following observations were summarized.

- ❖ Norconsult International AS project layout has been finalized mainly based on desk studies and reconnaissance survey without conducting field investigation.

- ❖ Norconsult international AS has considered FRL at EL 830m. However, the tail water level of Punatsangchhu-I HE project which is on the upstream of this project have been revised and fixed at EL 843m. This leaves an unutilized head of 13m.

- ❖ Presence of thick soil overburden and poor geological conditions along the abutments of proposed dam location were also envisaged based on observations of exposed rocks along the road cut slope.

- ❖ The occurrence of thick debris for a considerable distance on the proposed dam axis is expected to result in longer dam length.

- ❖ Presence of multiple shear zones in the underground power house complex was also envisaged from the rock outcrop observed rocks along the road.

In absence of any geological investigation results, there remains high degree of uncertainties. To get better knowledge on geology to help take better decision in reliable design to harness optimum output from the projects, further detailed engineering geological investigations were recommended to be carried out during feasibility study.

3.2.2 Feasibility study

After finalizing the need for the review of the project study, more detailed site investigation to explore all possible alternate options for different project components were done. However the main project sites have not changed much from the earlier studies. The overall view of the project location is given in the figure 3.1 and following were the conclusion of feasibility study.

Figure 3.1 Map showing the location of project components (Google earth, 2/2012)

Dam axis proposed in PFR

The dam axis is located 2km downstream of tail water outlet of Punatsangchu I project (figure 3.2). The geological features of the sites are as discussed below.

Left bank: - Talus/scree is present from the river edge to about 30m-35m above it. Beyond this gneiss is exposed up to the top of hill. Rock mass is expected at a shallow depth below the tallus/scree on this bank. The river bed is covered with fluvial deposits. Gneissic rock outcrop is seen at 150m-200m upstream of this location on the left bank.

Right bank:- There is 50m-60m wide slide debris from the river edge up to nearly 35m-40m above the river bed level. Thereafter, towards the abutment, partly weathered gneissic rock having near vertical escarpment is present.

The occurrence of thick debris for a considerable distance on the right bank will result a longer dam axis. Besides, the outfall level of the tail water from the Punatsangchu –I project is revised to El. 845m which could result to the loss of available head.

Alternate Dam axis-I

This dam axis is located 50m upstream of PFR site, at Lat. 27^0 18’ 58.7”, Long. 89^0 56’ 50”. It is located upstream of a small Brooke confluence with main river (Refer fig. 3.2). Here, the river flows is S40^0E and the direction of dam axis will be N55^0E.

Left bank: - The geological condition on the left bank shall be almost similar to that of the PFR site.

Right bank:- The rock line has shifted nearer to the right bank. A 30m wide debris zone is present between the right river edge and the rocky scarp (partly weathered biotite gneiss) on the hill side.

The rock bottom line is seen at 20 to 25m above the river edge. The intake structure is located almost at same site of site in PFR dam site. The initial intake structure may pass through debris where cut and cover sections is required.

Here the rock mass is partially weathered and traversed by four sets of joints/fractures. The details of the foliation and cross jointing are given in table 3.2. The upstream dipping foliation of the rock with dam axis is considered favorable condition for the stability of the dam.

Table 3-2 Foliation and cross jointing details of alternate dam axis I

Type of jointing	Strike direction	Dip direction	Remark
Foliation joint Jf_1	$N15^0E$	25^0-40^0 SE	Foliation joint
Foliation joint Jf_2	$N120^0$-140^0E	30^0-50^0 SW	Swing of foliation
Cross jointing Jz_1	$N20^0$ - 60^0E	20^0 – 25^0 SE	Shear planes associated with gouge (5 to 20m thick) and slicken sliding are commonly observed.
Cross jointing Jz_2	$N110^{0-}130^0$ E	20^0 – 25^0 NE	Main scarp on the right bank.

Alternate Dam Axis –II

This dam Axis is located 400m downstream of the PFR dam site, at Lat. 27^0 18’ 44.9”, and long. 89^0 57’ 13.8”, (refer Fig.3.2). Here, the river flows towards $S50^0E$ and the direction of dam axis shall be at $N40^0E$.

<u>Right bank:-</u> Partially weathered gneiss is exposed 25m away from the river edge, 10m above the road bench (≈20-25m above the river edge). The stretch between the river and rocky escarpment is covered with debris. The rocky scarp extends to a height of 60-70m from the river edge.

<u>Left bank</u>: Talus/scree is present 40-50m above the river level and thereafter partially weathered gneissic rock is exposed. The rock is expected at a shallow depth below the talus/scree.

The dip and strike direction of the gneissosity/foliation and the prominent cross joints are given in Table. 3.3 below.

Table 3-3 Foliation and cross jointing details for alternate dam axis II

Sl. No.	Strike	Dip, dip direction	Remarks
Foliation joint Jf_1	$N120^0$-140^0E	30^0-50^0SW	Foliation joints
Shear joints Jz_1	$N50^0$-60^0E	20^0- 25^0SE	Shear planes associated with gouge (5 to 20m thick) and slicken sliding are commonly observed
Cross joints Jz_2	$N115^{0-}150^0E$	60^0 - 80^0 NW	

Beside the above joints, minor shears trending parallel to foliations have also dissected the rock mass.

Dam Axis –III

This dam axis is located at 60m - 70m downstream of dam axis-II, at Lat. 27^0 18' 42.2" and Long.89^0 57' 17.8 (refer fig. 3.2). Here, the river flows at S60^0E and the direction of dam axis shall be at N30^0E. The geological features are same to that of axis-II.

Right bank:- Partially weathered gneiss rock occurs at distance of 15m-20m from the river edge to the road level and upslope. The rock is closest to river edge compared to other sites. This avoids need for cut and cover sections for the intake tunnels.

Due to the closeness of rock line to the river edge, this dam axis is geotechnically favored over other locations. However final selection of the dam axis should be made after considering intake level and other geotechnical results of field core drilling investigation results.

Table 3-4 Details of foliation and cross jointing details at alternate dam axis III

Sl. No.	Strike	Dip, dip direction	Remarks
Foliation joint Jf_1	N15^0E	75^0SE	Foliation
Foliation joint Jf_2	N120^0-140^0E	30^0-50^0 SW	Foliation swing
Cross joints Jz_1	N20^0-60^0E	20^0-25^0SE	5cm - 20cm wide shears marked with slicken slides at places.
Cross joints Jz_2	N110^0-130^0E	20^0- 25^0NE	Main scarp on the right bank.

Beside the above joints, a number of minor shears tending parallel to foliation also dissects the rock mass.

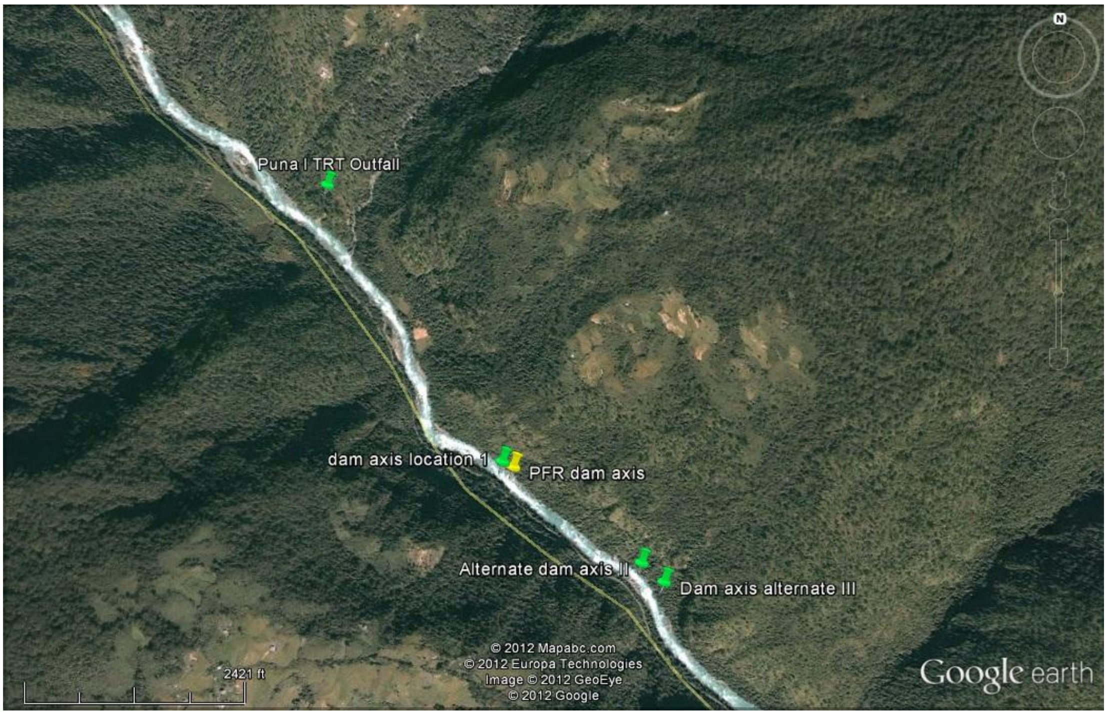


Figure 3.2 Location of different dam axis (Google earth, 2/2012)

Power House

An underground powerhouse is recommended on the right bank of Punatsangchhu, at about 1km downstream of the confluence of Di Chu River. Here mainly leuco-granite with small-undigested bands of schist is seen exposed from road bench to the river edge, and talus/debris present above the road bench. The proposed powerhouse cavern is located inside this spur. The gouge thickness in the shear zones ranges from few centimeters to 2m. The steeply dipping joint sets Jz_1 & Jz_2 may not be present at the powerhouse cavern. The shear joint Jz_3 is likely to be encountered at the powerhouse level. At the TRT outfall, shear zones Jz_1 & Jz_2 are also present. The attitudes of the shear zones are presented in table 3.5 along with other major jointing. For location refer fig.3.3

Table 3-5 Details of major joint sets at power house location

Sl. No.	Strike	Dip, dip direction	Remarks
Foliation Jf_1	N120^0-130^{0}E	30^{0}SW	foliation
Shear zones Js_1	N70^0-80^{0}E	70^0-80^0 SE	Shear zones
Cross joints Jz_1	N70^{0}E	80^{0}NW	Conjugate/often sheared
Cross joints Jz_2	N120^0-160^{0}E	30^{0}SW	Conjugate/often, shear/thrust

There is high likelihood of encountering the shear zones at the power house location. The power house cannot be shifted downstream due to worsening geological conditions. So this site is not good for power house location.

Surface power house

A surface powerhouse site was tentatively selected just upstream of the proposed TRT outfall in between two nala depressions. Here steeply dipping shear zones are present in close vicinity of

TRT outfall. The rock is exposed from river edge till road bench. A bench in rock has to be excavated for the powerhouse with adequate slope stabilization. Number of shear zones mentioned above is likely to intersect the surge shaft and penstock tunnels.

Underground power house

Alternatively, an underground powerhouse may be considered below the centre line of the spur of the ridge opposite to the confluence of Di Chu River (refer fig. 3.3). Reconnaissance traverses along the road cutting indicate no major shear zones likely to encounter at power house location. TRT outfall may be considered at same location of PFR study location to avoid head loss. This will result into longer length of TRT passing through shear zones described in table 3.5. However, detailed geological mapping on larger scale is required to confirm the detailed projections of shear zones to power house, surge shaft and TRT area.

Figure 3.3 Locations of alternate power house sites (Google earth, 2/2012)

Head Race Tunnel (HRT)

The HRT alignment starts in Gneiss rock belonging to Thimphu formation. In the later reach, tunnel alignment will be dominated by rocks of Schistose and leuco-granite belonging to Chukha formations. About 2.5 km length of HRT pass through gneiss rock of the Thimpu formation and rest will pass through schist rock intruded by leuco-granite of Chukka Formation. The HRT alignment will intersect the foliation at angle varying from 10^0 to 45^0 considering the foliation trend from N 15^0 E to N 120^0E of gneiss rocks belonging to Thimpu formation. The shear/fault/thrust encountered at the powerhouse complex may intersect the HRT. Detailed geotechnical investigation is required to delineate the tunneling condition along the proposed HRT alignment.

3.2.3 Conclusion on feasibility report

The results of the feasibility report for different components of the project are summarized hereunder.

Dam Site

After a through comparison of all the possible alternate sites, the alternate dam axis-III (ref. 3.2.1.4) is found suitable and recommended for detailed geotechnical investigation.

Powerhouse site

The initial site proposed at PFR study level is intruded by multiple shear and weakness zones. This site is not feasible for power house location. The other two power house options may be considered for further field investigation.

Head Race Tunnel

The traversing along the road cutting indicates that initial 2.5 km of the HRT will go through gneiss rock of Thimphu formation. The remaining reach will pass through schist and leuco-granite of Chukha formation with leucogranite occurring as intrusive in behavior. The slide debris covering this formation made it difficult to delineate the rock boundary between the two rock types. Considering the incidences and attitudes of the shear/fault/thrust zones near the powerhouse complex, it is apprehended it will intersect the HRT and require confirmation by detail study.

3.3 Detailed Design Phase

The possible different alternative sites for all the different components of the project were reviewed in the feasibility study. The final sites were selected considering their advantage over the other alternate sites. Accordingly, the required geotechnical field investigations were carried out at the respective sites. The main field and laboratory studies includes following.

- Geological mapping of all project sites.
- Core drilling including permeability test for various project sites.
- Drifting at the dam site and
- Other geotechnical laboratory tests.

The different types of investigation carried at different sites are given in table 3.6. The detail of findings from different field and laboratory studies is discussed in the following sections.

Dam Site:-

After examining all possible alternate sites, the alternate site located at 825m d/s of PFR dam axis is chosen. To study the physical geological condition of rocks along the proposed de-silting chamber, an exploratory drift was excavated during the preconstruction stage. In addition, some rock mechanics tests were also conducted. The results from the test are discussed in following chapters.

Figure 3.4 Location of final dam axis (Google earth 3/2012)

Table 3-6 List of geological and geotechnical investigation done at different sites

Project Component	Field Investigation
Dam Site	Geological Mapping Core drilling 15 nos. with permeability tests. Laboratory Tests Drifting on both abutments
Intake	Geological Mapping Core drilling 1 nos. with permeability test Laboratory tests
Head Race Tunnel (HRT)	Geological mapping
Adits	Geological mapping
Pressure Shaft	Geological mapping Core drilling 2 nos.
Surge Shaft	Geological mapping Core drilling 1 no. Laboratory Test
Power House Complex	Geological mapping Core drilling 5 nos. Three for surface and 2 nos. for underground power house. Laboratory Test
Tail Race Channel	Geological mapping

Power House:-

After examining all alternatives, the surface power house option was found preferable. Here, Leucogranite with gneiss and schist enclaves was exposed along the proposed pressure shaft area. These rocks were traversed by two major joints sets trending NW-SE, dipping steeply NW direction. Besides these, some shallow dipping random joints sets are also presents. Other details are discussed in following sections

3.3.1 Head works

Geological mapping of the final Dam axis, located 825m d /s of the Prefeasibility dam site, was carried out on 1:2000 scale covering 500m d/s and 700m u/s of proposed site and up to elevation El. 900m on either side. The river flows at N140^{o}E and the 0rientation of the dam axis is finalized at N38^{o}E. The riverbed level at the proposed dam site is El. 788m.

The Left Bank shows a gentle slope of 30^o-35^o. This bank is covered with thick debris up to El. 880m, above it hard and moderately weathered quartzo felspathic gneiss and Biotite gneiss are exposed. The rock foliation near El. 881 shows wide variation from N-S/40^{o}SE to N70^{o}E /15^o SE. In the river bed, fluvial deposits consisting of pebbles, cobbles, boulders and very big rock blocks (>10m) are present. The width of the river Channel at the proposed dam axis is around 70m.

The right bank is sloping near vertical up to elevation ± 830m and the slope gets gentle at 35^o-40^o up to elevation 910m. This bank forms rocky escarpment. Quartzo felspathic gneiss/Quartz biotite gneiss is found exposed mainly along the road cut level. Small patch of gneiss rock is also exposed below the road level downstream of dam axis. Here the foliation in gneiss varies from N100^{o}E/10^{o}SW to N50^{o}E/30^o-35^{o}SE.This swing of foliation is due to warping of the rock. However the general trend of the foliation is N60^o-70^{o}E/30^{o}South east.

The rocks are traversed by six joints sets. A few thin shear zones mainly foliation shears varying in thickness from 5cm to 20cm with 2cm to 3cm thick gouge infill have also traversed the rocks. here biotite gneiss/quartzo-feldspathic Gneiss form the foundation rock, so no major problem is anticipated except the possibility of conspicuous shear zone running parallel to the river channel at the left bank where seepage through the dam foundation may be anticipated.

Drilling

Sixteen boreholes were drilled at the dam complex to establish the rock profile, rock mass condition and porosity of rock for dam foundation, along the intake and spillway sites. The summarized logs of these boreholes are given in table 3.11. From core logging results, at about 20m along the dam axis on the left bank, the depth to bedrock has suddenly increased to 42m i.e. R.L.763m (DH-5) which is 5-6m below the rock level of DH-3. This could be indication of a buried channel formed due to either a master joint/shear zone along the left bank. The very poor core recovery in the drill holes may indicate the presence of possible shear zones. The exact geometry of the shear zones could only be established during the excavation of the foundation. Suitable treatment should be provided to prevent the leakage/seepage of water and improve stability of the dam.

Figure 3.5 Gneiss rock outcrop at Dam site

Water Percolation Test

Water percolation tests were carried out in drilled holes. The permeability value observed in general was very high. This suggests the need for foundation treatment by grouting to minimize seepage. Groutability tests needs to be conducted to ascertain the rock mass groutability.

Drift

The drifting work was carried out on both banks of dam axis to demarcate the depth of stripping and to get better information on the rock mass. From the result of geological logging of drifting works, the Q value of the rock mass varies from 0.6 (Class –V) to 4.7 (Class-III) with average range in Class-IV.

Intake

All four intakes were located within 125m upstream of dam axis on the right bank. Here the rock is exposed at the hill side road edge near the axis but gradually the rock line moves away towards the hill in the u/s of the dam axis. At 125m upstream, the exposed rock was seen at EL. 822m i.e. at about 45 m away from the river edge. There are debris present between the rock line and the river edge. Due to the rock line moving towards the hill side in the u/s, the first intake portal will be in rock while other three intake portals need cut and cover sections for shorter distance. On examination of the surface exposure and the log results of drill hole (DH-21) it is interpreted as fairly good quartzo-feldspathic gneiss in the area.

Under Ground Desilting Chambers

Four underground Desilting Chambers with dimension 17/19mX24.5mX420m is designed on the right bank. Rocky outcrops of biotite gneiss, quartzose gneiss belonging to Thimphu Shumar Formation occasionally intruded by pegmatite were seen in this area. The foliation varies from N140^{o}E to N80^{o}E dipping 5^{o}-30^{o} towards east. The general foliation is N60^{o}-80^{o}E/ 30^{o}SE. Besides this, six set of prominent joints are also recorded in the area. However, the outcrop pattern in the area is seen to be controlled mainly by 2 master joints trending N65^{o}E/50^{o}-60^{o}NE and N65^{o}E/80^{o}

SE. A tentative desilting chamber orientation along L-axis at N130°E was provided. The joint N110°E/80°NE is seen as a conspicuous joint which is often clay filled. If this joint set is present in the crown of desilting caverns, the desilting chamber orientation may have to be slightly modified.

Diversion Tunnels

The four number of Intakes and Underground Desilting Chambers were located on the right bank, as such the Diversion Tunnel was proposed on the left bank to avoid interference with these structures located at different levels on the right bank.

Reservoir Competency

The reservoir is located in a very narrow gorge. As such, the lateral spread of the reservoir is less. In general the slope of both banks is very steep. High hills and uprising slopes are present on both sides of the reservoir for considerable distance. No conspicuous saddle is seen within the limit of the reservoir rim so there are little chances of any leakage from the reservoir to adjacent river basin through joints and other lineaments.

Colluviums deposit / slope wash material was present at few places on the banks of the river, especially along the right bank. Slope stability problem is not anticipated in the terrace on the left bank. The possible occurrences of landslides within the saturated colluviums/ slope wash debris during drawdown condition cannot be completely ruled out.

3.3.2 Head Race Tunnel

The head race tunnel (HRT) is 8.5km long and 11.00m diameter with slope angle of 1:309 designed on the right bank of Punatsangchhu river to carry water from the reservoir to the proposed power house for generating power. To ensure adequate lateral and vertical rock cover while crossing the conspicuous stream channels, fournumber of kinks were provided at Ch 1735m, Ch 4821.5m, Ch 6113m and Ch. 8426.9m (refer fig.4.5). One construction adits each at intake and near surge shaft were provided. In addition, three more adits located at Rd 1053.3m, RD 4740.1m and Rd 8482m were provided to facilitate effective construction. The Invert Level of the HRT at the intake is kept at El. 803.50m while at the Surge Shaft location the Invert Level is El.764.5m.

Geological map along the proposed tunnel alignment was prepared on scale 1: 5000. The major part of the tunnel alignment is covered in debris and soil. It is established from the traverses survey that the minimum and maximum vertical rock cover varies between 80m and 600m. The rock cover of about 80.00m is available at the intersection zones of the HRT with Petshochhu at Ch. 1765m, and about 128.00m at Bisokha at Ch. 4176m. The average rock cover along the major section of the HRT is above 300m. From the topography map it is seen that sufficient lateral rock cover will also be available throughout the entire stretch of the proposed tunnel alignment.

The geology, rock mass quality and tunneling condition along the proposed tunnel alignment are discussed in the following three Sections.

3.3.2.1 Dam axis (0.0m) to Adit-I (Ch. 1053.3m)

The initial 980m tunnel stretch will pass through fresh to partially weathered qurartzo-feldspathic gneiss. The rocks are traversed by two sets of conspicuous joints. From Ch.980m to Adit-I the rock is represented by fresh grey biotite gneiss with increasing amount of quartzo-feldspathic materials. Leucogranite are found exposed at some places along road cutting. The Leucogranite appears to be intrusive into the Biotite gneiss and Quartzo-feldspathic gneiss. The general gneissosity of the rocks and the joint sets traversing the rock mass is presented in table 3.7. Five numbers of Shear Zones were recorded in this stretch.

Table 3-7 Foliation details of the HRT between Ch 0m to Adit I

Type of Jointing	Strike direction	Dip direction	Remark
Foliation joint Jf_1	$N45^0$-80^0E	20^0-30^0 SE	Grey biotite gneiss and quartzo feldspathic
Foliation joint Jf_2	$N130^0$-140^0E	20^0-30^0SE	gneiss.
Cross Joints Jz_1	$N0^0$-5^0E	Sub vertical	Conspicuous joints.
Cross joints Jz_2	EW	Sub vertical	

The tunnel is oriented slightly askew to the gneissosity of the rock, so fair condition of tunneling is expected along it. The low dipping of the foliation and the cross joints will result into frequent over breakages from the crown of the tunnel. Wedge formation on the crown of the tunnel due to intersection of joint sets is also anticipated.The tentative RMR and Q values of the above rocks are as given in table 3.10

3.3.2.2 Adit -I (Ch. 1053.3) to Adit-II (Ch. 4740.1m)

This 3686.81m stretch of the Tunnel alignment will pass through alternate bands of fresh to partially weathered biotite gneiss/ quartzo-feldspathic gneiss and leucogranite (predominant), the latter appears to be intrusive into the former. The most common foliation and cross joint orientations are given in table. 3.8

Table 3-8 Foliation details of the rock mass of HRT between Adit I and Adit II

Type of Jointing	Strike direction	Dip direction	Remark
Foliation joints Jf_1	$N125^0$-150^0E	20^0-50^0SE	Weathered biotite gneiss and quartzo feldspathic gneiss with leucogranite intrusion.
Shear joint Js	$N65^0E$	65^0SE	150cm thick shear with 160m shattered width. At Ch.3839m

The major part of tunnel alignment passes through high debris making it difficult to establish the actual tunneling condition. More shear zones covered under the debris cannot be ruled out. The tunnel alignment is oriented askew to the foliation indicating a fair to favorable condition of tunneling. The tunnel alignment crosses the two nalas i.e Petsochu and Bickhachu at Ch. 1762m and Ch. 4176m respectively in this reach. Due to presence of fractured shear zone, and the nala crossing, occurrence of heavy seepage into the tunnel cannot be ruled out. A hidden shear zones under thick debris is suspecteded which may pose a great threat to tunneling especially when charged with seepage water. The contacts between biotite gneiss/quartzo feldspathic gneiss and the intrusive leucogranite may represent zones of weakness or may be sheared as observed along the road cut sections. The rock mass parameters and rock class are given in table 3.10.

3.3.2.3 Adit-II (Ch.4740.1m) to Adit-III (Ch.8482m)

The tunnel alignment is covered with debris in most of this length. The rocks are represented by well foliated partially weathered to fresh biotite gneiss with numbers of both concordant and discordant bands of intrusive leucogranite. The incidences of leucogranite are likely to be more towards the adit-III. The general foliation and joint orientations are given in table 3.9. Besides the main joint, few random sets of joints were also recorded in the leucogranite.

Table 3-9 Foliation and cross joint details for rock mass between Adit II and Adit III

Type of Jointing	Strike direction	Dip direction	Remark
Foliation joints Jf	$N125^0$-145^0E	30^0-60^0SE	Biotite gneiss/leucogranite foliation direction.
Cross joints Jz_1	$N\ 0^0$-15^0E	Sub vertical SE	Conspicuous joins
Cross joint Jz_2	$N50^0$-$85^{00}E$	Sub vertical SE	Conspicuous joins
Shear zone JS_1	$N75^0$-80^0E	45^0-50^0NW	Shear zone

One set of shear zone may cross the tunnel alignment at Ch.4989m and Ch.8359m along with a number of shear zones which may be present under the debris covered area.

As the tunnel alignment in this stretch also runs askew to the foliation trend, it is interpreted as fair condition of tunneling. One sub vertical $N105^oE$ trending joints in the leucogranite running almost parallel with the tunnel alignment may give rise to over breakages problems at tunnel crossing. The wedges are likely to be formed by the intersection of the above three conspicuous sets of joints in the biotite gneiss along with some over breaks from tunnel crown. In view of massive nature of leucogranite and very high rock cover, the possibility of rock bursting in this stretch particularly between Ch.5319m and Ch.6339m is anticipated. The rock mass parameters and rock class is given in table 3.10

3.3.2.4 Adit-III (Ch.8482m) To Surge Shaft (Ch.8584.3m)

This last 102m stretch of the tunnel will pass through predominantly fresh to partially weathered biotite gneiss and few inter beds of quartzite with numbers of discordant and concordant bands of leucogranite. As the major part of this stretch of the tunnel is covered with debris, the geological mapping was done by traversing from the nearby nala and the surface power house location. The projected rock mass parameters and rock class is given in table 3.10.

At least five numbers of shear zones will be intersecting the HRT at different locations. If the shear zones and crushed rock zones crosses the charged water bodies, possibility of heavy water seepage is expected requiring advance draining, fore poling and grouting.

Table 3-10 Rock mass classification along the HRT alignment (WAPCoS, 2011)

Location	Rock type	Q-Value	RMR Value	Rock class
Dam axis (Ch. 0.0m) to Adit-I Ch.1053.3m)	Bioitie gneiss	6.01	50-65	Class III
	Quartzo feldspathic gneiss	0.5-3.76	54-77	Class-III to Class V
	Leucogranite intrusion	1.3	58	Class IV
Adit I (Ch.1053.3m) to Adit II(Ch. 4740.1m)	Biotite gneiss	1.39 -3.47	58-67	Class III to Class IV
	Leucogranite intrusion.	0.65-5.21	49-67	Class III To Class V
Adit II(Ch.4740.1m) to Adit III(8482m)	Biotite gneiss	0.80-3.77	43-69	Class III To Class V
	Leucogranite intrusion.	1.15-5.9	57-70	Class IV to Class III
Adit III (Ch.8482m) to Surge Shaft (Ch.8584.3m)	Biotite gneiss	0.32 –3.27	50-63	Class III to Class V
	Leucogranite intrusion.	1.31- 5.47	33-70	Class III to Class IV

3.3.3 Geotechnical evaluation of adit portals

To facilitate the construction of various project components, many construction adits were provided. The geology along these construction adits are discussed in this section.

Portal for Adit to HRT Intake

The length of this adit would be around 374.60m aligned N68^{o}E direction. At the proposed portal site, partially weathered to fresh quartzo-feldspathic gneiss is traversed by two conspicuous sets of joints. The geotechnical appraisal revealed no major problem likely to be faced in and around this portal.

Portal for adit to Gate Chamber

The adit is aligned in N40^{o}E direction. At the adit portal good rock face of partially weathered quarzo-feldspathic gneiss traversed by two conspicuous sets of joints is projected. No major problem would be encountered for construction of this portal and driving of the adit.

Portal of Adit -1 to HRT

Partially weathered to fresh biotite gneiss sub-horizontal in thin bands of leucogranite is exposed at the proposed adit portal site. The rock is traversed by 3 conspicuous joint sets besides foliation joint. Intersection of these joints may give rise to formation of wedges in the crown. However it is tentatively established that the adit portal site is located on a stable rock face.

Portal of Adit -2 to HRT

This adit portal is located on rock face comprising partially weathered to fresh biotite gneiss with thin concordant/discordant bands of leucogranite with two joint sets. The geotechnical evaluation of the site condition reveals less likelihood of any major problems at this site.

Portal of Adit-3 to HRT

At the proposed portal site partially weathered to fresh leucogranite with patches of boitite gneiss is exposed. The rock is traversed by two conspicuous sets of joints besides the foliation joint. No major problem for construction of the adit portal is anticipated.

3.3.4 Power house Complex

The possible option of one underground power house and two surface power house sites were selected through geotechnical mapping. The rocks in this area are represented by biotite gneiss/ quartzo-feldspathic gneiss/ thin bands of quartzite with both concordant and discordant bodies of intrusive leucogranite, the latter predominating over the former.

Underground Power House

From result of reconnaissance and geological mapping survey, a possible underground power house site is located in between two nalas due east of Surge Shaft location. The length axis of the power house cavern is tentatively placed in N-S direction based on available joint and shear data. The rock exposed at this site is mainly leucocratic granite with enclaves of schist, gneiss and quartzite of Chukha formation. No foliation is seen in the exposed leucocratic granite and the foliation in the enclaves of the Chukha formation is highly disturbed. The rocks are traversed by a numerous joints sets and affected by number of prominent shear zones.

Figure 3.6 Surge Shaft location

Surface Power House

Based on geological mapping, two alternative sites were selected for the proposed surface power house with same surge shaft location. The first site is located on a terrace at the right bank of Punatsangchhu river immediately downstream of Dichu confluence and the other at the TRT outfall of the underground power house site. The TRT outfall site is located in a depression where rock is exposed under shallow overburden cover. However a nala flowing through the middle part of this site has to be diverted to have sufficient space for the proposed surface power house. In this case a number of major and minor shears will intersect the penstock /pressure shaft tunnels for a considerable length. It is not desirable to drive the pressure tunnels through this structurally disturbed zone, hence this site was not considered suitable.

It may be mentioned here that, the location of the power house was however changed during the last phase of tendering stage. The final power house was shifted upstream to perhaps avoid the many shear zones crossing power house cavern. The final location is given in fig. 3.3

Analysis of rock samples

The rock samples were collected from the core drilled holes both from dam complex area and the power house complex. The important tests include uniaxial compressive strength, Youngs modulus of elasticity values and poissons values. The results of the laboratory test are presented in table 3.11 both for dam site and power house complex.

Table 3-11 Laboratory test results for core drilling at different project sites

Location	B H No. & location.	Depth of hole (m)	Υ_{dry} (KN/m^3)	Υ_{sat} (KN/m^3)	Porosity (%)	Sp. Gr.	Point load test: Axial (MPa)	Point load test: Diametric (MPa)	Brazilian test (MPa)	UCS MPa	E-Modulus MPa	poisson ratio υ
Rock type Quartz gneiss												
Values at the Dam site	DH-03. L/B	93	27.2	27.3	0.25	2.70	2.98	3.15	8.42	50.09	6696.8	0.32
	DH-08, R/B	45	25.60	25.70	0.37	2.67	4.87	2.26	7.82	72.14	12534.6	0.36
	DH-21. R/B	45	25.40	25.40	0.35	2.65	4.75	3.17	9.38	40.55	5843.3	0.34
	DH-06. R/B 150m D/S	40	25.4	25.5	0.34	2.64	4.96	2.62	8.30	52.53	6412.30	0.26
Average Values	All holes	55.75	25.90	25.97	0.33	2.67	4.39	2.80	8.48	53.83	7871.75	0.32
Rock Type Leucogranite												
Values at dam site	DH07-03, L/B	40	26.2	26.3	0.37	2.65	3.81	3.12	7.95	42.83	3435.5	0.31
	DH-01. R/B	95	25.70	25.80	0.40	2.65	4.84	2.81	7.46	78.73	8532.8	0.40
Values at power house site	DH-19. Pressure shaft	60	25.20	25.30	0.46	2.67	4.49	3.41	6.44	43.77	5645.6	0.24
	DH-17. Surge shaft	84	27.30	27.40	0.42	2.65	1.63	3.30	5.71	51.79	3878.90	0.21
	DH-15. Power house	90	24.60	24.70	0.32	2.69	3.27	2.53	7.78	25.20	2805.6	0.20
Average values	All holes	73.80	25.80	25.82	0.39	2.66	3.60	3.0	7.10	48.46	4859.68	.27

4 REVIEW OF THE HEAD RACE TUNNEL ALIGNMENT

The water from the head works will be conveyed to the power house through 8.5km long, 11m diameter modified horse shoe type head race tunnel. The whole length of the head race tunnel will be concrete lined with thickness varying from 500mm to 675mm depending on the type of rock mass. Further, based on rock mass competency, steel bar reinforcement and structural steel supports were designed in the areas of poor and extremely poor geological reaches respectively. This chapter deals with discussion on geology along the HRT alignment, possible stress related and other problems along the HRT alignment and ends by proposing an alternate alignment to the existing one.

4.1 Geological features along the HRT alignment

The proposed head race tunnel has a total length of 8584.28m long with an excavation diameter varying from 12m in very good rock to 12.35m in poor and extremely poor rock mass. The tunnel has finished diameter of 11m.

The rock type in initial reaches of the tunnel through is partially weathered quartzo feldspathic gneiss and fresh biotite gneiss. The rock class slowly changes to biotite gneiss with bands of intrusive leucogranite into biotite gneiss in the middle reach. In the last reaches, leucogranite rock is dominat. The rock mass classification was done using both Q and RMR system. The Bartons Q value of the rock mass along the tunnel ranges from lowest of Q=0.32 to maximum of Q=6. The summarized values of the rock mass parameters are given in table 4.4. A brief geotechnical aspect of HRT along different chainage is discussed hereunder.

4.1.1 Dam axis (0.0m) to Adit-I (Ch. 1053.3m)

The tunnel passes through mixture of fresh to partially weathered qurartzo-feldspathic gneiss to biotite gneiss with intrusive leucogranite into biotite gneiss. The strike and dip details of the foliation and cross jointing are tabulated in table 4.1. The foliation and jointing are also presented in joint rosette along with the tunnel alignment orientation.

Table 4-1 Foliation and joint details of the rock mass for HRT between Ch. 0.0m to Adit I

Type of Jointing	Strike direction	Dip direction	Remark
Foliation joint Jf_1	$N45^0$-80^0E	20^0-30^0 SE	Grey biotite gneiss and quartzo feldspathic gneiss.
Foliation joint Jf_2	$N130^0$-140^0E	20^0-30^0SE	
Cross Joints Jz_1	$N0^0$-5^0E	Sub vertical	Conspicuous joints.
Cross joints Jz_2	EW	Sub vertical	

The dip direction of both the foliation and cross joints is along the same direction. This may be considered favorable. If the dip direction of the two were in opposite direction, it may result in rock falls from the crown and the walls of the tunnel.

The tunnel is aligned with the foliation jf1 at an angle of 84^0 and the tunnel is aligned at 23^0 with the main gneissosity. However, the tunnel alignment is going semi perpendicular with the cross joints.

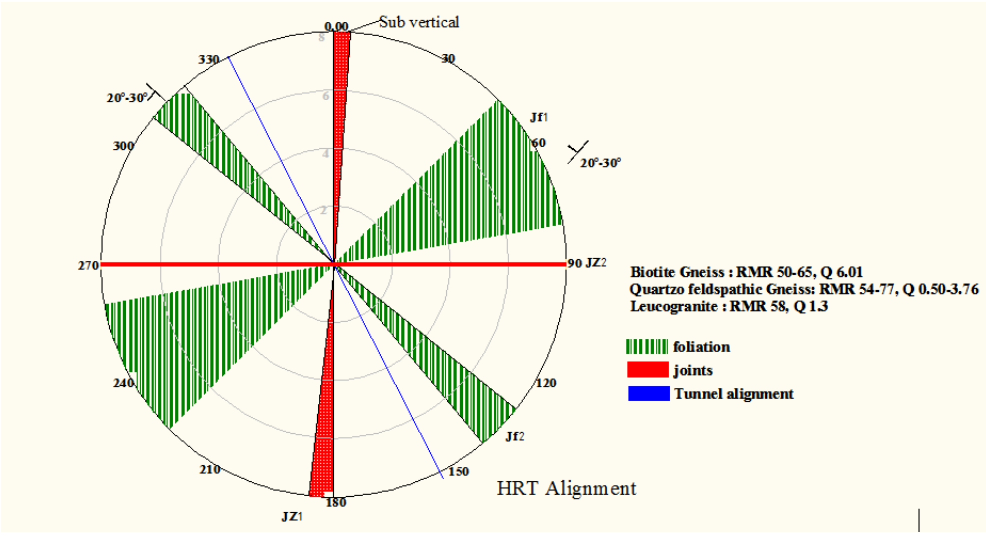


Figure 4.1 Joint Rosette for rock mass from Ch. 0.0m to Adit I

The vertical rock cover in this reach on average is 300m. The stretch is intersected by a sub vertical shear zone at Ch.110m-120m. Another shear zone intersects the tunnel alignment at Ch.750m. dipping 50^0-60^0SW. Bartons Q values are given in table.4.4

4.1.2 Adit-I (Ch. 1053.3m) to Adit-IICh. 4740.10m

This stretch of the Tunnel will probably pass through alternate bands of fresh to partially weathered Biotite Gneiss/ Quartzo-feldspathic gneiss and Leucogranite (predominant), leucogranite occurring as intrusive into former. The most common foliation and cross joint orientations are given in table. 4.2

Table 4-2 Foliation and joint details for rock mass between Adit I and Adit II

Type of Jointing	Strike direction	Dip direction	Remark
Foliation joints Jf_1	N125^0-150^0E	20^0-50^0SE	Weathered biotite gneiss and quartzo feldspathic gneiss with leucogranite intrusion.
Shear joint Js	N65^0E	65^0SE	150cm thick shear with 160m shattered width. At Ch.3839m

The foliation and shear joints are both dipping in same south east direction which is favorable from the risk of rock falls from the crown and the walls. But, the tunnel orientation is going askew with the main foliation direction. But, the tunnel orientation cannot be changed much since the stretch has to be aligned accounting the alignment of preceding and succeeding tunnel stretch.

The tunnel alignment is at 36^0 angle with the main foliation direction. The cross joint orientation is also favorable with the tunnel alignment.

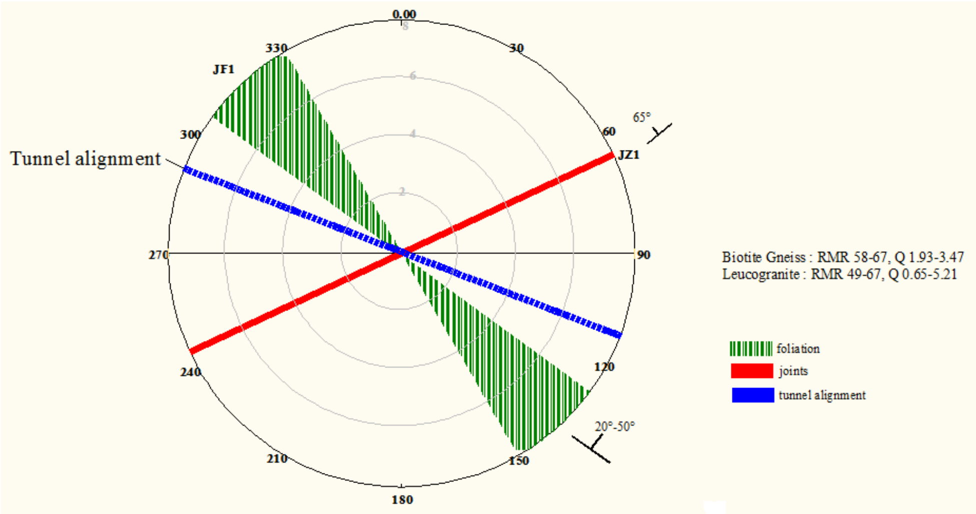


Figure 4.2 Joint Rosette for the rock mass between Adit I and Adit II

The vertical rock cover varies from 250m at adit junction to 80m at Ch.1795m where the first nala crosses the tunnel alignment. From Ch. 1795m to Ch. 4176m the vertical rock cover is highest with average of 900m and reducing to 124m at Ch. 4176 where second nala intercepts the tunnel alignment.

The tunnel alignment in this reach is intercepted by numerous shear zones. A sub vertical shear at Ch.1280 and Ch.1400m dipping 60^0-70^0 SE, another shear dipping 50^0-60^0 SE at Ch. 3950m and at Ch.4176m. last shear dipping sub vertical at the point of second nala crossing. The Q values are given in table 4.4

4.1.3 Adit-II (Ch.4740.10m) to Adit-III (Ch.8482)

The rocks are represented by well foliated partially weathered to fresh Biotite gneiss with numbers of both concordant and discordant bands of intrusive leucogranite. The Tunnel will pass through alternate bands of Biotite gneiss and Leucogranite of varying thickness. The incidences of leucogranite are likely to be more towards the Adit-III. The general foliation and joint orientations of foliation and cross joints are given in table 4.3 besides the main joint, few random joints sets were also recorded in the leucogranite.

Table 4-3 Foliation and joint details for rock mass between Adit II and Adit III

Type of Jointing	Strike direction	Dip direction	Remark
Foliation joints Jf	N125^0-145^0E	30^0-60^0SE	Biotite gneiss/leucogranite foliation direction.
Cross joints Jz_1	N 0^0-15^0E	Sub vertical SE	Conspicuous joins
Cross joint Jz_2	N50^0-85^{00}E	Sub vertical SE	Conspicuous joins
Shear zone JS_1	N75^0-80^0E	45^0-50^0NW	Shear zone

Here too, the tunnel alignment is not in preferred orientation with the foliation direction. But the alignment options are limited by the constraints of the topography and the location of the power house and surge shaft.

In this reach the tunnel is going semi parallel with the main foliation direction which may pose stability problems. However the cross joints are favorable with the tunnel alignment.

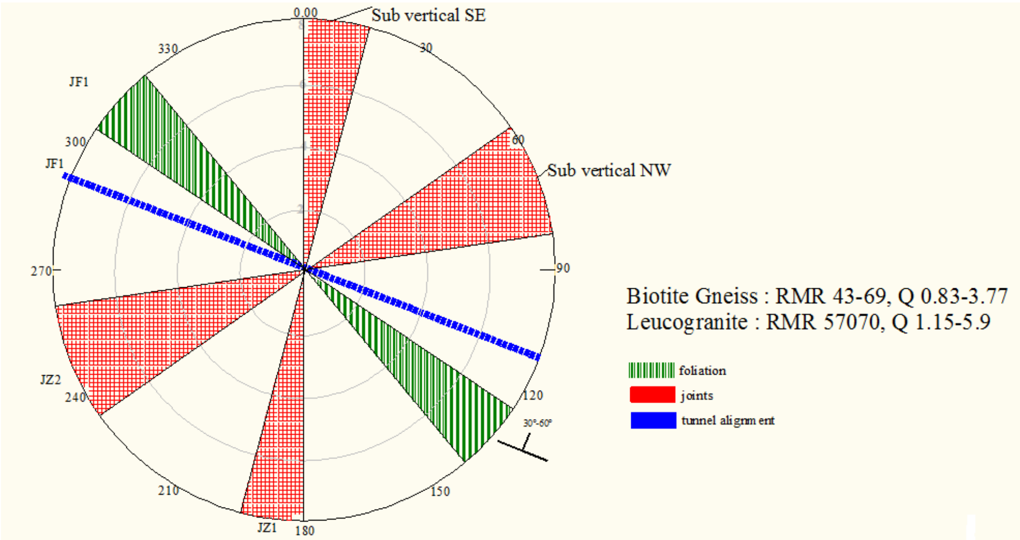


Figure 4.3 Joint Rosette for rock mass between Adit II and Adit III

Since the Tunnel alignment runs askew to the foliation trend, it is interpreted as fair condition of tunneling. The vertical rock cover varies from 300m at adit junction II to 120m at Ch.6000m. The overburden in the balance reach varies from 150m to 200m. One shear zone crosses the alignment at Ch. 5010m with dip angle of 60^0-70^0SW. rock mass parameters are given in table 4.4

4.1.4 Adit-III (Ch.8482m) To Surge Shaft (Ch.8584.3m)

This last 102m stretch of the tunnel would pass through predominantly fresh to partially weathered biotite gneiss with few interbeds of quartzite and numbers of discordant and concordant bands of leucogranite. As the major part of this stretch is covered with debris/soil, the geological mapping was done by traversing from the nearby nala and the power house location. The projected rock mass parameters and rock class for the whole HRT is given in table 4.4

Table 4-4 Rock mass parameters and rock class along the HRT alignment

Location	Rock type	Q-Value	RMR Value	Rock class
Ch. 0.0m to Adit-I Ch.1053.3m)	Bioitie gneiss	6.01	50-65	Class III
	Quartzo feldspathic gneiss	0.5-3.76	54-77	Class-III To Class V
	Leucogranite	1.3	58	Class IV
Adit I Ch.1053.3m) to Adit II (Ch.4740.1m)	Biotite gneiss	1.39 -3.47	58-67	Class III To Class IV
	Leucogranite	0.65-5.21	49-67	Class III To Class V
Adit II(Ch.4740.1m) to Adit III(8482.m)	Biotite gneiss	0.80-3.77	43-69	Class III To Class V
	Leucogranite	1.15-5.9	57-70	Class IV To Class III
Adit III (Ch.8482.m) to Surge Shaft (Ch.8584.3.m)	Biotite gneiss	0.32 –3.27	50-63	Class III to Class V
	Leucogranite	1.31- 5.47	33-70	Class III to Class IV

First Adit

The first adit junction with the HRT alignment is at Ch 1053.297m and the invert level at the junction is at El. 798.7m. The length of the adit is 807.598m and the adit portal is located at El. 772m. The gradient of the adit is 1 in 30.32 or 3.30%.

Second Adit

The second adit meets the HRT alignment at Ch. 4740.099m and the invert level of the adit junction HRT is at El.781.965m. The adit is 544.5m long and the portal is located at El. 765m. The gradient of the adit tunnel is 1 in 32.09 or 3.12%.

Third Adit

The third adit meets the HRT alignment at Ch. 8481.90m and is very close to the end of the HRT junction with surge shaft. The invert level of the adit junction is at El. 764.965m. The adit is 284.8m long taking off from the main adit to the surge shaft and Butter valve chamber.

4.2 Discussion on features of the head race tunnel alignment

The total length of the tunnel is 8.5km. It is divided into five segments. A bend each is provided at Ch.1768m, Ch.4871.6m, Ch.6135m and Ch.8460.1m respectively. The tunnel is finished with concrete lining and have internal area of 95m2 i.e. 11m internal diameter.

The rock cover along the tunnel alignment varies from minimum of 80m to maximum of $\approx$ 700m at certain reaches of the tunnel.

The construction of the HRT will be done through three construction adits. The chainages of adit locations are adit –I at Ch. 1053.3m, adit-II at Ch.4740.10m and adit-III at Ch. 8481.9m. The features of HRT along each stretch is discussed and reviewed in the following sections.

4.2.1 Head loss

The primary head loss in the tunnel is accounted from the frictional loss between the tunnel and water surface. This loss is function of surface roughness and length of the tunnel, velocity of flow and the size of cross sectional area of the tunnel. Besides, frictional loss, head losses can occur at bends and transition in cross sectional areas. 5 numbers of bends provided in the alignment will add to the head loss. The frictional head loss is calculated using Mannings formulae.

$$\Delta h_f = \frac{Q^2 * L}{M^2 * A^2 * R^{4/3}} \text{........................4(1)}$$

The Mannings M value of 60 is considered for concrete lined tunnel. The design discharge Q is 466 m^3/sec, length of tunnel L is 8584.3m, cross sectional area A of tunnel is 95 m2 and R is the hydraulic radius calculated from ratio of cross section area to perimeter of the tunnel. the total head loss calculated with above input values is 14.89m. This is not high considering the size and length of tunnel. However, there are rooms for reducing these losses within the same system. This could be done by reducing the length and numbers of bends. This is discussed in alternate alignment.

4.2.2 Hydraulic fracturing of the rock mass

In a pressurized tunnel, when the hydro static head in the tunnel is greater than the weight of the rock mass, the rock mass surrounding the tunnel may be subjected to hydraulic fracturing. The water pressure fractures the rock mass and escapes into the surrounding. This causes stability problems and loss of power generation. To avoid this, adequate vertical and lateral rock cover has to be provided. The equilibrium condition is given by

$\gamma_w * H \leq \gamma_r * h \cos\alpha$ ……..4(2) for vertical rock cover and

$\gamma_w * H \leq \gamma_r * L * \cos\beta$ …….4(3) for lateral rock cover

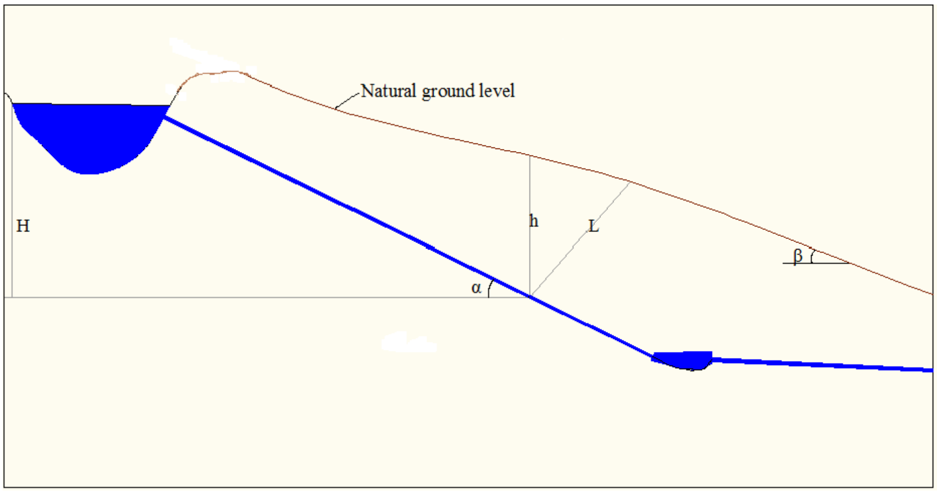


Figure 4.4 Figurative illustration of different parameters for hydraulic fracturing

The FRL of the dam is at El.843m, invert level of HRT at Ch.0.0 and end of HRT is at El. 803.5m and El.764.5 respectively. From the above relation using these water levels, minimum of 30m and 46m rock cover is needed to avoid hydraulic fracturing. The actual rock cover along the HRT alignment is significantly higher than these values. It can be safely concluded that the tunnel is free from hydraulic fracturing due to static water pressure.

4.2.3 Adit locations

Construction adits are provided to facilitate access to the tunnel work fronts. The number of such adits is decided depending on the length of HRT, the topography along the alignment and etc. The decision is made based on the cost, criticality of the tunnel work in context of overall completion the projects.

In a drill and blast tunneling, the length of the tunnel from an adit junction are restricted by the effectiveness of ventilation arrangements, increase in lead for removing the excavated materials, the length of pipes required for dewatering seepage waters in tunnel from downstream face etc.
In an idle homogeneous rock condition, adits are normally provided at equidistances along the tunnel alignments. But in field, this is seldom true. Adit locations can be influenced by the dictates of topography, location of bends and most importantly location of possible problematic zones. Easy access to problematic zones gives the constructor sufficient time to plan and treat the problem.

For this project, topography limits the location of adits. The distance of the adits from possible weakness zones are far. The distance from adit I to 2nd nala crossing is 3123m. The geological sections along these reaches show semi vertical joints. This may cause possible stability problems. In an event of geological problems at face near to 2nd nala while excavating from adit II, the distance to the same is very far from adit I, and in addition it has to cross 1st nala crossing which may again give rise to problems. These long distances between the adits and the possible problematic zones may prove very costly for the project. This aspect is optimized in the alternate alignment.

4.2.4 Squeezing problems

Deep seated tunnels through weak rock mass can be susceptible to instability problems. It will occur as rock bursting and spalling in good competent rocks and squeezing in poor weak rock. The rock cover along the tunnel alignment varies from 80m to 700m. The empirical approach given by Singh et al (1992) is used to evaluate possible squeezing. The relation is

$$H \geq 350Q^{1/3} \qquad \text{……………………………………….4(4)}$$

Using above relation, and substituting relevant parameters of Q (the average Q value of 3.7 is used which is the average of all Q values along the tunnel alignment), the result show possible squeezing along some sections where the rock cober exceeds 540m.

From the Norwegian experience (Nelson & Palnstrom, Engineering geology and rock engineering hand book 2), squeezing is possible when the rock cover exceeds 500m with valley ward slope angle exceeding 25^{0}. The geology along the tunnel alignment is through mixture of biotite , schist and quartzite rocks. Possible rock burst may occur in quartz where the overburden exceeds 500m and squeezing when the rock type is through schistose formation.

The complexity of geology, topography and locations of other project components makes elimination of all problems virtually impossible. However designs can be optimized to minimize possibility of problems. This has been explored in the alternate alignment.

Tangential stresses

The HRT alignments at different sections are checked for tangential stresses. The tangential stresses for the roof and the walls are calculated separately using the equation proposed by Hoek and Brown (1980). The other parameters are used from relevant figures and calculations.

$\sigma_{\theta r} = (AxK - 1)\sigma_v$ ....4(5) Tangentia stress in roof

$\sigma_{\theta w} = (B - k)\sigma_v$ ....4(6) for the tangential stress in walls

The main calculation sheet is attached in the Appendix A. The tangential stress values along the HRT alignment in three selected reaches is shown in table. 4.5

Table 4-5 Tangential stress values in the roof and walls at different HRT sections

Description	Ch.0 to Adit I	Adit I to Adit II	Adit II to s/shaft
Tangential stress roof σθr	20.26	17.92	14.49
Tangentia stress wall σθw	11.69	22.20	12.22
Rock mass strength Average rock mass strength	11.09	9.11	6.79

For calculating the horizontal stress, a tectonic stress value of 5 MPa is used from Panthis case study in Parbathi project in India. From the calculated values, the tangential stresses values in walls and roofs are more than the average rock mass strength, so minor rock falls from both wall and roof can be expected.

Squeezing prediction by Hoek & Marinos approach

The squeezing in the tunnels at the different reaches was calculated using Hoek & Marinos (2000) approach. The details calculation sheet and theory is discussed in Appendix A and chapter 5 respectively.

The deformation result of for the rock mass in different tunnel alignment and their tangential stress values are calculated and presented in table 4.6 below.

Table 4-6 Rock deformation values along the HRT sections.

Description	Ch.0 to Adit I	Adit I to Adit II	Adit II to s/shaft
Deformation without support pressure εt in %	0.10	0.33	0.22
Deformation with support pressure εt %	0.003	0.068	0.053

The result shows the deformation without support is 1.25cm from each side in stretch between Ch. 0.0 to Adit I, around 5cm from each wall in the middle reach between adit I to adit II, and 3cm from each wall side in the last reach. All the deformation values are very negligible. It and can easily be encountered with flexible support like shotcrete and rock bolts. These values are very small according to Hoek and Marinos interpretation chart, so there is no serious risk of squeezing or rock spalling problems even without the support.

Bend location at Ch.1053.3m

The 1st bend at Ch.1053.3m is located just below the crossing of the nala with the HRT alignment. The rock mass below the crossing of the nala is expected to be poor due to the presence of shear zone. This shear zone could further be weakened by weathering affects intruded by seepage through it. Locating the bend at this location may not be advisable from the stability point. This might give instability problems during construction time.

4.2.5 Tunnel alignment and foliation orientation

The tunnel between Ch.0.0m and Ch.1053m and the main gneissosity is aligned at $\approx 23°$ only. This aanle is small and may give rise to minor problems during excavation. However, the cross joints are aligned with the tunnel at $\approx 84°$ which is favorable for tunneling.

In the second stretch between Ch. 1750m 70 4821m, the HRT is aligned at $\approx 36°$ with the main foliation which is improved from the earlier stretch. The tunnel alignment is favorable with cross joint orientation. The third 1292m reach from Ch. 8421m to Ch. 6113m is almost parallely orientated with the main gneissosity with $\approx 5°$. The last reach from Ch.6113m to 8342m is also aligned at $\approx 31°$ with the main foliation, but the cross joints may not cause serious problem to the tunnels.

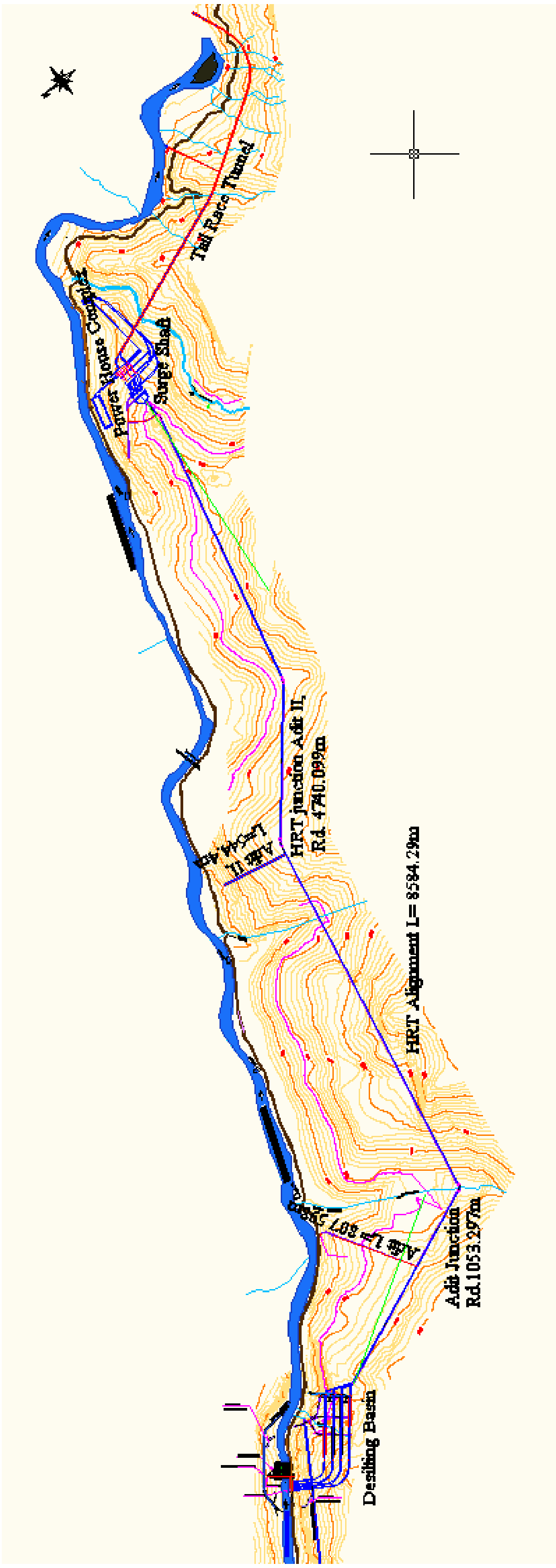


Figure 4.5 Layout plan of HRT alignment

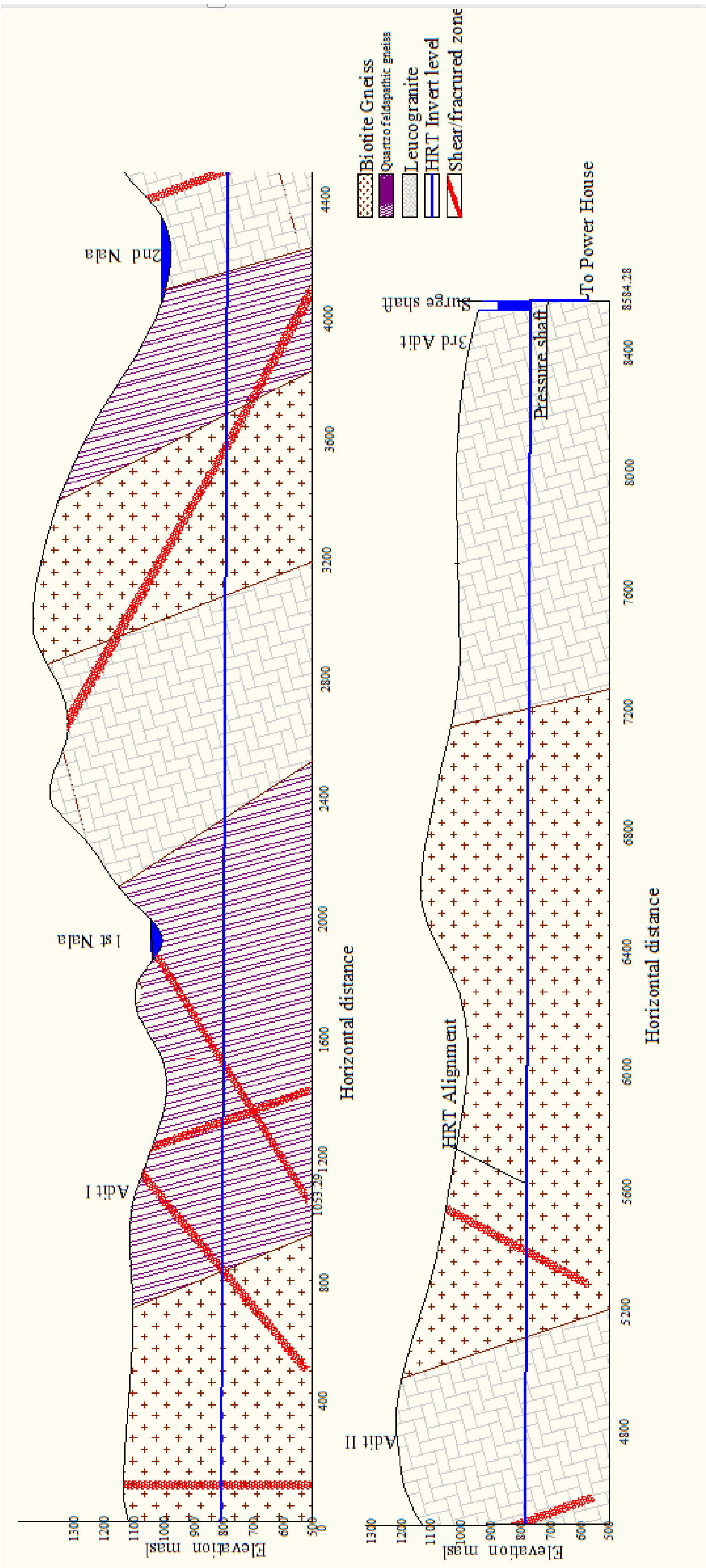


Figure 4.6 Geological section along HRT

4.3 Alternate HRT alignment

The take off and the end point of HRT alignment was kept same with the existing design. Within the limits of these constraints, an exercise was made to optimize the alignment to reduce the deficiencies in the existing alignment. The number of bends in the alignment was reduced without compromising other geotechnical requirements. The locations of the construction adits were also optimized according to the needs of the geological features and its locations. The final layout of the realignment is shown in figure 4.11. The details of each feature are discussed in the following sections.

4.3.1 Calculation of head loss due to friction

The frictional head losses along the new alignment are calculated using equation 4(1) given by Mannings.

The head loss value is 14.45m. The difference in the head loss is due to the reduction of tunnel length from 8584.28m to 8342m. Besides this frictional loss, the number of bends is also reduced. This reduction in head loss between the two alignments is almost 1m, which is equivalent to 4MW of power. This is extra benefit in the new alignment.

4.3.2 Hydraulic fracturing and water leakage problem

The possible hydraulic fracturing to the rock mass is checked using the Selmer –Oslens (1970) equation given in 4(2) & 4(3).

Since the start and end location of the HRT alignment was kept same with the designed layout, the minimum vertical and later rock covers remain unchanged. The minimum rock covers were calculated considering a factor of safety value of 1.5. The new alignment has more than the minimum rock covers.

The new alignment has shifted towards the valley side, both vertical and lateral minimum rock covers are well above the required limits. This value is also considering a safety factor of 1.5.The details of the calculation along the new alignment is tabulated and shown in table 4.7.

Table 4-7 Minimum rock cover calculated from Thimb rule for hydraulic fracturing

HRT Rd. (m)	HRT invert level	Hydrostatic head (MPa)	Slope angle β Degree	vertical rock cover (m)	lateral rock cover (m)	vertical rock cover with (m)	lateral rock cover with (m)
0.00	El. 803.5	0.395	63^0	15.01	33.08	22.65	49.62
4167	El.784.00m	0.59	32^0	22.43	26.45	33.65	39.67
8335	El. 764.50m	0.785	19^0	29.84	31.56	44.76	47.34

The figures from figure 4.7 to figure 4.9 shows the rock covers available at three different locations along the new tunnel alignment. The rock covers are more than adequate.

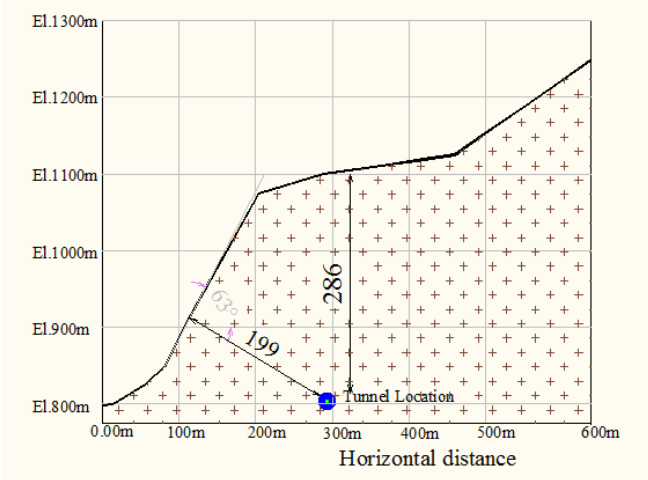


Figure 4.7 Rock cover at Ch. 0.0m

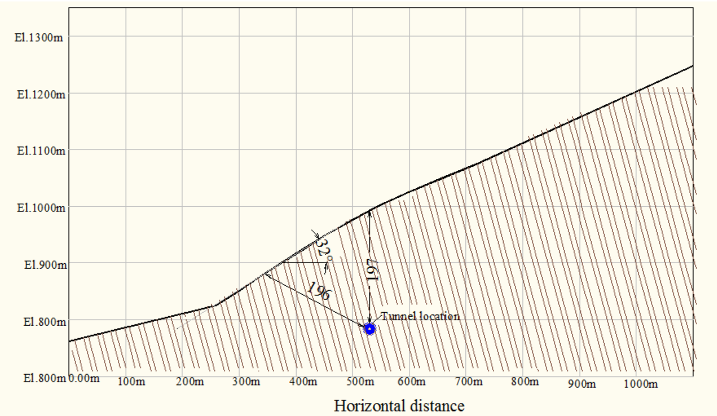


Figure 4.8 Rock cover at Ch.4167m

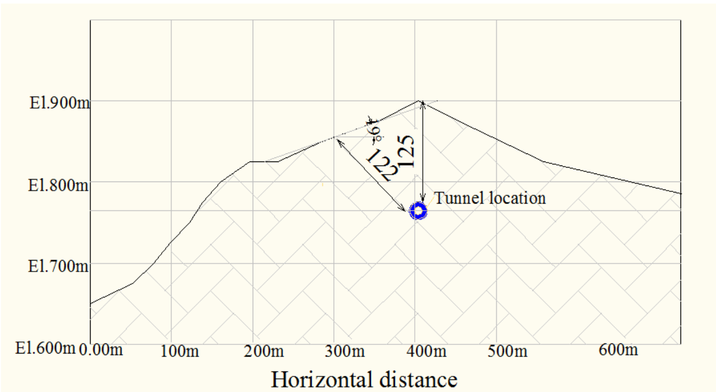


Figure 4.9 Rock cover at Ch.8584.28m

4.3.3 Location of Adit portals

The possibly best adit locations are explored within the limits of existing topography. More focus was given for optimizing the location of adits with respect to the 2nd nala crossing with the tunnel alignment. The minimum gradients for adits were maintained to discharge seepage waters through the drainage arrangements. The natural topography was carefully considered with regard to the practicability of providing access roads from the existing road points. The length of the adits was also thoroughly considered so that it does not unnecessarily add to high cost of project. The final locations of different adits were as given in fig. 4.11. Other features of the adits are discussed individually here under.

Adit 1

The topography restricts the portal location in a very small stretch of area. The portal location is not very far from the old portal location but placed closer to 2nd nala crossing.

- Elevation of the portal location= E. 775m
- Length of the Adit = 797
- Junction Ch.1309m
- The invert elevation of the adit with the HRT =El. 797.38m
- Gradient of the adit tunnel 2.8%

Adit 2

The distance between adit junctions and the 2nd nala is reduced. Other features include

- Elevation of the adit portal location El.750m
- Length of the adit tunnel = 844m
- Junction Ch.4468m
- Invert level of the adit with the HRT junction El.783.1m
- Gradient of the tunnel 3.9%.

Adit 3

The location of the adit 3 was not changed since it is bifurcating from the main access tunnel to the butterfly valve chamber and surge shaft area. The junction could not be shifted close to the surge shaft end considering the interference with the works of surge shaft widening. The overall dimension of the adit tunnels is kept 7m x7m which is the original size of the adit tunnels as designed.

4.3.4 Squeezing problem due to high rock stress

The new alignment is checked for possible squeezing problem by the thumb rule proposed by Singh et al. with the average rock cover reduced compared to the existing alignment, the squeezing problems in major portion of the tunnel alignment is reduced but minor problems can still be expected in some smaller stretches where the rock cover is above 500.

The deformation results along the new alignment calculated based on Hoek & Marinos (2000) is presented in table 4.8

Table 4-8 Values of rock stresses and deformation at different locations along HRT alignment

Description	Ch.0 to Adit I	Adit I to Adit II	Adit II to s/shaft
Tangential stress roof	20.10	17.47	14.66
Tangentia stress wall	11.12	17.12	9.35
Rock mass strength			
Average rock mass strength	11.09	9.11	6.79
Deformation without support pressure εt in %	0.09	0.24	0.16
Deformation with support pressure εt %	0.01	0.03	0.11

The tangential stresses in the roof are lower than the rock mass strength but the tangential stress in the walls are higher so there can still be stress problems in the walls. However the tangential stress values are smaller compared with existing alignment condition.

In the realigned location, the deformation in the different tunnel reaches has reduced to negligible limits. This instability situation can be easily countered without need for heavy support. Light flexible rock support like shotcrete and rock bolting is sufficient for instability measures.

4.3.5 Cross section shape of the Tunnel

The designed cross section of HRT is modified horse shoe shape. This shape is difficult for excavation and gets worse with larger size and poor rock quality. Rock over breaks is common in the poor Himalayan geology and it will get worse with complicated shapes. This over breaks in excavation leaves room for confrontation between the client and contractor where the later often gets benefited. Instead, a simple D-shaped HRT cross section can be adopted. This will be easy for blasting and reduce over breaks and related supports cutting down the cost. The proposed shape for the HRT is shown in fig. 4.10

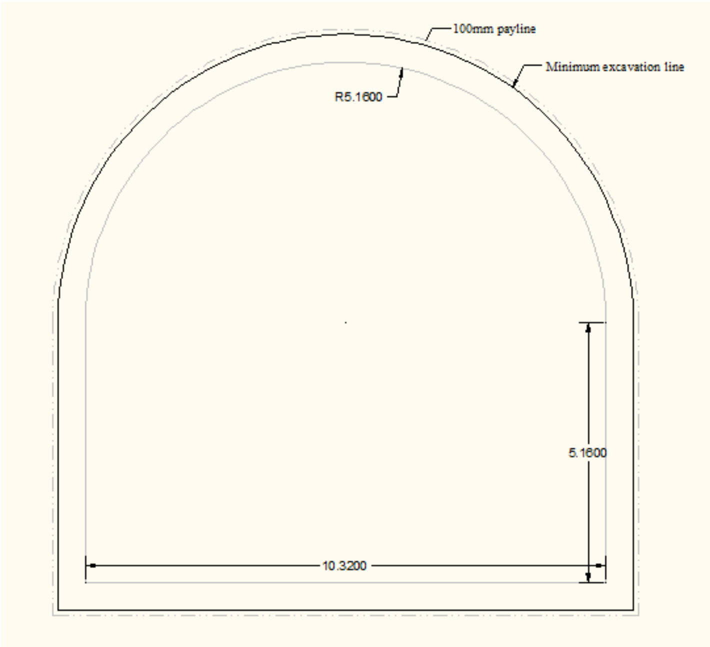


Figure 4.10 Alternate cross section for HRT

4.3.6 Location of bends

The area around the nala crossing can be a possible weak zone. The deep weathering affects penetrating the rock mass can be further worsened by the ingress of water through the jointing and foliation at these locations. To avoid this location of the 1st bends have been shifted away from the nala crossing. This will avert the possible weak zones for tunneling.

4.3.7 Tunnel alignments and the gneissosity

The first 1543m of the tunnel is not favorably aligned with the foliation directions with just $\approx 21°$ and $\approx 54°$ with the tunnel direction. However the remaining tunnel reaches have improved orientation with the main foliations directions. The second stretch of 4025m from Ch. 1543 to Ch.5568m has $\approx 65°$ orientation with the main foliation direction and the last 2767m from Ch 5568 to Ch. 8342m makes an angle of $\approx 76°$ with the main foliation. This has improved orientation of tunnel alignment with main foliation and jointing. This improved tunnel orientation with the main foliation and cross jointing will make a huge impact in improving the tunneling through this poor geology.

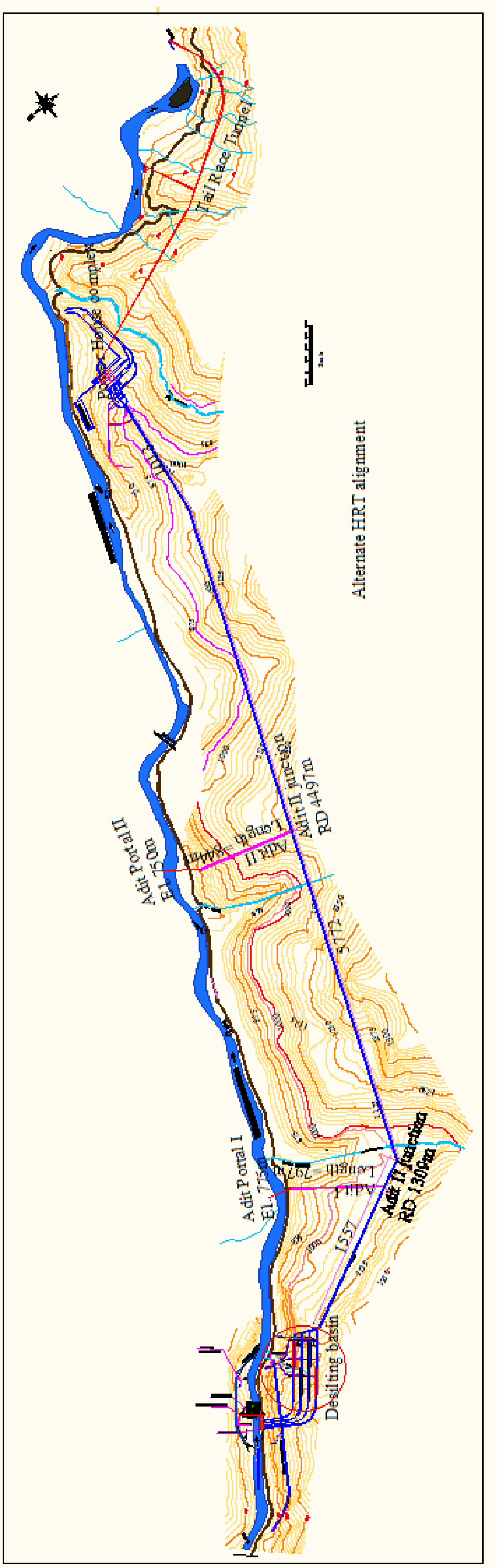


Figure 4.11 Alternate HRT alignment layout

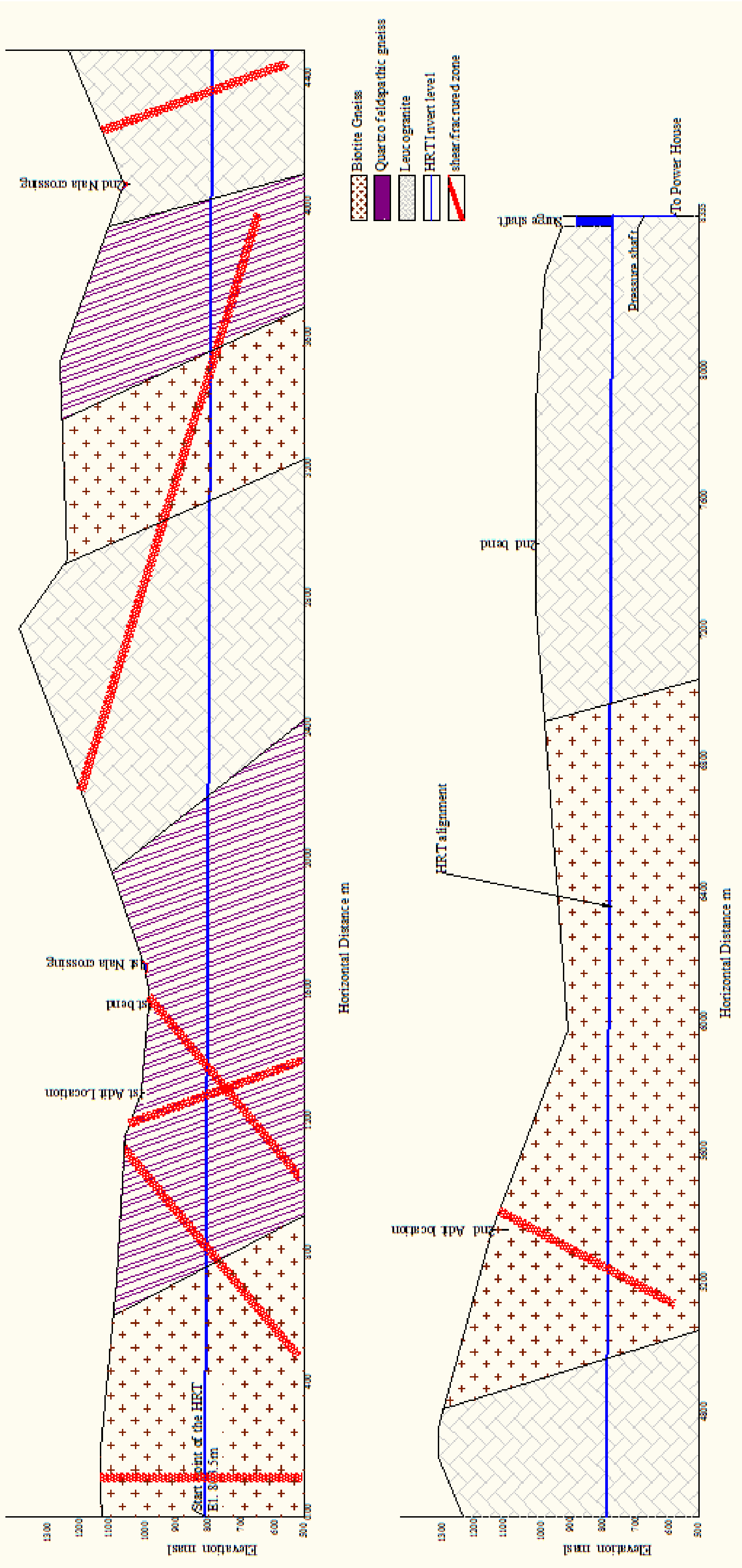


Figure 4.12 Geological section along the alternate HRT alignment

4.4 Conclusion

The advantages and benefits derived from the revised tunnel alignment are

- ❖ The overall length of the head race tunnel is reduced by 250m

- ❖ Due to reduced length, there is reduction of 0.45m in head loss.

- ❖ The numbers of kinks required in the HRT alignment is reduced to 2 nos. which will reduce head loss.

- ❖ The overall vertical rock cover is reduced which changed squeezing condition from severe to minor squeezing.

- ❖ The total increase in length of the adit is more than reduction of length in HRT, but the adits do not require expensive support system like the HRT thereby reducing the time and cost of the project.

- ❖ The distance to the expected problem zones from each adits is reduced which will give advantage during the construction of the tunnel.

The geological instability is common in the Himalayan geology. The choice of machines and methodology adopted will make lot of difference to the cost and time of project completion. The instability in the Himalayas is the result of high tectonics, deep weathering and young rock formation of the Himalayas. The stability problems cannot be eliminated totally but when right measures and technology are adopted; its seriousness can be reduced. The new tunnel alignment proposed may help reduce stability problems, save cost and time in successful completion of the project. The new alignment is shown in figure 4.12 and 4.13.

Table 4-9 Comparative features between existing and alternate alignment layout

Description of component	Original layout	Revised layout	Remarks
Length of HRT (m)	8584.3	8342	242.3m less
Numbers of kinks (Nos)	4	2	2 bends less
Minimum vertical overburden (m)	80	124	
Maximum vertical over burden (m)	600	584	
Squeezing in the tunnels	Severe squeezing	Minor squeezing	

The new alternate alignment is better than the planned alignment. However since the project is already under execution, this exercise is done more as a academic exploration. But the process involved can be successfully used to optimize benefits in future tunneling projects.

5 STABILITY ANALYSIS

Rock mass comprises of many minerals and discontinuity features. This affects the mechanical property of the rock mass, reducing it compared with intact rock. The mechanical strength properties of the rock mass are smaller than the intact rock. The information on rock mass properties is required for planning and design purposes. It is difficult to collect rock sample with all discontinuity features. This is due to constraints in laboratory size and financial limitations. As such, normally the laboratory test is carried out on intact rock specimen in the field. The result of this is extrapolated using different relations to get the material properties of rock mass. This chapter deals with the study of some of the characters of the rock mass and its behavioral trends.

Continuous and discontinuous rocks

In an underground work, the behavior of the excavated rock mass is influenced by the shape and size of opening. The ground behavior in an excavation is related to size of opening to the rock mass block size. The ground behaves either as continuous or discontinuous material during excavation; the continuity behavior of ground is assessed by continuity factor (CF). $CF = \frac{D_t}{D_b}$, where D_t is the diameter of the tunnel and D_b block diameter of the rock mass.

The limit between continuous and discontinuous is matter of judgment. Palmstrom (1995) has suggested the following limits.

- For CF ≈ approx. 5 – 100, the ground is considered discontinuous. The behaviors are likely to be anisotropic, dominated by individual discontinuities.

- For CF ≈ <5, the rock properties dominate and for highly jointed rocks with CF > 100, the material behaves like a soil.

Continuous and discontinuous ground behaves differently. It is important to determine the type of rock mass and the ground condition. Only after knowing the ground condition, an appropriate design principle can be effectively applied.

In general, the stability assessment of the tunnels and caverns are carried out using three different approaches. The three methods are discussed separately in the following sections.

Empirical method

Analytical method and

Numerical analysis

5.1 Empirical method

Empirical methods are developed mostly from the observation gained by different authors through field experience. This method gives the indicative trends in the behavior of rock mass rather than definitive accurate results. These methods are more descriptive in nature than calculative. These methods are more easy to use in field. Some of the empirical methods used in rock engineering are discussed in this section.

5.1.1 Rock mass classification

There are different methods used for the classification of the rock mass. Two of the most commonly used methods in Bhutan are the NGIs Q method and Bieniaswkis RMR method. These two methods are extensively used in most projects in Bhutan and the support systems were designed on the basis of these two methods. The following section discusses the two methods.

Bartons Q-System of rock mass classification

Bartons Q system of rock mass classification was developed at the Norwegian Geotechnical Institute (NGI) by Barton et al. in 1974. It is a quantitative classification system for estimation of support system. The rock mass is classified based on the following six rock mass parameters.

- Rock quality designation (RQD)
- Numbers of joints (Jn)
- Roughness of most unfavorable joint or discontinuity (Jr)
- Degree of alteration or filling along the weak joints(Ja)
- Water inflow (Jw)
- Stress condition given as the stress reduction factor (SRF).

The above six parameters are grouped into three quotient to give an overall rock mass quality.

$$Q = \frac{RQD}{Jn} x \frac{Jr}{Ja} x \frac{Jw}{SRF} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 5(1)$$

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

The second quotient describes an indication of the inter block shear strength and

The third quotient described the active stresses.

Each of the six parameters gives the description of the rock mass in terms of its jointing, spacing, the infill materials, its properties and the seepage conditions. The six parameters and its ratings as given by are Barton et. al (1974) reproduced in Appendix B. The combined result of all the above six parameters defines the rock mass and its quality. This is called the rock mass quality index Q.

Based on the different Q values ranges, the rock mass are classified into different class of rock. The different rock classes based on the rock mass rating index Q is given in Table.5.1.

Table 5-1 Rock mass classification based on NGIs Q method

Sl. No	Rock Class description	Rock Class	Q value range
1	Very to extremely good rock	Class-I	100-1000
2	Good rock	Class-II	10 – 100
3	Fair to good rock	Class-III	4 – 10
4	Poor rock	Class-IV	1 - 4
5	Very poor rock	Class V	0.1 - 1
6	Extremely poor rock	Class VI	0.01 – 0.1
7	Exceptionally poor	Class VII	0.001 – 0.01

The rock mass rating index Q is used in combination with the excavation support ratio in the standard chart for designing the rock support system. This is dealt in chapter 7 in supports.

Bieniawski's RMR system of rock mass classification

The RMR or the geomechanics system is developed by Bieniaswki in 1973. It is also one of the most commonly used classification method in Bhutan. In this method the rock mass is classified using the following six rock mass parameters.

- Uniaxial compressive strength of intact rock material
- Rock quality designation (RQD)
- Spacing of discontinuities
- Condition of discontinuities
- Ground water condition
- Orientation of discontinuities

In this method, the rock mass along the tunnel alignment is divided into zones having similar or uniform geological features. The above six parameters are measured in the field or obtained from the results of the bore hole data along these zones. Once these parameters are determined, the rock mass is classified using the standard RMR rating charts given in Appendix B.

The RMR rating chart is used along with the classification guide and rock tunnels support given by Bieniawski in 1989. This aspect is discussed in chapter 6.

Based on the values of the classification rating parameters, the rock mass is classified into different calsses based on their RMR values range. This is presented in table 5.2.

Table 5-2 Rock mass classification based on RMR ratings

RMR value	100-81	80-61	60-41	40-21	<20
Rock class	I	II	III	IV	V
Description	Very good	Good	Fair	Poor	Very poor

5.1.2 Prediction of tunnel squeezingl

One of the empirical methods of predicting squeezing in tunnels is given by Singh et al (1992) based on a relation between the rock mass quality Q and over burden H for the Himalayan tunneling. The equation is

$$H \geq 350Q^{1/3} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 5(2)$$

According to this approach, when the over burden is equal to or higher than the value given by the above equation, the tunnel is likely to be affected by squeezing. The figurative presentation of the approach is given in fig. 5.1

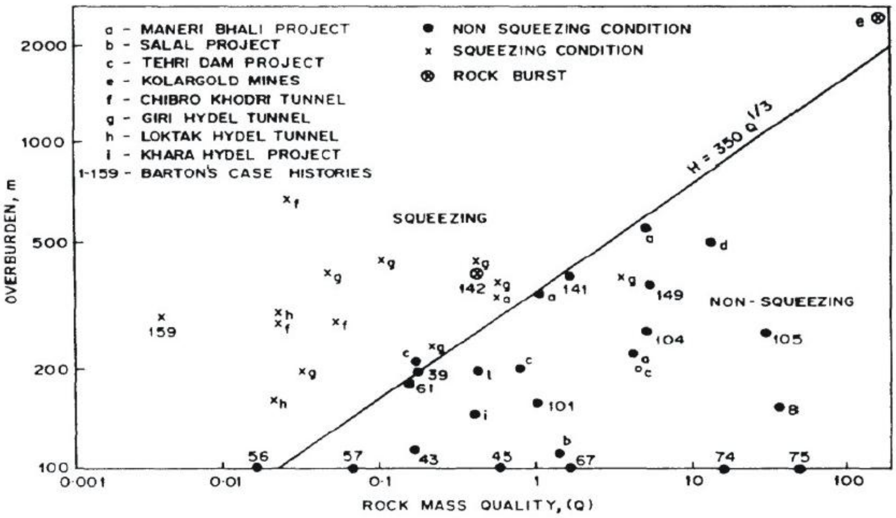


Figure 5.1 Tunnel squeezing prediction chart by Singh et al (1992)

The possible squeezing along the HRT alignment based on this approach is reviewed. The results are presented in table 5.3

Table 5-3 Minimum rock cover for squeezing from Singh et al relation

HRT reach	Ave. Q	H value from Singhs relation (m)	Ave. over burden in the field (m)	Remark
Ch. 0 to Adit I	3.31	521	295	The available overburden is much less than the minimum height for squeezing so squeezing may not be a problem along the HRT alignment from Singhs prediction.
Adit I to Adit II	3.7	541	455	
Adit II to S/shaft	3.64	538	276	

Based on results from this method and calculations, there are some possibility of squeezing in the middle reach where the available rock cover is higher than the minimum threshold value. However, the tunnel in the starting and ending reaches should not have squeezing problem.

Goel et al (1995)

Goel et al. has developed the squeezing prediction on the same line with Singh et al except that they used rock mass number (N). The proposed equation is given below.

$$H \geq 270 \mathrm{x} H^{0.33} \mathrm{x} B^{-.1} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5(3)$$

H is rock overburden, B is the width of the tunnel and N is the rating Q value without SRF. There can be possible squeezing problems when the available over burden exceeds the value given by the equation. The results of the tunnel squeezing along the HRT based on Singh were discussed in the alternate alignment of the HRT. And it has shown some minor squeezing possibilities when the rock cover exceeds 500m plus in height.

5.2 Analytical method

The analytical methods have evolved on the foundation of the empirical methods. However the results are presented more as calculations in analytical methods. since the results are in calculated figures it gives better guidance for comparison and a better understanding of the rock mass and its behavior. This sections deals with some of the analytical methods used in rock engineering and analysis of the results.

5.2.1 Stresses in the rock mass

The rock mass is subjected to in-situ stresses in undisturbed form. The in-situ stress gets redistributed during excavation. The different types of stresses and their origin are discussed in this section.

Origin of stresses

The in-situ rock masses are subjected to virgin stresses from its surroundings. The most important rock stresses are caused by the following.

- Gravitational stress
- Topographic stresses
- Tectonic stresses and
- Residual stress.

Of all stress, the topographic and gravitational stress is most prominent stress affecting underground structure. The tectonic stress is responsible for the incidents such as faults and folding and is significant in the tectonically active regions in shallow depths. Bhutan is location in active tectonic regions of eastern Himalayan range. As such the tectonic stresses should be carefully considered for design of underground structures in Bhutan.

The vertical gravitational stress at any depth H is proportional to the weight of the over lying rock mass and is calculated by

$\sigma_v = \Upsilon H = \rho g H$ (MPa) ……………………………5(4)

Where ϒ is the density of rock in MN/m^3, and H is the height of over burden in m. knowing the vertical stress, the horizontal stress can then be calculated using the equation

$\sigma_h = \frac{\upsilon}{1-\upsilon} X \sigma_v + \sigma_{tec}$ ………………………………………5(5)

Where υ is poisons ratio, and σ_{tec} is the tectonic stress in MPa. A tectonic stress value of 5MPa the value used by Panthi for Parbati project in India (Panthi, 2011) is assumed since the two projects are located within the same region. Using formulas, the vertical and horizontal stresses along the HRT alignment is calculated and presented in table. 5.4.

Table 5-4 Vertical and horizontal stress values along HRT sections

Location/Stresses	Ch.0 to Adit I	Adit I to Adit II	Adit II to S/shaft end.
Vertical stress MPa	7.66	11.77	7.11
Horizontal stress MPa	9.01	9.58	6.97

The horizontal stress is higher than the vertical stress. This holds true because the height of the vertical cover is less than 600m. As per the studies conducted by different authors, the horizontal stresses value will be higher till depth of 1000m. Beyond it the vertical stress will dominate.

Stress distribution

The rock mass is under the virgin stress condition in its undisturbed state. When excavated, the stresses in the surrounding rock gets redistributed around the periphery of the opening readjusting to the changed surrounding. The stress distribution in the rock mass before and after excavation is given in fig.5.2.

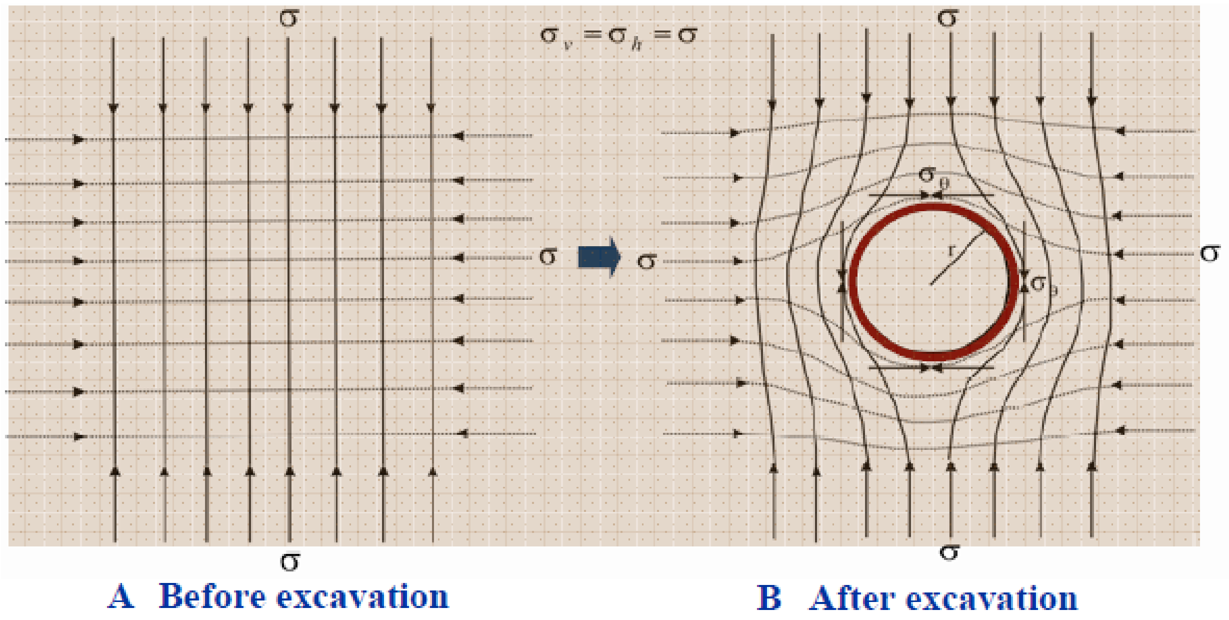


Figure 5.2 Stress distribution before and after excavation (Panthi, 2011)

Circular openings in iso-static condition

In an ideal homogeneous, elastic material with iso-static stress condition where ($\sigma_1 = \sigma_2 = \sigma_2 = \sigma$) is the virgin stress, the tangential and radial stress distribution along the contour of opening with radius r_i will follow the trend shown in fig. 5.3, and the corresponding relation with the radial variation is given in the same figure.

The graphical representation of the stresses variation along the periphery of the opening with changing radius from the center of the opening is shown in fig.5.3. as presented, the radial stress is zero at the distance of 1r from center of opening and tangential stress is maximum. With increasing distance from center, the radial and tangential stresses values increases and decreases respectively. The stress values stabilises and takes a constant value at distance roughly 1D from center.

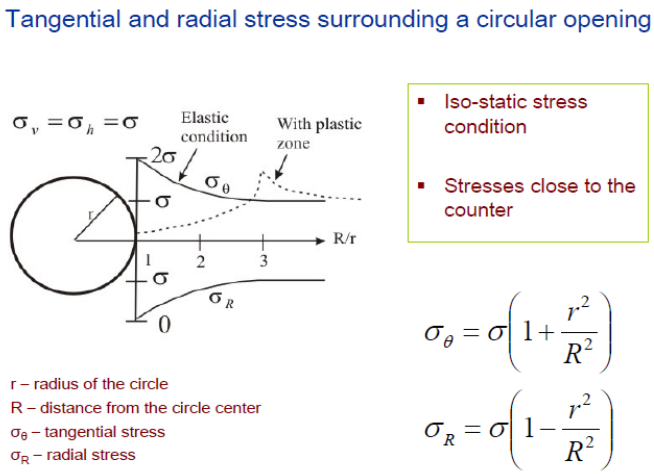


Figure 5.3 Tangential & radial stress along circular opening in isostatic field (Panthi,2011)

Kirsch approach

In the field, the stresses are hardly isotopic. The tangential stresses vary along the periphery of the circular opening. Kirsch's equation gives the value of maximum and minimum stress along the opening periphery in an-isotopic stress condition.

$$\sigma_{\theta(\max)} = 3\sigma_1 - \sigma_3 \text{ ……………5(6)}$$

$$\sigma_{\theta(\min)} = 3\sigma_1 - \sigma_3 \text{ ……………5(7)}$$

The stress distribution is strongly influenced by degree of anisotropy. After excavation, the induced stresses in the rock mass are redistributed along the periphery of the opening. When the stress exceeds the rock mass strength the rock faces instability problems. In general the instability induced by stresses occurs in the areas of maximum tangential stress. However when the tangential stress values are very low, problem of rock fall in the jointed rock mass occurs.

Hoek and Brown approach

Hoek and Brown (1980) has, based on large number of detailed boundary element stress analysis developed a correlation to estimate tangential stresses as given below.

Tangential stress in roof $$\sigma_{\theta r} = (A \; x \; K - 1)\sigma_v \text{ …………………..5(8)}$$

Tangential stress in wall $$\sigma_{\theta w} = (B - k)\sigma_v. \quad \text{…………………5(9)}$$

Where A and B are factors given by Hoek & Brown (1980) and is shown in table 5.5. K is ratio of horizontal to vertical stresses.

Table 5-5 A & B values in underground openings (Hoek & Brown, 1980)

VALUES OF CONSTANTS A & B									
A	5.0	4.0	3.9	3.2	3.1	3.0	2.0	1.9	1.8
B	2.0	1.5	1.8	2.3	2.7	3.0	5.0	1.9	3.9

Based on the above A and B values, the tangential stresses in roof and the walls of the tunnel along different reaches of the HRT alignment is calculated and presented in table. 5.6

Table 5-6 Tangential stress values in roof & walls along the HR alignment

Location/Stresses	Ch.0 to Adit I	Adit I to Adit II	Adit II to S/shaft end
Rock mass strength MPa	11.09	9.11	6.99
Tangential stress in Roof MPa	20.26	17.92	14.49
Tangential stress in walls MPa	11.69	22.20	12.22

The rock mass strength values used are the average values calculated from relations proposed by different authors. It can be seen that the tangential stress in both the roof and walls are greater than the rock mass strength, as such there can be minor stability problems in the roof and walls throughout the HRT alignment.

5.2.2 Rock mass failure criterions

Rock mass comprise of many geological features that shape the overall strength and behavior of it. Different factors contribute to the instability in the rock mass that cause failure in the underground tunnels and openings. The most widely used failure criterions proposed by Hoek & Brown and the Mohr-Coloumb failure criterions were discussed here.

The Hoek & Brown failure criterion for the rock mass

The strength of the rock mass is an important factor for the stability of the underground structures. Hoek & Brown (1980) has given the relation for estimating the strength based on the interlocking of the blocks and the surface conditions between the blocks. This relation is revised many times over the years and a generalized Hoek – Brown failure criterion by Hoek et. al. (2002) is given by

$$\sigma_1{}' = \sigma_3{}' + \sigma_c(m_b\frac{\sigma_3}{\sigma_c})^a \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 5(10)$$

Where σ_1, and σ_3' are the effective major and minor principal stresses. σ_c is the uniaxial compressive strength of the intact rock. m_b is the reduced value of material constant m_i, s and a is constants which depends on the rock mass characteristics. mb, s and a values of the rock mass is calculated from following equation

$$m_b = m_i\ exp^{\frac{GSI-100}{28-14D}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5(11)$$

$$s = \exp\left(\frac{GSI-100}{9-3D}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5(12)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5(13)$$

Where, D is degree of disturbance of the rock mass due to blast damage and stress relaxation. The value varies from 0 for undisturbed rock mass to 1 for highly disturbed rock mass. GSI is the geological strength index and mi is the material constant, the different values are calculated and shown in table.5.7. The disturbance factor D for different tunneling conditions is given in fig. 5.7

Table 5-7 Rock mass constant values for the HRT

Location/rock mass constant values	Ch.0 to Adit I	Adit I to Adit II	Adit II to S/shaft end
mb	2.19	2.19	2.19
s	0.0006	0.0006	0.0006
a	0.51	0.51	0.51

These calculated values were checked with similar cases in the region and the results are found within the similar range. It is also comparable to the values computed by Rocdata.

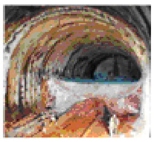
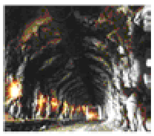
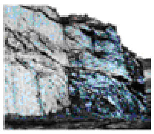
Appearance	Description of Rock Mass	Suggested Value
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	D = 0
	Mechanical or hand excavation in poor quality rock masses (no blasting results in minimal 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 3m, 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 slope is less	D = 1.0 Production blasting D = 0.7 Mechanical excavation

Figure 5.4 Disturbance factor for different tunneling (Hoek & Brown, 2002)

Mohr Coloumb failure criterion

Mohr –Coulomb is one of most commonly used failure criterion based on normal stress and shear strength of the intact rock. Shear strength of the rock mass can be defined by the friction angle (**Φ**) and cohesive strength (C).To determines these two parameters, Hoek et. al. (2002) derived a relation between Hoek & Brown failure criterion and Mohr –Coulomb criterion by fitting an average linear curve balancing the area above and below the Mohr-Coulomb plot as shown in fig. 5.5.

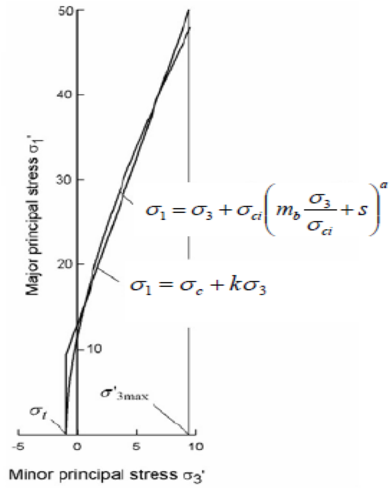


Figure 5.5 Relation between major & minor principal stresses for Hoek & Brown and equivalent Mohr-Coloumb criterion (Hoek et. al, 2002)

According to Hoek (2007), when the structure being analyzed is large compared to the block size, the rock mass strength can be estimated using Hoek & Brown failure criterion, and when discontinuity spacing is larger compared to structure dimension, Mohr-coulomb failure criterion can be used for stability analysis of the underground structures. Fig. 5.6 shows the transition of the rock mass properties from isotopic intact rock to heavily jointed anisotropic rock mass and applicable failure conditions.

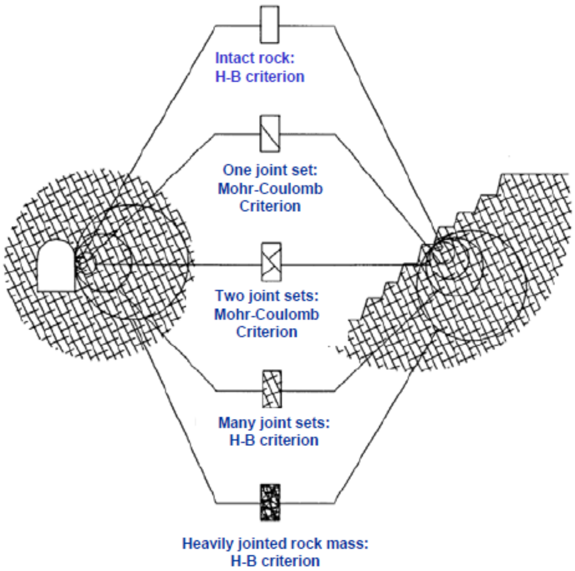


Figure 5.6 Choice of failure criterion (Panthi, 2011)

5.2.3 Rock mass properties and estimation.

Rock mass is a heterogeneous medium with many associated variables. The two main features characterizing the rock mass are 1) rock mass quality and 2) the mechanical processes acting on the rock mass. These two features are interlinked with each other. The rock mass quality is related to rock mass strength, deformability, strength anisotropy, presence of discontinuities and weathering affects. The mechanical properties effect tunnel stability and is linked with stresses in rock and ground water. The stability is further influenced by project specific like shapes, sizes and location and orientation. Due to heterogeneity of rock mass, it is very difficult to get representative rock sample for laboratory tests, so laboratory tests are performed on intact samples. The results of the intact rock samples do not directly give rock mass strengths. However experiments and studies conducted by different authors have shown scale effect relation between the two. Different authors have developed different equations for calculating rock mass strength parameters. Hoek-Brown has developed one such relation with following input parameters.

σ_{ci} , the uniaxial compressive strength (UCS) of intact rock piece.

m_i, Hoek-Brown constants of intact rock

GSI, Geological strength index of the rock mass and

E_m, Deformation modulus of the rock mass

These parameters are used as inputs to calculate other rock mass properties and stresses. Different authors have given different equations for calculating different properties of rock. Some of the important properties are discussed here.

Uniaxial compressive strength of the rock mass

The effect of weathering reduces the rock mass strength. Many authors have developed relations for estimating the rock mass strength relating with different rock mass parameters. Some of the most commonly used relations are tabulated in able 5.8 and their values calculated and compared.

Table 5-8 Empirical formulas used for estimating rock mass strength

Author	Empirical relation
Bieniawski (1993)	$\sigma_{cm} = \sigma_{ci} x \exp(\frac{RMR-100}{24})$.........................5(14)
Hoek and Brown (2002)	$\sigma_{cm} = \sigma_{ci} x [\exp(\frac{GSI-100}{9})]^a$.....................5(15)
Barton (2002)	$\sigma_{cm} = 5 * \gamma * Q_c^{1/3} = 5 * \gamma * [\frac{\sigma_{ci}}{100} * Q]^{1/3}$............................5(16)
Panthi (2006)	$\sigma_{cm} = \frac{\sigma_{ci}^{1.5}}{60}$..5(17)

Where σ_{cm} is the unconfined compressive strength of rock mass in MPa , σ_{ci} is the uniaxial compressive strength of the intact rock with 50mm diameter in MPa. RMR is the Bieniawski's rock mass rating, s and a the material constants related to Hoek-Brown failure criterion(the value of a ranges from 0.5 for GSI value of 100 to 0.58 for GSI value 10).GSI is the geological strength indeed, ϒ is rock density (tons/m3), Q_c normalized rock mass quality rating and Q rock mass quality rating. The rock mass strength calculated using the above relations is presented in table 5.9

Table 5-9 Rock mass strength values calculated from empirical formulas

Author	Ch. 0.0 to Adit-I		Adit I-Adot-II		Adit-II to S/shaft.	
	σci (MPa)	σcm (MPa)	σci (MPa)	σcm (MPa)	σci (MPa)	σcm (MPa)
Bieniaswki	58.30	25.97	49.27	20.61	40.25	15.60
Hoek et al	58.30	2.61	49.27	2.20	40.25	1.80
Barton	58.30	8.363	49.27	7.86	40.25	6.29
Panthi	58.30	7.42	49.27	5.76	40.25	4.26
Average Values		11.09		9.11		6.79

All formulas have calculations linked with the UCS value of the intact rock respecting the scale factor effect between the strengths. The values given by Bieniawski are very high whereas Hoek et al's gives more conservative values. Since the values vary from one author to another, choice of the method must be made keeping in view, the region, rock mass properties and the appropriateness to the specific project. The value from Panthis and Bartons formula are in comparable range which is in the middle range. Since every formula gives differing values, for further calculations, the average values of the four is considered.

Affect of weathering on strength

Weathering is a natural process that affects the rock mass. Weathering reduces the rock mass properties like strength, durability and frictional resistance. Panthi (2006) illustrates the reduction effect of weathering on rock mass as given in fig. 5.7.

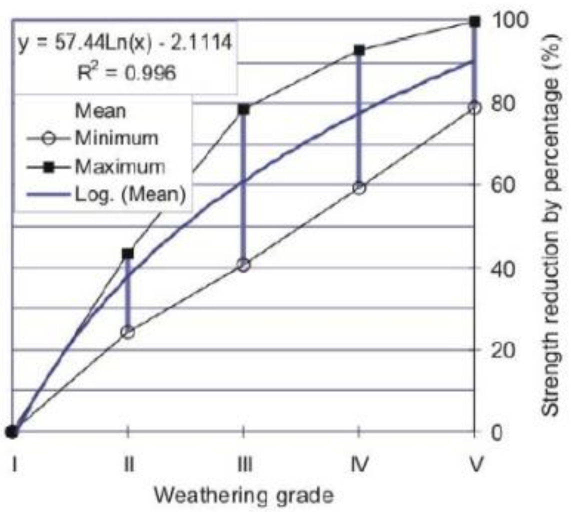


Figure 5.7 Strength reduction by weathering in percent (Panthi, 2006)

Hoek-Brown constant m_i

Hoek-Brown constant, m_i is computed from the statistical analysis of a set of triaxial tests. When laboratory test are not available for the rock mass, material constant mi are used from table 5.10

proposed by Hoek. The range of values quoted in table 5.10 for each material depends on the granularity and interlocking of the crystal structure, the higher values being associated with tightly interlocked and more frictional characteristics.

Table 5-10 Material constant mi values from Hoek

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates* (21±3) Breccias (19±5)	Sandstone 17±4	Siltstone 7±2 Greywackers 18±3	Claystones 4±2 Shales (6±2) Marls 7±2
	Non clastic	Carbonates	Crystalline limestone (12±3)	Sparitic limestone (10±2)	Micritic limestones (9±2)	Dolomites (9±3)
		Evaporites		Gypsum 8±2	Anhydrites 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)	Slate 7±4
IGNEOUS	Plutonic	Light	Granite 32±3	Diorite 25±5		
			Grandorites (29±3)			
		Dark	Gabbro 27±3 Neorite 20±5	Dolerite (16±5)		
	Hypabyssal		Porphyries (20±5)		Diabase (15±5)	Peridotite (25±5)
	Volcanic	Lava		Rhyolite (25±50 Andesite 25±5	Dacite (25±3) Basalt (25±5)	Obsidian (19±3)
		Pyroclastic	Agglomerate (19±3)	Breccia (19±5)	Tuff (13±50)	

*Conglomerates and breccias may present a wide range of mi 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 the intact rock specimen tested normal to bedding or foliation. The value of mi will be significantly different if failure occurs along a weakness plane.

Geological strength index (GSI)

Geological strength index provides a system for estimating the reduction in the rock mass strength σ_{ci} and material constant m_i for laboratory values to appropriate in-situ values under identified geological conditions. GSI is the combination of two fundamental parameters of geological process, the blockiness of the mass and condition of discontinuities. It respects the main geological constraints that govern formation and is thus estimated from visual examination of the rock mass exposed in surface excavation such as road cuts, in tunnels faces and in borehole core (Hoek and Marinos, 2000). Standard GSI table is presented in Figure 5.8.

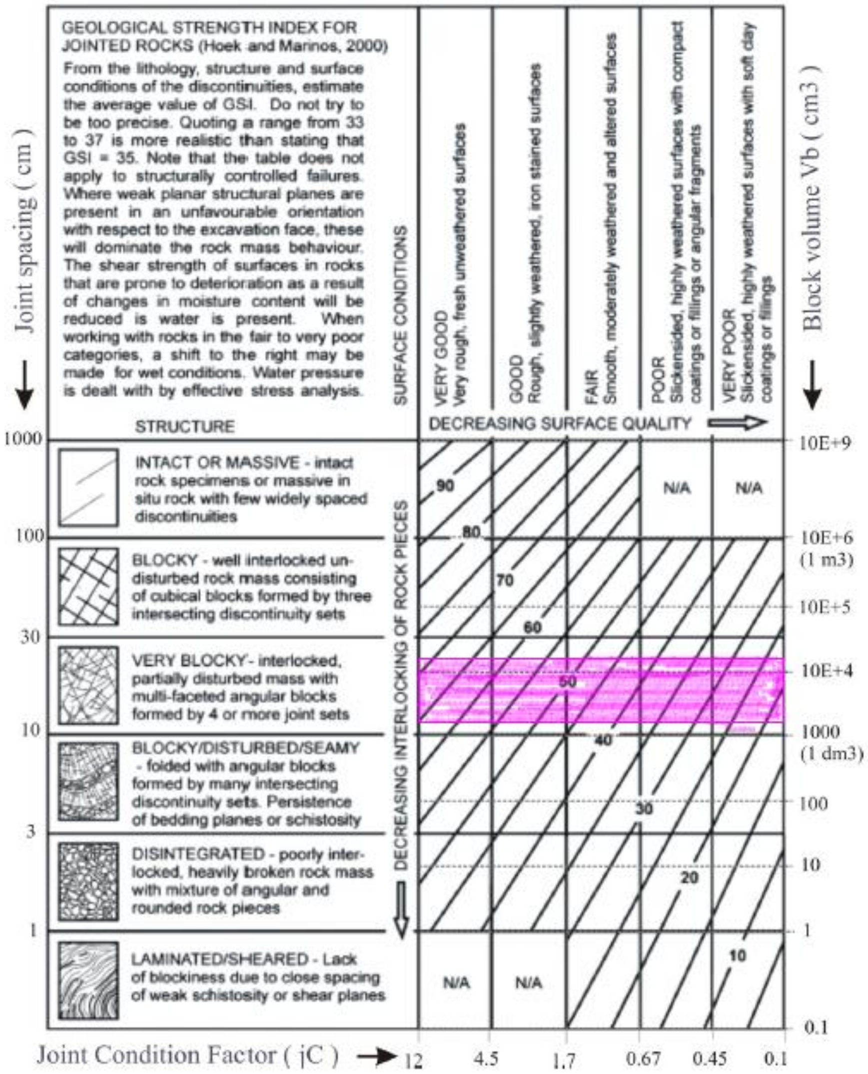


Figure 5.8 Geological strength Index for jointed rock mass (Hoek & Marinos, 2000)

The GSI values can be correlated with Rock Mass Rating (RMR) using the relation

$$GSI = \mathrm{RMR} - 5$$

Deformation modulus

Jointed rock mass do not behave elastically which nesseciated the use of modulus of deformation E_m rather than modulus of elasticity (E_{ci}). According to ISRM (1975), E_m is the ratio of stress to corresponding strain during loading of rock mass under elastic and inelastic behavior and E_{ci} is the same ratio under elastic limits. It can be measured directly in the field or in the laboratory but is time consuming and costly. Even otherwise, the laboratory results differ as high as 100%. Therefore it will save time and money if the values are calculated empirically. Various empirical equations proposed by different authors are presented in table.5.9

Table 5-11 Empirical formulas used for calculating deformation modulus

Proposed by	Empirical relation.
Bieniawski (1978)	$E_m = 2 * RMR - 100$ (GPa) for RMR >50…….5(18)
Barton (2002)	$E_m = 10x(\frac{Qx\sigma_{ci}}{100})^{1/3}$ (GPa) ………………… 5(19)
Hoek and Brown (1997)	$E_m = \sqrt{\frac{\sigma_{ci}}{100}} x 10^{((GSI-10)/40)}$.................................5(20)
Panthi (2006)	$E_m = \frac{E_{ci}}{60} x \sigma_{ci}^{1.5.}$...5(21)

The results using the above relations are calculated and presented in table 5.12

Table 5-12 Deformation modulus values calculated using above formulas

Author	Ch. 0.0 to Adit-I		Adit I-Adot-II		Adit-II to S/shaft.	
	Eci (GPa)	Em (GPa)	Eci (GPa)	Em (GPa)	Eci (GPa)	Em (GPa)
Bieniaswki	6.6	61.2	5.5	58.2	4.4	54.5
Hoek et al	6.6	5.7	5.5	5.3	4.4	4.7
Barton	6.6	6.5	5.5	6.1	4.4	4.9
Panthi	6.6	0.8	5.5	0.6	4.4	0.5

The calculated results by different authors vary considerably from each other. The relation given by Bieniawski gives very high values for rock mass with GSI vales above 60. The deformation modulus in the range one tenth of the intact rock mass value can be acceptable. But the relations give very different values. The result given by Panthi (2006) is considered here, especially since his is the outcome of compilation of many results from the Himalayas region. As such in the further calculations the modulus of deformation given by Panthi's relation is considered.

5.2.4 Water leakage problems

Water occurs in the rock in different forms. Ground water is the most common way of occurrence of water in the rock mass. Ground water is the unconfined water that occurs below the ground water table in the rock mass. The other ways in which water may occur are as:-

- Chemically bonded water to the crystal structure eg. Gypsum ($CaSO_{4\,X}2H_2O$)
- Absorbed water by crystals structure in some minerals. eg. Smectite and
- Capillary water in thin fissures and pores.

The occurrence of freely movable ground water in the underground in presence of joints and discontinuities creates seepage problem during construction, and water leakage problem during operation of tunnels. Since the degree of discontinuity differs in the rock mass, the water seepage problem too will vary from rock mass to rock mass.

Estimation of seepage and water leakage in underground tunnels

The presence of discontinuities makes the seepage problems even worse. To overcome this problem, it is important to estimate the water leakage into and from the tunnels. It's very difficult to predict the occurrence and location of seepage in the underground but there are different methods devised to estimate the quantity of seepage water in the underground rock mass.

One such technique is developed by Tokheim and Janbu(1984). This method was initially developed for evaluating the potential air loss in the unlined compressed air cushion surge chamber in jointed rock. However it was also found well suited for estimating the water leakage. The relation is given by

$$G_w = \frac{2 * \pi * K * L * P.}{\mu_w * G} \text{..} 5(22)$$

Where Q_w inflow rate, k is specific permeability (m2), L length of tunnel/cavern (m), p potential active head, μ_w dynamic viscosity of water (kg/m) = Density X cinematic viscosity and G geometry factor.

Geometry factor describes the flow pattern relatively to the geometry of the tunnel and is given by

$$G = \frac{\ln(2D - r) * (L + 2r)}{r[L + 2r(2D - r)]} \text{..} 5(23)$$

D is distance between center line of excavation line and the ground water table, r equal radius. i.e radius of cylinder with surface area equal to that of actual excavation.

In absence of the required parametric values, the quantity of possible ground water could not be calculated for the Punatsangchu-II project. It may be measured during the excavation time.

Lugeon Test

Lugeon test is another such method to test the leakage problem and it is described below.

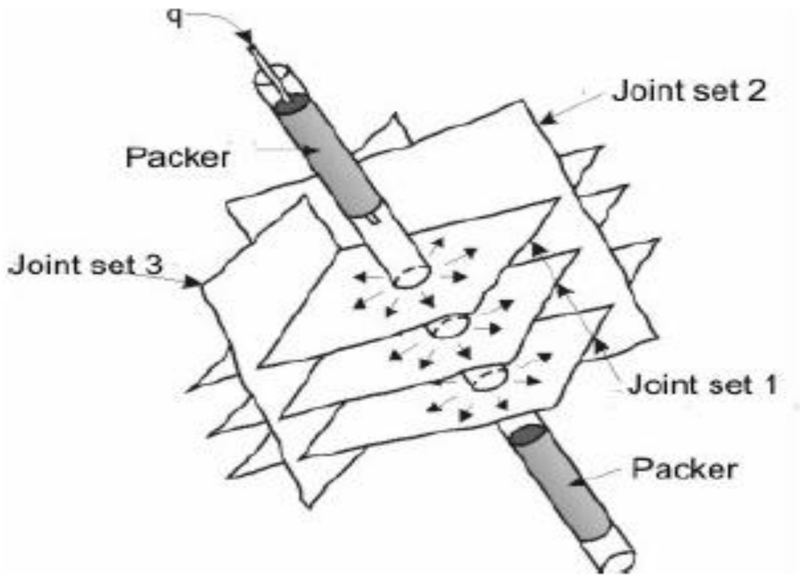


Figure 5.9 Figurative presentation of Lugeon test

The lugeon test is carried out in the bore holes drillings. The bore holes are divided into sections of 1 m length by providing packers (see fig. 5.9). The test is carried out from the lowest to the upper most section. Once the holes are compartmentalized by the packers, water is pumped into the section of borehole measuring 1m length under pressure of 1 MPa(10 bars). 1 Lugeon is the quantity of water lost through 1m length of hole in one minute under 1MPa pressure. The presence of high seepage water in the tunnels has negative effect on the tunneling works. It reduces the tunneling advance rate and increases the instability in the tunnels.

The prediction of water leakage in the tunneling works is important. It helps to plan preventive support measures. The HRT alignment for Punatsangchu II project is intersected by two nalas with numerous shear zones and prominent joints, as such seepage related problems can be expected in this project. To counter leakage problems, provision of drainage holes are provided in the design during construction. Consolidation and contact grouting are provided throughout the HRT alignment. However, apart from the drainage holes, there is no mention of probe drilling and pre consolidation grouting ahead of excavation in seepage zone. This is very important since there are two nalas crossing the tunnel alignment; besides the rock cover is less and has some major joints in these reaches. Panthi (2006) has done a comparative study for effectiveness of grouting in the tunnels in Nepal and found pre injecting grouting to be more effective than post injection grouting in dealing with seepage problems. Therefore provision for probe drilling and pre consolidation should be done in areas where seepage problems are expected.

5.2.5 Tunnel squeezing

Rock mass is heterogeneous medium and rock mass properties vary within space. When tunneling through poor rock medium dominated by poor schistose and fractured rock mass, the strength of the rock mass is less compared to the tangential stresses exerted in the rock. This results in formation of micro cracks along the schistocity or foliation plane. As a result, a visco-plastic zone of micro fractured rock mass is formed deeply into the wall as shown in fig. 5.10 and the induced maximum tangential stresses are moved beyond the plastic zone (Panthi, 2006).

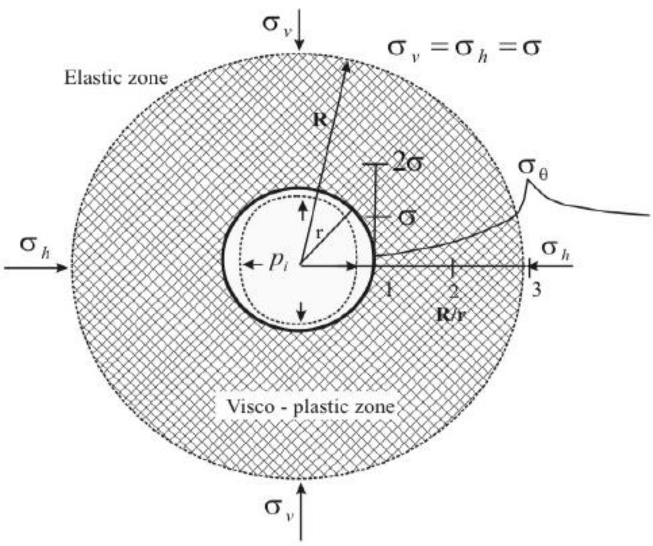


Figure 5.10 Illustration of squeezing in circular tunnels based on Bray, 1976 (Panthi, 2006)

σh= σh=σ is the normal stresses in the rock mass. R is the radius of visco- plastic zone and r is the tunnel radius. Pi is the support pressure. As a result of time dependent inward movement of the rock mass, the support material will be subjected to high stresses and sometimes when the support fails to sustain the large deformation caused by high tangential stresses, the tunnels squeezes inward reducing the size of opening. In extreme cases, new equilibrium are reached after the complete closure of tunnel, eg Kovari in 1998 and Steiner 1996 (Panthi, 2006). Many authors have developed different approaches to predict tunnel squeezing. Hoek and Marinos approach is discussed below.

Hoek and Marinos (2000)

Over burden alone is not responsible for squeezing. The changing strength and deformability properties of the rock mass over time have far greater consequences on squeezing (Kovari, 1998). In Hoek and Marinos approach, rock mass strength and over burden are the two parameters considered responsible for squeezing. With this view, Hoek and Brown (2000) have suggested a relation that gives total strain (ratio of tunnel closer to tunnel diameter), which is a function of rock mass strength and in-situ overburden pressure. The criterion is based on iso-static circular stress condition.

The support pressure can be calculated using different equations. The RMR method and Unal method are used here.

The rock support pressure from the RMR relation.

$$P = \frac{100 - RMR}{100} * \rho * Dt \quad\text{..5(24)}$$

The support pressure can also be calculated using equation proposed by Unal (1983) as shown below.

$$Pi = (100 - RMR) * \gamma * \frac{B}{100} \quad\text{...5(25)}$$

Where ϒ is rock density in t/m3, and B is tunnel width in m. accordingly the deformation in the tunnels were calculated first without support pressure and later with support pressure. The rock support pressure is tabulated in table 5.13.

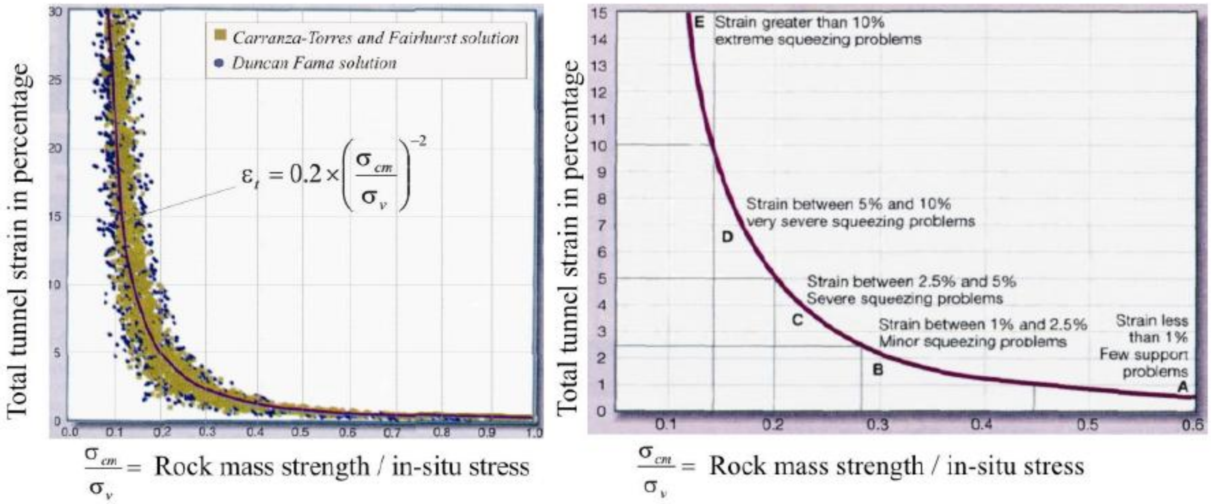


Figure 5.11 Tunnel convergence and degree of difficulties associated with tunnel squeezing (Hoek & Marinos, 2000)

Hoek and Marinos defined the plastic zone (R) and total tunnel strain (ε_t) by following equations.

$$R = r x \left(1.25 - 0.625 x \frac{p_i}{\sigma_v}\right) \left[\frac{\sigma_{cm}}{\sigma_v}\right]^{\left(\frac{p_i}{\sigma_v} - 0.57\right)} \quad\text{……………………..…..5(26)}$$

$$\varepsilon_t = \frac{\delta_t}{2r} x100 = \left(0.2 - 0.25 x \frac{p_i}{\sigma_v}\right) \left[\frac{\sigma_{cm}}{\sigma_v}\right]^{\left(2.4 x \frac{p_i}{\sigma_v} - 2\right)} \quad\text{………………5(27)}$$

Where δt is total inward deformation and εt is total inward strain. Pi is the support pressure in MPa. When the support pressure is taken as zero, the squeezing condition in the rock mass may be rewritten as

$$R = 2.5 x r x \left(\frac{\sigma_v}{\sigma_{cm}}\right)^{(0.57)} \quad\text{…………………………………………………5(28)}$$

$$\varepsilon_t = (0.2)\left[\frac{\sigma_{cm}}{\sigma_v}\right]^{-2} \quad\text{………………………………………..5(29)}$$

Hoek ans Marinos have assumed that weak rock mass are incapable of sustaining significant differential stress and failure will occur till in-situ horizontal and vertical stresses are equalized. This justifies why they considered over burden instead of tangential stresses, which is always greater than over burden pressure for estimating tunnel squeezing. For defining the approximate degree of difficulty at different level of tunneling, the figure 5.11 right is used.

The squeezing results from the Hoek and Marinos relations for the different reaches along the HRT alignment is given in table 5.13. The squeezing chart states that, degree of squeezing will be with few support problems when strain is less than 1%. Minor squeezing problem when strain is between 1 %and 2.5%. When the strain exceeds 2.5% but less than 5% there can be severe squeezing problem in the rock mass. The tunnel will face extreme squeezing when the strain exceeds 10%.

Table 5-13Support pressure and deformation along the HRT alignment

Description	Ch.0 to Adit I	Adit I to Adit II	Adit II to S/shaft
Support pressure MPa	6.23	6.68	7.23
Deformation without support pressure %	0.1	0.33	0.22
Deformation with support pressure %	0.003	0.06	0.05

From the results, very less squeezing problems is expected along the HRT. All the possible squeezing can be solved by adopting simple flexible support measures. However, since all these input parameters are calculations based on rock mass parameters on the surface, it should not be a reason to relax. There should be adequate readiness for any worsening squeezing problem since the rock mass is intersected by numerous joints. The HRT alignment crossing two nala along its layout should also be cautionary tunneling reach. However, till the actual tunneling and its results are obtained, there is no severe squeezing problem. Tunneling design is a dynamic process and the design is best done by considering the actual conditions at the face, so this approach has to be taken and appropriate support designed during construction stage. It is stated that the values of the support pressure is very high since it is calculated from the RMR method. The support pressure given by RMR formula is unrealistically high which no support combination can possibly achieve. In actual cases, the support pressure should be within 2.5 MPa.

5.3 Numerical methods

The rock mass is very complex medium. Its material properties and other discontinuities keep changing even within very short stretch. Since the input parameters are numerous and vary a lot, numerical analysis method of analysis becomes very cumbersome and time consuming; it is therefore more convenient to use fast computers for the same analysis. Numerical method gives quantitative assessment of the problems. Due to the advantage of speed and space, it can be used for testing the results with varying conditions of input parameters giving a better understanding of the mechanism in the rock mass. Further it can be used to verify the traditional thumb rule applications. Due to the availability of space and speed, it can be used to find results for extended conditions.

5.3.1 Types of numerical models

The numerical methods can be fundamentally divided into continuous modeling and discontinuous modeling.

Continuous models: - In the continuous model, the rock mass is treated as a continuous medium and includes very minimum of discontinuities. In continuous models we have three different models namely 1) Finite Element Method (FEM), 2) Finite Difference Method (FDM) and 3) Boundary Element Method (BEM).

Discontinuous models: - In the discontinuous model, the rock mass is treated as a discontinuous material with individual blocks free to rotate, translate and interact along its boundaries. Some of the software available in discontinuous model is 1) Distinct Element Method (DEM) and 2) Discontinuous Deformation Analysis (DDA).

The Phase2 numerical modeling is discussed in chapter 6

5.4 Conclusion

The design of underground structures involves the use of all three methods of empirical, analytical and numerical methods. The empirical methods does not give stress distribution and deformation around the tunnels, but due to the simplicity and ease for use, its preferred by the field engineers.

The analytical and the numerical methods are dependent upon the strength parameters of associated rock masses that are the input parameters derived from the field observations and empirical results. Therefore the reliability of the results of analytical and numerical methods is only as reliable as the input parameters fed from the field and empirical calculations.

It is therefore more appropriate to use all the three methods for the design and better results. Laboratory tests are also important input for any of the above calculations.

From the different stress calculation results, it can be said that the middle reach of the HRT alignment is more susceptible for squeezing problem. Since the tangential stress in the roof and walls are higher than the rock mass strength, stability problems can be expected in roof and walls as such required supports should be provided to meet such challenges.

6 ROCK SUPPORT ESTIMATION

The in-situ rock mass are subjected to different stresses originating from gravitational, topographic and tectonic stresses. These stresses cause instability to the underground structures. The scale of instability is higher in the Himalayan geology where the tectonic activity is high. Rock supports are provided to create safe working space during the construction, and stable structure for the long time operation of the tunnels and caverns. The degree of the support system varies depending on the purpose of structures, technical and financial capability of the projects. Different types of support can be provided to counter the instabilities. This chapter discusses on the different types of support systems and design procedures

6.1 Types of support system

Rock support is provided to improve safety and stability of unstable underground openings. Different types of support are adopted based on the nature of stability problems. There cannot be a single standard support for any stability problem. It has to be customized depending on rock mass parameters, technology, time and money available. The best support system can be one which provides reasonably best stability within minimum time and cost. Some of the commonly adopted support systems in tunnel stability problem are discussed below.

6.1.1 Scaling

Scaling involves removal of disintegrated loose hanging rock fragments triggered by vibration impact of blasting around the tunnel periphery. It is carried out immediately after every cycle of blast at the face of the tunnel. Normally 20-30 minutes is required to remove the loose fragments rocks. Different methods of scaling are

Manual scaling

Manual scaling is one of the oldest methods of scaling. The scaling is done from the piles of blasted material or from wheel loaders as a platform. It is risky and has poor safety standards. It is not much in use these days except in small drifts and shafts where machines cannot be used. Figure 6.1 and 6.2 illustrates manual scaling.

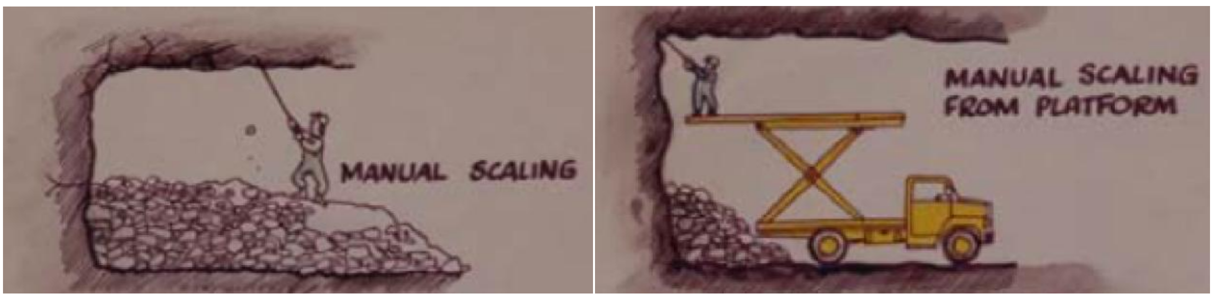


Figure 6.1 Mannual scaling from muck pile (L) & from scissor plateform (R)

Mechanical scaling

The scaling is done using the tunneling rig machines. This has improves safety standards and consumes less time. In Bhutan, mechanical scaling is most widely practiced. Figure 5.2 illustrates how mechanical scaling is done.

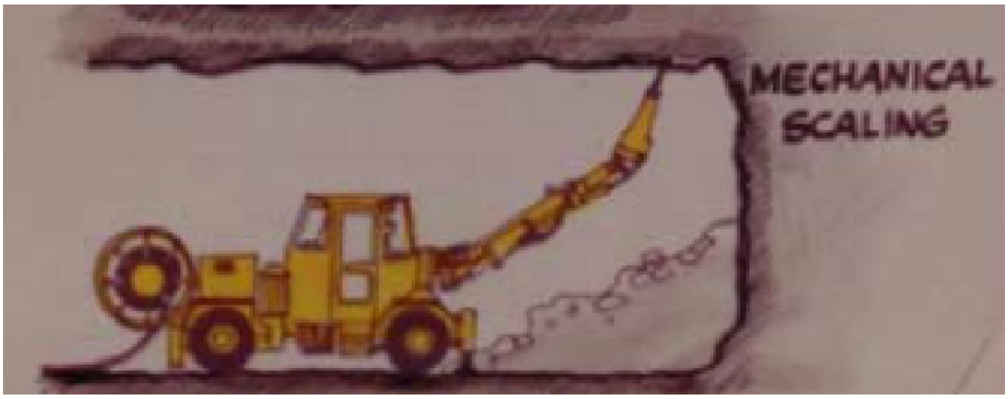


Figure 6.2 Mechanical scaling using drilling rigs

6.1.2 Rock bolts

Rock bolting is a flexible rock support which often forms the first line of support. The rock bolting material stitches disintegrated loose fragmented rocks to the surrounding parent rock mass. It is applied both as temporary and permanent support. Rock bolts can be classified with different names based on its function, type and etc. some of the rock bolt classifications are discussed below. An experienced construction engineer can make a sound judgment how and where to install bolts according to the orientation of foliation and jointing.

Based on end anchoring

End anchorage provides early strength to the end length of rock bolts with its surrounding rock mass. Different types of mechanism for bolt anchoring are devised. Some of the most commonly methods in use are resin capsules, cement capsules or grout and expansion shell type.

Resin capsules give accelerated strength to the bolts. Normally resin capsule is provided in last one third of the bolt length. . However the experts have discouraged the use of resin capsule in squeezing ground condition

End anchorage is also provided by use of cement capsule or by cement grouting of the rock bolt. Cement grouted rock bolt is better option in squeezing condition. But grouting the rock bolt in inclined reach is difficult. Specific care should be taken so that the area between the bolt and drilled space is filled with cement grout.

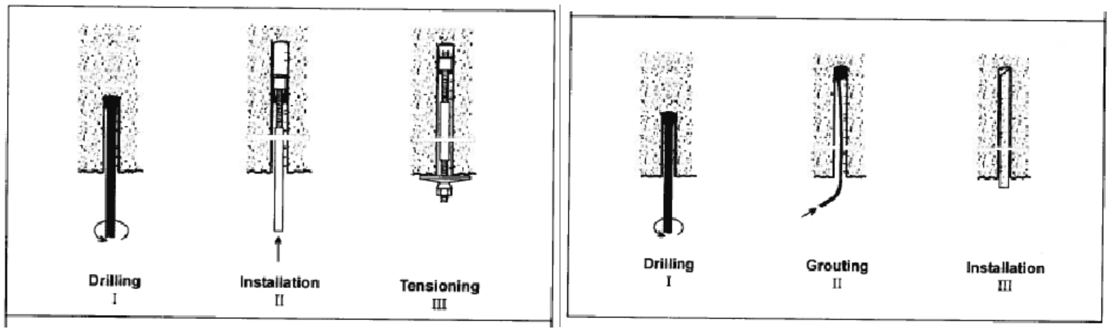


Figure 6.3 Principal of installing expansion shell and grouted rock bolts

Based on bolt spacing

When the excavated rock face is competent with very little jointing and weakness zones, spot bolting is provided only in localized area. A figure 6.4 shows spot bolting.

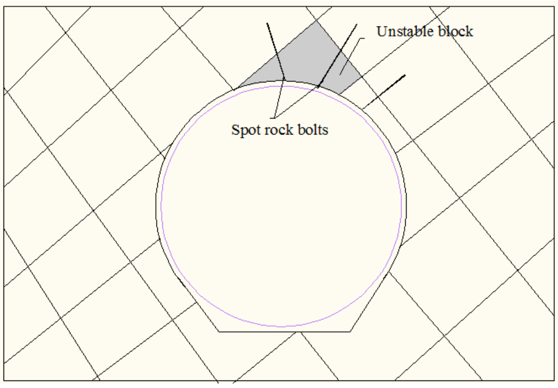


Figure 6.4 Illustration of spot bolting

When the rock mass is poor, a systematic bolting is provided throughout the cross sectional of the tunnel. The spacing and the size of the bolting across the section will depend on the rock mass parameter. Fig. 6.5 shows an example of pattern bolting. The pattern bolting may be provided only in the crown, walls or the entire section of the tunnels depending on the rock mass.

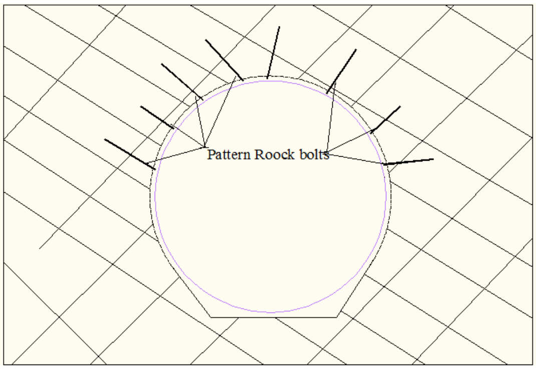


Figure 6.5 Systematic pattern bolting

Other types of bolts

Friction bolts are those bolts which use the friction between the bolt length and the surrounding rock mass to take the load of the instable rock blocks. swellex bolts and split bolts are the examples of this type of bolts.

Spilling bolts are used at the junctions and portals of the adits where the rock mass is poor. They are installed ahead of the tunnel opening. The diameter and the length of the bolts vary from 20mm to 32m and 4 m to 6m respectively.

6.1.3 Shotcrete

A matrix of cement and fine aggregates is prepared and sprayed on the rock face. This matrix can be sprayed either in dry form or mixed with water and sprayed as wet shotcrete. Depending on the rock mass character, shotcrete can be used either as temporary or permanent support system. It can be used alone or in combination with rock bolts, wiremesh and concrete lining. The thickness of shotcrete can vary from few centimeters to 10-25 centimeter depending on the type of the rock mass. Due to the safe working condition, wet shotcrete is mostly preferred for used in the tunnels.

Few centimeters of shotcrete is preferably used in very weak rock as the first line of support. This can later be combined with rock bolts and other permanent supports systems. Just like the rock bolts, shotcrete can also be applied at localized spots when the rock is good and throughout the profile section when the rock mass is poor.

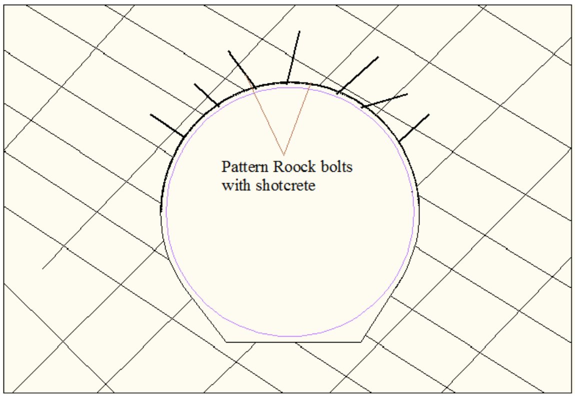


Figure 6.6 Systematic rock bolts with shotcrete

Use of 25mm-30mm long needle sized steel fiber is gaining acceptance as one of the most reliable fast and flexible support system. it reinforces the strength property of the shotcrete. Micro silica is used to improve the workability of the matrix. The experience has shown that use of 1% steel fiber by the volume of concrete in the shotcrete improves the strength immensely.

6.1.4 Steel supports

Structural steels support is used in poor rock condition. The arching action of the structural steel takes the loads and is used as the main support principle. The steel structures can also be prefabricated and placed readily at the site. The space between the steel rib and the rock profile is filled with metal block welded to the steel rib or some concrete blocks to get better contact with the rock mass. The space behind the steel section and excavated profile may be left un-concreted or embedded in concrete lining at later date.

The Norwegian method uses steel reinforcement and fiber reinforced shotcrete as the permanent support. It have been successfully used all over Norway and proved very versatile even in poor rock mass conditions. The thickness and the spacing between the ribs can be adjusted depending on the local condition of the geological strata. This method is found fast and cost effective. Fig. 6.7 illustrates the principle of Norwegian method.

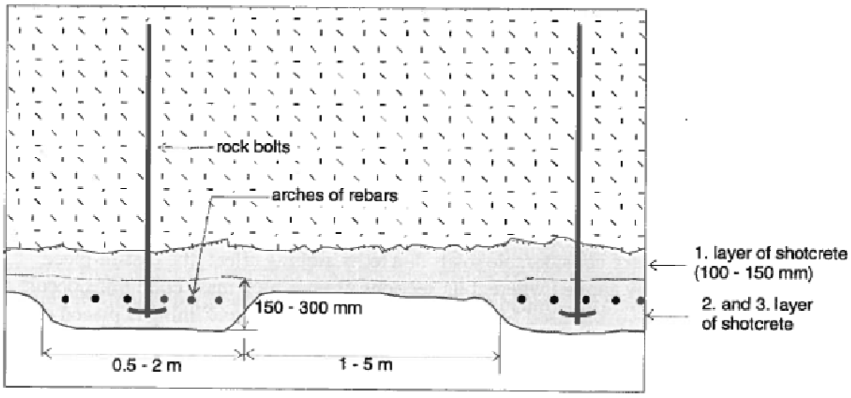


Figure 6.7 Norwegian method of support

6.1.5 Cast in place concrete

It is permanent support system mostly used in poor and extremely poor rock mass. The concrete is cast at site parallel with excavation. This can take considerable load compared to any other supports. But it is also most time consuming and expensive methods of support. The cost increases when structural steel sections are used in extremely poor rock strata. Now there are prefabricated form works which can be readily installed at site making concreting work fast and cost effective. Figure 6.8 shows the concrete lining supports.

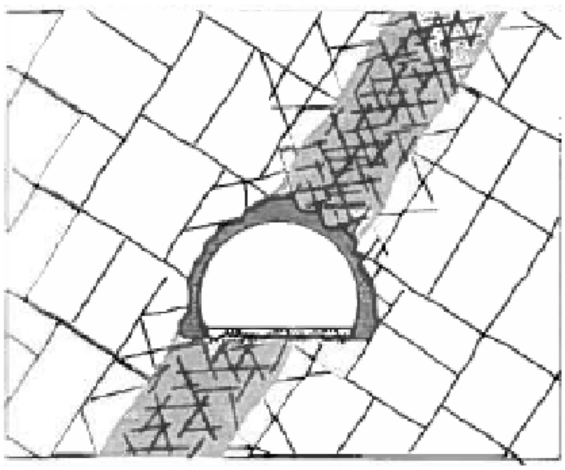


Figure 6.8 Steel rib support with back fill concrete

Concrete lining with heavy structural steel sections form the last support system in extremely poor rock strata. If this cannot provide stability, then the tunnel alignments may have to be realigned to get better rock condition. Normally probe drilling are done ahead of tunnel excavation and the advance rate reduced with cautionary supports when the poor rock mass are detected.

6.1.6 Grouting.

Grouting is injection of matrix prepared from cement, fine aggregates and water into the surrounding weak rocks. This improves the structural stability of the rock mass by filling the empty voids. There are different grouting methods used in different tunneling conditions.

Pre-injection grouting is pumping grout into the weak rocks mass ahead of excavation. This improves the strength and stability of the rock. Use of pre-injection grouting as permanent support in the Himalayas has been successfully done in Nepal (Panthi, 2006).

In concrete lined tunnels, the concrete shrinks over time due to the heat of hydration. This creates gap between the concrete and the surrounding rock. It leaves room for rock expansion and may experience rock falls in future. In contact grouting; normally 1 foot deep hole is drilled into the rock mass from the excavation line. Grouting is done through these holes eliminating the void created by concrete shrinkage. Contact grouting is done behind concrete lined section and was done extensively in Tala Project and Punatsangchu-I projects in Bhutan.

6.2 Designed support of Punatsangchu-II HRT.

The rock mass classification for the project is done using NGIs Q method and Bieniawskis RMR method. Accordingly the rock mass parameters were calculated from the field observations and rock mass was classified into following classes for the HRT alignment as tabulated in table 6.1.

Table 6-1 Rock mass rating value Q and rock classification (WAPCoS,2011)

Sl. No	Rock Class description	Rock Class	Q value range	RMR value range
1	Very good rock	Class-I	40 – 100	77 - 100
2	Good rock	Class-II	10 – 40	64 - 77
3	Fair rock	Class-III	4 – 10	56 - 64
4	Poor rock	Class-IV	1 - 4	50 - 56
5	Very poor rock	Class V	0.1 - 1	35 - 50
6	Extremely poor rock or squeezing rock	Extremely poor or Squeezing rock	< 0.1	< 35

The support system was worked out based on the above system of rock mass classification. Different support combinations were designed for different values of rock mass rating value Q. The rock mass classification and support system as designed for Punatsangchu-II project by the consultant is presented in table 6.2.

Table 6-2 Different rock class and support design for Punatsangchu II project

Sl.No	Rock class	Excavation diameter (m)	Rock bolts	Shotcrete	Steel section	Concrete lining
1	Class I	12.10	25 **Φ** spot R/B , 5500mm long as required	Local application of 75mm thick shotcrete as required	nil	550mm thick plain concrete
2	Class II	12.10	25 mm **Φ** spot R/B 5500mm long as required	75mm think shotcrete in the upper 120^0 crown area	nil	550mm thick plain concrete
3	Class III	12.15	25mm **Φ** R/B 5500mm long @1500x1500 staggered.	100mm thick shotcrete till the tangent point.	nil	575mm thick plain concrete
4	Class IV	12.25	25mm **Φ** R/B 5500mm long @ 1500x1000 staggered.	125mm thick shotcrete in crown and walls.	nil	625mm thick steel reinforced concerete
5	Class V	12.35	25mm **Φ** R/B staggered @1500x1000	125 mm thick shotcrete in crown and walls.	ISMB 250 @600 c/c in heading area.	675mm thick steel reinforced concerete
6	Extremely poor	12.35	25mm **Φ** R/B staggered @1500x1000	125mm thick shotcrete in crown and walls	ISMB 250 @400 c/c in heading area	675mm thick steel reinforced concerete
7	Squeezing	12.35	25mm **Φ** R/B staggered @1500x1000	150mm thick shotcrete in crown and walls	ISMB 250 @400 c/c in crown and throughout walls.	675mm thick steel reinforced concerete

Provision for 6m long 76mm **Φ** drainage holes were provided during the construction for all rock mass condition. 38mm **Φ**, 300mm deep holes into the rock with alternate spacing of 3m for contact grouting and 38mm **Φ**, 10m deep holes at 3m spacing in alternate sections were provided for consolidation grouting. The figurative representation of support systems is shown in the figures from fig. 6.9 to fig 6.11.

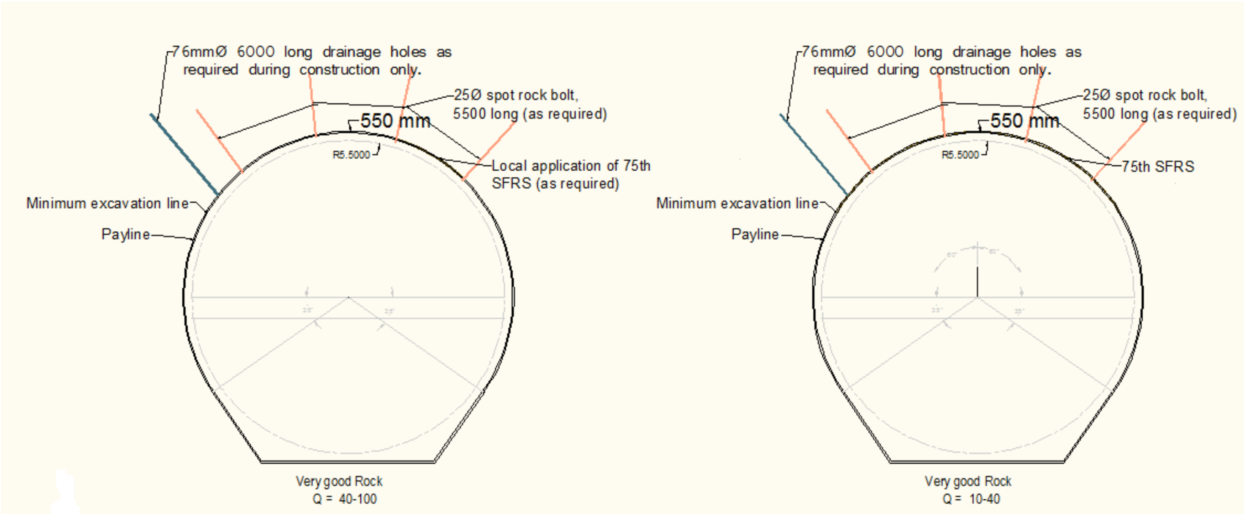


Figure 6.9 Rock support system for rock class I & class II

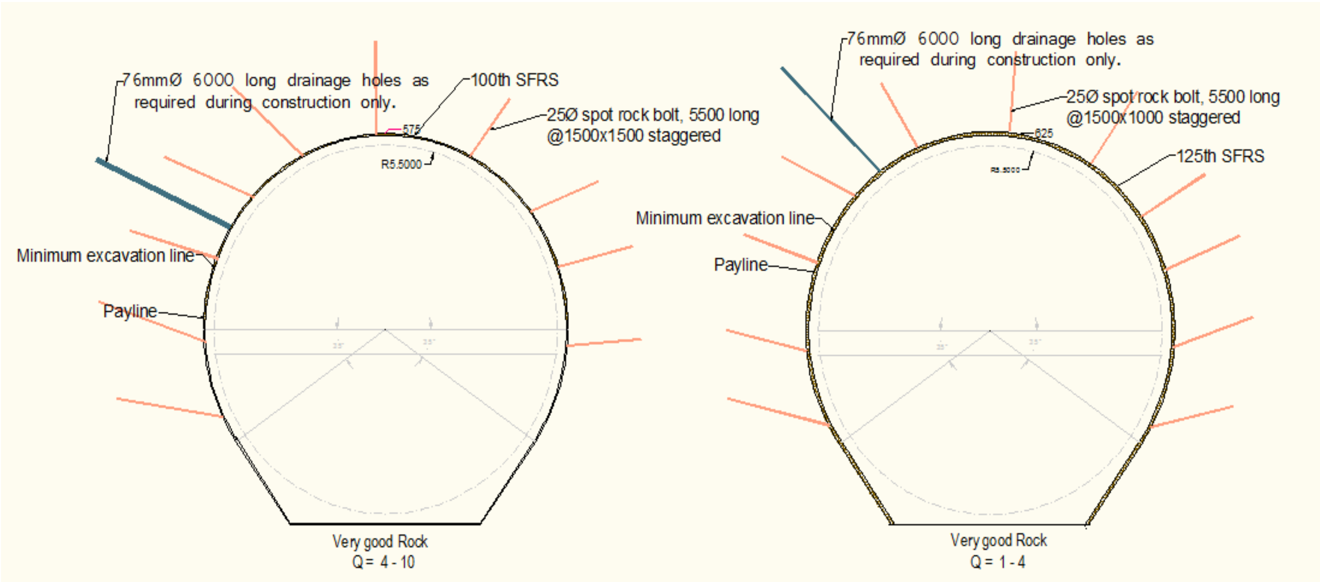


Figure 6.10 Rock support system for rock class III & class IV

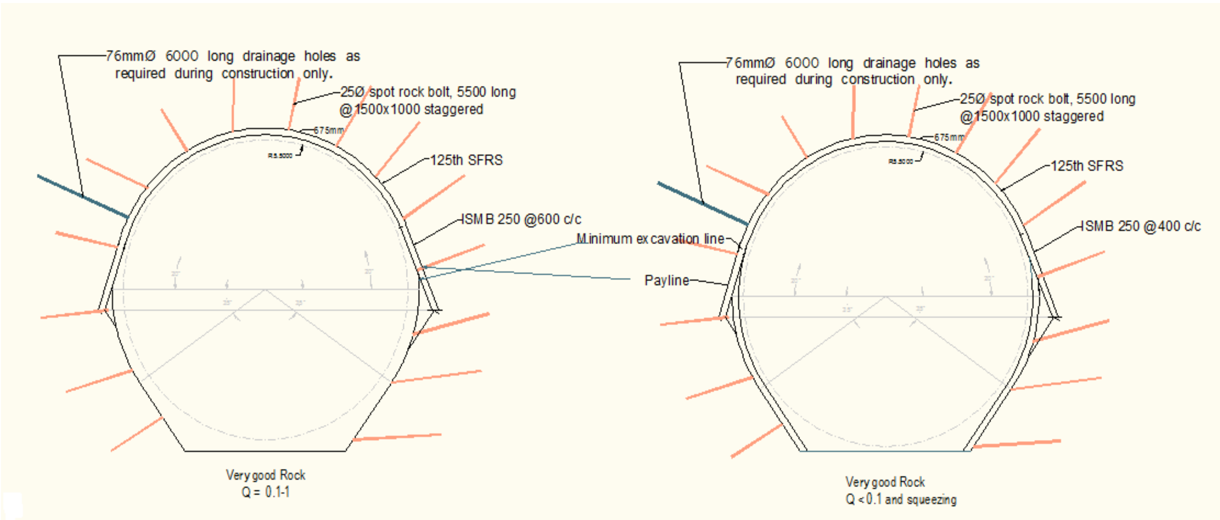


Figure 6.11 Rock support for rock class V & extremely poor to squeezing condition

6.3 Methods of rock support design

There are different types of rock mass classification systems practiced at different countries and regions around the world. Each of these rock mass classification systems has its own approach towards the design of support system. Some of the approaches and methods are widely followed in certain countries and regions while few have gained acceptance internationally. The two methods which have international recognition and acceptance are NGIs Q method and the BIeniawskis RMR method. These are the two methods of rock mass classification used for Punatsangchu II project. As such these two methods of rock support design is discussed in this section.

6.3.1 NGIs Q method

The procedure for the classification of the rock mass is discussed in the chapter 5. From the results of field inspection, the ratings for the different parameters of the rock mass are assigned to different lengths or reaches along the tunnel alignment. Based on results of the six parameters, the rock mass is classified into different classes as discussed in chapter 5. This rock mass classification is used in combination with other properties of the tunnel.

Excavation support ration

There are different types of tunneling works carried out for different purposes. Each work, based on their purpose and stability requirement are assigned with different rating values called the “excavation support ratio” (ESR). In general, the total tunneling works are classified under six different rating values (Barton et al. 1974) which is reproduced in Appendix B.

Equivalent dimension

The equivalent dimension (De) is the function of the size and type of tunnel excavation. It is the ratio of the tunnel span, diameter or the wall height of the tunnel excavation to the excavation support ratio (ESR).

$$De = \frac{Dt}{ESR} \qquad \text{6(1)}$$

After the rock mass parameters are defined, the rock is classified into different classes using their Q values. Then excavation support ratio (ESR) of the tunnel is defined and the equivalent dimension (De) calculated. The NGIs standard support chart developed by Grimstad & Barton in 1993 is used for support design. This chart gives the different combination of support system depending on the different values of the Q and ratio of De/ESR. The best optimum choice of support from these different combinations can be adopted for the project. The support chart is given in figure 6.12.

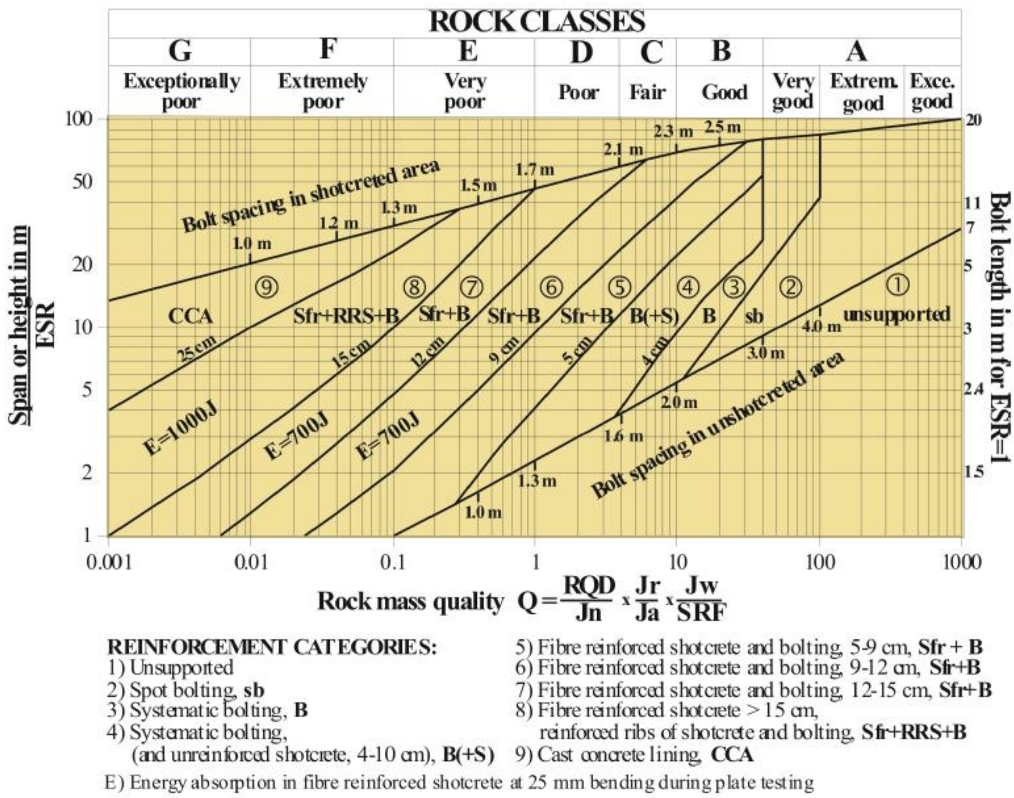


Figure 6.12 NGIs support chart (Grimstaed & Barton, 1993)

Support system for Punatsangchu II project using NGIs Q method

The rock mass classification along the HRT varies from class II to class V with Q values ranging from as small as o.3 to 6. The different class of rock for the HRT is separated into different regions on the standard support charts as shown in figure 6.13 for class I to class VII.

The excavation support ratio for hydropower tunnels is 1.6 which is from standard excavation support ratio table (Appendix B)

The excavation diameter of the head race tunnel is 6.175m and the excavation support ratio (ESR) of 1.6, the equivalent dimension value is calculated as 3.86. This value is marked on the vertical axis along Y on the support chart.

Length of rock bolts

The typical length of the rock bolts normally used in tunnels are 2-4 m with diameter ranging from 20-25mm. in cavern the bolt length normally used is 6m with 25-32 mm diameter. The Norwegian tunneling uses the following expression to find bolt length.

$Lb = 1.4 + 0.184\ Dt$ ………………………….6(2)

Lb = 3.7m, where Dt is the diameter of the tunnel. However, the ideal way of estimating the bolt length is from the rock mass conditions especially their block size in the field. Palmstrom (2000) has suggested following expression for estimating the bolt lengths for roof and walls of tunnels.

$$Lb_{roof} = 1.4 + 0.16Dt(1 + \frac{0.1}{Db}) \quad \text{.......} 6(3)$$

$$Lb_{wall} = 1.4 + 0.08(Dt + 0.5Wt)(1 + \frac{0.1}{Db}) \quad \text{.......} 6(4)$$

Where, block diameter value of $1m$ is considered and Wt the tunnel wall height is ($\approx 6m$). According to these relations, the bolt length calculated is $\approx 3.6m$ for the roof and $\approx 2.75m$ for the wall. The values for Db, Wt and Dt used is 1m, 6m and 12.35 m respectively.

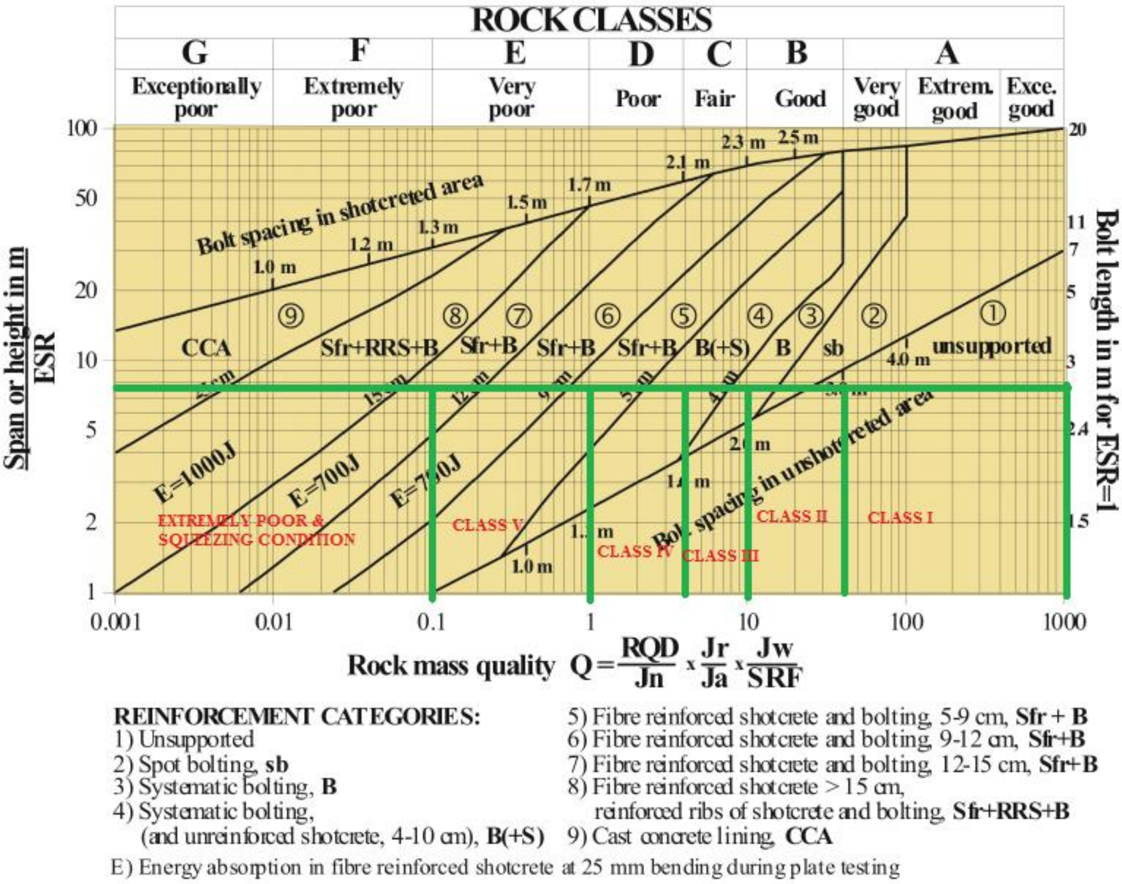


Figure 6.13 Rock class & support combinations for Punatsangchu II according to Q method

Based on the combination of the above calculations and the procedures, the final support system is designed using the standard support chart. The different support combinations arrived accordingly is presented in Table 6.3

However, it has to be noted that the above support procedures are for the total temporary and permanent supports in the roof only. For the walls, the height of the walls may be considered and slight modification for the Q values as given below may be adopted. Rest of the procedures remains same.

For Q > 10 use Qwall = 5Q

For Q 0.1 < Q < 10 use Qwall = 2.5Q

For Q < 0.1 use Qwall = Q

Table 6-3 Support combination for Punatsangchu II HRT based on Q method

Rock class	Q value	Support as per NGI system
Class I	40-100	Unsupported
Class II	10-40	Spot bolting at localized areas to systematic R/B with maximum spacing @2000x2000 and no shotcrete.
Class III	4-10	Systematic R/B with maximum spacing of @2500 c/c and plain shotcrete with thickness from 40mm to100mm.
Class IV	1-4	Systematic R/B @2500 c/c with 40mm plain shotcrete to fiber reinforced shorcrete with maximum thickness of 90mm depending on the Q value.
Class V	0.1-1	Systematic R/B with spacing varying from 2500to 1700 c/c. in addition minimum 40mm thick plain shotcrete to maximum of 150mm thick fiber reinforced shotcrete depending on the Q value.
Extremely poor	<0.1	Systematic R/B with spacing from 1700 to1000 c/c. in addition shotcrete support varying few mm thick of plain shotcrete to 150mm thick fiber reinforced shotcrete with concrete lining depending on the Q value.
Squeezing	< 0.1	Systematic R/B with spacing from 1700 to1000 c/c. in addition shotcrete support varying few mm thick of plain shotcrete to 150mm thick fiber reinforced shotcrete with concrete lining depending on the Q value.

The size of the rock bolt is 4m long with 25mm diameter

6.3.2 RMR method of support design

The rock mass classification based on the RMR system is discussed in previous chapter. Once the classification is complete, the RMR methods give only the guide for excavation and rock support in tunneling. The support system described by RMR method does not have a comparison support chart like the Q method. The supports guide can at best be used as a tool to check rather than a standard for support design. The excavation guide and support system as given by Bieniawski (Bieniawski 1989) is as presented in table 6.4 below.

Table 6-4 Support combination for Punatsangchu II based on RMR method

Shape: Horse shoe; width: 10m; Vertical stress: below 25 MPa; Excavation by drill & blast.

Rock Mass class	Excavation	Support: Rock bolt (20mm dia, fully grouted)	Support: Shotcrete	Support: Steel sets
1 V.good rock. RMR: 81-100	Full face: 3m advance	Generally no support required except for occasional spot bolting		
2 Good rock: RMR : 61-80	Full face 1.0-1.5m advance; complete support 20m from face	Locally bolts in crown, 3m long, spaced 2.5m with occasional wire mesh.	50mm in crown where required.	None
3 Fair rock ; RMR: 41-60	Top heading and bench: 1.5- 3m advance in top heading; commence support after each blast; commence support 10m from face.	Systematic bolt 4m long; spaced 1.5-2m in crown and walls with wire mesh in crown.	50-100mm in crown and 30mm in sides.	None
4 Poor rock ; RMR : 21-40	Top heading and bench; 1.0-1.5m advance in top heading; install support concurrently with excavation 10m from face.	Systematic bolt 4-5m long; spaced 1-1.5m in crown and walls with wire mesh.	100-150mm in crown and 100mm in sides.	Light ribs spaced 1.5m where required
5 very poor rock; RMR < 21	Multiple drift; 0.5-1.5m advance in top heading; install support concurrently with excavation; shotcrete as soon as possible after blasting.	Systematic bolt 5-6m long; spaced 1-1.5m in crown and walls with wire mesh. Bolt invert.	150-200mm in crown, 150mm in sides and 50mm on face.	Medium to heavy ribs spaced 0.75m with steel lagging and forepoling if required. Closed invert.

6.4 Comparison and comments on the support systems

The following observations were made from the above two support standard charts and the designed support for the project.

Neither Bartons Q support chart nor the Bieniawskis support guide gives any provision for increased excavation diameter in poor and squeezing condition to compensate for the squeezing. There has to be some change in the excavation size in differing rock condition. This aspect was taken care of in the support system in Punatsangchus case.

The rock mass classification system adopted for the project is from Q and RMR system. But, the supports were on very conservative side. This could be due to the erratic changing nature of the rock mass in the Himalayas.

In the Q methods, the support is mainly focused on the combination of bolting with shotcrete adjusting with spacing in bolting; thickness and fiber reinforcement in the shotcrete, but the RMR do not say anything on reinforced shotcrete but is supplemented by the use of wire mesh. The use of wire mesh is almost obsolete in the tunnels except in inclined shafts excavated manually.

The RMR system can be used only as a guide and not as a design reference. Even the supports in Q system may be very optimistic for the Himalayan geology. The support designed based on this could be in adequate for the Himalayan geology.

The provision of concrete lining for the whole reach of the tunnel may not be required from stability but only to reduce frictional head loss. The Q method specifies concrete lining only in rock class VI and below where squeezing is imminent but it’s provided throughout the length of the tunnel for this project. The possibility of avoiding full concrete lining and replacing by shotcrete lining may be worth a try.

The advantage of self supporting capacity of the rock is not used much effectively. This could be tried in combination with pre consolidation grouting instead of using very expensive concrete lining. This could save lots of time and money.

A very conservative approach of support system was adopted for the project. This may help keep geological problems at bay, but the cost implication can be very high. This could be due to high variability of the rock mass even within short span of tunneling length. This can however be optimized by following a dynamic continuous process of design based on the actual rock mass condition in tunneling during the execution stage.

6.5 Measures to reduce stability problems by shape and size of tunnel opening

The magnitude of the stresses in the underground cannot be changed. It depends on the character of the rock mass and its discontinuities. But influence of stresses on the opening can be reduced by adjusting the size and the shape of the underground opening. Knowing the magnitude and direction of the stresses, we can adjust our opening geometry to minimize the affect of stress in the opening. Some recommendation for the shape and size of the opening which can be adopted in specific condition of stresses is given in fig. 6.14.

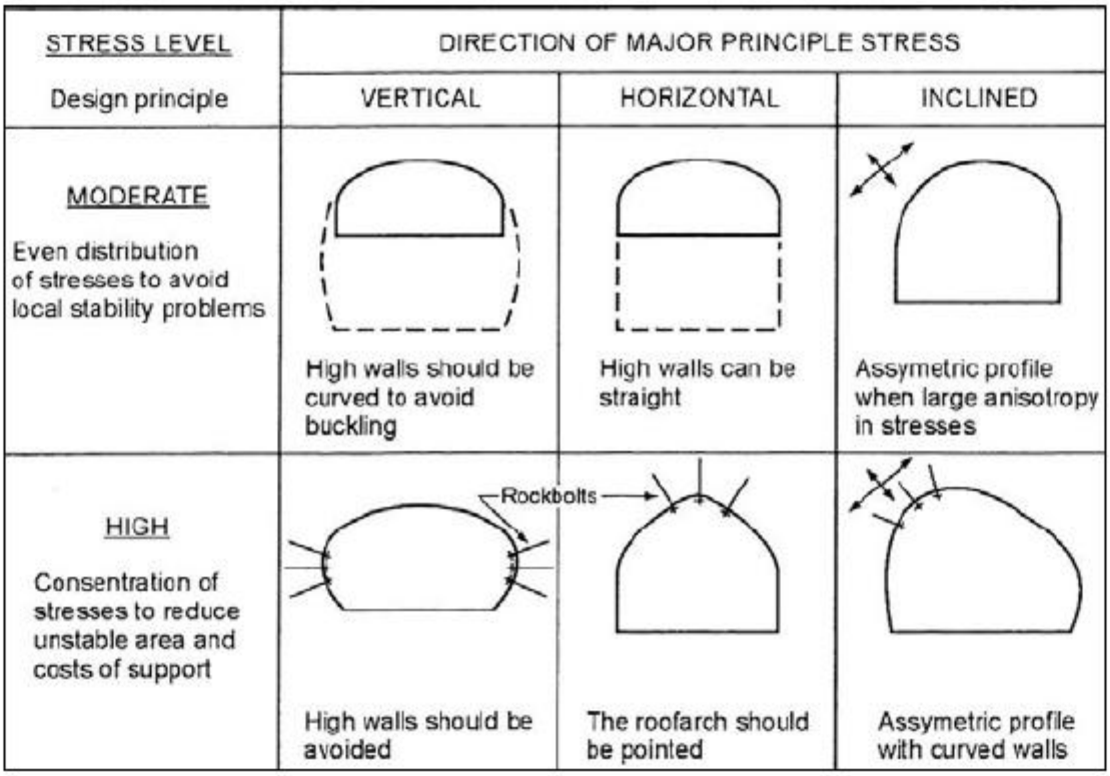


Figure 6.14 Recommended shape of tunnels according to stress condition

6.6 Conclusion and discussion on rock classification and support system

Use of more than one system of rock mass classification in the field has good advantage. It will help in events of discrepancies of values during later dates. NGIs Q system and Bieniawskis RMR system was used for Punatsangchu-II project.

The Himalayas rocks are normally very poor with lots of discontinuities and weakness zones created by the active tectonic activity. The geology in the Himalayas changes greatly within very short distances exposing uncertainties. From the geotechnical report, the HRT passes through mixture of good to very poor rock mass intercepted by few prominent shear and numerous cross joint sets. A conservative support system was adopted for the tunnels as compared with the supports derived from Q method. The whole length of HRT is provided with concrete lining which is a traditional approach followed in projects in India and Bhutan.

Probe drilling and pre injection grouting are very important to be carried out in weak zones where seepage problems can be encountered. This can save lots of time and money instead of waiting for the tunnel to collapse and going for remedy measures at later date. The self supporting capacity of the rock mass is not explored well. Pre-injection grouting technology was found to be effectively used in Nepal (Panthi, 2006) as such the same may be explored in tunnels in Bhutan which lies in the same Himalayan range. This could save lots of time and money.

In the good rock mass reach, lighter flexible support system as required by the site condition could have been adopted instead of conservative support. Provision of consolidation grouting in whole tunnel reach may not be required, contact grouting should be adequate. However the seepage in tunnel should be pre explored and pre-injection grouting adopted where high seepage are expected. Exploratory probing ahead of the excavation in the suspected weak zones may be adopted and necessary support system adjusted. This will save lots of time and money.

7 Numerical modeling

The designs of underground structures are traditionally done using empirical methods before the advent of computer programs. These empirical methods were found easy and handy by field geologist and construction engineers. However, due to complex nature of the rock mass, the empirical methods face limitation in its application in complex geometries. Empirical methods are still effective for simple and regular geometry with homogeneous rock masses. Numerical analysis is sub group of analytical method. It uses computers to analyses stress on models prepared representing the rock mass. This gives enhanced results on models at very minute details with two dimension visual outputs. The fast and powerful computers facilitate discretization of rock mass into large number of smaller elements. The numerical analysis can be used for stress analysis, deformations, elemental and support yielding details and for cross checking the support systems. Use of more than one method increases advantage in design of underground structures. More design methods will compliment the effectiveness of each other.

Despite all advancement, the accuracy of results from computer software and codes are still dependent on the accuracy in defining the input parameters. The input rock mass data's required are direction, magnitude of virgin stresses and elasticity parameters of rock mass which still is challenging. The effectiveness of the analysis depends on the quality of input parameter and right interpretation of the results by an experienced person. The reliability of the analysis will never be more than the reliability of input parameters.

The numerical models can be grouped into two types.

1. Continuous models:- In continuous models, the rock mass is considered as a continuous medium with only limited numbers of discontinuity. Different computer software available in continuous models include
    - Finite element method (FEM)
    - Finite differential method (FDM) and
    - Boundary element method (BEM)
2. Discontinuous models:- In discontinuous models, the rock mass is modeled as individual elements blocks which is free to rotate, translate and interact along their boundaries. Different computer software available in discontinuous models includes
    - Distinct element method (DEM)
    - Discontinuous displacement Analysis (DDA).

7.1 Phase2

Phase2 is a two dimensional elasto-plastic finite element programme. It is used for estimating the stresses and displacement around the underground openings (reference manual, rock science). It can also be used to solve wide range of mining, geotechnical and civil engineering problems. The basic features of Phase2 for the application includes

- Excavation in rocks or soil
- Multi stage excavation
- Elastic or plastic material analysis
- Bolt support
- Liner support (shotcrete, concrete/piles/geosysthesis)
- Constant or gravity field stresses
- Jointed rocks
- Plain stress or axisymmetry
- Ground water (pore pressure is inclusive in analysis)
- Finite element slope stability and load split etc.

7.1.1 Assumptions

Phase2 uses a plane strain analysis where two principal in-situ stresses are in the plane of excavation and the third principal stress is out of plane. This assumption is to dissolve the 3-D stress tensor into three orthogonal stresses which are aligned with the 2-D model of the excavation. The axisymmetric option of phase2 program can be used to analyze three dimensional excavations which are rotationally symmetric about an axis. But, hydropower tunnels are rarely symmetric.

The excavation section is assumed constant and the excavation is of finite length in out-plane direction, therefore three dimensional end effects are not encountered. The shear stresses and strains in the out of plane are considered nil.

7.1.2 Phase2 working

The phase2 programme mainly comprise of three important features. The first task is creating the module by providing necessary input parameters from the field. After feeding the required input data's, the module is simulated and parametric results are calculated. The results of calculation are finally presented both graphically and figuratively. The interpretation of the result by an experienced engineer is important to get better understanding of the stress situations and solutions thereof. The working of the phase2 can be illustrated figuratively as shown in figure 7.1.

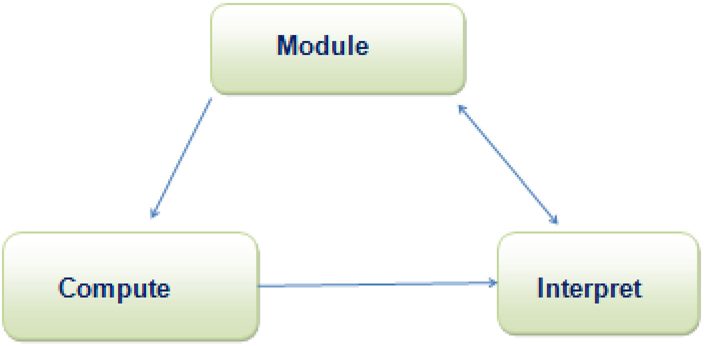


Figure 7.1 Three module operation of

Module

Module is the creation of field replica of the rock mass providing input parameters. The boundary conditions, in-situ stresses, material properties, meshing, staging sequences, excavations, and any other relevant input parameters required for creating the module are done at this stage. To get better results, it's important to create models that are closest and representative of rock mass condition. The remaining steps from here are calculation of outputs based on inputs provided in this step. Other conditions in the models like creation of elastic or plastic models, combination of supports etc can be customized accordingly.

Compute

Once all the required input parameters for generating module is complete, the module is ready for simulation, the computer carries out computation of stresses and deformation pertaining to this module.

Interpret

The computed results of the module are displayed graphically for the stress conditions, strength factors, displacement vectors and yield elements etc. as required. The experienced interpretation of the results is very important to get logical conclusions from the modeling. If the results are not interpreted properly, the beautiful display of results has no meaning, so it needs an experienced person to interpret the results of the simulation.

7.2 Module generation

This section briefly describes the process involved in module generation giving step by step procedures and describing the significance of each step.

Project setting

In this stage, the user sets the name of the projects, the type of measurement unit used, analysis type and the ground water methods. the number of stages etc.

The analysis can be performed either using Plane strain or by axisymmetric option. The plane strain module assumes the excavation as infinite length normal to the section of analysis and assumes the out-of-plane stress as zero. This analysis computes major and minor in-plane principal stresses, out-of-plain principal stresses and in-plain displacement and strain.

Boundary conditions

The boundary conditions for the excavation and external limits are defined. This is mandatory for generating the module. These boundaries are formed by close poly lines. In large openings, staged excavation may be necessary, so staged boundary have to be created. The external boundary encompasses all other mesh boundaries. It can be defined either as rectangular box or circular shape around the excavated boundry. Other boundaries include material, joints, structural interface and piezometric line. To separate the different material type in the rock mass, material boundary is used. The end boundaries can also be restricted or free depending on the nature of analysis.

Meshing

The element mesh can be of graded type or uniform types which can be customized. The finite elements can be generated either in triangular (three nodes or six nodes) or quadrilateral by an automatic two dimensional finite element mesh generator. The meshing discretize the boundaries to build a framework of the finite element mesh. After discrediting, mesh set up option generates the finite element mesh within the defined external boundary. If required advanced mesh set up can be used to fine tune the grading for better results.

Field stresses

The in-situ stress conditions and their values are defined prior to excavation. The field stresses is defined by either constant or gravity. Constant field stress is used for deep seated openings to define the in-situ stress condition which do not vary with depth. The gravity field stress defines in-situ stress condition for surface or shallow seated openings where the stress conditions vary with the variation of depths in topography of the surface.

Material properties

The materials can be defined as either elastic or plastic in material properties. The elasticity or plasticity of the material and its strength parameters are customized. The initial element loading defines the initial loading of the material which can be under field stress or body force. For constant field stress, the initial element loading is from field stress only but for gravity field stress the field stress and the body force are the initial stress element. The body force is the load due to the self weight of the material derived from the unit weight of the material. The elastic property of the material can be defined as isotopic, transversely isotopic, orthotropic and Duncun-Chang hyperbolic material. The Youngs modulus and poisons ratio are required to define isotopic material. The strength parameter allows user to define failure criterion of the material and the material type. The failure criterion can be chosen from different options available like Mohr-Columb, Hoek and Brown, Drucker-Prager, Generalised Hoek and Brown etc. and other input parameters defined accordingly. In generalized Hoek & Brown and Mohr-Columb the input parameters can be imported using RocData.

The elastic materials do not fail but the failure criteria are used to calculate and plot its degree of strength factors. But in plastic material, when the materials yields its strength parameter is used for stress analysis. Residual and dilation parameters are required for plastic materials. When the residual strength factor is equal to its peak strength, the material is considered ideally elastic plastic. The dilation parameter defines the increment in volume of the material due to shear.

Support

There are different types of support options available in Phase2. The types of supports include different rock bolts, shotcrete and concrete liners. The support if not available in the list of supports in Phase2, can be customized depending on the needs. The support parameters pertaining to compressive and tensile strength, youngs modulus, etc have to be defined. The support is assigned to the model at different stages of excavation. The model is simulated and analyzed first without support and then with support. The results of this exercise guide the need for supports depending on the changing values of principal stresses, strength factor and deformation values. The values of deformation, strength factor and principal stresses are studied for different combinations of support and an optimum supports is decided when the best combination is achieved. This decides the stability of the tunnel opening.

7.3 Input parameters

Three sections at different locations is selected representing the whole length of the HRT. The first section is at the take off point and the second at the end of HRT alignment. The third section is taken at the mid length where the rock cover is also in the range of maximum values. There is no core drilling works conducted along the HRT layout. As such the laboratory test of the rock mass properties available for the Dam complex and power house are used for the HRT sections. For the first sections due to its closeness to the dam complex, the laboratory results of rock mass properties at dam is considered, on similar reasoning the rock mass properties for the power house area is used for the last section. In the middle section, the average values at dam and power house locations are adopted for all calculations and numerical modeling purpose. The final values of the rock mass properties for the three sections are tabulated in table 7.1

Table 7-1 Input parameters for numerical model

Properties	Ch.0.0m	Ch. 4167m	Ch. 8584.3m
Intact strength σci (MPa).	58.3	49.27	40.25
Youngs Modulus Eci (MPa)	6623.4	5511.6	4409.8
GSI	45	45	45
Disturbance factor D	0.5	0.5	0.5
Material constant mi	30	30	30
Unit weight of rock ϒ ((KN/m3)	2.60	2.59	2.57
Poisons ratio	0.34	0.28	0.22
mb	2.19	2.19	2.19
s	0.0006	0.0006	0.0006
a	0.51	0.51	0.51
Rock mass strength σcm (MPa)	11.09	9.11	6.79
Deformation modulus (MPa)	841.60	648.80	466.30

The above input parameters are used for generating models at each section. The other parameters are also provided to each section according to its requirement.

Field stresses

The stress condition is considered an-isotopic which is normal in the rock is mass. The in plane and out of plane stress components of the horizontal stress values were calculated considering the tunnel orientation with reference to the north direction. The locked in horizontal stress at the ground surface for both the planes were calculated by dissolving the tectonic stress value of 5 MPa which is oriented north south direction along the tunnel alignment and other parameters are considered accordingly.

Support parameter

Different combinations of supports were used for the models. The different support and their support parameters used in the model is shown in table 7.2.

Table 7-2 Rock supports and their strength parameters used in models

Rock Bolt		Fiber reinforced shotcrete	
Diameter (mm)	25	Youngs modulus (MPa)	30000
Length (m)	5	Compressive strength Peak (MPa)	35
Youngs Modulus (MPa)	200000	Compressive strength residual (MPa)	5
Tensile strength Peak (MPa)	0.1	Poissons ratio	0.2
Tensile strength Residual (MPa)	0.01	Liner type	Standard beam
Type	End anchorage		

7.4 Interpretation of results

Once the model is prepared the relevant input parameters, the phase2 displays the results in different formats like graphic and tabulated Excel sheets. Some important results are discussed in this section.

Principal stresses.

The phase2 simulation result gives the values of the three main principal stresses. The major principal stress $\sigma 1$ and minor principal stress $\sigma 3$ are mutually perpendicular to each other and σz is perpendicular but in out plane direction. The value of $\sigma 3$ can be either smallest or in medium range depending on the value of σz. The magnitude and direction of the principal stresses are shown by the cross bars in stress trajectory. The values of all three can be displayed along the periphery of the tunnel opening. As heavier support systems are adopted, the values of the principal stresses reduce indicating improved stability along the tunnel periphery.

Strength factor

Strength factor indicates the stability of the tunnel around its periphery. Strength factor is the ratio between the rock mass strength to the induced stress at the location. This is influenced by the major principal stresses. In elastic material, the strength factor is less than 1 since the over stressing of the material is allowed. However in plastic materials, the strength factor has to be above 1 since over stressing is not allowed. The strength factor for the models improves as the support intensity increases but after certain optimum support value, the change in support decreases the strength factor. This is used for deciding the optimum combination of supports.

Displacement

The phase2 results give the displacement values in horizontal, vertical direction and the total displacement. The total displacement value is the resultant displacement of the horizontal and vertical displacement given by

$$Total displacement = \sqrt{X^2 + Y^2}$$

The total displacement values also decreases with the increase in support intensity. But after certain limits , increase in support intensity do not decrease the deformation values. This behavior of the rock mass is also used in deciding the optimization of support combination.

Yielded elements

The yielded element is applicable only in plastic material. It gives the number of yielded elements, the rock bolts and liners in finite elements. The elements that fail in shear is represented by x and the elements that fail in tension is represented by o. where the failure is due to both, it presents the over lapping of the x and the o. the number of yielded elements and supports are used for the optimum design of the rock support system.

7.5 Numerical modeling results

Three sections, one at the take off point, end point and the middle point of the head race tunnel was considered for numerical modeling. The models at each section were prepared with the input rock mass parameters as detailed in input parameters. The models were then checked for possible hydraulic fracturing problems. Different support system as detailed in e ach case was tried for each model. The final support system is decided with the optimum results of the principal stress values, strength factor and total deformation values for each section of tunnel location. This chapter discusses the results of numerical modeling at the three selected reach of the head race tunnel alignment for Punatsangchu II project.

7.5.1 Support analysis at Ch.0.0m (Take off point)

The valley slope model was prepared for the section at Ch. 0.0m. The elastic model with the gravity stress condition was analyzed for getting the values of principal stresses. The minor principal stress at the tunnel location is checked for Hydraulic fracturing. The result is given in figure 7.2.

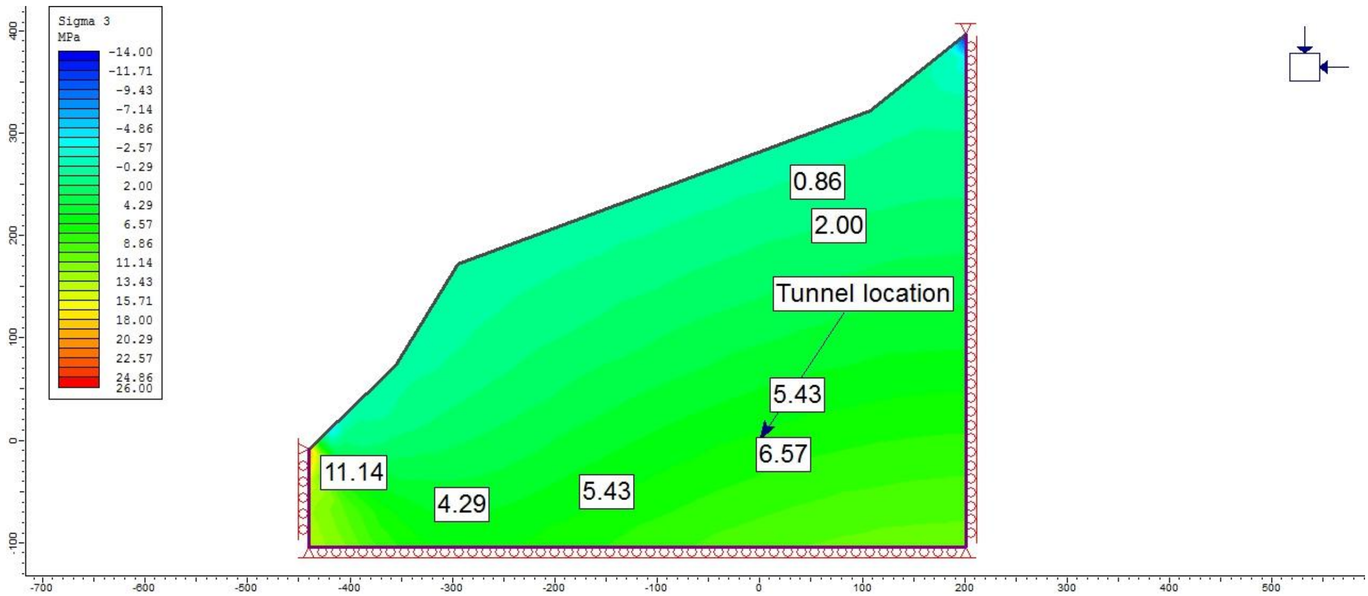


Figure 7.2 Valley slope model for hydraulic fracture and minor principal stress at Ch.0.0m.

The minimum principal stress at tunnel location is 6.29 MPa. The hydrostatic pressure at the same location is only 0.39 MPa, which is very small compared with the minimum principal stress value. Therefore it can be concluded that the tunnel at this section is free from hydraulic fracturing problem.

The principal stresses at this location from the valley slope model are given below.

$$\sigma 1 = 8.93 MPa \qquad \sigma 3 = 6.57 MPa \qquad Orientation : \theta = 16^0$$

For further computation purpose, the value for minor principal stress and intermediate principal stress is taken same given by the minimum of σ3 or the value given by kirschs equation

$$\sigma_{t\max} = 3\sigma 1 - \sigma 3$$

Based on this constant stress inputs and other rock mass parameters given in input parameter table, the models at this section is prepared and simulated for stability analysis as described here under.

Elastic analysis

Elastic analysis is carried out for calculation of the strength factor along the periphery of the tunnel opening. Since the elastic does not permit over stressing, the strength factor value is less than 1. The strength factor for elastic modeling is given in figure 7.3

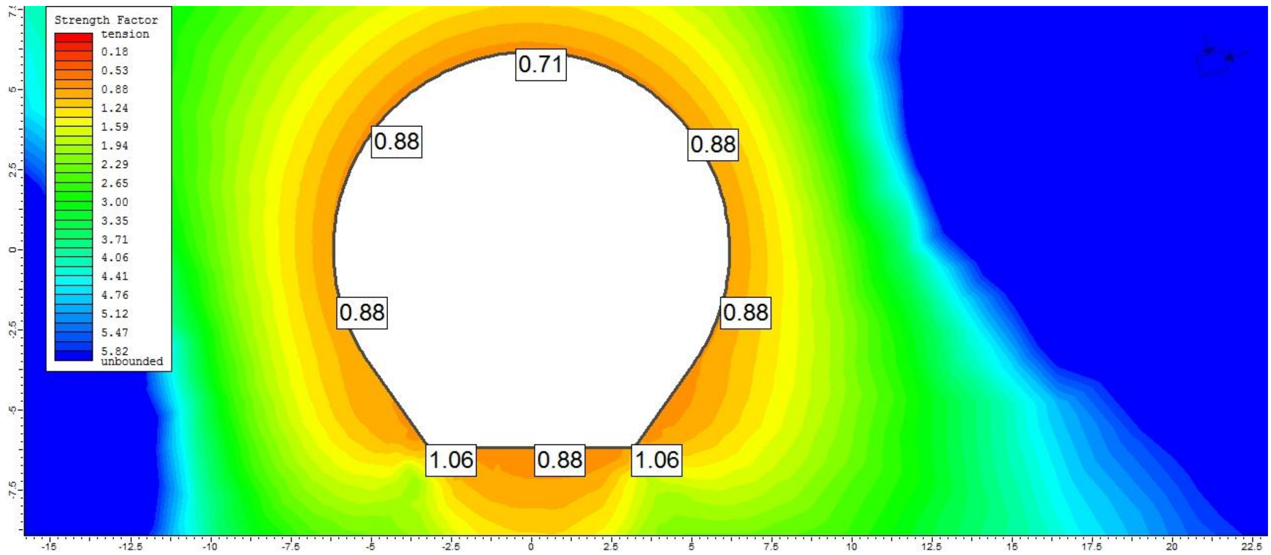


Figure 7.3 Strength factor diagram at Ch. 0.0m for elastic analysis.

It can be seen from the figure that the strength factor values are less than 1 in most of the areas. The region of over stressing are concentrated at the crown and up to the spring level. In the invert level the strength factor is greater than 1 signifying higher rock mass strength than induced strength.

Plastic analysis

The results of the major principal stress, strength factor and the total deformation for the plastic analysis at Ch.0.0m are given in figure 7.4 to figure 7.6

The principal stress value ranges from 0.5 MPa in the left heading level to a maximum of 9MPa in the left invert.

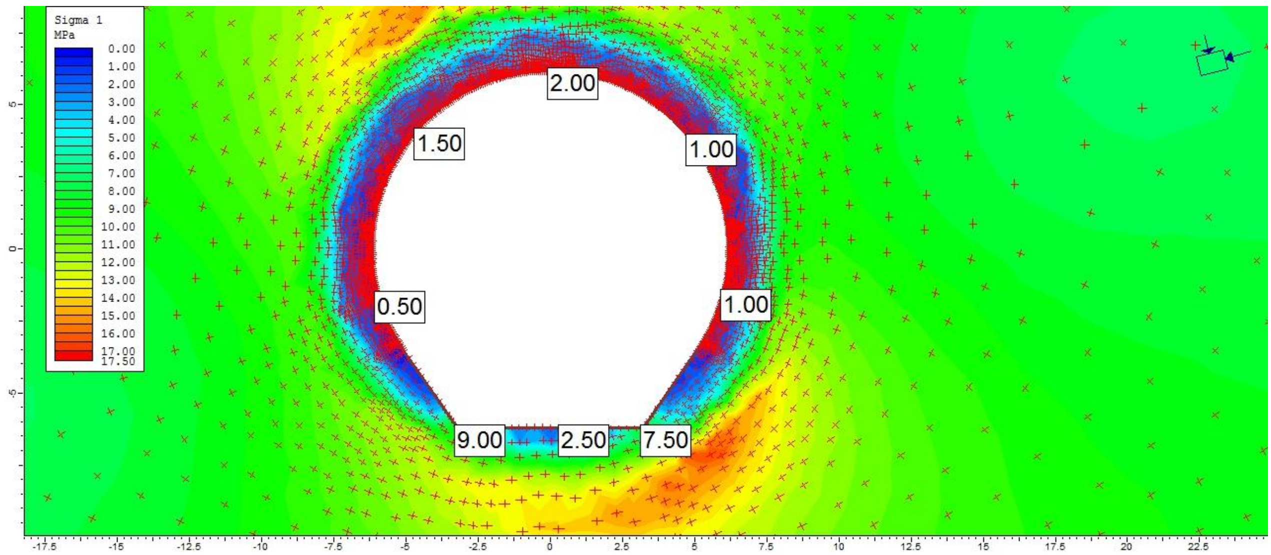


Figure 7.4 Principal stress sigma1 value for plastic analysis for tunnel section at Ch.0.0m.

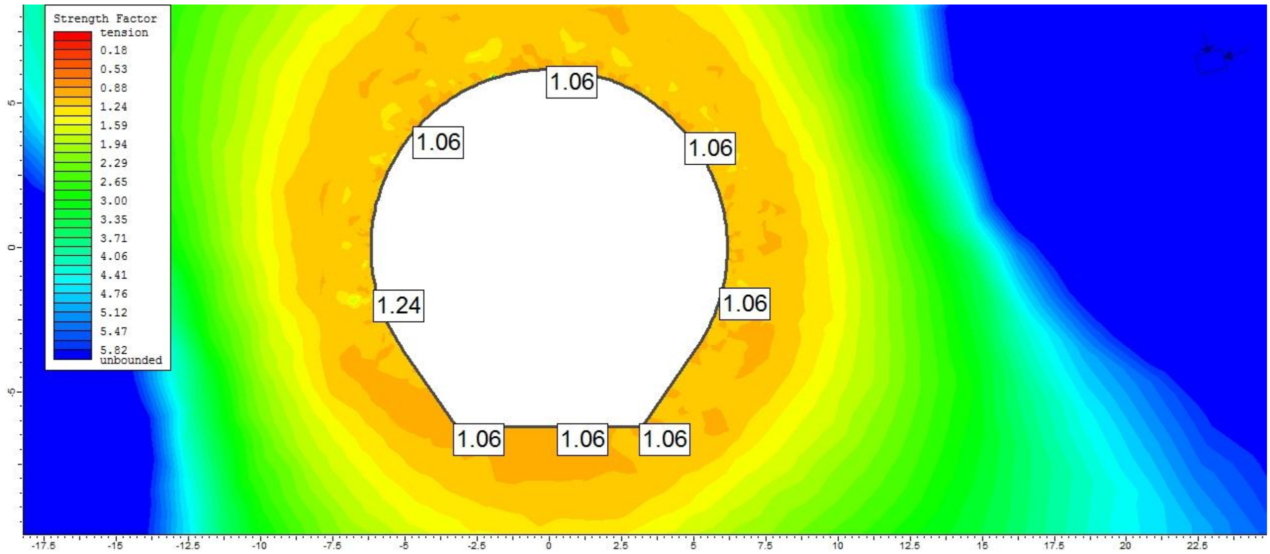


Figure 7.5 Strength factor value for plastic analysis for tunnel section at CH.0.0m

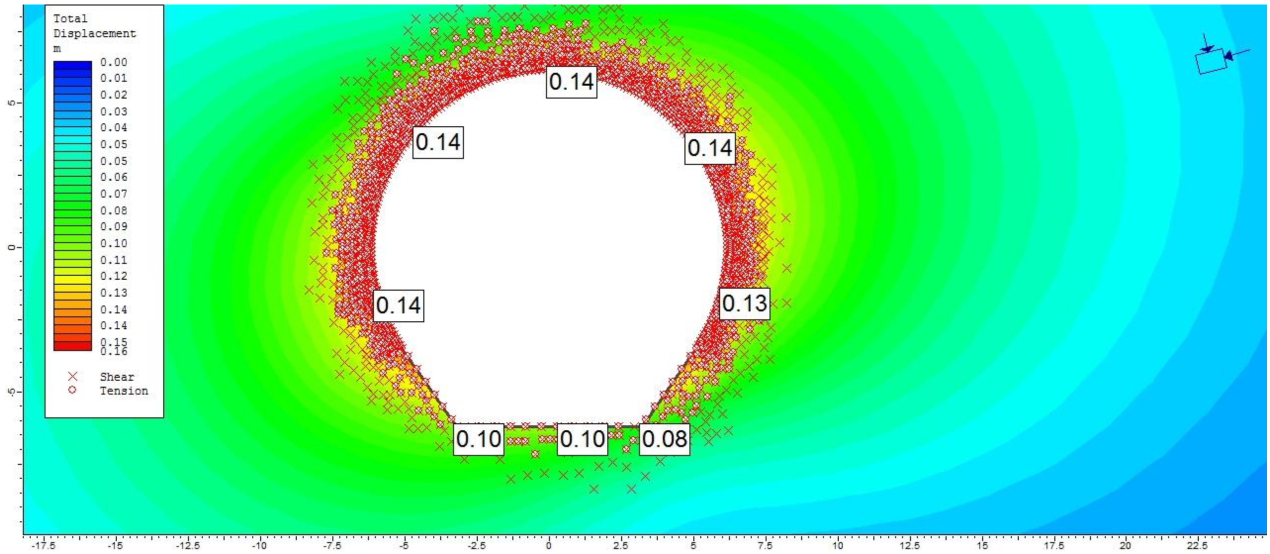


Figure 7.6 Total displacement value for plastic analysis for tunnel section at Ch.0.0m

The strength factor is just above 1 which says that the induce stress values are equally high comparable to the rock mass strength.

The total displacement of the tunnel without the support arrangement is 0.14m with 1345 numbers of yielded elements. The total displacement is not very high so no serious support systems may be required. The maximum displacement is seen on the crown area.

Plastic analysis with rock support

Plastic analysis with different combinations of support system was carried out. The total displacement from plastic analysis without support is very small so the need for heavy support does not arise. However different combinations of support system was tried to optimize on stability by comparing the different values of principal stress, strength factor, total displacement , yielded elements, bolts and liners with each combination. The final support was decided considering all this factors.

Table 7-3 Support combinations and their stability results

Support combination	Sigma1 MPa	SF	TD (m)	Y-E Nos.	Y-Bolts (Nos)	Y- Liners (Nos)
R/B @ 2x2 & 75mm thk SFRS	4.5	1.06	0.16	1352	18	217
R/B @1.5x1.5 & 100mm thk. SFRS	9.35	1.06	0.14	1245	24	218
R/B@1.5x1.5 & 120mm thk SFRS	9.40	1.06	0.14	1526	24	214
R/B @1.2x1.2 & 120mm thk SFRS	10.69	1.06	0.14	1190	30	200
R/B @1.2x1.2 & 150mm thk SFRS	8.5	1.06	0.15	1265	30	218

Note: R/B: Rock bolts, thk : thickness, SFRS: fiber reinforced shotcrete, ISMB: Indian standard m. beam, SF: strength factor ,TD: total Displacement, Y-E: yielded elements, Y-Bolts: yielded bolts, Y-liners; yielded liners

The results for the stability for various combinations of supports are shown in figure 7.7 to figure 7.9

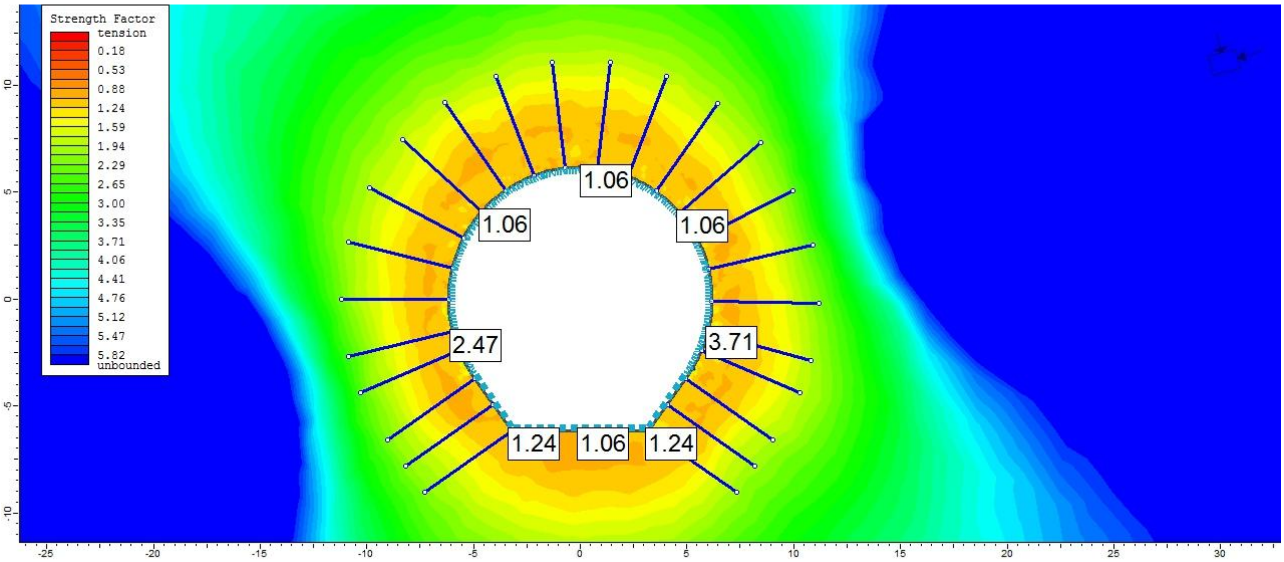


Figure 7.7 Sigma1 for 1.5mx1.5m R/b and 100mm thick shotcrete support at Ch. 0.0m

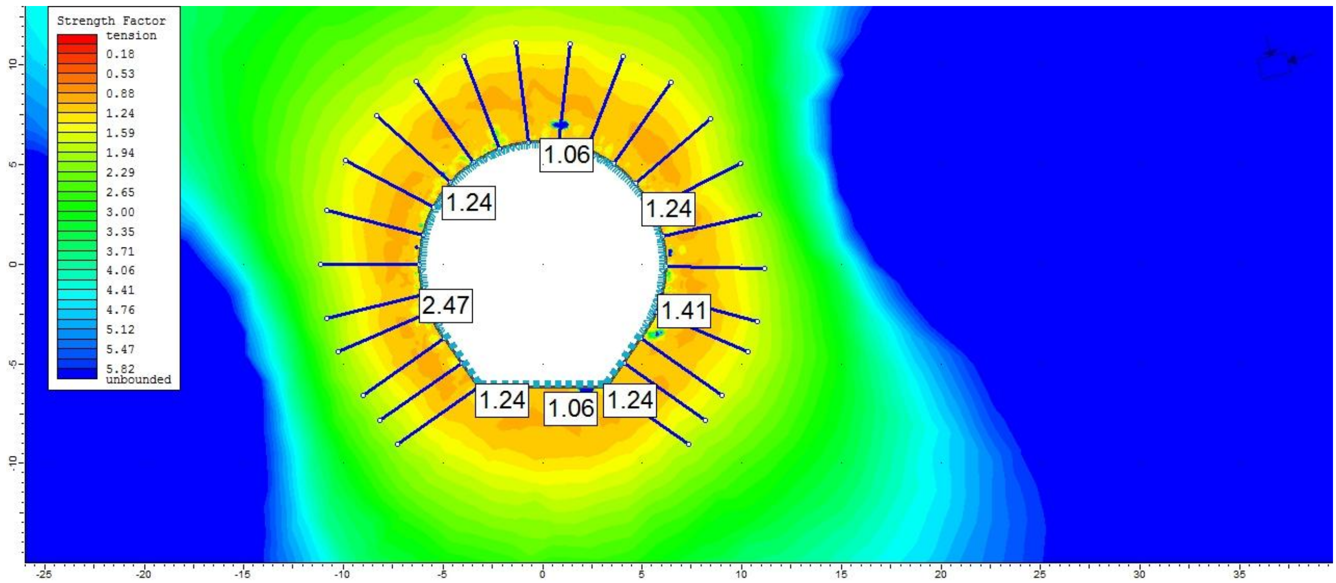


Figure 7.8 Strength factor for 1.5mx1.5mR/B and 100mm thick Shotcrete support at Ch. 0.0m.

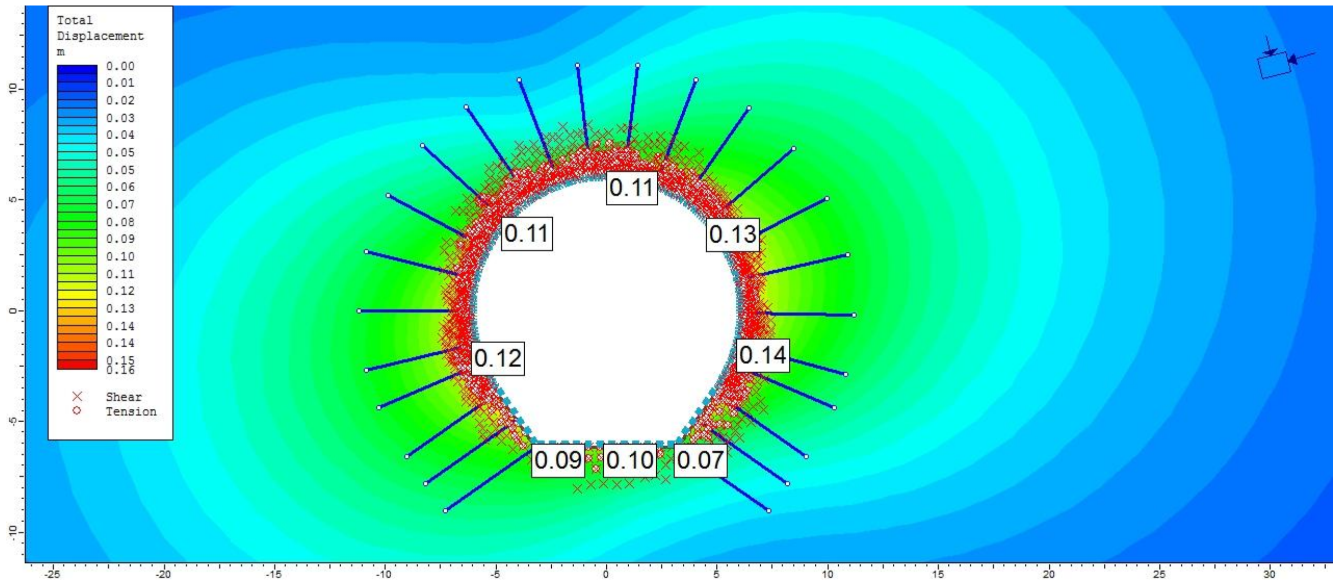


Figure 7.9 Total displacement vector for 1.5mx1.5m R/B and 100mm thick shotcrete support.

The final support is chosen based on the deformation values, number of yielded element and supports. The total displacement of 0.14m for a tunnel diameter of 12.35 m is within the acceptable limit. Attempt for heavier reinforcement will only increase the cost and does not change the stability condition any better. As such a final reinforcement of 5m long rock bolts spaced at 1.5mx1.5m and reinforced shotcrete of 100mmm thick throughout the whole length of walls and crown is found adequate.

7.5.2 Support analysis at Ch. 4167m

The valley slope model was prepared for the section at Ch. 4167m. The elastic model with the gravity stress condition was analyzed for getting the values of principal stresses. The minor principal stress value at the tunnel location is checked for Hydraulic fracturing. The result is given in figure 7.10

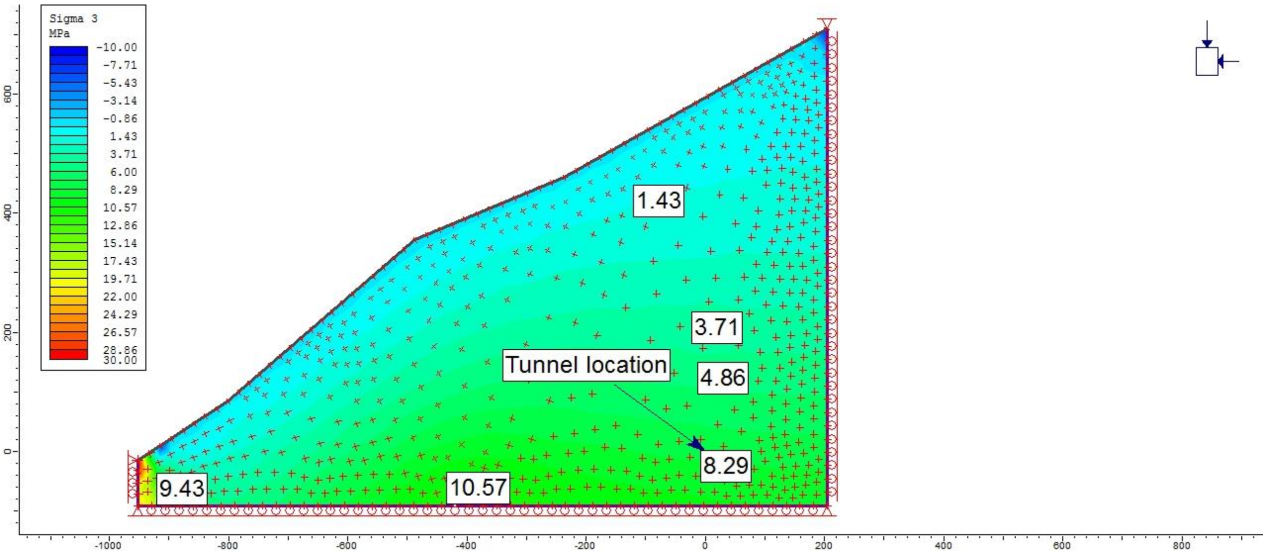


Figure 7.10 Valley slope model for hydraulic fracture and principal stress values at Ch.4167m

The minimum value of principal stress at the tunnel section is 8.29 MPa. The hydrostatic pressure at the same location is only 0.58 MPa, which makes it less likely to have any hydraulic fracturing problems due to hydro static head.
The principal stresses at this location from the valley slope model are given below.

$\sigma 1 = 14.5 MPa \qquad \sigma 3 = 8.29 MPa \qquad Orientation : \theta = 0$

For further computation purpose, the value for minor principal stress and intermediate principal stress is taken same given by the minimum of σ3 or the value given by kirschs equation

$\sigma_{t\max} = 3\sigma 1 - \sigma 3$

Elastic analysis

Elastic analysis is carried out for calculation of the strength factor along the periphery of the tunnel opening. Since the elastic does permit over stressing, the strength factor value is less than 1. The strength factor for elastic modeling is given in figure 7.3

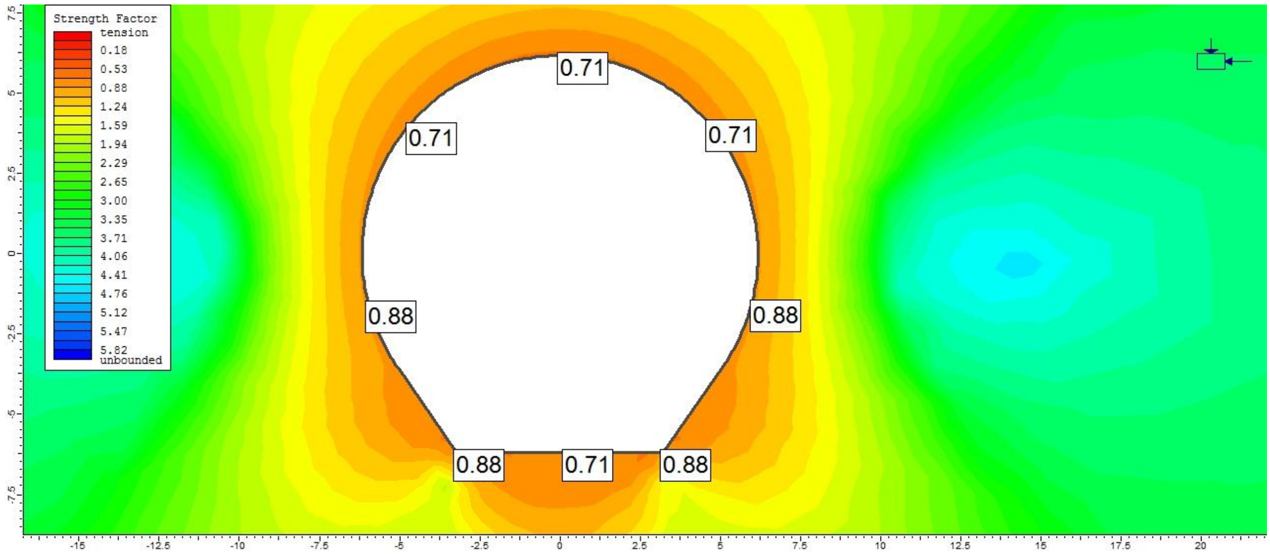


Figure 7.11 Strength factor for elastic model at Ch.4167m

The stressing is more on the crown area with the minimum strength factor at these locations. In the invert areas the strength factor is close to 1 meaning the induced stress in the rock equivalent to the rock mass strength.

Plastic analysis

The plastic analysis is carried out for the tunnel at the same section. The results of various results are given in figures from figure 7.12 to figure 7.14.

The principal stress value ranges from 0.8 MPa in the left heading level to a maximum of 9MPa in the left invert. The principal stress in the crown area is not very high.

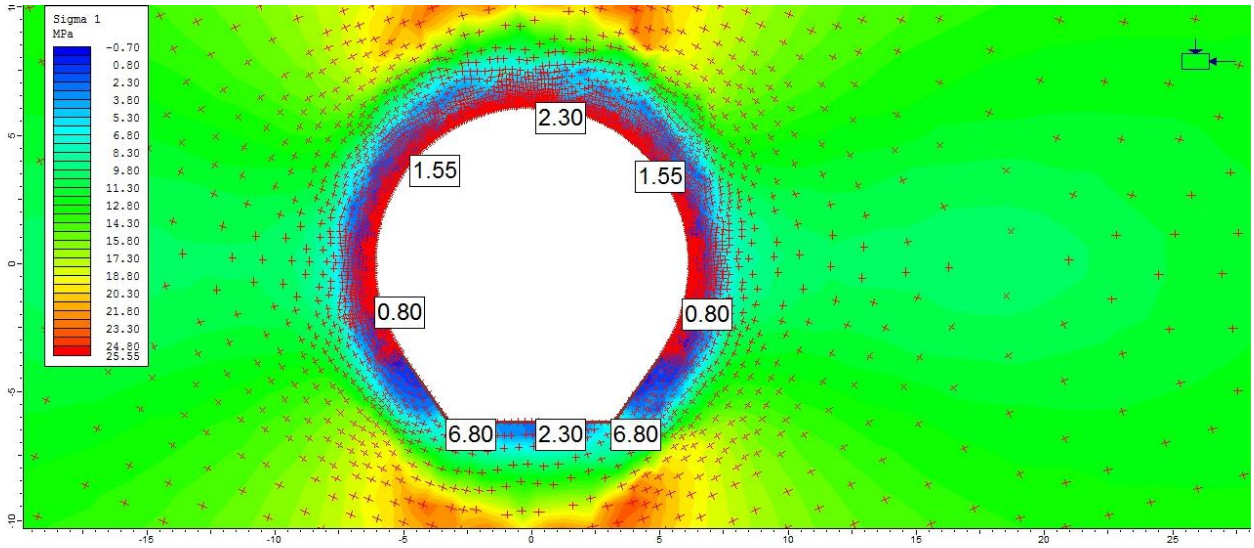


Figure 7.12 Principal stress value for plastic analysis at Ch.4167m

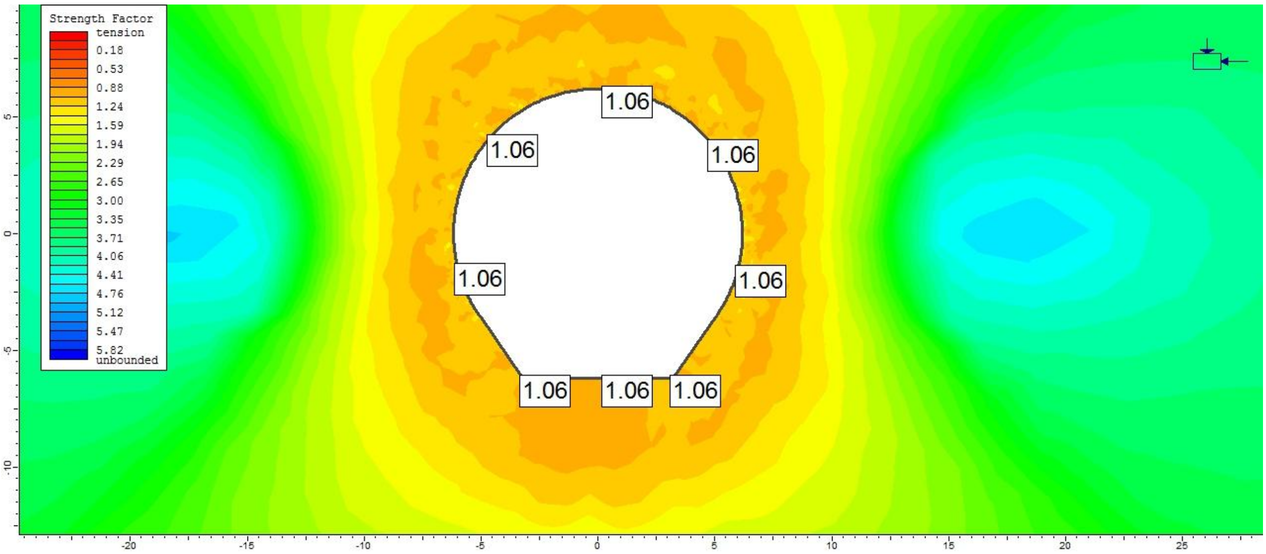


Figure 7.13 Strength factor for plastic analysis at Ch.4167m

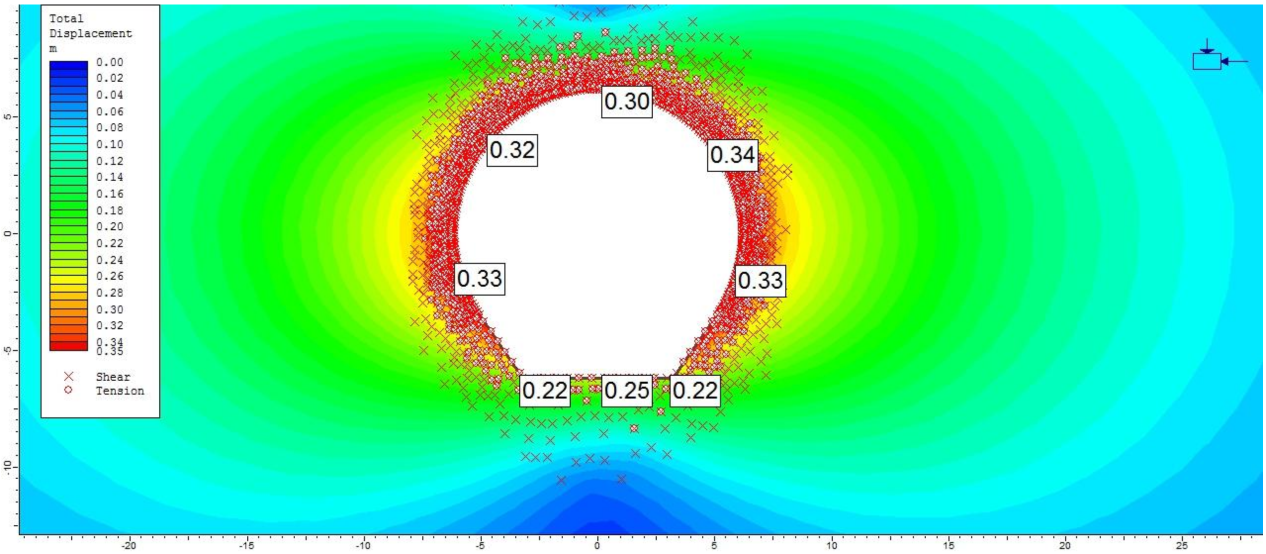


Figure 7.14 Total displacement for plastic analysis at Ch.4167m

The strength factor around the tunnel periphery is just above 1 meaning that the induce stress values are equal to the rock mass strength.

The total displacement of the tunnel without the support arrangement is 0.349m and has 1404 yielded elements. The displacement is higher than 2% of the tunnel dimension. There can be possible instability problems due to high deformation values. As such rock supports is required

Plastic analysis with rock support

The models are analyzed for stability assessment with different support combinations. The total displacement in plastic analysis is high necessitating supports. Different support combinations were tried to bring down the deformation and yielded elements. Different values are presented in table 7.4. The final support combination is decided based on the total displacement, major principal stress and strength factor values. The result for the final support arrangement is presented in figures 7.15 to figure 7.17.

Table 7-4 Support combination and stability results for section at Ch.4167m

Support combination	Sigma1 MPa	SF	TD (m)	Y-E Nos.	Y-Bolts (Nos)	Y -Liners (Nos)
R/B @1.5x1.5 & 100mm thk. SFRS	6	1.06	0.39	1379	24	222
R/B@1.5x1.5 & 120mm thk SFRS	3.75	1.06	0.41	1378	24	222
R/B @1.2x1.2 & 120mm thk SFRS	3	1.06	0.42	1402	30	222
R/B @1.2x1.2 & 150mm thk SFRS	6.8	1.06	0.35	1330	29	222
R/B @1.2x1.2 & 200mm thk SFRS with ISMB 250 @750 c/c.	23.4	1.06	0.19	731	29	Nil
R/B @1.2x1.2 & 250mm thk SFRS with ISMB 250 @500 c/c.	22	1.06	0.19	656	30	nil

Note: R/B: Rock bolts, thk : thickness, SFRS: fiber reinforced shotcrete, ISMB: Indian standard m. beam

It can be seen from the behavior trend that total deformation value is very high to be contained by simple combination of rock bolt and shotcrete. However with increasing support intensity, the major principal stress increases while the strength factor remains same. The yielding elements and the supports reduce with introduction of structural steel support. The support combination of 5m long rock bolts at 1.2mx1.2m spacing with 250mm thick fiber reinforced shotcrete and structural steel bean ISMB 250 @ 500 c/c has brought down the total displacement within permissible limit of 1.5%. The results are displayed in figure 7.15 to figure 7.17

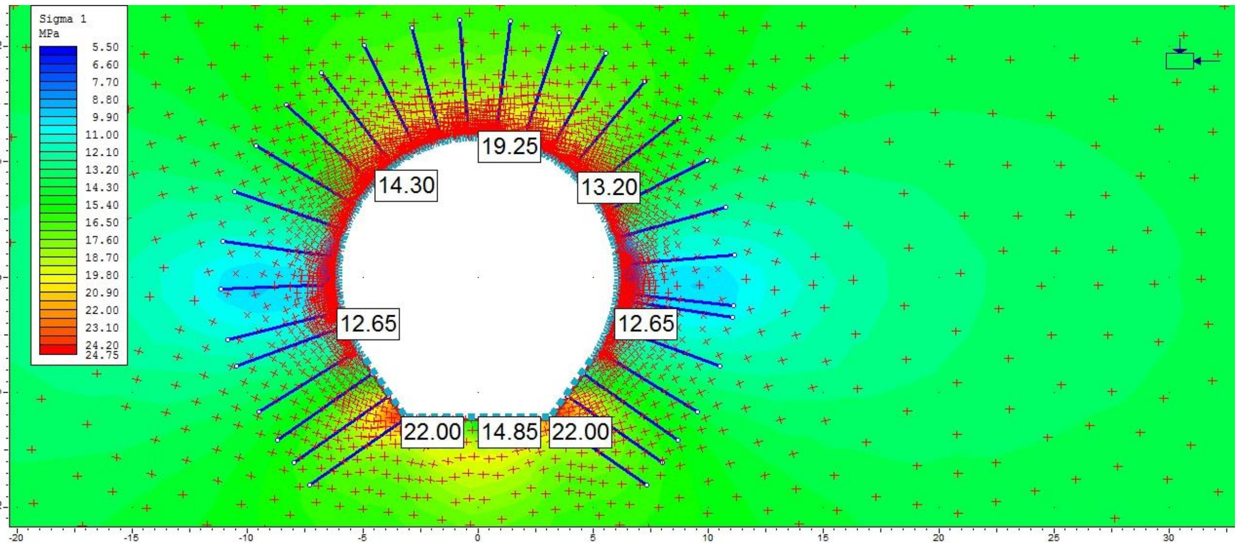


Figure 7.15 Sigma1 value for 1.2x1.2 R/B , 250mm thk SFRS & ISMB 250@500 at Ch.4167m

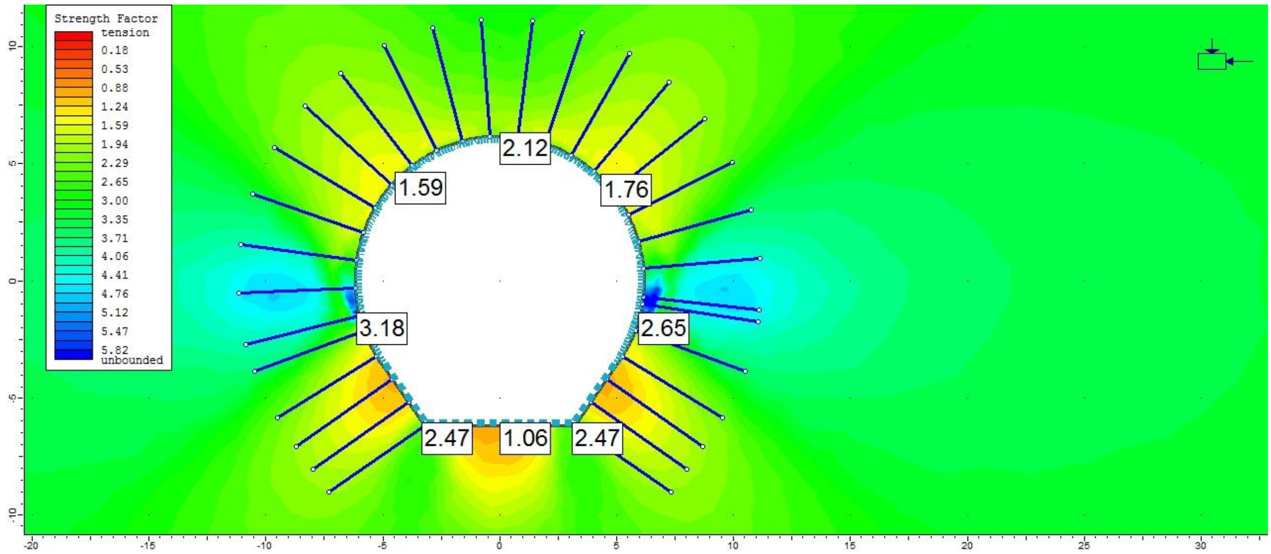


Figure 7.16 Strength factor for 1.2x1.2 R/B, 250mm thk SFRS & ISMB 250 @500at Ch.4167m

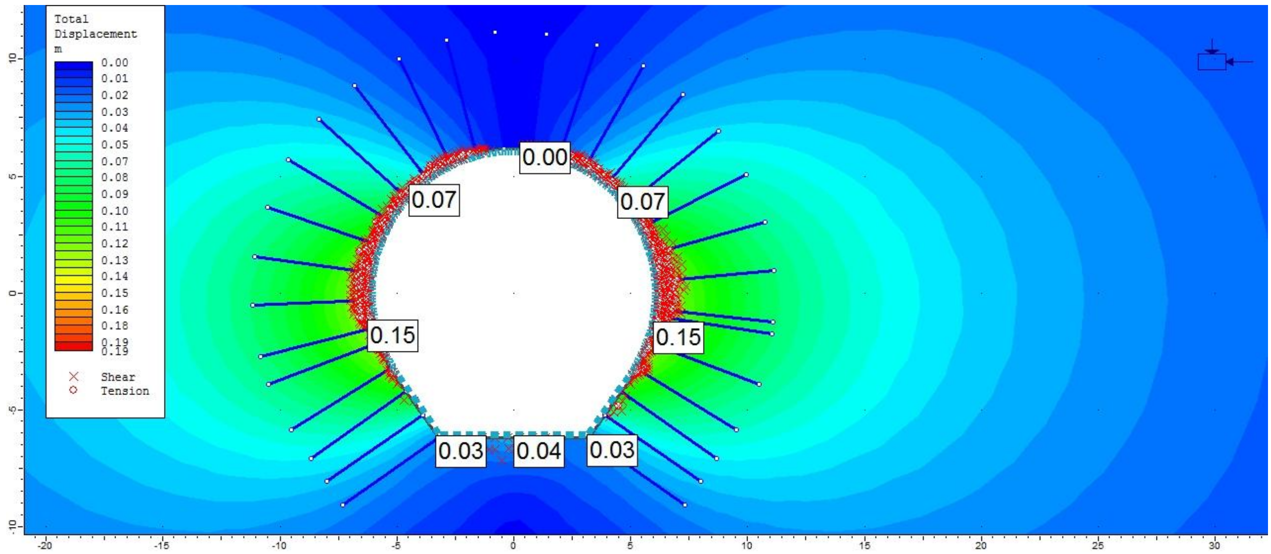


Figure 7.17 Total displacement for1.2x1.2 R/B, 250mm thk SFRS & ISMB @500at Ch.4167m

7.5.3 Support analysis Ch.8584.3m

The valley slope model for the section at Ch. 8584.29m is presented in figure 7.18. The principal stress values are generated by loading the model under gravity stress. The section is tested for possible hydraulic fracturing problem due to stress accumulation

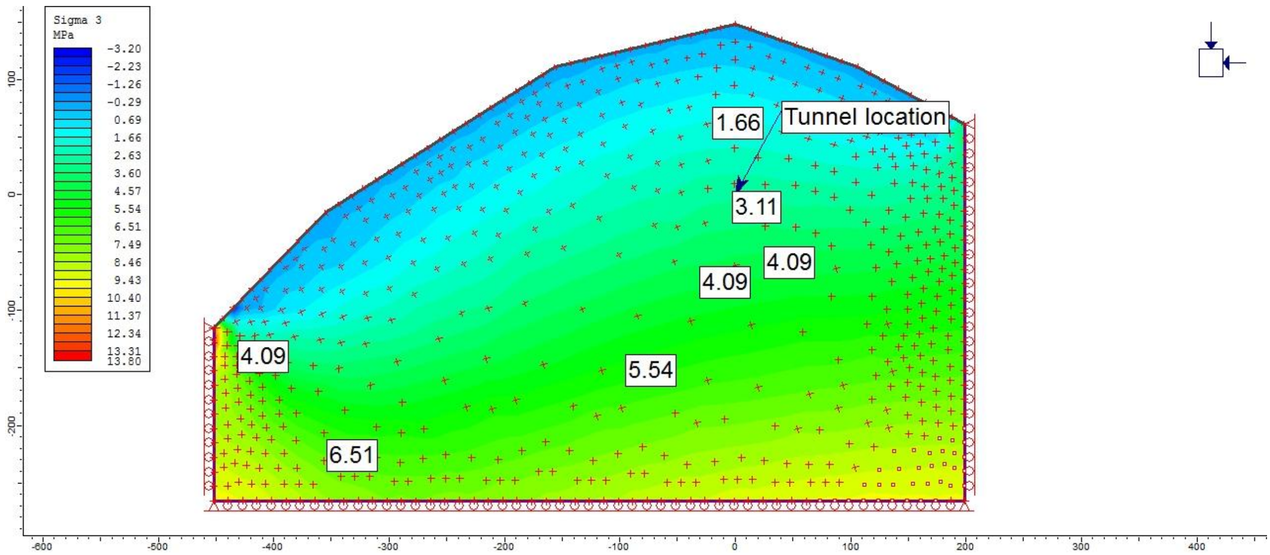


Figure 7.18 Hydraulic fracturing problem test at section at Ch. 8584.3m

The minimum principal stress at the tunnel location is 3.11 MPa. The hydrostatic pressure at the same location is only 0.78 MPa. The difference between two values is not very large, so if there is tectonic activity exceeding the tectonic stress of 5MPa or other geological uncertainties; possibilities of hydraulic fracturing due to high hydro static pressure cannot be over ruled. However since the whole length of the tunnel is provided with concrete lining, there is no possibility of instability due to static water pressure.

$\sigma 1 = 3.43 MPa \qquad \sigma 3 = 3.11 MPa \qquad Orientation : \theta = 11^0$

For further computation purpose, the value for minor principal stress and intermediate principal stress is taken same given by the minimum of σ3 or the value given by kirschs equation

$\sigma_{t\max} = 3\sigma 1 - \sigma 3$

Elastic analysis

Elastic analysis is carried out for calculation of the strength factor along the periphery of the tunnel opening. Since the elastic does permit over stressing, the strength factor value is less than 1. The strength factor for elastic modeling is given in figure 7.19

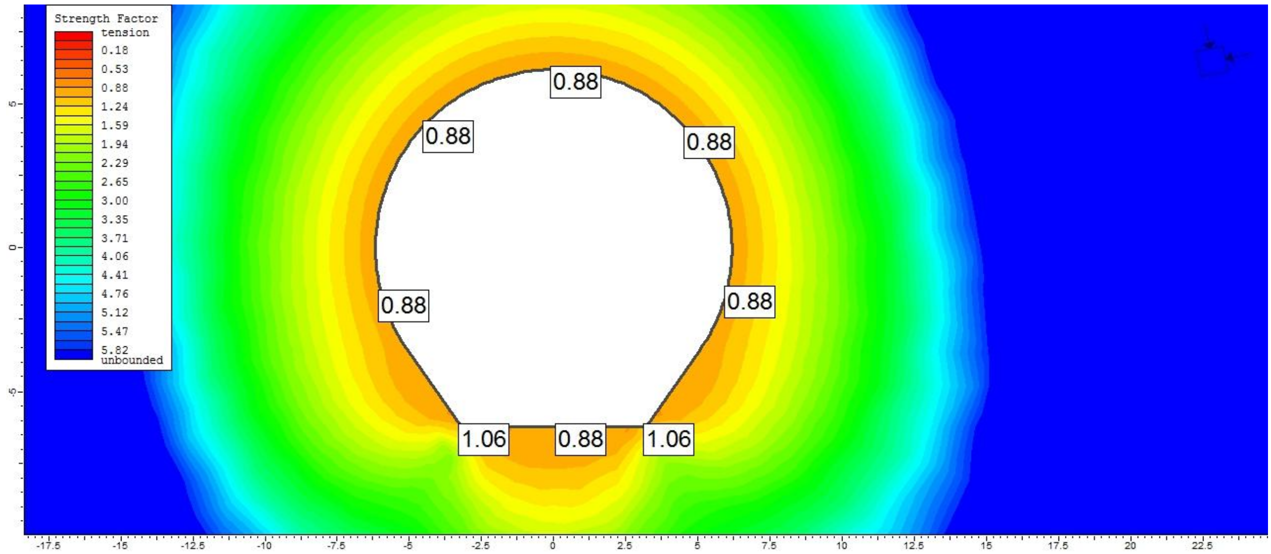


Figure 7.19 Strength factor for elastic model at Ch.8584.3m

The sections along the periphery of the opening are equally stressed except at few locations on the invert at two corners where the strength factor is above 1.

Plastic analysis

The plastic analysis is carried out for the tunnel at the same section. The results of various results are given in figures from figure 7.20 to figure 7.22.

The major principal stress value ranges from 0.6 MPa in the left heading level to a maximum of 4.40MPa in the invert. The principal stress in the crown area is not very high.

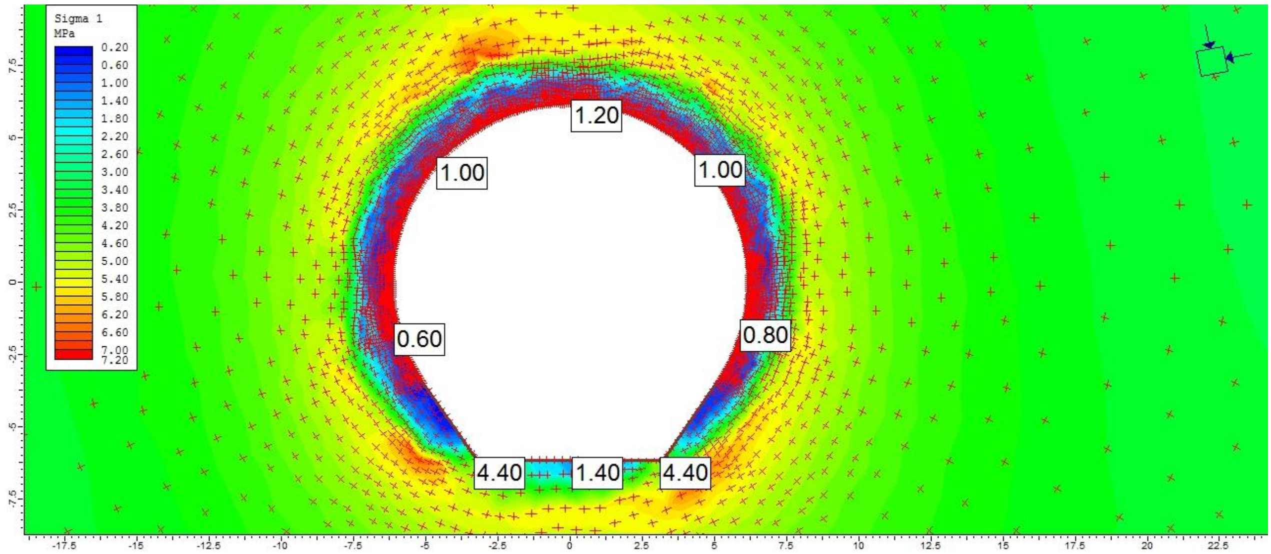


Figure 7.20 Major Principal stress value for plastic analysis at Ch.8584.3m

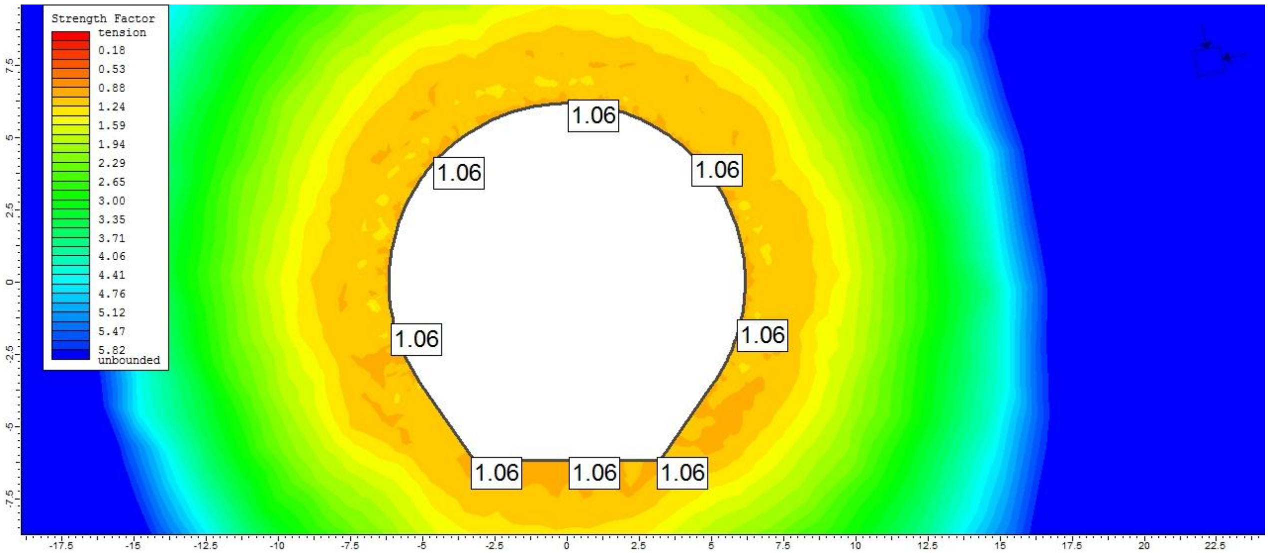


Figure 7.21 Strength factor for plastic analysis at Ch.8584.3m

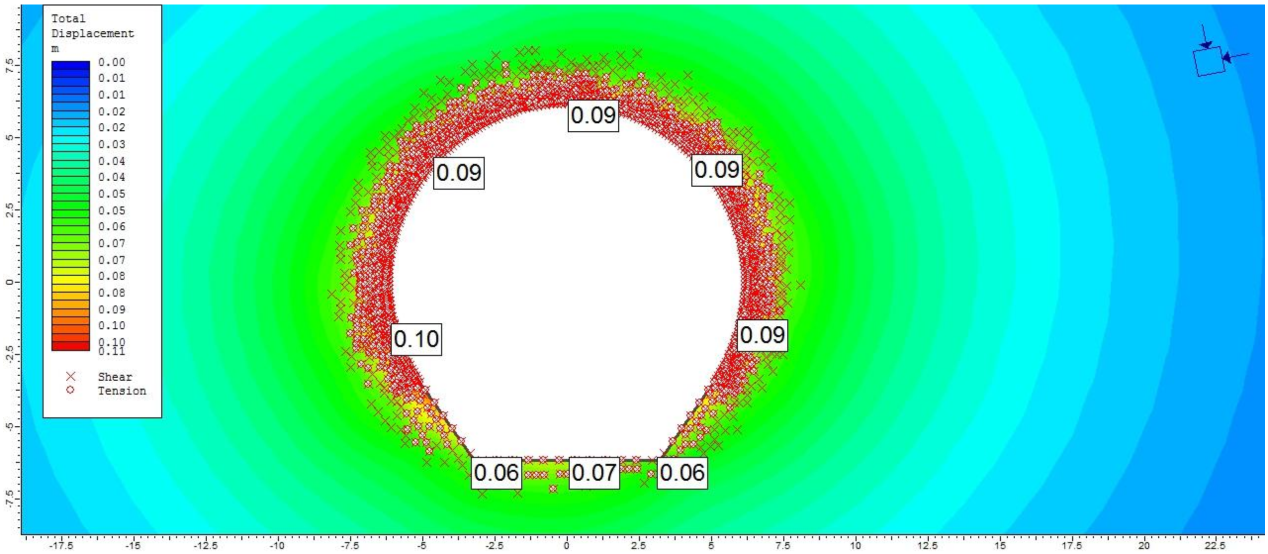


Figure 7.22 Total displacement for plastic analysis at Ch.8584.3m

The strength factor is just above 1 meaning that the induce stress values are equally high comparable to the rock mass strength.

The total displacement of the tunnel without the support arrangement is 0.11m and has 1247 numbers of yielded elements. The displacement is very small even without support. Due to low deformation, the stability can be achieved by very light flexible supports.

Plastic analysis with rock support

The models are analyzed for stability assessment with different support combinations. The total displacement without support in plastic analysis is very low; as such no heavy support combinations may be required. Different support combinations and the corresponding stress and deformation values are presented in Table no.7.5. providing of heavy support is not required since the deformation values are minimal. The support of 5m long rock bolts, 1.2mx1.2m spacing with 100mm thick shotcrete is found sufficient for this section. The results of other stress and deformation situations are shown through figure 7.23 to figure 7.25. and the support combination shown in table 7.5

Table 7-5 Support combinations and their stress and deformation values at Ch.8584.3m

Support combination	Sigma1 MPa	SF	TD (m)	Y-E Nos.	Y-Bolts (Nos)	Y -Liners (Nos)
R/B @1.5x1.5 & 100mm thk. SFRS	5.25	1.06	0.094	1068	18	192
R/B@1.5x1.5 & 120mm thk SFRS	5.4	1.06	0.104	903	24	176
R/B @1.2x1.2 & 120mm thk SFRS	5.75	1.06	0.096	896	24	196
R/B @1.2x1.2 & 150mm thk SFRS	5.1	1.06	0.086	962	29	197
R/B @1.2x1.2 & 200mm thk SFRS with ISMB 250 @750 c/c.	7.8	1.06	0.075	659	29	179

Note: R/B: Rock bolts, thk : thickness, SFRS: fiber reinforced shotcrete, ISMB: Indian standard m. beam

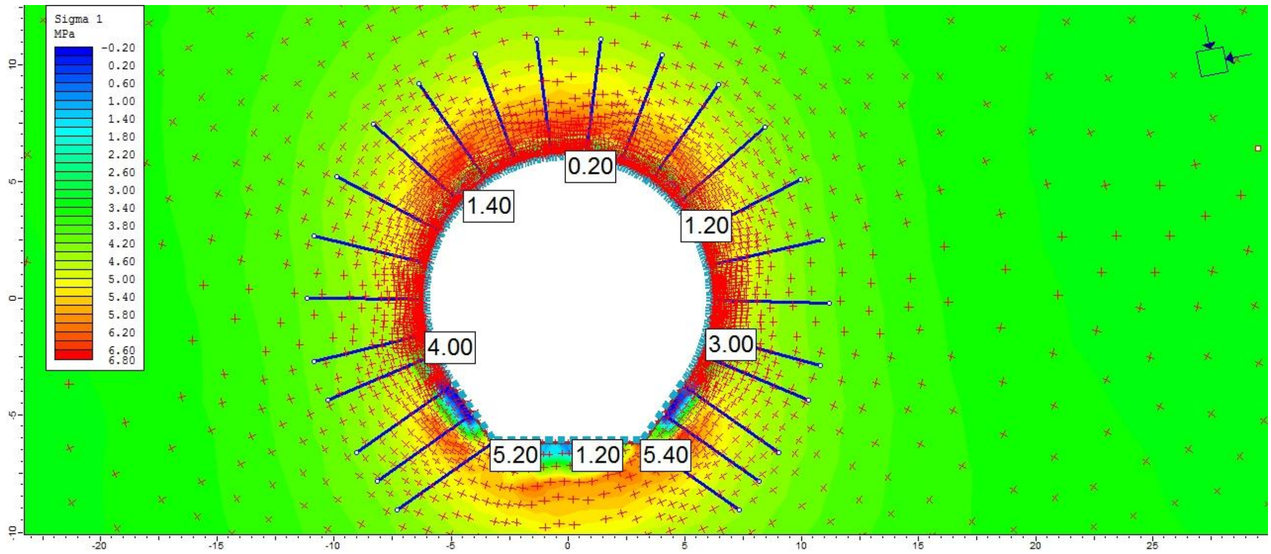


Figure 7.23 Sigma1 value for 1.2x1.2 R/B , 250mm thk SFRS & ISMB 250@500 at Ch.8584.3m

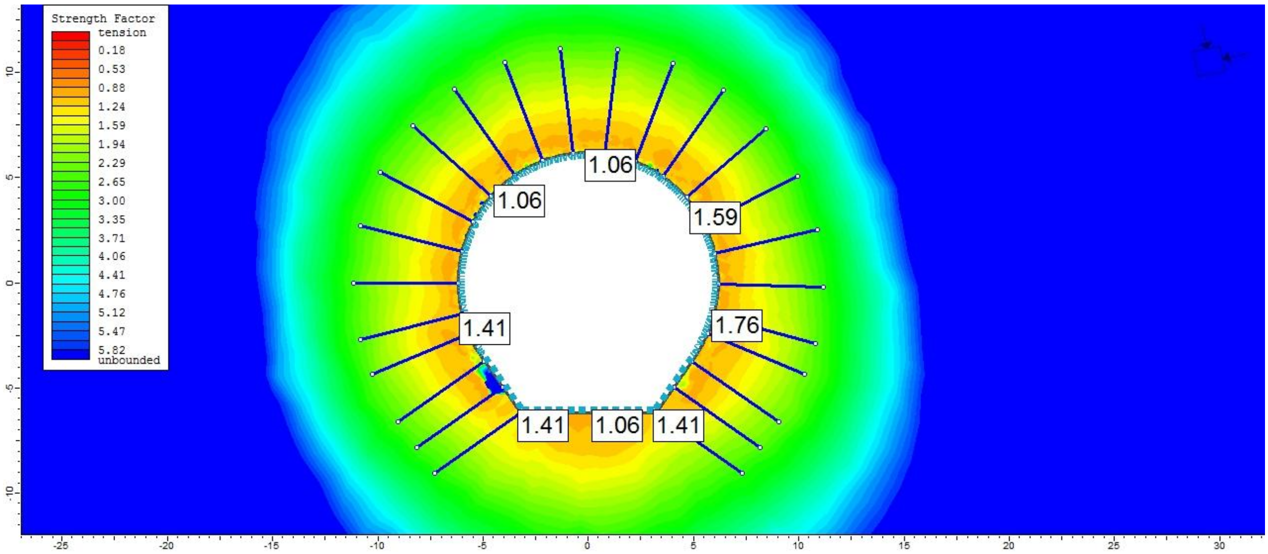


Figure 7.24 Strength factor for 1.2x1.2 R/B, 250mm thk SFRS & ISMB 250 @500at Ch.8584.3m

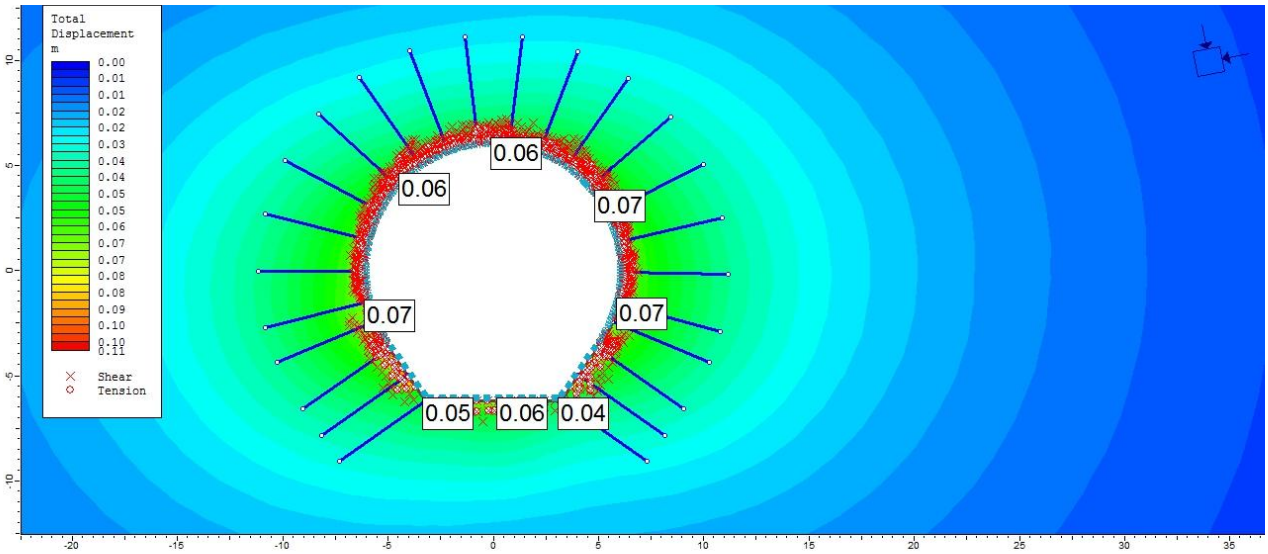


Figure 7.25 Total displacement for1.2x1.2 R/B, 250mm thk SFRS & ISMB @500at Ch.8584.3

The major principal stress is concentrated on the invert levels with smaller values on the crown. No rock failures from the crown are likely. The strength factor is same along the periphery of the opening. The deformation values are very small and have the maximum at the spring level and minimum on right invert.

7.5.4 Concluding Remarks

The possibility of hydraulic fracturing to the rock mass due to high static water pressure results was checked by empirical methods. The results give the tunnels safe from instability due to high static water pressure. This was further confirmed by the numerical valley model where minor principal stress values were found much higher than the hydrostatic pressure along all the three sections. Providing of final concrete lining in the whole reach of HRT sections will further add to the stability of the tunnels. However during the tunnel excavation, the actual site conditions can be updated and pre injection grouting will be needed where the tunnel seepages are high.

The orientation and the magnitudes of the principal stresses along the periphery of the tunnels are influenced greatly by the height of rock cover. The numerical modeling in the first and last sections where the rock over burden is not high and the topography of the sections influences the orientation of the principal stresses. In the middle section at Ch.4167 where the rock cover is fairly higher, the topography affect on the principal stress orientation is very small.

The tunnel deformation at the two sections at Ch.0.0m and Ch.8584.3m is small which needs very light support combination. Adopting heavy support system in these reach will only add to higher cost without affecting stability situation significantly. However the middle section at Ch.4167m due to high rock cover has high deformation values around the tunnel periphery. The empirical stability analysis also predicted some degree of deformation in this reach. Various rock support combination with systematic rock bolting, reinforced shotcrete and structural steel was carried out. The total deformation, principal stress and the yielded elements, and supports were considered while deciding the final support at different. However, the support systems calculated based on these results are not very realistic, for better reliable support design, a continuous design approach parallel with the excavation with actual stress and discontinuities inputs will be the best method of design approach.

8 Conclusions

Tunneling through the young Himalayan geological formation is very challenging job. The rock mass properties keep changing even within short distances propelling more challenges and uncertainties to the designers and executors alike. No single standard approach can be applied in such diverse Himalayan geological condition. The best approach could be one that adapts to the dictates of the field condition and comes up with adequate degree of safety standards within the available funding. The role of more detailed geotechnical investigation during the prefeasibility and feasibility study phase is very important. However due to the constraints of time and fund only the basic necessary investigations are carried out for most of the projects in developing countries. Following are some of the observations and recommendations from the findings of this thesis study.

The main foliation orientations were adequately taken care in the layout design of the HRT alignment. However there are still room for optimization to tunnel alignment with cross joints and other discontinuities within the same limits of the total project layout. These features were discussed under alternate layout chapter.

The overall rock mass parameters along the HRT alignment is through reasonably fair condition. The possibility of instability to the rock along the tunnel opening is ruled out both by the empirical analysis and further confirmed by the numerical analysis on hydraulic fracturing.

Minor rock falls and spalling from the crown and overt areas of the tunnel opening is possible since the rock mass strength in some cases along the alignment is lower than the induced stresses. This can be taken care by providing timely support combinations as deliberated in the support systems.

No major squeezing due to high stresses is expected during the tunnel execution. However minor squeezing problems in the reaches where high rock covers can be anticipated. Probe drilling and pre-injection grouting in susceptible areas of weakness zones can safe both time and money.

The support systems adopted for the HRT alignment is on higher side compared to the support requirement as per the NGIs Q method which infact was the basis of rock mass classification for this project. The numerical methods can only be used as a measure to cross check the designed supports rather than as method for designing support system. The reliability of the numerical modeling and analysis can be best achieved when the actual rock mass input parameters are considered from the actual tunneling conditions at site.

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Appendix A

A1: Stress calculation sheet

Description	Ch.0 to Adit I	Adit I to Adit II	Adit II to s/shaft
UCS of intact rock MPa	58.30	49.27	40.25
Hydrostatic Head (m)	39.50	58.43	78.50
Average Q	3.31	3.70	3.64
Average RMR	80.59	79.08	77.26
Average poisson υ	0.34	0.28	0.22
Average rock cover(m)	295.00	455.00	276.00
Rock density ϒr (tons/M^3)	2.60	2.59	2.58
E- modulus (Mpa)	6613.35	5511.60	4409.85
GSI value	45	45	45
mi material constant	30.00	30.00	30.00
Disturbance factor D	0.50	0.50	0.50
mb	2.19	2.19	2.19
s	0.0006	0.0006	0.0006
a	0.51	0.51	0.51
Vertical stress σv (Mpa)	7.66	11.77	7.11
Horizontal stress σh (Mpa)	9.01	9.58	6.97
K	1.18	0.81	0.98
Tangential stress roof σθr	20.26	17.92	14.49
Tangentia stress wall σθw	11.69	22.20	12.22
Rock mass strength			
Bieniawski MPa	25.97	20.61	15.60
Hoek and Brown 2002 Mpa	2.61	2.21	1.80
Barton 2000 MPa	8.36	7.86	6.29
Panthi 2006	7.42	5.76	4.26
Average rock mass strength (Mpa)	11.09	9.11	6.79
E- modulus			
Bieniawski 1978 Gpa	61.18	58.15	54.51
Barton 2002 Gpa	6.44	6.08	4.89
Hoek and Brown 1997 Gpa	5.73	5.26	4.76
Panthi 2006 Gpa	0.84	0.64	0.47
Deformation modulus value used is from Panthis realtion	0.84	0.64	0.47
Support pressure from RMR classification (KN/m2)	6.23	6.68	7.23
Deformation without support pressure εt in %	0.10	0.33	0.22
Deformation with support pressure εt %	0.003	0.06	0.05

A2: Project features of revised prefeasibility study report 2003 (WAPCoS, 2011)

Type of Development	Run-of-the-River
Catchment area:	7,007 km^2
Mean Annual Yield	11.027 mill m^3
Riverbed Elevation at dam site	788 m.a.s.l
HRWL	830 m.a.s.l
Live Storage HWL/LWL el. 830/el 813	5.176 mill m^3
Dead Storage Below 813 m.a.s.l	2.813 mill m^3
Tailwater level	563 m.a.s.l
Gross head	267 m
Design Discharge (8 units of @54.6m3/s)	437 m^3/s
Net head	254 m
Total efficiency	87.50%
Installed capacity 8 x 124 MW n= 375 rpm	992 MW
Estimated utilization (of mean annual yield)	69.80%
Mean Annual Energy Production	4,667 GWh
Firm Flow (available 90%) 0.222 x mean flow	77.6 m^3
Firm Power Continuous 24 h/day	165 MW
Firm Peaking Power 4 h/day	888 MW
Firm Peaking Energy	1297 GWh
Baseload outside peaking hours	85 GWh
Roads construction	3 km
Diversion Dam and quantity	Concrete gravity, 225,000 m^3
Dam height above Riverbed	42.5 m
Maximum Height Above Foundation	70 m
Crest Length	190 m
Design Flood (Q_{1000})	10128 m^3/s
Radial Crest Gate	4 nos. B-12 m , H=19 m
Low Level Radial Gates	4 nos .8.5x 10 m
Sediment basin	6 nos. Fnet= 242.8 m2, L= 250 m
Headrace Tunnel incl. sedimentation basin	3 nos each 11,500 m & A= 88.02 m2
Pressure shaft including horizontal part	8 Nos each 365 m and diameter D= 3.5 m.
Power House and Transformer Cavern	Underground, 130,000 m^3
Access Tunnel	1 No. 250 m long ,c/s area A= 50 m2.
Tailrace Tunnel including manifolds (submerged)	3 Nos. 350 m each with c/s area A= 88.02 m2.
Construction cost IPL 2003, including Transmission	875.1 mill USD
Construction time	5 years

A3: Project features of the detailed study report (WAPCoS,2011)

1. LOCATION	
District	Wangdue Phodrang
Dam site	Latitude 27^0 18.44'11", Longitude 89^0 57' 13.8"
	About 22.5 km D/S of Wangdi Bridge
Power House	Near Village Kamechhu
2. RESERVOIR	
Catchment Area	6835 km^2
Average Annual inflow	10092 MCM
MWL/FRL	El. 843 m
Gross Storage Capacity	7.0 MCM
Live Storage Capacity	4.64 MCM
Reservoir Area	0.3538 km^2
MDDL	825 m
Storage Capacity at MDDL	2.36 MCM
Storage Area at MDDL	0.1746 km^2
3. DIVERSION TUNNEL	
Location	Left bank
Length	828.25m
Size	12.0 m dia. circular (finished dimension)
Design Discharge	1118 cumecs
Gate	Vertical lift fixed wheel type gate; 2 Nos.; 5m (L) x 12m (H).
Intermediate adit	Length 131m, 7.5m x 7.5m – D-shaped
4. U/S COFFER DAM	
Type	Colcrete Dam
Length (Top)	170 m
Height	17.5 m
5. D/S COFFER DAM	
Type	Colcrete Dam
Length (Top)	150 m
Height	14.5 m
6. DAM	
Type	Concrete Gravity Dam
Top of Dam	EL 846 m
River Bed Elevation	EL 784 m
Length of Dam (top)	213.5 m
Max. Height of Dam	86 m (from deepest foundation level)
7. AUXILLARY SPILLWAY	
Type	Chute with gates

Elevation of overflow Crest	EL 839 m
Energy Dissipater	Bucket Type
Type of Gate	Vertical Lift.
Number of Gate	One service gate
Size of Gate	Width 4 m x Height 4 m
8.SLUICE SPILLWAY	
Design Flood	11723 m^3/s PMF + 4300 m^3/s GLOF
Type	Radial Gate
Number	Seven
Size of Gate	Width 8 m x Height 13.2 m
Crest Level	EL 797.0 m
Energy Dissipater	Bucket Type
9. POWER INTAKE	
Type	Horizontal (Circular)
Number	Four
Type	Straight Intake with bell mouth
Discharge Capacity	138.00 m^3/s per line x 4
Inner Diameter	6.4 m
Intake Center line level	EL 814.5 m
Gates	4 nos., 5.5 m x 6.4 m vertical lift fixed wheel type gate.
	One set 5.5 m x 6.4 m vertical lift fixed wheel emergency gate.
10. DESILTING BASIN	
Type	Underground
Number	Four
Size	Width 19 m x height 24.70 m x length 420m
Alignment	N $52^{0\,W}$ – S 80^0 E
Particle size to be removed	0.2 mm and above (suspended sediment)
Construction Adit	
Common length	120.6 m, 7.5x7.5 m – D shaped
Top branch	385.5 m, 7.5x7.5 m – D shaped
Bottom Branch	343.5 m, 7.5x7.5 m – D shaped
Silt Flushing Tunnel	
Size	5.1 m (W) x 5.5 m (H) – D shaped
Length	300 m
Flushing Discharge	0.2
Gates	4 nos., 3.60 m X 2.5 m Vertical Lift Slide Gate
Gate Chamber	
Adit to Gate Chamber	7.5 m x 7.5 m (D shaped), Length 452 m
Gates for Desilting Chamber	4 nos., 5.0 m X 6.4 m Vertical Lift Gate

11 HEAD RACE TUNNEL	
Number	One
Shape	Circular
Design Discharge	466.00 m^3/s
Inner Diameter	11.00 m
Length	8584.40 m
Slope	1 in 220.11
Invert level of HRT at 0 RD or at start	El.803.5 m
Invert level at junction with Surge Shaft	El. 764.50 m
Construction adits	
1) Adit I	Length 856.35 m, 7.5 m x 7.5 m – D Shaped; Invert level of HRT at Adit junction 798.715m.
2) Adit II	Length 556.70 m, 7.5 mx7.5 m – D Shaped; Invert level of HRT at Adit junction 781.965 m.
3) Adit near Surge Shaft	Common length with BVC – 282.244m, 8 x 8 m D-shaped; Balance length 248.8m, 7x7m D shaped; Invert level of HRT at Adit junction 764.965m.
Gate	1 no. flap gate hinge type 2.5m x 2.5m at Adit – II junction.
12. SURGE SHAFT	
Type	Orifice Type (Open to sky)
Number	One
Size	31.0 m Diameter
Orifice Size	2.8 m
Max Up/down surge	EL 895 m
Top Elevation	EL 900 m
Bottom Elevation	EL 763 m
Height of Surge Shaft	137.00 m
Gates	3 nos. each of size 5.50m x 4.35m
13. PRESSURE SHAFT	
Number	Three, each bifurcated to two branches, which feeds to individual turbines
Inclination	Vertical/Horizontal
Max. Diameter	5.5 m
Length of Pressure Shafts - I, II & III before bifurcation	997 m
Penstock Diameter	3.86 m
Length of penstocks	400m
Steel liner	ASTM 537 CL II from starting point at EL 770m to EL 610 m; Thickness varies from 22

	mm to 36 mm
	ASTM 517 Gr-F from EL 610m to EL 571.5 m, Thickness varies from 28 mm to 32 mm
Construction adit to Pressure Shaft top near vertical Drop.	8 m x 8 m – D shaped; Length 88.9m
Construction adit to Pressure Shaft Bottom near bifurcation.	8 m x 8.5 m – D shaped; Length 423.3m
14. BUTTERFLY VALVE CHAMBER	
Size	120m (L) x 12 m (W) x 21m (H)
Butterfly Valves	3 Nos. 5.5m dia. each.
Adit to BVC	Common length with HRT Adit – 282.244m, Balance length to BVC-192.8m, 8x8m D-shaped
15. POWER HOUSE	
Type	Underground
Size of Main cavern	Length 236 m x Width 23 m x Height 51 m
Rock pillar between Powerhouse and Transformer hall	40 m
Installed Capacity	6 x 170 MW (1020 MW)
Service Bay Level	EL 584 m
Center line of Machine	EL 571 m
Gates	6 nos., 5.0 m x 6.0 m vertical lift, fixed wheel type
16. ACCESS TUNNEL	
MAT	
Type	D-Shape
Size	Width 8 m x Height 8.5 m, Length 863.9 m
Other Tunnels	
Connecting Tunnel from Power House to Transformer Hall	D-Shape; Width 8m x Height 8m, Length 40 m
CAT to Transformer Hall cavern	D-Shape; Width 7m x Height 7m, Length 94 m
CAT to Power house	D-Shape, Width 7 m x Height 7 m, Length 228.2 m
Escape Tunnel from Power house to Transformer Hall	D-Shape, Width 4 m x Height 4.5 m, Length 40 m
17.CABLE TUNNEL	
Type	D-shape Tunnel + Cut & Cover Section
Size	Width 5 m x Height 7.0 m
Number	One
Length	Tunnel 241 m + 80 m cut & cover section
18.EHV CABLE	

Type	XLPE
Voltage	400 kV
Single/Three phase	Single
19. TRANSFORMER HALL	
Type	Underground
Size	14m (W) x 26.5 m (H) x 215.40 m (L)
Bus Duct	3 Bus Ducts 11 m x 7.75 m (D Shaped)
20. TAIL RACE TUNNEL	
Type	D Shaped
Number	One
Max. Discharge	460.00 m^3/s design discharge
Size	11 m diameter
Length	3000 m
TRT Adit	
Type	D Shape
Size	7 m x 7 m – D Shape, Length 473.80 m
Gate	2 nos, 6.5m x 11m each, vertical lift fixed wheel type
21. DOWN STREAM SURGE CHAMBER	
Number	One
Size	Length 319m x width 18m x height 58.5m
Bottom Elevation	565m
Top Elevation	623.5m
Access Tunnel to Draft Tube Gate Gallery	D-Shape, 7m x7 m, Length 592 m
22. GIS	
Type	Gas Insulated Switchgear (GIS)
Bus System	Double bus
Nos. of bays	12 + 2 for future
Voltage	400kV
23. POTHEAD YARD	
Number of bays	4 + 2 for future
Voltage	400 kV
Number of transmission lines	1 no. 400 kV D/C Transmission line to Alipurduar
Size of Pothead yard	200m x 40m
24. MAIN TRANSFORMER	
Type	Single phase, ODWF
Number	20 (including two spare)
Rating, Voltage ratio	70 MVA, 13.8/400/√3 kV
25. DESIGN PARAMETER-ELECTROMECHANICAL	

Gross Head	264 m
Design Head	236 m
Design Discharge/unit	76.67 m^3/s (with provision of 10% overloading)
Number of Unit	Six
Installed Capacity	990 MW (6 x 165)
Tail Water level Max.	581 m
Tail Water level Min.	579 m
26. TURBINE	
Type	Vertical Shaft, Francis Turbine
Synchronous Speed	250 rpm
Design Head	236.00 m
27. GENERATOR	
Type	Three Phase Alternating Current, Synchronous
Rated Output	165 MW
Synchronous Speed	250 rpm
Frequency	50 Hz
Generator Voltage	13.8 kV
28. ANNUAL ENERGY PRODUCTION	
Annual Energy in 90% dependable year	4214.56 GWh
Design Energy	4105.26 GWh
29. PLANT LOAD FACTOR	
Lean Period load factor	15.43% 90% dependable year
Av.Annual Load factor	48.60% 90% dependable year

Appendix B Standard charts and table

B1: NGIs Q Rating Parameters (Grimstad & Barton ,1993)

	RQD (Rock Quality Designation)	RQD
A	Very poor rock (> 27 joints per m^3)	0-25
B	Poor (20-27joints per m^3)	25-50
C	Fair (13-19 joints per m^3)	50-75
D	Good (8-12 joints per m^3)	75-90
E	Excellent (0- 7 joints per m^3)	90-100
Note: i) where RQD is reported or measures as ≤ 10 (including 0) the value 10 is used to evaluate the Q- value. ii) RQD interval of 5 i.e. 100, 95 90 are sufficiently accurate.		

Table for Joint Set Number

2. Joint set number		Jn
A	Massive, no or few joints	0.5- 1.0
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint set plus random joints	6
F	Three joint set	9
G	Three joint set plus random joints	12
H	Four or more joint sets, random heavily jointed " sugar cube " etc.	15
J	Crushed rock, earth like	20
Note: i) for intersection, use 3xJn, ii) For portals, use 2xJn.		

Table for Joint Roughness Number

3. Joint Roughness Number		Jr
	Rock-wall contact, b) Rock wall contact before 10cm	
A	Discontinuous joints	4
B	Rough or irregular , undulating	3
C	Smooth , undulating	2
D	Slickensided, undulating	1.5
E	Rough, irregular, planer	1.5
F	Smooth planer	1.0
G	Slickensided planer	0.5
Note: description refers to small scale features and intermediate scale features, in that order		
c) No rock-wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock-wall contact	1
J	Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact.	1
Note: i) Add 1 if the mean spacing of the relevant joint set is greater than 3m. ii) Jr = 0.5 can be		

used for planer slickensided joints having lineations, provided that the lineations are oriented in the estimated sliding direction.

Table for Joint Alteration Number

4) Joint alteration number		**Φ**r (approximate)	Ja
a)Rock-wall contact (No mineral filling , only coatings)			
A	Tightly healed , hard, non-softening, impermeable filling, i.e quartzite or epidote		-.75
B	Unaltered joint walls, surface staining only.	25^0-35^0	1
C	Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	25^0-30^0	2
D	silty or clay mineral coating , small clay fraction (non-softening)	20^0-25^0	3
E	Silty or clay coating, small mineral coating, i.e Kalonite or Mica, also chlorite, talc, gypsum, graphite, etc. and small quantities od swelling clays.	8^0-16^0	4
b) Rock-wall contact before 10cm shear (thin mineral filling)			
F	Sandy particles, clay-free disintegrated rocks, etc.	250-30^0	4
G	Strongly over consolidated, non-softening , clay mineral filling (continuous but <5mm thickness)	16^0-24^0	6
H	Medium or low over consolidated , softening , clay mineral filling (continuous but <5mm thickness)	12^0-16^0	8
J	Swelling clay filling, i.e. monmorillonite (continuous but <5mm thickness, values Ja depends on percent of swelling clay sized particles)	6^0-12^0	8-12
c) No rock-wall contact when sheared (thick mineral filling)			
K	Zones of bands of disintegrated or crushed rock. Strongly over-consolidated	6^0-24^0	6
L	Zones or bands of clay, disintegrated or crushed rocks. Medium or low over-consolidated or softening fillings	12^0-16^0	8
M	Zones or bands of clay, disintegrated or crushed rock. swelling clay. Ja depends on percent of swelling clay-size particles.	6^0-12^0	8-12
N	Thick continuous zones or bands of clay, strongly over-consolidated.	6^0-12^0	10
O	Thick continuous zones or bands of clay. Medium to low over-consolidation.	16^0-24^0	13

P	Thick continuous zones or bands of clay. Swelling clay. Ja depends on percent of swelling clay-sixe particles	12^0-16^0	13-20

Table for Water reduction Factor

5. Joint Water Reduction factor		Jw
A	Dry excavation or minor flow i.e < 5l/min. locally (humid or a few dripping)	1.0
B	Medium inflow or pressure, occasional outwash of joints filling (many dripping)	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	0.5
D	Large inflow or high pressure , considerable outwash of joint filling	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time.	0.2-0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay.	0.1-0.05
Note: i) factor C to F are crude estimates. Increase Jw if drainage measures are installed. ii) special problems caused by ice formation are not considered.		

Table for Stress Reduction Factor

6) Stress reduction factor		SRF
Weakness zones intersection excavation, which may cause loosening of rock mass when tunnel is excavated.		
A	Multiple occurrence of weakness zones containing clay or chemically disintegrated very loose surrounding rock (any depth)	10
B	Single weakness zone containing clay or chemically disintegrated rock (depth of excavation ≤ 50m).	5.0
C	Single weakness zone containing clay or chemically disintegrated rock (depth of excavation > 50m)	2.5
D	Multiple shear zone in competent rock (clay free) loose surrounding rock (any depth)	7.5
E	Single shear zone in competent rock (clay-free) (depth of excavation ≤ 50m).	5.0
F	Single shaer zone in competent rock (clay-free) (depth of excavation > 50m)	2.5
G	Loose, open joints, heavily jointed or “ sugar cube” , etc. (any depth)	5.0
Note : i) Reduce these values of the SRF by 25-30% if the relevant shear zones only influence but do not intersect the excavation.		

b) competent rock, stress problem		$\frac{\sigma_c}{\sigma_1}$	$\frac{\sigma_{c\theta}}{\sigma_{c1}}$	SRF
H	Low stress near surface open joints	>200	<0.01	2.5
J	Medium stress favorable stress condition	200-10	0.01-0.3	1
K	Highly stressed very tight structure. usually favorable to stability, may be unfavorable to wall stability	10-5	0.3-0.4	0.5-2
L	Moderate slabbing after > hour in massive rock.	5-3	0.5-0.65	5-50
M	Slabbing and rock burst after a few minutes in massive rock.	3-2	0.65-1.0	50-200
N	Heavy rock burst and immediate dynamic deformation in massive rock	< 2	1.0	200-400
Note : ii) for strongly anisotopic virgin stress fields (if measures): when $5 \leq \sigma 1/\sigma 3 \leq 10$, reduce σc to 0.75σc .when σ1/σ3 >10 , reduce σc to 0.5σc, where σc = unconfined compressive strength , σ1 and σ3 are major and minor principal stresses, and σθ is maximum tangential stress (estimated from elastic theory). iii) Few cases record available where depth of crown below surface is less than span with. Suggest SRF increase from 2.5 to 5 for such cases.				
C) Squeezing rock : plastic flow on incompetent rock under the influence of high pressure.		σθ/σc		SRF
O	Mild squeezing rock pressure	1-5		5-10
P	Heavy squeezing rock pressure	<5		10-20
Note: iv) cases of squeezing rock may occur for the depth $H < 350Q^{1/3}$ (Singh et al.199) , Rock mass compression strength can be estimated from $\sigma cm = 0.7 \Upsilon Q^{1/3}$ (MPa) where ϒ = rock density in KN/m3 (Singh ,1993)				
d) Swelling rock: Chemical swelling activity depending on the presence of water.				SRF
R	Mild swelling rock pressure			5-10
S	Heavy swelling rock pressure			10-15

B3: Excavation Support Ratio

Types of excavation		ESR
A	Temporary mine openings, etc	Ca. 3.5
B	Vertical shaft : i) circular sections ii) Rectangular /square sections	Ca 2.5 Ca 2.0
C	Permanent openings, water tunnel for hydropower exclude high pressure penstocks), pilot tunnels, drifts and heading for large openings.	1.6
D	Storage rooms, water treatment plants, minor roads and railway tunnels, surge chamber, access tunnels etc.	1.3

E	Power station, major road and railway tunnels, civil defense chambers, portals , intersections etc.	1.0
F	Underground nuclear power station , railway stations, sports and public facilities, factories, etc.	0.8
G	Very important caverns and tunnels with long lifetime, tunnels for gas pipe lines.	0.5

B3: RMR classification of rock mass rating (Bieniawski, 1989)

A Classification parameters and their ratings

Parameter			Range of values// Rating						
1	Strength of intact rock.	Point load strength index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range uniaxial compressive strength is preferred		
		Uniaxial compressive strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality		90-100%	75-90%	50-75%	25-50%	<25%		
	Rating		20	17	13	8	5		
3	Spacing of discontinuities		>2m	0.6-2m	200-600mm	60-200mm	<60 mmm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities	Length, persistence	<1 m	1-3m	3-10m	10-20m	>20m		
		Rating	6	4	2	1	0		
		separation	none	<0.1mm	0.1-1mm	1-5mm	>5mm		
		Rating	6	5	4	1	0		
		Roughness	Very rough	rough	Slightly rough	smooth	slickensided		
		Rating	6	5	3	1	0		
		Infilling (gouge)	None	hard filling <5mm	>5mm	Soft filling <5mm	>5mm		
		Rating	6	4	2	2	0		
		Weathering	unweathered	Slightly w	Moderately w	Highly w	Decomposed		
		Rating	6	5	3	1	0		
5	Ground	Inflow per	none	<10	10-25	25-125	>125 lit/min		

	water	10m tunnel length		lit/min	lit/min	lit/min	
		P_w/σ_1	0	0-0.1	0.1-0.2	0.2-0.5	>0.5
		General condition	Completely dry	damp	wet	dripping	Flowing
		Rating	15	10	7	4	0
P_w= joint water pressure, σ_1= major principal stress							

B Rating adjustment for discontinuity orientation

		Very favourable	favourable	fair	unfavourable	Very unfavourable
Rating	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	slopes	0	-5	-25	-50	-60

C Rock mass classess determination from total rating

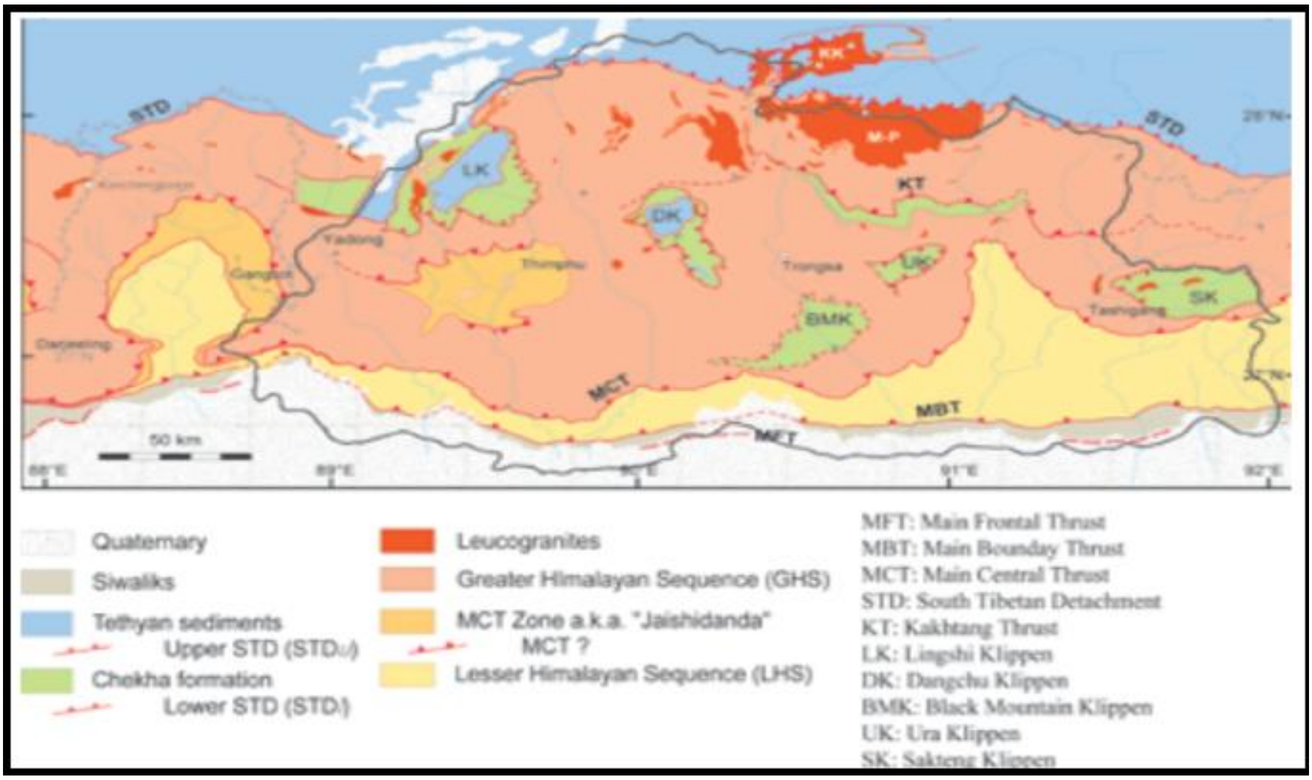
Rating	100-81	80-61	60-41	40-21	<20
Class no.	I	II	III	IV	V
Description	Very good	Good	Fair	Poor	Very poor

D Meaning of rock mass classes

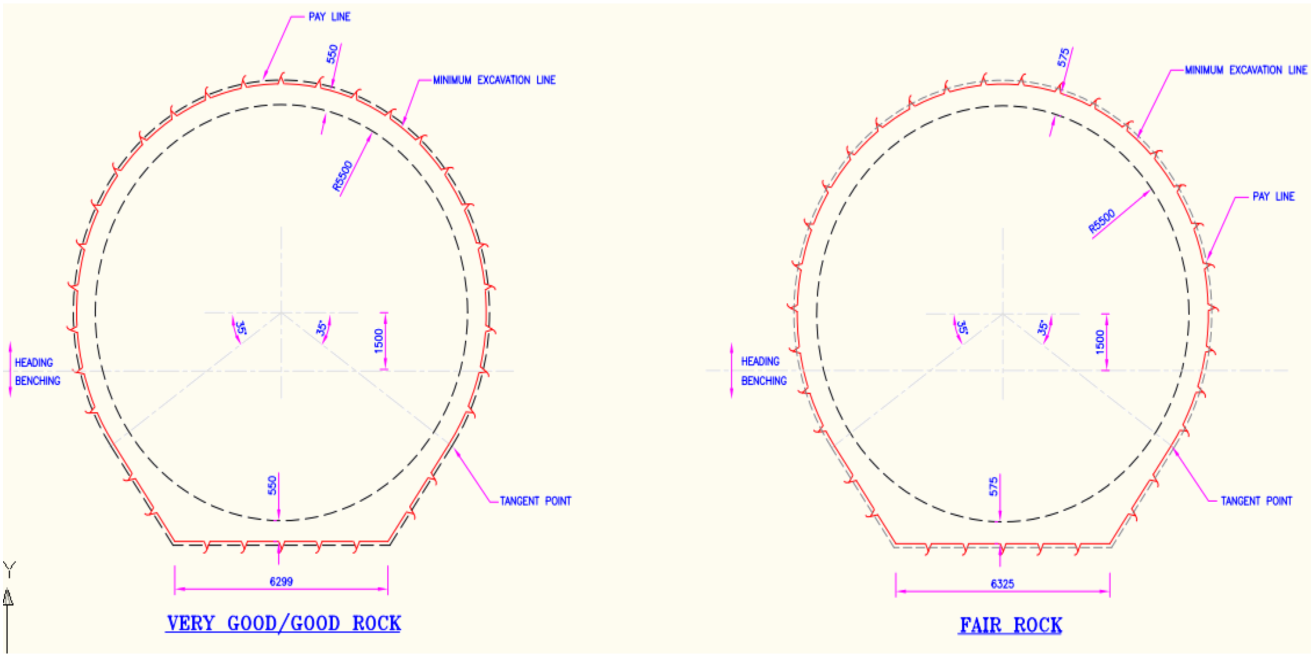
Class No	I	II	III	IV	V
Average stand up time	10 years for 15m span	6 months for 8m span	1 week for 5m span	10 hours for 2.5m span	30 minutes for 1m span
Cohesion of the rock mass	>400 Kpa	300-400Kpa	200-300 Kps	100-200 Kpa	<100 Kpa
Friction angle of the rock mass	$<45^{0}$	$35\text{-}45^{0}$	$25\text{-}35^{0}$	$15\text{-}25^{0}$	$<15^{0}$

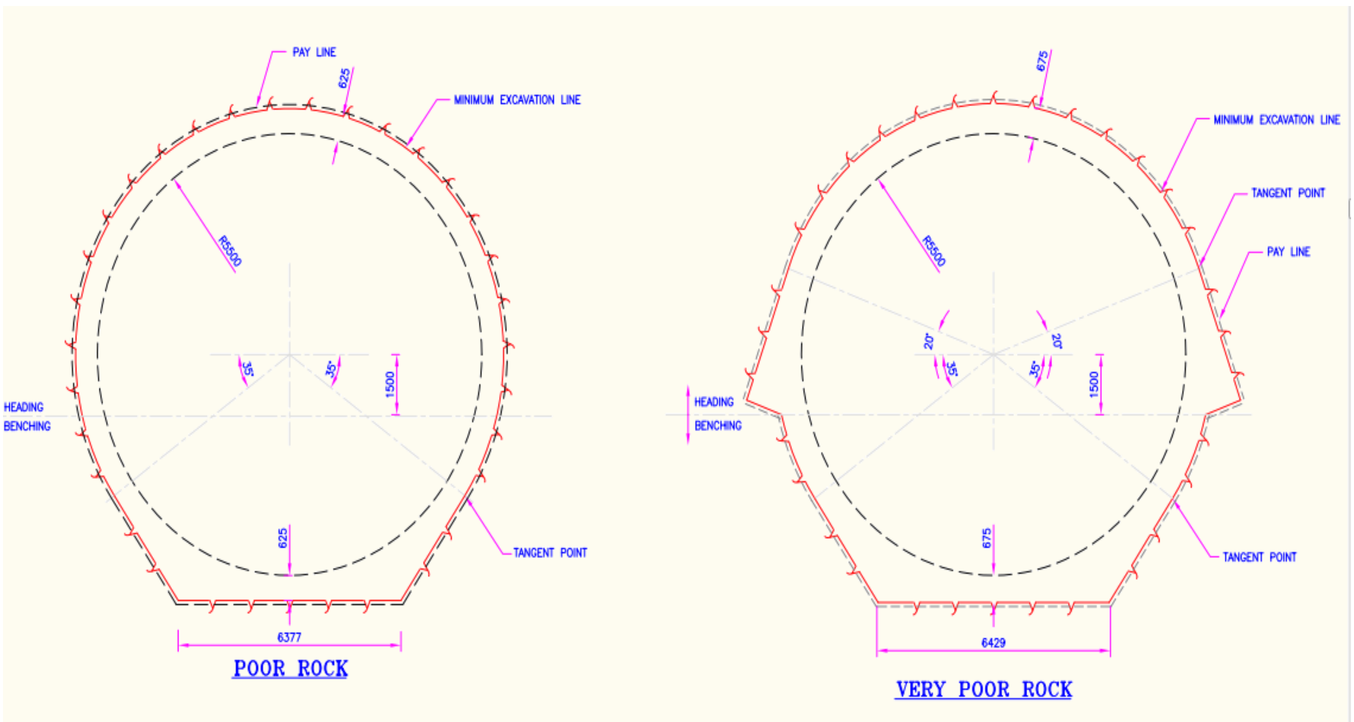
Appendix C Maps and figures

C1: Geological map of Bhutan (Gucci,2000)

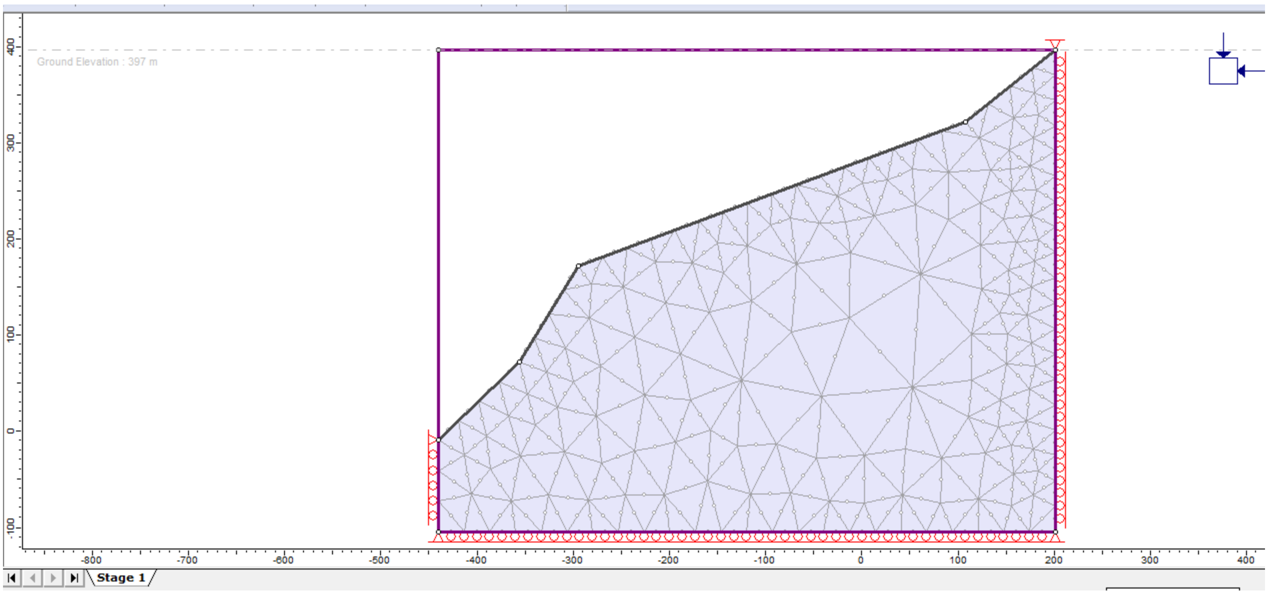


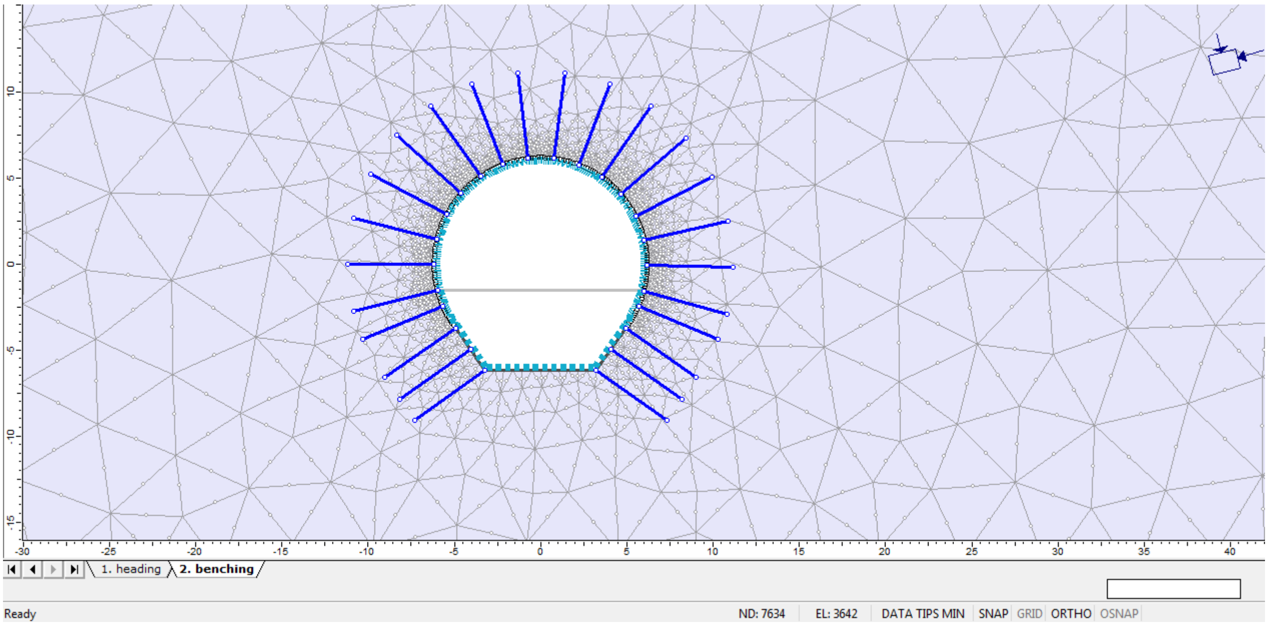
C2: Excavation section for HRT in different class of rocks



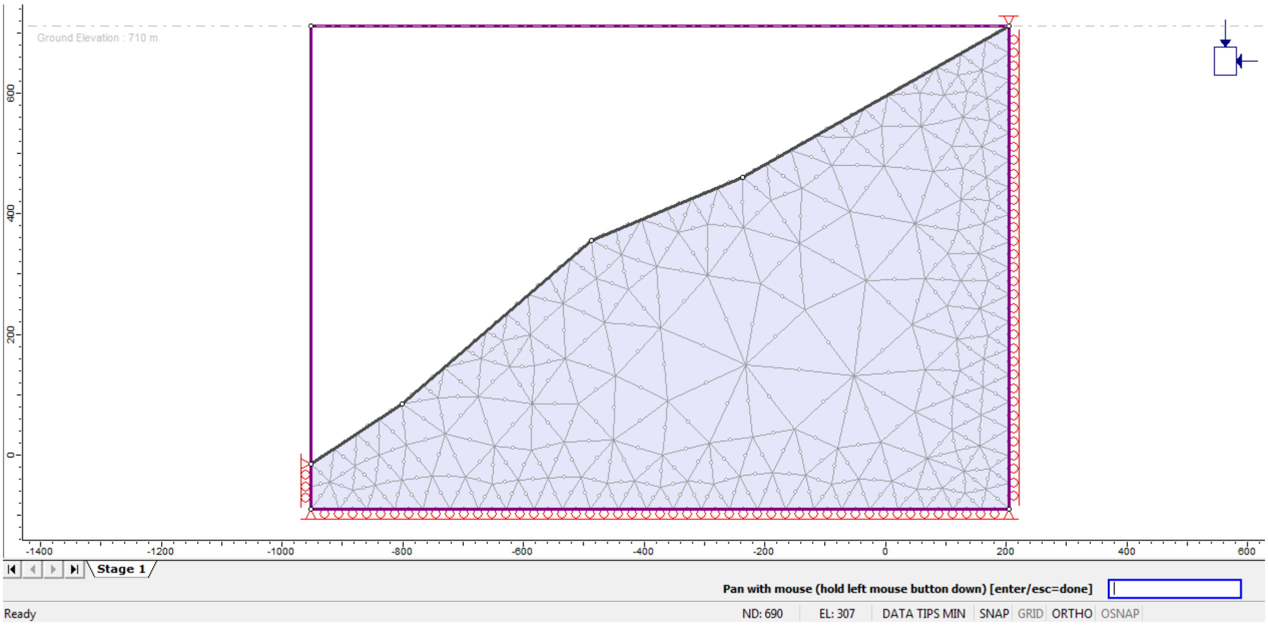


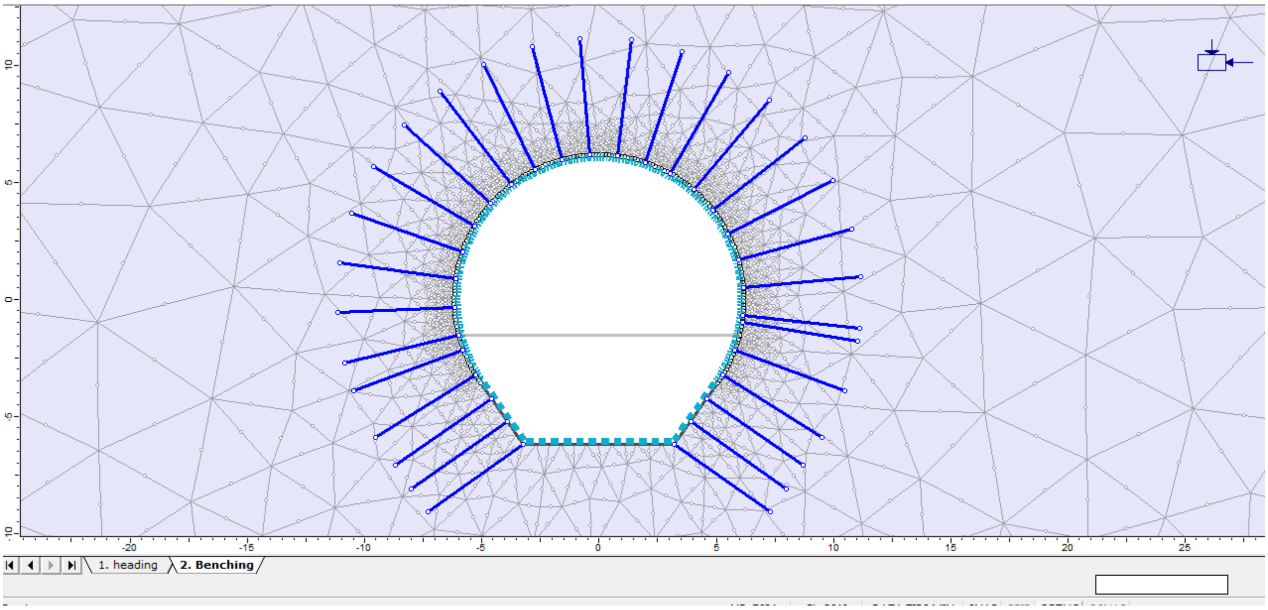
C3: Valley slope models and excavation support model at Ch 0.0m





C4 Valley slope model and excavation support model at Ch.4167m





C5: Valley slope model and excavation support model at Ch.8584.28m